DEVELOPMENT OF SATURATION FLOW AND DELAY MODELS FOR SIGNALISED INTERSECTION IN DHAKA CITY

A Thesis

Submitted by

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January, 2008 **Md. Hadiuzzaman**

This thesis is dedicated to my mother Hazera Khatun and my wife Tamanna Sharker. Their continuous inspirations made this effort possible.

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List of Abbreviations

c = Capacity of intersection approach

C = Traffic signal cycle length (sec)

CF = Adjustment factor for control type,

d = Average delay (sec/veh)

d_1 = Average uniform delay (sec/veh)

d_2 = Incremental, or random, or stopped delay (sec/veh)

d_3 = Residual demand delay to account for over saturation queues that may have existed before the analysis period (sec/veh)

d_{ad} = Acceleration/Deceleration Delay

d_f = Average field delay

DF = Delay adjustment factor for quality of progression and control type

f_a = Adjustment factor for area type,

f_{bb} = Adjustment factor for blocking effect of local buses,

f_e = Adjustment factor for parking activity,

f_g = Adjustment factor for approach grade

f_{HV} = Adjustment factor for heavy vehicles,

f_{Lpb} = Pedestrian adjustment factor for left turn,

f_{LT} = Adjustment factor for left turn,

f_{LU} = Adjustment factor for lane utilisation,

fp = Supplemental adjustment factor for when the platoon arriving during green.

f_{Rpb} = Pedestrian adjustment factor for right turn.

f_{RT} = Adjustment factor for right turn,

f_w = Adjustment factor for lane width,

g = Effective green time for lane group (sec)

g_s = Saturated green

h = Saturation flow headway

HCM = Highway Capacity Manual

I = Upstream filtering/metering adjustment factor

l_1 = Start-up lost time (sec)

k = Incremental delay factor dependent on signal controller setting

L = Total lost time in cycle (sec)

m = Incremental delay calibration term

N = No. of lanes in a approach,

NMV = Non Motorized Vehicle

PCU_{LB} = Passenger Car Unit for large bus

PCU_{MB} = Passenger Car Unit for mini bus

PCU_C = Passenger Car Unit for car

PCU_{AR} = Passenger Car Unit for autorickshaw

PCU_{MC} = Passenger Car Unit for motor cycle

PF = Adjustment factor for the effect of the quality of progression in coordinated system

P_s = Proportion of vehicles required to stop in the intersection approach

Q = Expected overflow queue length (veh)

q = Arrival flow rate

RAJUK= Rajdhani Unnayan Katripakhya

RHD = Roads and Highway Department

r = Effective red time (sec)

R^2 = Coefficient of Determination

RMSE = Root Mean Square Error

s = Saturation flow rate

s_o = Base saturation flow rate per lane, pc/hr/lane,

T = Duration of analysis period in time dependent delay models

t_i = Incremental headway for i^{th} vehicle

V_{iq} = Sum of vehicle-in-queue counts, veh,

V_{tot} = Total number of vehicles arriving during the survey period, Veh

w = Width of approach in meter

X = v/c ratio for a approach

TEDI = Two dimensional editing interface

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Abstract

Traffic in Bangladesh consists of both motorized and non-motorized vehicles, as in many other developing countries. The static and dynamic characteristics of the different vehicle vary widely even within the same class. Also, the lack of lane discipline and unrestricted mixing of the various types of vehicle in the same right of way makes the traffic stream heterogeneous in nature. The equation of delays and optimization of the signal timing which involve in predicting passenger car unit (PCU) and saturation flow, developed for use in developed countries, do not take into account the non-lane based traffic conditions prevalent in Bangladesh. However, the increasing number of vehicles and limited new road construction makes the correct prediction of these parameters important so as to make the most efficient use of existing road system by providing better traffic operation and control.

Intersections are nodal points in transportation network and their efficiency of operation greatly influences the entire network performance. Capacity analysis of signalized intersections basically consists of estimating saturation flow and delay. Pretimed signals are most commonly used in developing countries. Present study deals with development of saturation flow and delay models for pretimed signalized intersections with reference to non-lane based traffic condition prevailing in Bangladesh.

To carry out analysis traffic data were collected at five signalized intersections in Dhaka city. Video camera was used to collect field data because it provides permanent record of data with minimum manpower. Recorded film was replayed in the laboratory and required information for saturation flow and delay estimation was retrieved.

Passenger car unit (PCU) values for each vehicle is found out using synchronous regression technique and a range of site-specific PCU values were obtained. The saturation flow for each survey approach was calculated using the average PCU values and multiple linear regression techniques were then used to derive predictive saturation flow models. The saturation flow values for full approach were regressed against several intersection characteristics such as signal timing, % right turn, approach width and exit

width. This model shows good correlation with field observed saturation flow and can be used to estimate saturation flow in non-lane based traffic condition.

Field delay for each approach is calculated based on HCM 2000 guidelines. Theoretical delay is estimated by various delay models and compared with field delay. Based on results obtained from analysis, HCM 2000 delay model has been selected to modify. It has been observed that HCM 2000 delay model consistently over estimate delay at degree of saturation more than 1.0. It has been suggested from the analysis that theoretical incremental delay (due to random arrival and over saturated queues) in HCM 2000 delay model be reduced by 70 % to better reflect field conditions. Recommended delay model shows good correlation for the present case but needs to be refined further to become suitable for all range of input variables.

Finally, simulation technique has been adopted to estimate number of stops incurred by each vehicle facing an over saturated signalized approach. The research effort also deals with the generation of a stop estimation analytical model for over-saturated conditions through the establishment of an upper bound for the number of vehicle stops at oversaturated conditions using observed simulated data.

Chapter 1

INTRODUCTION

1.1 General

The growth of traffic in the road network of large cities in developing countries, like Bangladesh, is a serious concern from the traffic engineer's point of view. The congestion at the intersection is most crucial because the performance of intersection affects the performance and productivity of the whole road network most significantly. To reduce conflicts and ensure orderly movement of traffic at the urban intersections, it is common practice to introduce fixed time traffic signals at uncontrolled or priority controlled or traffic police controlled intersections if the conditions warrant its choice.

In Bangladesh, to control the traffic at Dhaka and several other large cities, fixed time traffic signals are being used for a significant number of years. Unfortunately, rather than using any traffic engineering knowledge, these signals have been timed by traffic police from arbitrary judgment only. As a result, they became ineffective to serve the purpose properly and efficiently. Recently, the traffic signals of Dhaka city have improved by introducing Multiplan signal controller along with necessary road marking and chanelisation. This improvement has been done by hiring consultants from abroad. As a result, the optimization has become compatible to the situation of western countries.

The important parameters in the planning, design and control of a signalized intersection are saturation flows, lost times and passenger car units(PCU). These factors have been traditionally measured, in most western countries, based on the research carried on test tracks and on public roads (RRL 1963; HRB 1965; Webster et al 1966; Miller 1968) where traffic is typically car-dominated with vehicles moving in clearly defined lanes.

However, the traffic movement in Bangladesh and in other developing countries is rendered in more complex due to the heterogeneous characteristics of the traffic stream using the same right of way. The stream includes slow push carts at one extreme and the fast moving passenger cars at the other, with many intermediate types of vehicles depicting a wide variation in static and dynamic characteristics. As in heterogeneous traffic operation no single vehicle clearly dominates the traffic stream, prediction of saturation flow is more sensitive to the vehicle mix than in western countries where the traffic is largely motorised and car dominated.

Moreover, because of increasing number of vehicles on Bangladeshi roads and the lag in roads and the lag in road construction, attention is to be given to traffic operation and control measures to make the most efficient use of existing road systems. Traffic control in Bangladesh has long been dependent on the use of fixed cycle traffic signals, due to cheap installation and maintenance cost, but this demands more critical calculation of signal setting and hence accurate measurement of signal design parameters.

Another striking feature of the road traffic operating condition in developing countries is that, despite having lane markings, most of the times lane discipline is not followed no matter whether non motorized vehicle is present or not. At intersection, there is notable lateral movement and vehicles tend to use lateral gaps to reach the head of the queue.

Due to these fundamental differences, the standard western relationships for predicting the values of saturation flows, lost time, and PCU factors are not appropriate for developing countries. For correct signal design these parameters should be estimated based on the local prevailing traffic conditions and hence requires a different approach to analysis.

Delay is an important parameter for the measurement of the level of performance of signalized intersection. Well-defined procedure is available to measure delay for developed countries. Highway Capacity Manual (HCM) is widely used for capacity analysis of signalized intersection in North America and other developed countries. HCM and all delay models have been developed assuming lane disciplined and more or less uniform traffic. Their applicability to non-lane based traffic conditions needs to be checked.

Again, for many years measurement of the level of performance of signalized intersections has primarily focused solely on vehicle delay. Besides delay, other performance measures such as the number of vehicle stops and the spatial extent of queues on intersection approaches have also been found to play an important role in the performance evaluation of signalized intersections of developing countries, where most of the times the intersection remains near or at oversaturated condition. Hence, stop and go situation is very frequent at intersection. These measures not only relate to the level of service that is provided to the drivers, but also to the level of fuel consumption and air pollution that is generated by the vehicles traversing the signalized intersections. In particular, while vehicle stop estimates play an important role in determining vehicle fuel consumption and emissions on intersection approaches, queue length estimates are important not only for the design of pocket lanes, but also to ensure that traffic signal operations do not result in vehicle queues that spillback onto upstream intersections.

1.2 Problem Definition

Highway Capacity Manual (HCM) is widely used for the design of signalized intersection in North America and other developed countries. Again capacity analysis of an existing signalized intersection mainly includes determination of saturation flow and delay. HCM and other works assume homogeneous and lane based traffic for analysis, which exists in those countries. But traffic flow in countries like Bangladesh consists of different classes of vehicles having no lane disciplined. Present practice is to convert all classes of vehicles into single unit called passenger car unit (PCU). Due to fundamental differences, the standard western relationships for predicting the values of saturation flows, lost time, and PCU factors are not appropriate for developing countries. This problem needs to be solved. There are no proper guidelines available to estimate saturation flow for non-lane based traffic conditions. All delay models have been developed assuming disciplined and more or less uniform traffic. Their applicability to non-lane based traffic conditions needs to be checked. Effect of lack of lane discipline on capacity analysis needs to be considered. HCM 2000 has given a range of control delay values to find out level of service (LOS) of signalized intersections. This range may not be applicable to Bangladeshi traffic conditions.

1.3 Objectives and Scope

The research mainly attempts to study parameters that influence the design and performance of signalized intersections and to suggest modification with respect to non-lane based traffic condition. The specific objectives are summarized as follows:

1. To measure appropriate passenger car unit (PCU) values for different types of vehicle.

2. To develop saturation flow model by regression method.

3. To develop delay model for non-lane based traffic condition.

4. To study applicability of delay as a level of service parameter and to redefine LOS parameter for non-lane based traffic condition.

5. To estimate number of stops per vehicle at signalized intersection by micro-simulation technique

It is expected that the outcome of this research will be able to develop analytical tools for the design and evaluation of performance of new and existing isolated signalized intersections in urban areas under non-lane based traffic conditions.

1.4 Methodology

Three types of research approach are generally used in the field of traffic engineering. They are:

(i) Empirical approach
(ii) Analytical approach
(iii) Simulation approach

In this study part of analyses is carried out by empirical approach and remaining part by simulation approach.

Digital video camera was used to collect data on the field. Video based technique overcomes many of the difficulties of collecting traffic data. The video camera takes continuous picture of the traffic flow. A number of intersections are selected in Dhaka city, for the analysis. All signals were pre-timed signals. Simultaneously data on signal timing i.e. cycle length, number of phases and phase length was collected manually using stopwatch.

The recorded film was replayed in the laboratory on a large screen and required information was retrieved. PCU values were calculated using synchronous regression technique and using these values saturation flow was estimated in PCU per hour for each approach. Finally, saturation flow model is developed for non-lane based traffic condition by regression analysis.

Field measurement of delay has been done by the method suggested in HCM 2000, which is based on direct observation of vehicle in queue counts at the intersection. The method does not directly measure delay during deceleration and acceleration. Time-in-queue is measured by counting number of vehicles in queue at regular interval of 10 to 20 seconds. Acceleration and deceleration delay is estimated as suggested in HCM 2000 and added to time-in-queue to get the control delay.

The delay that a particular vehicle experience, when it travels through signalized intersection approach depends on number of factors such as arrival flow rate, distribution, signals timings, etc. In a real application environment, many of these factors are random variables, which make the accurate prediction of delay a very complicated process. There are number of theoretical models available for delay estimation. The delay at each intersection is estimated by the following models.

- Webster delay model
- TRANSYT model
- Ackelik's model
- Reilly's model
- Highway Capacity Manual (HCM) 2000 model

These models predict the delay satisfactorily in developed countries. To check the applicability of these models to non-lane based traffic conditions, values obtained by these models are compared with the field observed values. Based on regression analysis modified delay model is suggested for non-lane based traffic conditions prevailing in Dhaka metropolitan.

In this study two approaches for computing vehicle stops at signalized intersections have been introduced. The first approach that is introduced is a microscopic model that computes stops for over-saturated conditions by AIMSUN 5.0 micro simulator where a vehicle is considered "stopped" when its speed decreases below the user specific queue threshold value (in meter/sec). Partial calibration of the model was carried out by entering different vehicle characteristics into the model through TEDI (Two Dimensional Editing Interface) that was investigated by Hoque 1994 for Bangladesh traffic condition. Validation of the model was carried out from field observed delay. The second model that is introduced is an analytical formulation derived from the microscopic model that computes the number of vehicle stops for over-saturated approaches considering signal timings, approach arrival rate, approach saturation flow rate, and analysis period as model independent variables.

1.5 Organization of the Thesis

The thesis consists of eight chapters including the present one. The first chapter deals with the introduction, problem statement, objectives and methodology. Chapter two introduce traffic flow characteristic at signalized intersections. Chapter three gives insight into the capacity and level of service of signalized intersections. This chapter also presents the review of various delay models. Effect of non lane based traffic condition in capacity analysis has been discussed at the end of chapter. Chapter four describes the selected intersections and data collection technique. This chapter starts with the discussion of the study area and then gives details about selected intersections. Measurement of saturation flow and delay is presented at the end. Chapter five is devoted to the determination of PCU value for different types of vehicle and analysis of saturation flow using regression analysis. Chapter six deals with field measurement of delay, estimation of delay by theoretical models and developing modified delay model. Chapter seven gives detail about formulation of analytical equation to estimate number of stops for over saturated condition from simulation result. Summary, conclusions and future scope is presented in chapter eight.

Chapter 2

BASIC ASPECTS OF SIGNALISED INTERSECTIONS

2.1 General

Intersection may be signalized for number of reasons, most of which relate to the safety and effective movement of conflicting vehicular and pedestrian flows through intersection. Two concepts are of important in understanding signalized intersection design and operation: 1) The time allocation of the 3600 seconds in an hour to conflicting movements and to "lost times" in the cycle. 2) The effect of right-turning vehicles on the operation of the intersection. This chapter discusses the basic principles of traffic behavior at signalized intersection. At the end capacity analysis concept has been introduced.

2.2 Terminology and Key Definitions

The following terms are commonly used to describe traffic signal operation.

Cycle: one complete sequence of signal indications

Cycle length (C): total time for the signal to complete one cycle, generally express in second.

Phase: part of cycle allocated to any combination of traffic movements receiving the right of way simultaneously during one or more interval.

Interval: period of time during which all signal indications remain constant

Change interval (Y): the "yellow" and /or "all-red" intervals, which occur at the end of a phase to provide for clearance of the intersection before conflicting movement are released also known as "Amber Period".

Green time *(G):* time within a given phase during which the "green" indication is shown, stated in seconds.

Lost time: time during which the intersection is not effectively used by any movement, which occur during the change and clearance intervals (when the intersection is cleared) and at the beginning of each phase as the first few vehicles in a standing queue experience start-up delays.

Effective green time *(g):* time during which a given phase is effectively available for stable moving platoons of vehicles in the permitted movements, generally taken to be the green time plus the change and clearance interval minus the lost time for the designated movement, stated in seconds.

Green ratio *(g/C):* ratio of effective green time to the cycle length

Effective red *(r):* time during which a given movement or set of movements is effectively not permitted to occur, the cycle length minus the effective green time, stated in seconds.

Traffic signals may operate in three basic modes, depending upon the type of control equipment used:

1. ***Pretimed operation:*** In pretimed operation, the cycle length, phases, green times, and change intervals are all preset. The signal rotates through this defined cycle in constant fashion. Each cycle is same, with the cycle length and phase lengths constants.

2. ***Semi - actuated operation:*** In semi - actuated operation, the designated main street has a "green" indication at all times until detectors on the side street determine that a vehicles have arrived on one or both of the minor approaches. The signal then provides a "green" phase for the side street, after an appropriate change interval, which is retained until all vehicles are served, or until a preset maximum side-street green is reached. In this type of operation, the cycle length and green times vary from cycle to cycle in response to demand.

3. ***Full- actuated operation***: In full-actuated operation, all signal phases are controlled by detector actuations. In general, minimum and maximum green times are specified for each phase. In this type of control, cycle length and green times may vary considerable in response to demand.

Signal phasing can provide for either protected or permitted turning movements.

A ***permitted*** turning movement is made through a conflicting pedestrian or an opposing vehicle flow. Thus, a left turn movement that is made at the same time as the opposing through movement is considered to be permitted, as is a right-turn movement made at the same time as pedestrian crossing in a conflicting crosswalk.

Protected turns are those made without these conflicts, such as turns made during an exclusive left-turn phase or a right-turn phase during which conflicting pedestrian movements are prohibited. Permitted turns experience the friction of selecting and passing through gaps in a conflicting vehicle or pedestrian flow. Thus, a single permitted turn often consumes more of the available green time than a single protected turn.

Either permitted or protected turning phases may be more efficient in a given situation, depending on the turning and opposing volumes, intersection geometry, and other factors.

2.3 Signalized Intersection Flow Characteristics

For a given approach at signalized intersection, three signal indications are seen: green, yellow, and red. The indication may include a short period during which all indications are red, referred to as an all-red interval, which with the yellow indication forms the change and clearance interval between two green phases. Figure 2.1 presents some fundamental attributes of flow at signalized intersection. The diagram represents a simple situation of one-way approach to signalized intersection having two phases in the cycle (HCM 2000).

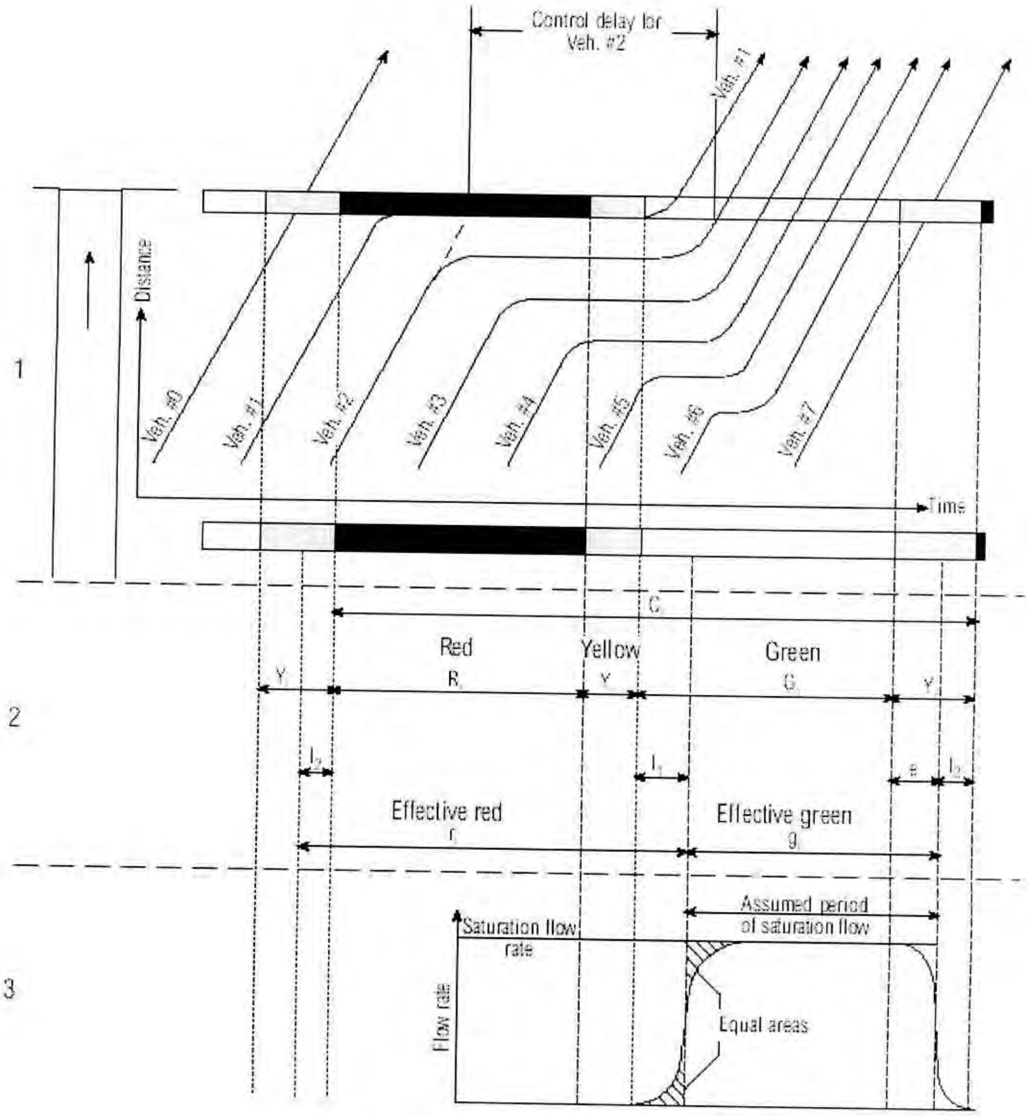

Figure 2.1 Fundamental Attributes of Flow at Signalized Intersection
Source: HCM 2000

The diagram is divided into three parts. The first part shows a time-space plot of vehicles on the northbound approach to the intersection. Intervals for the signal cycle are indicated in the diagram. The second part repeats the timing interval, and labels the various time interval of interest with the symbols. The third part is an idealized plot of flow rate past the stop line, indicating the saturation flow.

2.3.1 Performance measures

Common measures by which the performance of an intersection may be evaluated include (1) delay, (2) stops, and (3) queue length. Each of these may be expressed as values, which represent totals or averages for the entire intersection or for particular approaches or movements within the intersection. Averages are often expressed on a per vehicle basis. Other measures, which have been used to characterize the performance, are *throughput* and *total travel time* (Mcshane and Roess, 1990).

Delay, specifically the control delay is the measure used in the signalized intersection methodology of the 2000 HCM and the primary measure used in the number of signalization optimization procedure. Performance measures are critical part of all intersection design methodologies.

2.3.2 Discharge Headway, Lost time, and Saturation Flow

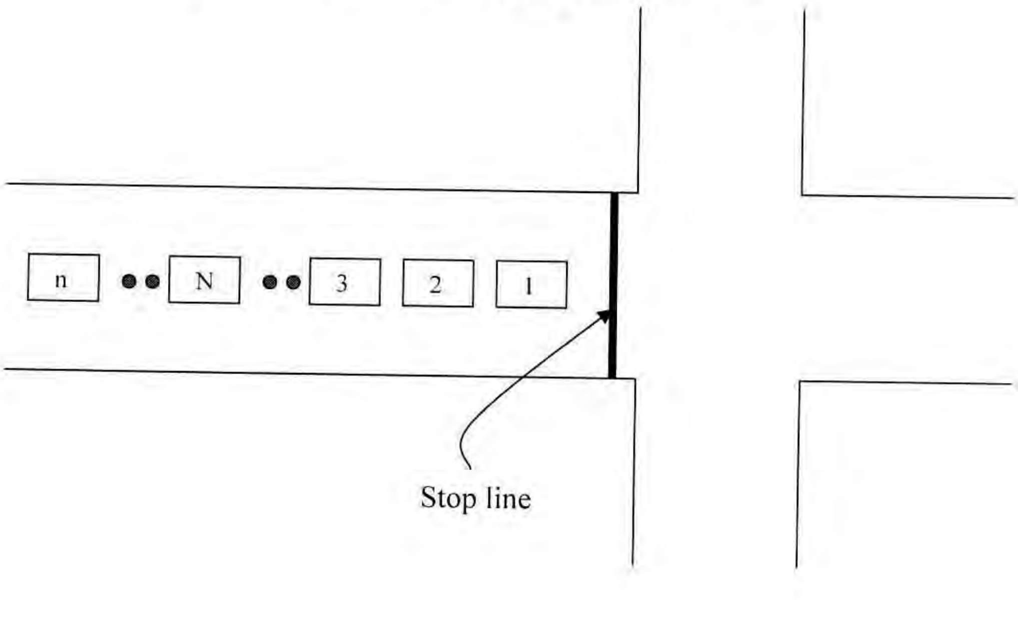

Figure 2.2 Condition at Traffic Interruption
Source: HCM 2000

Vehicle in queue	Headway
1	$h + t_1$
2	$h + t_2$
3	$h + t_3$
.	.
N	$h + t_N$
N + 1	h
N + 2	h
.	.
.	.
n	h

Figure 2.2 illustrates a queue of vehicles at signalized intersection. When the signal turns green, the queue begins to move into the intersection. The first headway is defined as the time between the initiation of the green signal and the first vehicle's front bumper's crossing the stop line. This first headway will be comparatively long, as the first driver must see the light turn green, then react and accelerate the vehicle into intersection. The second headway, measured as the time between the first and second vehicles bumper's crossing the stop line, will be some what smaller, as the second driver's reaction to the green overlaps the first driver's reaction. The third reaction would be smaller than the second, and so on.

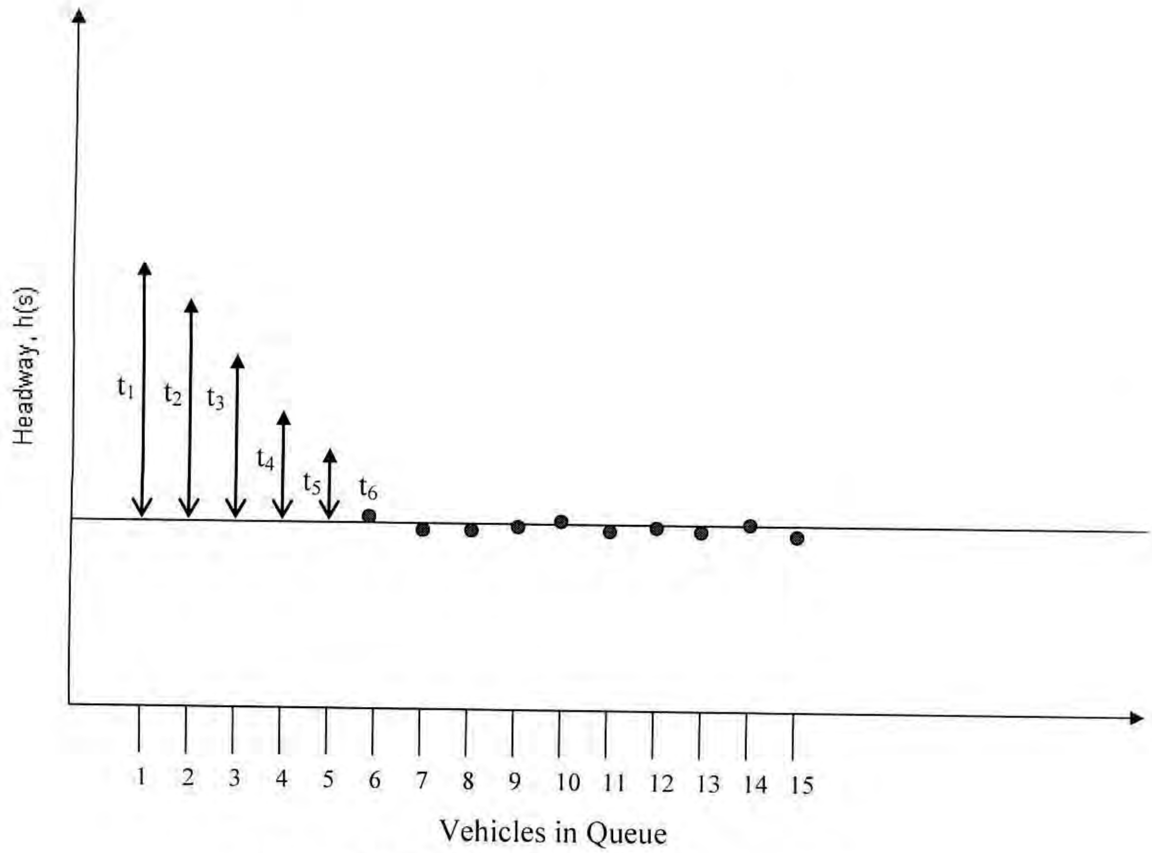

Figure 2.3 Headways at Traffic Interruption
Source: HCM 2000

Eventually (usually between the fourth and sixth headway), vehicles entering the intersection have fully accelerated by the time they reach the stop line, and approximately equal headways would then be observed. Figure 2.3 represents a plot of average headway of vehicles entering the intersection versus the position of the vehicle in the queue.

The constant headway achieved once a stable moving queue is established is called the ***saturation headway***. If it is assumed that each vehicle entering the intersection consumes h sec, then the number of vehicles that can enter the intersection in a lane may be computed as

$$s = 3600/h$$

Where, s = saturation flow rate (vehicle per hour of green per lane)

h = saturation flow rate headway (sec)

The saturation flow rate, s, is the number of vehicles that could enter the intersection in a single lane if the signal were always green for that lane, and the vehicles were never stopped.

Traffic stream at signalized intersection do stop periodically. When the traffic stream starts, the first several vehicles consume more than h sec/veh. Thus sum of the incremental headways (above h sec/v) for first several vehicles is called ***start-up lost time.***

$$l_l = \sum_i t_i$$

Where l_l = start-up lost time (sec)

t_i = incremental headway for the i^{th} vehicle (sec)

The ***clearance lost time l_2,*** is the time between the last vehicle from one approach entering the intersection and the initiation of the green signal for conflicting movements.

The capacity at signalized intersection is based ion saturation flow rate, the lost time and the signal timing.

2.3.3 Effect of Right- turning Vehicles

The impact of right turns on operations is the most significant factor considered in signalized intersection analysis, because it is so common that right turns and right- turn conflicts are the principal source problems. Based upon the headways in the rightmost lane it is observed that right turner vehicles takes as much time to process as 2, 4, even 10 through vehicles. That right tuner is "equivalent to" 2, 4, or even 10 through vehicles. As the opposing flow increase, the impedance to right turn increases and their operational impact also increases. If right turning traffic is heavy, it may require prohibiting the right turn or providing exclusive right-turn phase and/or lane.

2.4 Capacity and Level of Service Concepts

A principal objective of capacity analysis is the estimation of the maximum amount of traffic that can be accommodated by given facility. Capacity analysis is also intended to estimate the maximum amount of traffic that can be accommodated by a facility while maintaining prescribed operational qualities. The definition of operational criteria is accomplished by introducing the concept of level of service. Ranges of operating conditions are defined for each type of facility and are related to the amount of traffic that can be accommodated at each service level.

2.4.1 Capacity

The 2000 Highway Capacity Manual defines the capacity of facility as " the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions.

Prevailing roadway, traffic, and control conditions, which should be reasonably uniform for any section of facility, define capacity. Any change in the prevailing conditions will result in the change in capacity of the facility. Capacity analysis is conducted for segment or points (such as signalized intersections) of a facility having uniform traffic, roadway, and control conditions.

Capacity is defined on the basis of "reasonable expectancy." That is stated capacity for a given facility is a rate of flow that can be repeatedly achieved during every peak period for which sufficient demand exists and that can be achieved on any facility with similar characteristic.

The capacity of highway facility is an important characteristic. Operating conditions at capacity are, however, generally poor. Few facilities are designed to operate at or near capacity because of poor operating characteristics and the difficulty in maintaining capacity operations without breakdown. Thus, the ability to analyze the traffic carrying ability of facilities under better operating is major aspect of capacity analysis. Capacity may be defined in terms of persons per hour, passenger cars per hour, or vehicles per hour depending upon the type of facility and type of analysis.

2.4.2 Level-of-service

The 2000 Highway Capacity Manual defines level of service (LOS) as term, which denotes a range of operating conditions that occur on transportation facility when it is accommodating range of traffic volumes

Highway Capacity Manual describes service quality in following terms.

- *Speed and travel time.* One of the most easily perceived measures of service quality is speed, or travel time. On freeway, speed is very evident measure of quality, while on surface streets systems, the driver is very sensitive to total travel time.

- *Density.* Density is not often used in traffic analysis. Density describes the proximity of vehicles to each other in the traffic stream and reflects ease of maneuverability in the traffic stream, as well as psychological comfort of drivers.

- *Delay.* Delay can be described in many ways. It represents excess or additional travel time due to travel time of controls.

- *Other measure.* A variety of other measures are used to describe service quality. In some cases, measures used are not directly discernible to drivers or passengers. Such measures generally rely upon volumes or flow rates.

Six level of service (LOS) are defined for capacity analysis. They are given letter designations A through F, with LOS A representing the best range of operating conditions and F the worst. Safety is not included in the measures, which are used to establish level of service. The specific terms in which each level of service is defined vary with the type of facility involved. In general LOS A describes a free flowing condition in which individual vehicle of the traffic stream are not influenced by the presence of other vehicles. LOS F generally describes breakdown operations (except for signalized intersections), which occur when flow arriving at a point is greater than facility's capacity to discharge flow. Level of service B, C, D, and E represent intermediate conditions, with lower bound of LOS E often corresponds to capacity operations. Each facility has five service flow rates, one for each level of service (A through E). For LOS F, it is difficult to predict flow since stop-start conditions often occur. *Service flow Rate* is the maximum hourly rate at which person or vehicles can reasonable be expected to traverse a point or uniform segment of lane or roadway during given period under prevailing roadway, traffic, and control conditions while maintaining a designated level of service. The service flow rates are generally based on a 15-min period.

2.4.3 Factors affecting level of service

Base Conditions

Many of the procedures in 2000 HCM provide formula or simple tabular or graphic presentations for set of specified standard conditions, which must be adjusted to account for any prevailing conditions not matching those specified. Base conditions assume good weather, good pavement conditions, user familiar with facility, and no incident impending traffic flow.

Base conditions for uninterrupted flow facilities are

- Lane width of 3.6 m,
- Clearance of 1.8 m between the edge of the travel lane and the nearest obstruction or the objects at the road side and in the median,
- Free-flow speed of 100km/h for multilane highway,
- Only passenger cars in the traffic streams (no heavy vehicles),
- Level terrain,

- No no-passing zone on two-lane highway, and

- No impediment to through traffic due to traffic control or turning vehicles.

Base conditions for intersection approaches include

- Lane width of 3.6 m,
- Level grade,
- No curb parking on the approaches,
- Only passenger cars in the traffic streams and no local transit buses stopping at the travel lanes,
- Intersection located in a non-central business district area, and
- No pedestrians.

In most capacity analysis, prevailing conditions differ from the base conditions, and computation of capacity, service flow rate, and level of service must include adjustment to reflect this. Prevailing conditions are generally categorized as roadway, traffic, or control.

Roadway Conditions

Roadway conditions include geometric and other elements. These includes

- Number of lanes
- The type of facility and its development environment,
- Lane widths,
- Shoulder widths and lateral clearance,
- Design speed,
- Horizontal and vertical alignments, and
- Availability of exclusive turn lanes at intersection.

Traffic Conditions

Traffic conditions that influence capacities and service levels include vehicle type and lane or directional distribution.

Vehicle Type: whenever a vehicle other than passenger cars exists in the traffic stream, the number of vehicles that can be served is affected. Heavy vehicles adversely affect traffic in two ways:

- They are larger than passenger cars and therefore occupy more roadway space, and

- They have poorer operating capability than passenger car, particularly with respect to acceleration, deceleration, and the ability to maintain speed on upgrades.

Directional distribution and lane distribution also affect capacity, service flow rates, and level of service.

Control Conditions

For interrupted flow facilities, the control of the time available for movement of specific traffic flow is critical element affecting capacity, service flow rates, and level of service. The most critical type of control on such facilities is the traffic signal. Operations are affected by the type of control in use, signal phasing, allocation of green time, cycle length, and relationship with adjacent control measures.

2.5 Summary

This chapter introduces the basic aspects of the signalized intersection. The chapter covers some definitions, traffic flow, signal timing etc. At the end it gives the concept of capacity and level of service of transportation facility in general. Factors affecting capacity are also discussed. Next chapter presents the capacity analysis of signalized intersection in detail.

Chapter 3

CAPACITY AND LEVEL OF SERVICE OF SIGNALISED INTERSECTIONS

3.1 General

The concept of capacity and level of service are central to the analyses of intersections, as they are for all types of facilities. It is necessary to consider both capacity and level of service to evaluate the overall operation of signalized intersections. As per HCM 2000 level of service is based upon the average control delay per vehicle for various movements within the intersections. This chapter deals with theoretical aspects of capacity analysis of signalized intersections. Literature review of saturation flow, delay, level of service, etc is presented under respective headings. Various theoretical delay models have been discussed in this chapter. Chapter ends with discussing impact of mixed traffic conditions.

3.2 Signalized Intersection Capacity

Capacity at intersections is defined for each lane group. The lane group capacity is the maximum hourly rate at which vehicles can reasonably be expected to pass through the intersections under prevailing traffic, roadway, and signalization conditions. Traffic conditions includes volumes on each approach, distribution of vehicles by movement (left, through, right), vehicle type distribution, location and use of bus stops within intersection area, pedestrian crossing flows, and parking movements on approaches to the intersections. Roadway conditions include the basic geometry of the intersections, including number and width of lanes, grades, and lane use allocations. Signalization condition includes signal phasing, timing, and type of control.

3.3 Level of Service

In the 1965 Highway Capacity Manual levels of service at signalized intersection were related to load factor. Load factor presented some problem such problem as its insensitivity to low service volume, absence of any rational; basis for defining break points, and difficulty in identifying loaded cycle. **Sutaria and Haynes (1977)** used road user opinion survey that involves depicting and rating different traffic situation at carefully selected single signalized intersection. Over 300 drivers rated randomly arranged film sequences of two types - a driver view (micro view) and an overall view (macro view) of an intersection –and evaluated these films, segment by segment, in terms of appropriate level of service. Statistical analyses indicated that average individual delay correlated better with level of service. The hypothesis that Load Factor is better predictor of Level of Service was tested and was rejected.

Chandra et al (1996) studied the parameter to define level of service for mixed traffic at signalized intersection. Due to many problems associated with the measurement and interpretation of delay at signalized intersection LOS parameter were redefined. Degree of saturation and percent of vehicle stopping in the approach were considered the appropriate parameters. Data collected at eight signalized intersection in Delhi were analyzed and breakpoints for various level of service were determined. They developed the graphical relationship incorporating the average stopped delay, saturation green ratio and the Degree of Saturation (DOS). Break points in the range of DOS for different LOS have been determined based on these parameters. DOS was also related to the percent stopping to define six LOS for mixed traffic flow at signalized intersection.

Degree of Saturation

If v is the arrival flow rate (based on 15 min vehicle count at upstream where flow is not affected by intersection) at an approach; then the number of vehicles arriving at intersection for this approach per cycle is 'vC'. Maximum number of vehicles that can discharge from the approach is sg. If all vehicles arriving at intersection are to be cleared in their first green, then

$$sg >= vC$$

Or

$$\frac{vXC}{sXg} <= 1.0 \tag{3.1}$$

The left hand side in this equation is the ratio of flow ratio and green ratio and it is commonly known as the degree of saturation (X). For satisfactory performance X<=1.0.

Proportion of the Vehicles Required to Stop

This parameter is easy to measure in the field by simply counting all vehicles that stop and total traffic volume for selected period of time. It is also calculated mathematically knowing the condition of arrival, departure and signal timings. Proportion of vehicles required to stop is the ratio of stopped vehicles and total vehicles arriving at intersection in a cycle. Mathematically

$$P_S = \frac{v(r + g_S)}{v(r + g)}$$

$$g_S = \frac{vr}{s - v} \tag{3.2}$$

Where, P_S = proportion of vehicles required to stop in the intersection approach

r = red time (sec)

g_s = saturated green (s) is defined as at this time, traffic flow is saturated.

In HCM 2000 level of service for signalized intersection is defined in terms of control delay, which is a measure of driver discomfort, frustration, fuel consumption, and increased travel time. The average control delay per vehicle is estimated for each lane group, and aggregated for each approach and for the intersection as a whole. Level of Service is directly related to the control delay value. The criteria for LOS are given in Table 3.1.

Table 3.1 Level of Service Criteria for Signalized Intersection

Level of service	Control Delay per Vehicle (sec/veh)
A	≤10
B	>10-20
C	>20-35
D	>35-55
E	>55-80
F	>80

Source: HCM 2000

Level of service A describes operation with very low control delay. This level of service occurs when progression is extremely favorable and most vehicles arrive during green phase. Many vehicles do not stop at all.

Level of service B this level generally occurs with good progression, short cycle length, or both. More vehicles stop than LOS A causing higher level of delay.

Level of service C the higher delays may result from only fair progression, longer cycle length, or both. Individual cycle failures may begin to appear at this level. The number of vehicles stopping is significant.

Level of service D at this level the influence of congestion becomes more noticeable longer delays may result from some combination of unfavorable progression, long cycle lengths, or high v/c ratios. Many vehicles stop and individual cycle failures are noticeable.

Level of service E at LOS E delays will be high indicating poor progression, long cycle length, and high v/c ratios. Individual cycle failures are frequent.

Level of service F this level is considered to be unacceptable to most drivers often occurs with over saturation, i.e. when arrival flow rate exceeds the capacity of lane groups. It may also occur at high v/c ratios with many individual cycle failures. Poor progression and long cycle lengths may also be major contributing causes to such delay levels.

3.4 Saturation Flow

Saturation flow rate is basic parameter used to derive capacity. It is determined based upon minimum headway that the lane group can sustain across the stop line. Saturation flow rate is computed for each of the lane groups. Saturation rate for the prevailing conditions can be determined directly from the field measurements and be used, with no adjustments. If default value selected for the base saturation flow rate, it must be adjusted for various factors, which reflects geometric, traffic, and environmental conditions.

Sarna and Malhotra (1967) presented the results and analysis of the studies on saturation flow conducted at a number of different intersections with varying approach road widths. They developed the relationship between the saturation flow and the approach road width at signalized intersection. Effect of approach volume and increasing percentage of bicycles on the saturation flow has been studied. It was suggested that flaring of the approach should be done to increase discharging capacity. The study has shown that the saturation flow increases with the increase in approach volume.

Zegeer (1986) conducted field survey to find saturation flow throughout the United States at signalized intersection. He verified the saturation flow rates and traffic volume adjustment factors used in various capacity analysis procedures by collecting relatively extensive database. Saturation flow headways for more than 20,000 observations were collected for series 12 geometric, traffic characteristic, and environmental factors and compared with baseline saturation flow headways for various signal cycle length and phase combinations. Vehicle blockage and lane distribution surveys were conducted for 1900 additional observations. He suggested series of modified adjustment factors to determine modified saturation flow rates when calculating signalized intersection capacity.

Lee and Chen (1986) studied the entering headways in small city Lawrence, Kansas and six factors were examined. Entering headway values from total of 1,899 traffic queues were recorded by using video camera equipment. From the data, mean entry headway of vehicle 1 through 12 were found to be 3.80, 2.56, 3.25, 2.22, 2.16, 2.03, 1.97, 1.94, 1.94, 1.78, 1.64, and 1.76. He found that

- Signal type has little influence on entering headway at signalized intersections.

- Time of the day (a. m. or p. m.) has little influence on entering headways.

- The inside lane of an approach has slightly lower entering headways than does outside lane.

- The entering headways at approaches with speed limits of 20 mph are significantly higher than those at approaches with higher speed limits. (>=30 mph). For approaches with speed limits higher than 30 mph, the influence of speed limit on the headway is noticeable.

- In general, streets that have higher speed limits and less roadside friction have lower entering headway values.

- When queue lengths increases, the general observation is that the entering headway values decreases.

Taylor et al (1989) used video-based equipment to estimate the character speeds and headway. This technique provided cheap, quick, easy, and accurate method of investigating traffic systems. Investigation of headways on freeway traffic allows the potential of this technology in a high-speed environment to be determined. Its application to the study of speeds in parking lots enabled its usefulness in low-speed environments to be studied. The data obtained from the video was compared to traditional methods of collecting headways and speed data.

Hoque (1994) developed micro simulation model, MIXSIM to study the non-lane based mixed traffic in a comprehensive manner as well as to estimate the signal design parameters. The model was calibrated and validated on the basis of data collected from Dhaka, the capital of Bangladesh. The method adopted to consider non-lane disciplined traffic streams, where vehicles can occupy any position across the carriageway, is to divide the whole width of approach into smaller lanes or strips. Extensive field observations were made in order to formulate the mixed traffic behavior with a good approximation. Dynamic graphical display of simulated traffic stream is used to develop a collision-free traffic model and also to analyze some results instantaneously.

HCM 2000 has suggested the following equation for the determination of saturation flow rates

$$s = s_o N f_w f_{HV} f_g f_e f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb} \qquad (3.3)$$

s	=	saturation flow rate for the lane group, veh/hr
s_o	=	base saturation flow rate per lane, pc/hr/lane
N	=	no. of lanes in a lane group
f_w	=	adjustment factor for lane width
f_{HV}	=	adjustment factor for heavy vehicles
f_g	=	adjustment factor for approach grade
f_e	=	adjustment factor for parking activity
f_{bb}	=	adjustment factor for blocking effect of local buses
f_a	=	adjustment factor for area type
f_{LU}	=	adjustment factor for lane utilization
f_{LT}	=	adjustment factor for left turn
f_{RT}	=	adjustment factor for right turn
f_{Lpb}	=	pedestrian adjustment factor for left turn
f_{Rpb}	=	pedestrian adjustment factor for right turn.

3.4.1 Determination of Capacity and v/c Ratio

Capacity at signalized intersection is based upon the concept of saturation flow and saturation flow rate. The flow ratio for a approach is the ratio of actual or projected demand flow rate for the approach v and saturation flow rate (s). The capacity of given lane group is given by

$$c = s\,\frac{g}{C} \tag{3.4}$$

Where,

c = capacity of intersection approach (veh/hr)

s = saturation flow rate for approach (veh/hr), and

g /C = effective green ratio for the approach

Volume to capacity ratio (v/c) is referred as degree of saturation. For given approach it is computed using the equation

$$X = \frac{vC}{sg} \tag{3.5}$$

Where,

X = (v/c) = ratio for demand flow rate and capacity for a approach

v = actual or projected demand flow rate for the approach (veh/hr)

s = saturation flow rate for for the approach (veh/hr)

g = effective green time for for the approach (veh/hr)

C = cycle length, sec.

3.5 Delay

Delay is one of the key parameters that are utilized in optimization of traffic signal timings. Furthermore, delay is a key parameter in computing level of service provided to the motorists at signalized intersections. Delay, however, is a parameter that is difficult to estimate because it includes the delay associated with decelerating to a stop, stopped delay, and the delay associated with accelerating from stop. Delay can be defined in number of ways such as

- Stopped delay
- Approach delay
- Travel time delay
- Time-in-queue delay
- Percentage of vehicles stopping

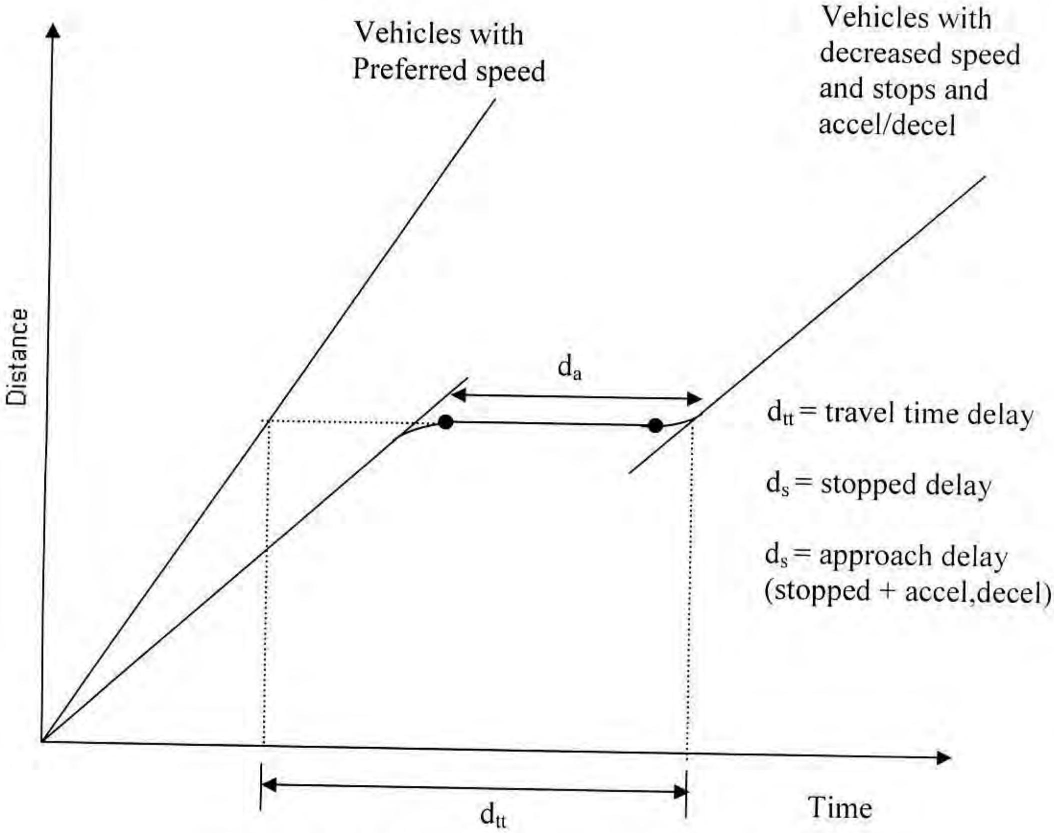

Figure 3.1 Delays at Signalised Intersection
Source: McShane and Roess (1990)

Figure 3.1 shows various delays occurred by the traffic. ***Travel time delay,*** d_{tt}, is difference between the time the vehicle clears the intersection and the time it would have passed through the intersection at the desired speed, without stopping. ***Stopped time***

delay, d_s, is only the time the vehicle spends stopped at the intersection. ***Approach delay***, d_a, adds the time lost in acceleration and deceleration to the stopped delay.

William (1977) presented simple, accurate technique for measuring vehicular delay on the approach to a signalized intersection. Precise definitions were established for four measure of performance: stopped delay, time-in-queue delay, approach delay and percentage of vehicles stopping. Approach delay was selected as being most representative of intersection efficiency. The values thus obtained were statistically compared with true values from time-lapse photography. The point sample, stopped delay procedure and the percentage of vehicle stopping method were selected as the most suitable methods for practical use and were performed in the field at three sites. Correction factors were developed to allow the field results to more accurately estimate the true values of stopped delay and percentage of vehicle stopping. Interrelationships among the four measure of performance were established so that approach delay can be estimated from a value of stopped time.

Correction factor of 0.92 was recommended for Point sample, stopped delay study. For the estimate of volume from the percentage of vehicle stopping study, no correction factor was recommended. Results from field study of percentage of vehicle stopping should be corrected by multiplier of 0.96. One other conclusion was reached that intersection studies should not, in most cases be performed on an individual lane basis. Rather, an entire approach should be studied at one time.

Hurdle (1984) presented a paper to serve as a primer for traffic engineers who are familiar with capacity estimation techniques but have not made much use of delay equations. The emphasis was not given on the mathematical derivation. Instead the discussion was concentrated on the assumption underlying the equations and the limitations that stem from these assumptions. This seemed necessary because the information is widely scattered, some of it material not readily available to the average traffic engineer. It was noted that the methods available at that time either ignore the way in which the delay vary with the time or try to cope with the variation in ways that are more mathematical application of common sense than mathematical models of traffic signal system. None of the model examined can be expected to give really consistent and accurate results. To obtain such results, one would need not just better models but better information about traffic patterns.

Feng-Bor Lin (1989) evaluated the reliability of the HCM 1984 procedure, based on field data, and discussed needed modifications. Stopped delay was measured for single lane movements at seven intersections. To compare the HCM estimates with observed delays, the cycle lengths, green durations, yellow durations, and saturation flow rates were also recorded using video cameras with built-in stopwatches. The evaluation reveals that the procedure tends to overestimate stopped delay at reasonably well-timed signal operations. The discrepancies between the HCM estimate and the observed delays can be very large even when correct cycle length and green durations were used as inputs. Given actual cycle lengths and green durations, the procedure's ability to correctly identify the level of service was found to be good. He found that large discrepancies between HCM estimates and some of the observed delays could be reduced significantly if no progression adjustment is applied to the estimates obtained from HCM delay equation.

Braun and Ivan (1996) studied the methods for determining the average stopped delay at signalized intersection. The average stopped delay encountered by vehicles at eight signalized intersections was measured during afternoon peak hour. The average stopped delay is then determined using the equations described in the 1994 version of the HCM. Average stopped delay was also computed using the 1985 HCM equations to identify improvements realized by applying these new techniques. During the field measurement, peak-hour flow rates are determined as well as intersection geometry and signal phasing. Then the stopped delay was calculated using 1985 HCM and the 1994 HCM equations. Error between field measurement and calculated values was examined and some explanations were given for the major difference. Some recommendations were also given concerning use of delay equations. It was showed that the 1994 HCM estimate intersection approach delay better than 1985 HCM.

Teply (1989) examined two approaches for measuring delay-- a time- space diagram and queuing diagram -- and explained various problems related to each. He concluded that, while delay cannot be measured precisely, it could be useful engineering tool if it is calculated properly. Some of his findings were

- Uniform delay formulas slightly underestimate overall delay because they neglect a portion of the acceleration delay.
- The fact that uniform delay formulas and delay surveys based on queue counts do not account for the rate of arrival a the end of the queue and the rate of discharge at the front of queue does not resort the resulting delay values.
- Delay surveys based on stopped queue count produce stopped delay values. In situations with low volumes and short red interval, these techniques may overestimate stopped delay to the point of exceeding overall delay values.
- The ratio between measured values of overall delay and stopped delay is not constant.

Hagen and Courage (1989) compared 1985 HCM delay computations with those performed by Signal Operation Analysis Package (SOAP) and by TRANSYT – 7F Release 5. They studied the effect of degree of saturation, the peak hour factor, and the period length on delay computations and on the treatment of left turns opposed by oncoming traffic. All of the models agreed closely at volume levels below the saturation point. When condition became over saturated, the model diverged; however, it was possible to make the delay values agree by the proper choice of period length and peak-hour factor. The computed saturation flow rates for right turns opposed by oncoming traffic also agreed closely. However the protected plus permitted right turns produced substantial differences. It was concluded that neither SOAP nor the HCM treats this case adequately. He proposed and evaluated alternative model based on deterministic queuing process.

Dowling (1994) tested the effect on accuracy of replacing most of the required field input data with the default values recommended in Table 9-3 of the HCM. The average stopped delay was calculated for six signalized intersections staring with basic volume (flow rate), lane geometry, and signal timing data. The HCM recommended default values for grades, heavy vehicles, and such were used in place of rest of the required input data. The calculations were repeated several times; each time one or more of the default values were replaced with field data. The resulting delay estimates were then compared with field measurements of delay. The results indicated that users could obtain reliable estimate of intersection level of service and delay using only field-measured turning movements, lane geometry, and signal timing plus the HCM-recommended defaults for rest of the required input data.

The 1997 update of the *Highway capacity manual* (HCM) changed the concept of delay for level-of-service determination from stop delay to control delay. **Powel (1998)** suggested a rational, reasonable way to survey delay in the field and then to translate this into total delay. A combined theoretical and empirical approach to measuring field delay on the basis of typical vehicle deceleration and acceleration profiles was taken. The profiles were related to the relatively easily surveyed quantity of vehicles in queue, which is equivalent to estimating time in queue of all vehicles stopped by the traffic signal. The results indicate that after vehicle in queue are sampled, correction factor can account, in practical terms, for the unsurveyed deceleration and acceleration delay. The results provided the basis for the updated Appendix III: Measurement of Control Delay in the Field in chapter 9, Signalized Intersection, of the 1997 HCM update.

3.5.1 Significance of delay as a measure of effectiveness

Vehicle delay is most important parameter used by transportation professionals to measure the performance of signalized intersections. This parameter is utilized in both the design and evaluation of traffic signalized intersections. As an example delay minimization is frequently used as primary optimization criteria when determining the operating parameters of traffic signals at both isolated and coordinated signalized intersections. In another example, the HCM uses the average delay incurred by vehicles on approaches to signalized intersections as a criterion for determining level of service. Delay is frequently used as an optimization and evaluation criteria because it is measure of performance that a driver can directly relate to. Moreover, delay is criteria whose meaning is easily comprehended by both traffic professional and the general public. However delay is also a parameter that is not easily determined.

3.5.2 Delay models

a) Delay model when demand is less than Capacity:

Figure 3.2 shows basic cumulative arrival pattern and cumulative service pattern assumed in most models. From the geometric construction average delay for cycle can be expressed as shown in equation 3.6.

$$\mathrm{UD} = \frac{C[1-(g/C)]^2}{2[1-v/s]} \tag{3.6}$$

Where,

UD = uniform delay (sec/vehicle)

C = cycle length (sec)

g/C = ratio of effective green to cycle length

v/s = ratio of demand flow rate to saturation flow rate.

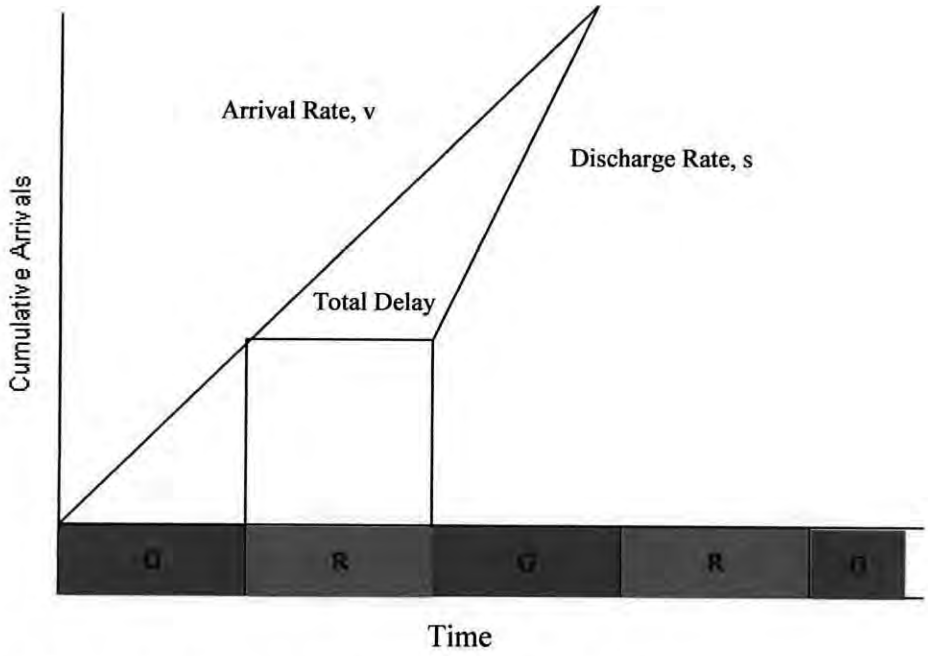

Figure 3.2 Cumulative Demand and Service Curve for v/c <1.0

Here it is assumed that the vehicles arrive uniformly at constant rate. Vehicle arrival patterns, however, are not uniform. They are more likely to be random. Number of stochastic models has been developed; the most used one is Webster Model. The assumption in these models are 1) the rates are constant for the analysis period, 2) the demand is less than capacity 3) the relation of delay to the pattern is deterministic, and 4) the arrival pattern of vehicles is generally Poisson distribution. Using deterministic queuing analysis, Webster (1958) developed model for estimating the delay incurred by motorists at under saturated signalized intersection that becomes the basis for all subsequent delay models. The model developed is

$$d = \frac{C[1-(g/C)]^2}{2[1-v/s]} + \frac{(v/c)^2}{2v[1-(v/c)]} - 0.65(c/v^2)^{1/3}(v/c)^{2+5(g/c)} \qquad (3.7)$$

Where, d = average overall delay per vehicle (seconds).

The first term of Webster model represents the uniform delay per vehicle. The second term attempts to account for the fact that the vehicles arrive randomly. This term increases rapidly with the degree of saturation. The third term is an adjustment factor to provide better mathematical fit to the theoretical curve. Following Webster's work, numerous studies were conducted on the subject of how to estimate delays at signalized intersection. As a result of these studies, a number of delay models based on deterministic queuing theory were proposed to suite the different field conditions. Among these, the most noticeable are the models developed by Miller (1963) and Akcelik (1981), and the models developed for use in Highway Capacity Manual (TRB, 1985, 1994, 1998) and Canadian Capacity Guide for signalized intersections (ITE, 1984, 1995).

b) Overflow delay model

For the cases in which v/c >1.0, the queue would grow to infinity over a long time. However, v/c >1.0 in practice exists for only finite time, say T. Figure 3.3 shows a situation for which the demand exceeds the capacity for an extended time. For uniform deterministic arrival rate v, and constant capacity c, the average delay is given by

$$d = \frac{C[1-(g/C)]}{2} + \frac{T}{2}[(v/c) - 1] \tag{3.8}$$

d = Average delay in sec/vehicle

T = Analysis period in seconds

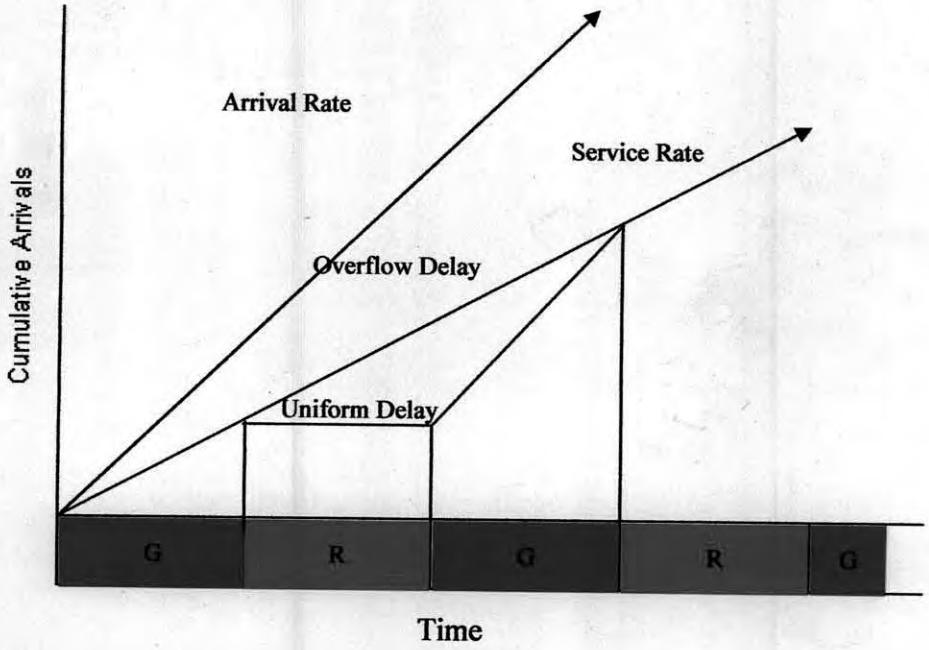

Figure 3.3 Cumulative Demand and Service Curve for v/c >1.0

c) Combining steady-state and oversaturated models

TRANSYT Model

The TRANSYT program is widely used signal-optimization program and can be applied to arterial and networks. TRANSYT-6 uses the delay relationship, which can be approximated by

$$\mathrm{OD} = \frac{15T}{c}\left[(v-c)+\sqrt{(v-c)^2+\frac{240v}{T}}\right] \tag{3.9}$$

Where T is analysis period in minutes, and v and c is in vehicle/hr. OD is "Overflow Delay" which must be added to the uniform delay to estimate total delay, d.

Ackelik Model

This model is in common use, as part of Australian procedure for Intersection analysis. This model is given as

$$\mathrm{OD} = \frac{T}{4}\left\{\left[(v/c)-1\right]+\sqrt{[(v/c)-1]+\frac{12[(v/c)-(v_o/c)]}{cT}}\right\} \tag{3.10}$$

For $v/c > v_o/c$, where $v_o/c = 0.67 + s\ (g/600)$ and OD = 0 otherwise.

Reilly's Model

In preparing model for the HCM 1985 Highway Capacity Manual, Reilly et al conducted extensive field studies to measure delays. They found that Akcelik equation consistently overestimated field-measured values, and recommended that the theoretical results be reduced by 50% to better reflect field conditions. The resulting equation is:

$$\mathrm{OD} = 450\left\{\left[(v/c)-1\right]+\sqrt{[(v/c)-1]+\frac{12[(v/c)-(v_o/c)]}{cT}}\right\} \tag{3.11}$$

Highway Capacity manual 1994 Model

In the HCM 1994, the average stopped delay per vehicle for a given lane group, with stopped delay defined as completely immobilized vehicles, is computed using following equations (TRB, 1994)

$$d = d_1 \text{ X (CF or DF)} + d_2 \tag{3.12}$$

$$d_1 = 0.38C \frac{(1-\frac{g}{C})^2}{(1-\frac{g}{C}X)} \tag{3.12 a}$$

$$d_2 = 173X^2 \left[(X-1) + \sqrt{(X-1)^2 + \frac{m}{c}X} \right] \tag{3.12 b}$$

Where,

d = stopped delay per vehicle (sec/veh)

d_1 = uniform stopped delay (sec/veh)

d_2 = incremental, or random, or stopped delay (sec/veh)

DF = delay adjustment factor for quality of progression and control type

CF = adjustment factor for control type,

X = volume to capacity ratio of lane group,

C = traffic signal cycle length (sec)

c = capacity of lane group (veh/h)

g = effective green time for lane group (sec)

m = an incremental delay calibration term

In above equation d_1 estimates the uniform delay. This parameter is based on the first term of Webster's delay formulation. This parameter is only valid for the analysis of delay on intersection approaches on which v/c ratio is less than 1.0. The second delay parameter, d_2, estimates the incremental delay caused by the randomness of vehicle arrivals. The equation for d_2 is only valid for oversaturated traffic conditions. The final element of delay equation is delay adjustment factor DF. This factor accounts for impact of the type of traffic signal control and of the quality of traffic progression between successive signalized intersections on the estimation of uniform delay. The adjustment factor for the quality of progression (PF) adjusts for the delay estimates depending on arrival pattern. This factor is calculated using following equation.

$$PF = \frac{(1-P)f_P}{1-\frac{g}{C}} \qquad (3.12\ c)$$

Where,

P = proportion of vehicles arriving during green interval

f_p = supplemental adjustment factor for when the platoon arriving during green.

Highway Capacity manual 2000 model

After the release of HCM 1994, numerous researches has been undertaken to asses the changes that were made in the delay estimated model with respect to 1985 version of the model. Fambro and Rouphail (1997) proposed the delay that corrected some of the problems found in the 1994 HCM model and that is now the delay model found in the HCM 2000. In the HCM 2000, average delay per vehicle for a lane group is given by equation (TRB, 1998).

$$d = d_1 \times PF + d_2 + d_3 \qquad (3.13)$$

$$d_1 = 0.5C \frac{(1-\frac{g}{C})^2}{(1-Min(1,X)\frac{g}{C})} \qquad (3.13\ a)$$

$$d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8kIX}{cT}}\right] \qquad (3.13\ b)$$

$$PF = \frac{(1-P)f_P}{1-\frac{g}{C}} \qquad (3.13\ c)$$

Where,

d = control delay per vehicle (sec/veh)

d_1 = uniform delay (sec/veh)

d_2 = incremental, or random delay (sec/veh)

d_3 = residual demand delay to account for over saturation queues that may have existed before the analysis period (sec/veh)

PF = adjustment factor for the effect of the quality of progression in coordinated system.

k = incremental delay factor dependent on signal controller setting (0.50 for pretimed signals; vary from 0.04 to 0.50 for actuated controllers)

I = upstream filtering/metering adjustment factor

(1.0 for an isolated intersection)

T = analysis period (hours)

P = proportion of vehicles arriving during the green interval

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

3.6 Heterogeneous Traffic

The composition of traffic in developing countries is mixed, with a variety of vehicles, motorized and non-motorized, using the same right of way. The motorized or fast moving vehicles include passenger cars, buses, trucks, auto-rickshaws, scooters and motorcycles; non-motorized or slow moving vehicles include bicycles, cycle-rickshaw, and animal drawn carts. Since 1950s, considerable research has been made to develop traffic flow models for roadways with mainly homogeneous traffic, representing the composition of traffic primary in developed countries (Khan and Maini, 1999). Very limited studies have been done to develop an understanding of traffic flow for non-lane-based heterogeneous or mixed traffic condition in developing countries. Some efforts have applied a variation of practices developed for homogeneous traffic by converting heterogeneous traffic by to equivalent passenger car-units and then applying procedures for homogeneous traffic. However, these efforts have produced mixed results. Recent efforts include the development of microscopic simulation models.

3.6.1 Difference between Heterogeneous and Homogeneous Traffic Flow

The differences that characterize mixed traffic system are mainly due to the wide variation in the operating and performance characteristic of vehicles. The traffic in mixed traffic flow can be classified as fast-moving and slow moving vehicles or motorized and non-motorized vehicles. In urban areas, mixed traffic flow often is also accompanied by substantial pedestrian movement, encroachment at intersection, street parking, business demand of abutting properties, and narrow roads.

Lane markings, if present, are typically not followed by mixed traffic flow. Figure 3.4 shows the homogeneous and heterogeneous traffic flow. Traffic does not move in single line. On the other hand, there is a significant amount of lateral movement, primarily by smaller-sized motor vehicles. Vehicles do not follow each other within lanes; hence the concept of relating headways and linear densities is not meaningful. Vehicles traverse in both the lateral and transverse directions. At intersection specifically, smaller vehicles such as bicycles, motorcycles, and scooters use the lateral gaps between larger vehicles in an attempt to reach the head of the queue.

Figure 3.4 a) Homogeneous Mix

Figure 3.4 b) Heterogeneous Mix

3.7 Passenger Car Unit (PCU)

The unrestricted mixing of various classes of vehicles along a road creates many problems to the traffic engineers and planners. One type of vehicles in the traffic stream cannot be considered equivalent to any other type, as there is large differences in their vehicular and flow characteristics (Justo and Tuladhar, 1984). The space of the carriage way is shared by vehicles depending upon their size, speed, headway and lateral gap maintained by them. The non-uniformity in the static and dynamic characteristics of the vehicles is normally taken into account by converting all vehicles in terms of common unit. The most accepted one such unit is passenger car unit (PCU). PCU values suggested for signalized intersection by different organizations and authors are given in Table 3.2

Table 3.2 PCU Values at Signalized Intersection

SL. No.	Organisation Or Author	TRRL	Justo and Reddy	Bhattacharya And Mandal	Justo and Tuladhar
	Class of Vehicles	**PCU values**			
1	Car, Jeep	1.0	1.0	1.0	1.0
2	Bus	2.25	2.8	2.25 (Single deck) 2.1 (Double deck) 1.5 (Mini)	2.8
3	Truck	1.74 (Medium) 2.25 (Heavy)	2.8	2.35	2.8
4	Auto Rickshaw	0.33	.4	.35	.4
5	Scooter, Motor Cycle		0.3		0.3
6	Pedal Cycle		0.35		0.4

Source: Justo and Tuladhar (1984)

3.7.1 Factors affecting PCU values

PCU value of a class of vehicle may be considered as the ratio of the capacity of a road with only that class of vehicles on the road to the capacity with passenger cars only, under identical conditions. PCU value depends on the following factor

i) Vehicle characteristics: physical and mechanical, such as length, width, power, acceleration, deceleration and breaking characteristic of vehicles.

ii) Stream characteristic:

- Mean stream speed
- Longitudinal and lateral clearance distribution
- Speed characteristic of the stream
- Percentage composition of different classes of vehicle

iii) Roadway characteristic

- Horizontal alignment, grade, location etc.
- Stretch: mid-block, signalized intersection etc.
- Pavement surface condition, pavement type, pavement width
- Environmental conditions
- Climatic conditions
- Control condition

Since the PCU values dependent on the traffic flow parameters, these values are subject to variations due to the factors influences the traffic flow parameters. Therefore it may not be precisely correct to adopt a constant set of PCU values under different roadway and traffic conditions (Justo and Tuladhar, 1984).

3.7.2 Determination of PCU

The passenger car unit factors are usually computed by comparing the departure times of cars at the stop line and identical measure for all other types of vehicles. Methods developed for the calculation of PCU factors are discussed in the following section.

A) Webster's method

This method is based on public road observations and controlled test track experiment, carried out by Webster at el. 1958, using cars, taxis, light, medium, and heavy commercial vehicles and some double-decker buses. The test track experiment was performed by varying the percentage of goods vehicle from zero to hundred. Subsequent public road observations in London by Miller 1968 and Glasgow by Kimber et al.1985 involved 70 signal controlled intersections, in the PCU analysis. Vehicle departures were recorded at the stop line by means of event-recorder techniques. The average numbers of light and goods vehicles per cycle were calculated as follows

$$\overline{nl} = \frac{1}{N}\sum_{i=1}^{N} nl_i \tag{3.14}$$

$$\overline{ng} = \frac{1}{N}\sum_{i=1}^{N} ng_i \tag{3.15}$$

Where, $\overline{nl}$ = average number of light vehicles per cycle

nl= number of departing light vehicles per cycle

$\overline{ng}$ = average number of goods vehicles per cycle

ng= number of departing goods vehicles per cycle

N= number of cycles in a set (taken as 12)

The PCU value was estimated as a reciprocal of the slope of straight line drawn through the values of nl and ng. This method is, consequently, unable to estimate more than two types of vehicles at a time.

B) The average headway method

The most common method of determining PCU factor is known as the headway ratio and used by many researchers viz. Scraggs 1964; Miller 1968; Kimber et al. 1986. In this method the PCU values of different vehicles types are obtained by comparing headway of these vehicle types with those of straight ahead passenger cars. For accurate results headways should be measured on a vehicle to vehicle basis of possible combinations for a large sample of each vehicle type. Because of complication involved in collecting data from a large number of possible combinations, a simpler calculation procedure has been developed, so long as there is no bias in doing so (Scraggs 1964). Assuming, in the traffic stream the proportion of heavy vehicles is low, the number of headways between heavy vehicles is small and headways are in any case almost dependent of the class of vehicle preceding and following it. The formula for this is given below and is a simplification of that used by Scraggs 1964.

$$\text{Passenger car unit of vehicle type h} = \frac{h_{1-h}}{h_{1-1}} \tag{3.16}$$

Where,

h_{1-h} = mean headway between light vehicle and a following vehicle of type h

h_{1-1} = mean headway between two consecutive light vehicles

C) Multiple regression method

In the recent years a number of alternative methods of processing the data collected in classified vehicle counts format have been developed, in an attempt to obtain simultaneous estimations of all the properties of the discharge process. These methods involved multiple linear regression techniques which have been used by a number of researchers (Branston et al. 1978, 1981; Holroyd 1963). By the use of extensive statistical manipulation, Branston et al.1978 introduced two methods known as 'asynchronous' and 'synchronous' multiple regression based on counting methods, which are described in the following articles. For both methods, the green period is divided into three time periods, the first period begins at the start of the green, contiguously, the middle period covers the time when the departure rate is constant and in saturate state, and the last period ends when the amber light shows. The end of last counting period of fully saturated cycles is fixed at the change to the amber light, but the ends of the first and middle may be chosen either

i) as arbitrary point of time, termed as asynchronous counting and
ii) to correspond with the instant of departure of a specific vehicle, termed as synchronous counting

Asynchronous Multiple Regression

In this method, vehicle departures are recorded over time periods T which begin and end at an arbitrary point of time. A typical mathematical expression of this model is given below.

$$L = S.T - S.\partial.l_1 - a_1.M - a_2.H - a_3.BC - a_4.MC - a_5.C \qquad (3.17)$$

Where

L = number of light vehicle recorded in time interval T.

M = number of medium vehicle recorded in time interval T.

H = number of heavy vehicle recorded in time interval T.

BC = number of buses or coaches recorded in time interval T.

MC = number of motorcycles vehicle recorded in time interval T.

C = number of bicycles recorded in time interval T.

S = saturation flow in time interval T (in PCU/hr)

l_1 = initial lost time in seconds

∂ = dummy variable (1 for the first counting interval or 0 for the other intervals). The time interval is typically 6 seconds or multiple of it.

a_1 to a_5 = passenger car equivalents for the corresponding vehicle types.

Synchronous Multiple Regression

A number of researches (Branston et al. 1978, 1981; Martin et al 1981) have used 'synchronous' regression in their studies as an alternative technique to asynchronous regression for the calculation of saturation flows and lost times. In this method the number of vehicle departures of each class is recorded over time periods beginning and ending with the instant of departure of a vehicle (the first few second due to start up loss time being excluded) and the counting periods are defined as:

$$T=h_0 + h_L L + h_M M + h_H H + h_{BC} BC + h_{MC} MC + h_C C \tag{3.18}$$

Where:

h_0 = y-intercept

T = Length of vehicle counting period

L= number of light vehicle recorded in time interval T

M= number of medium vehicle recorded in time interval T

H= number of Heavy vehicle recorded in time interval T

BC= number of buses or coaches recorded in time interval T

MC= number of motorcycles vehicle recorded in time interval T

C= number of bicycles recorded in time interval T

h_L to h_C = co-efficient representing average time headway for different vehicle class.

Then, Passenger car unit of vehicle type i = $\frac{h_i}{h_L}$

3.8 Summary

In this chapter capacity analysis of signalized intersection in particular has been discussed. Capacity analysis mainly includes the determination of saturation flow and delay. HCM has given equation to find saturation flow based on number of parameters. The delay models discussed include Webster model, Overflow delay model, TRANSYT model, Ackelik Model, Reilly model and HCM model. These models are based on uniform traffic. Chapter also focuses on the impact of mixed traffic on the analysis of signalized intersection. At the end of the chapter various method of PCU determination has been discussed. Study intersections selected and data collection have been described in next chapter.

Chapter 4

INTERSECTION SELECTION AND DATA COLLECTION TECHNIQUE

4.1 Study Area

The study area selected for the analysis is Dhaka city which is the capital and the biggest metropolitan in Bangladesh. Its population is more than 10 million and it is one of the mega cities of the world. Because of its wide spread commercial, industrial, government, private and other activities, Dhaka has become one of the most densely populated cities in the world. The road traffic facilities of the city have not improved to cope with this population boom. As a consequence, the road network and especially the intersections remain congested for significant period of time, which makes the road users the ultimate victim. Because of resource limitation, the only remedial option is to implement proper traffic management techniques. One of these techniques is the optimization of traffic signals

"In 1981, there were only 15 signalized intersections and another 15 were proposed in the Integrated Urban Development Plan. Prior to 1977, RAJUK was responsible for signal installation and control. In the early 1980s, control of signal was transferred to Traffic Police Division with RHD engineers assigned to help. This experiment lasted only a year before signals were reassigned to DCC. Traffic signals have increased over recent years with 12 signals being installed in just the last two years." (PPK consultants Ltd. others, 1994)

Dhaka City Corporation (DCC) is the sole authority for installing and implementing traffic signals within the Dhaka metropolitan area. DCC gets information about the warrant for a signal in a particular priority junction from Dhaka Metropolitan Police (DMP). DMP also determines the signal timings for different intersections. The method of signal timing determination has no scientific basis and involves no engineering practice. They do not conform to any guidelines of traffic signals and solely depends on intuitive judgment.

In the late 1990s, the traffic signal that were installed earlier without any engineering basis, not only started to become out of order very frequently but also deteriorated the congestion situation because of their non optimum setting and inflexible timing plan. By the end of the decade, the situation became such that virtually all the signalized intersections became traffic police controlled. This led DCC to officially take countermeasures to newly install traffic signals at key intersections of Dhaka city along with supplementary improvements like channelisation and provision of road marking.

It was BDT 240 million World Bank Project in joint association with Dhaka Urban Transport Project (DUTP) to install traffic signals at 59 intersection of Dhaka city. It also involved geometric improvements like channelisation, flaring, removal of unnecessary roundabouts etc. and other improvements like road marking, signing etc. The newly installed traffic signals were designed by hiring the consultants from Australia, who designed the traffic signals according to Australian standard. However, the road traffic situation of Dhaka city is quite different from that Australia. As a result, the timing plan designed by consultants was found to be inadequate. This has led the DCC to frequently change the timing plans on request from traffic police, which has increased the cycle time day by day with subsequent delays and spillback of queue to the upstream intersections.

4.2 Data Collection

Data from study area were collected during the period of March 2007 until June 2007. Digital video camera was used to collect data on the field. Numbers of intersections are selected in Dhaka City for the analysis. All signals are pre-timed signals. The intersections are selected considering the geometry of the intersection and availability of high-rise building near the intersection. The video camera was mounted at the roof of the building located near the intersection and was focused covering the one leg of the intersection. Care was taken to cover full queue formed on the study approach. At some study approaches it was not possible to cover the full queue because of difficulty in taking video due to some obstruction. Video at those approaches were conducted by focusing on stop line only. The recording was done for about 90 minutes to 120 minutes for each approach. Among the five intersections four were recorded at 12:00 PM to 2.00 PM and another one at 5.00 PM to 6:30 PM. Simultaneously data on signal timing i. e. cycle length, number of phases, phase length was collected manually using stopwatch. Number of approaches was noted down. The width of approaches was measured by measuring wheel. Following are the intersection selected for the study:

1. New Market Intersection
2. Panthapath Intersection
3. Science Lab Intersection
4. Sheraton Intersection
5. Bangla Motor Intersection

Figure 4.1 shows the selected intersections in the Dhaka city's map. Figure 4.2 through Figure 4.6 shows the geometric features of these intersections.

Among the above five intersections first two intersections comprise both motorized and non-motorized vehicle. While the last three intersections comprise only motorized vehicle. In this study, the approaches those comprise only motorized vehicle have been considered and for analysis purpose they have been grouped into five classes:

1) Car (Car, jeep, taxi, micro-bus, pickup, 8 seaters autorickshaws locally called tempo);
2) Mini-bus/truck;
3) Large Bus (single decker and double decker);
4) Motor cycles;
5) Autorickshaws (3 seaters locally called bay-taxi);

As the video was conducted during day time, trucks were not present as they are not allowed to enter within the city area at this time period. Again, NMV at the two intersections as stated earlier include bicycle, tricycle (Rickshaw), rickshaw van and push cart.

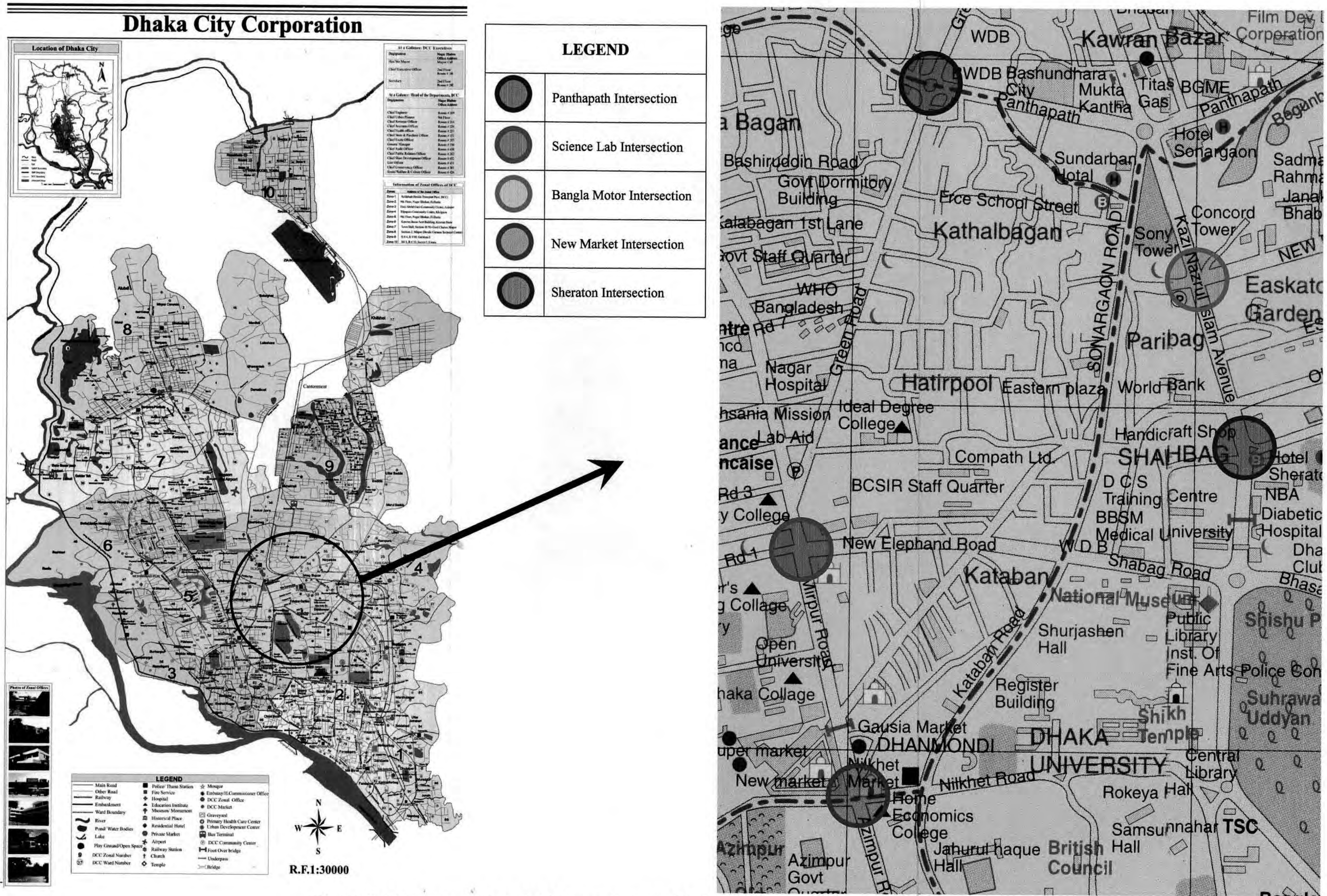

Figure 4.1 Location of Selected Intersections on Dhaka City's Map (*Source: Geo Consult, 2003*)

New Market Intersection

This is four-legged four phase intersection on Mirpur arterial. This is one of the most congested intersections in Dhaka City. During peak hours (morning peak and evening peak), the intersection gets over saturate (Demand more than capacity). The road surface condition at study approaches is not good, affecting the speed of vehicles. Traffic flow was recorded for the intersection, coming from Mirpur and also from Azimpur. Traffic consists of car, auto-rickshaw, mini-bus, large bus and motor cycle.

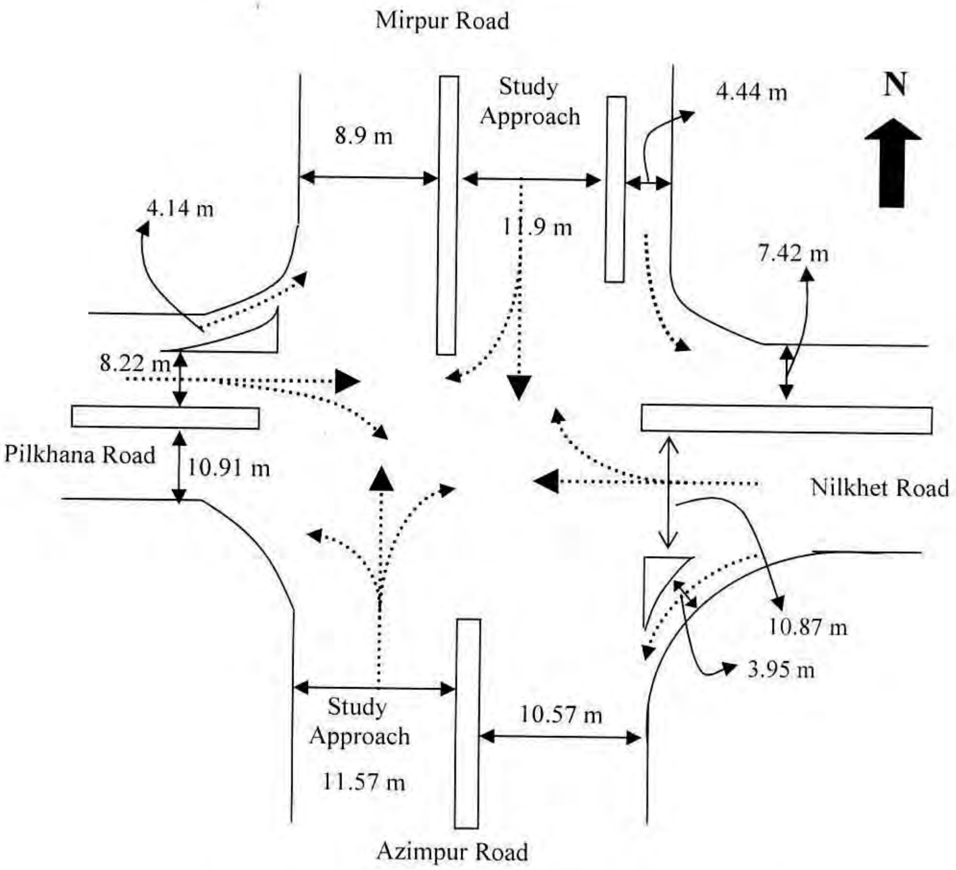

Figure 4.2 Geometric Features of New Market Intersection

Panthapath Intersection

This is also a four-legged four phase intersection on link road between two arterials namely Mirpur road and New Airport road. It becomes over saturate during peak hour mainly at evening. The surface condition and platoon speed at study approach is good. Vehicles in right turn from study approach mostly consist of car and motor cycle, whereas through movement contains mini-bus, large bus, car, auto-rickshaw and motor cycle. Traffic flow was recorded for the leg coming from Shonargaon intersection.

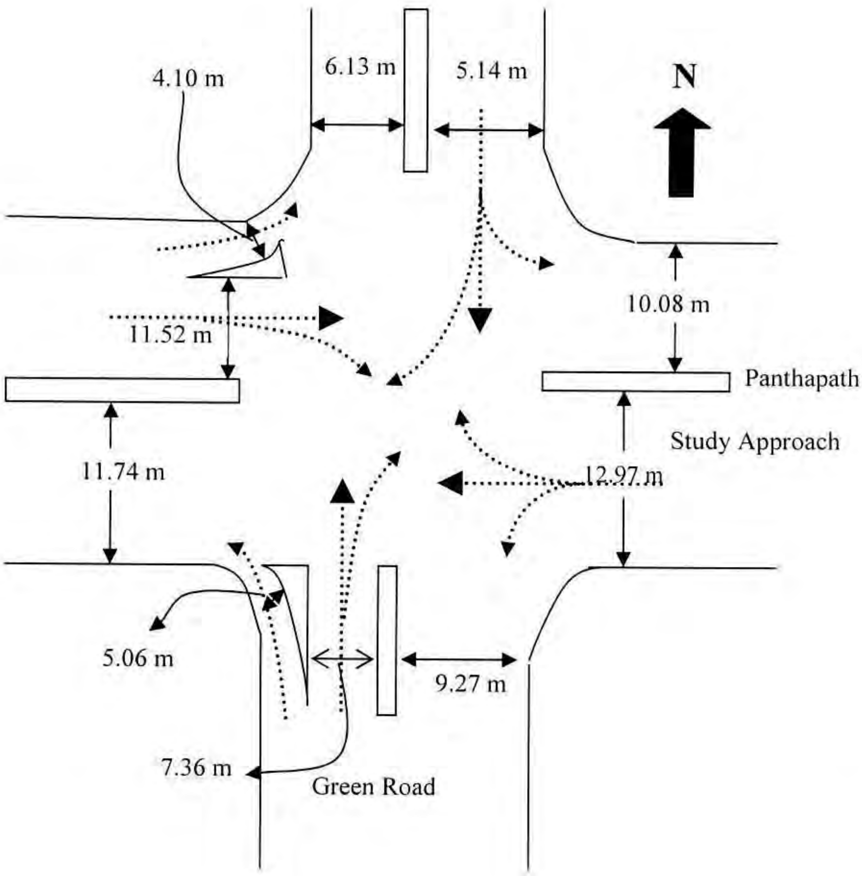

Figure 4.3 Geometric Features of Panthapath Intersection

Science Lab Intersection

This is three-legged three phase intersection on Mirpur arterial road. The surface condition and platoon speed at both study approaches of the intersection is good. On north approach right turn is banned and on east approach only right turn movement exist. Vehicles on both approaches comprise car, autorickshaw, large bus, mini-bus and motor cycle, having dominant amount of car.

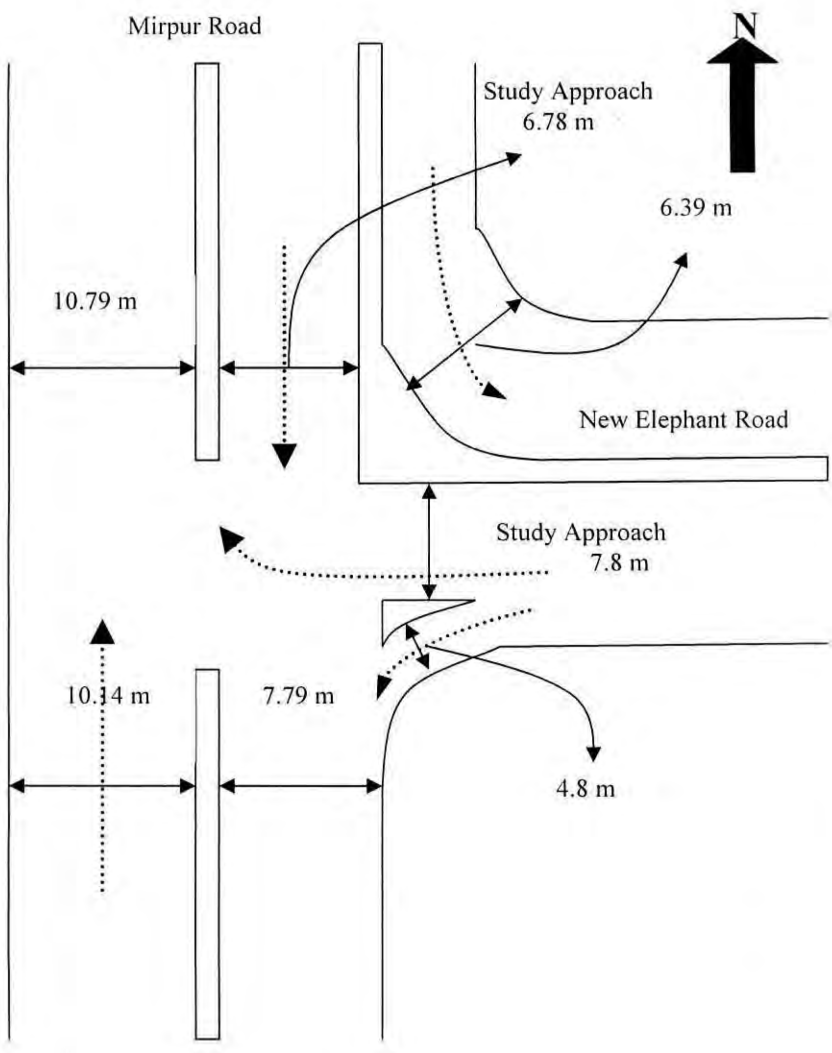

Figure 4.4 Geometric Features of Science Lab Intersection

Sheraton Intersection

This is also a three-legged three phase intersection on New Airport road. And one of the most congested intersections in Dhaka City. The surface condition and platoon speed at study approach of the intersection is good. On study approach only right turn movement exist. Vehicles on approach comprise car, autorickshaw, large bus, mini-bus and motor cycle. During video recording bus was totally absent.

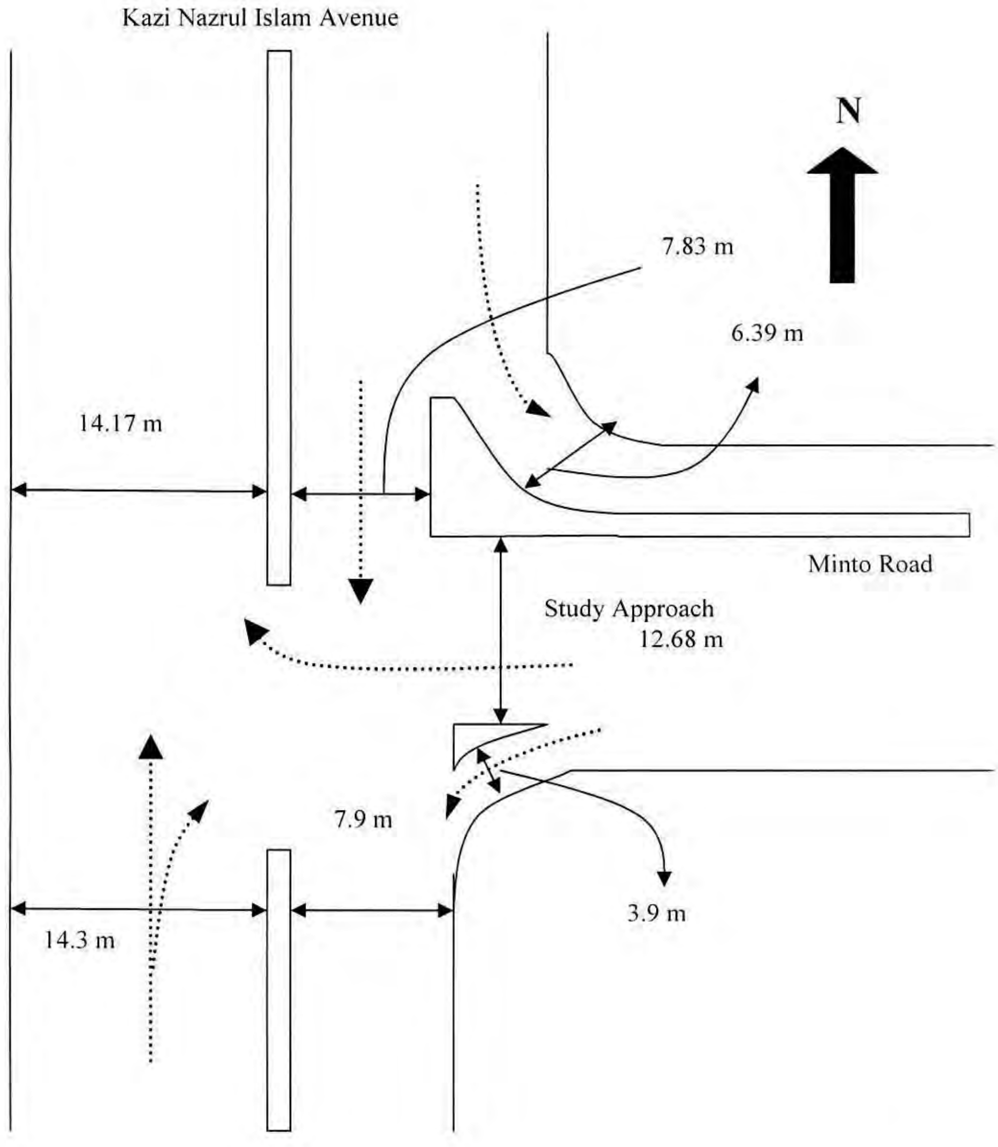

Figure 4.5 Geometric Features of Sheraton Intersection

Bangla Motor Intersection

This is a four-legged two phase intersection on New Airport road. Most of the time it remains over saturate. The surface condition and platoon speed at study approach is not satisfactory. On both study approaches right turn is banned. Also, exclusive left turn lane exists on south approach. Vehicles comprise large bus, mini-bus, car, autorickshaw and motorcycle, having dominant amount of auto rickshaw.

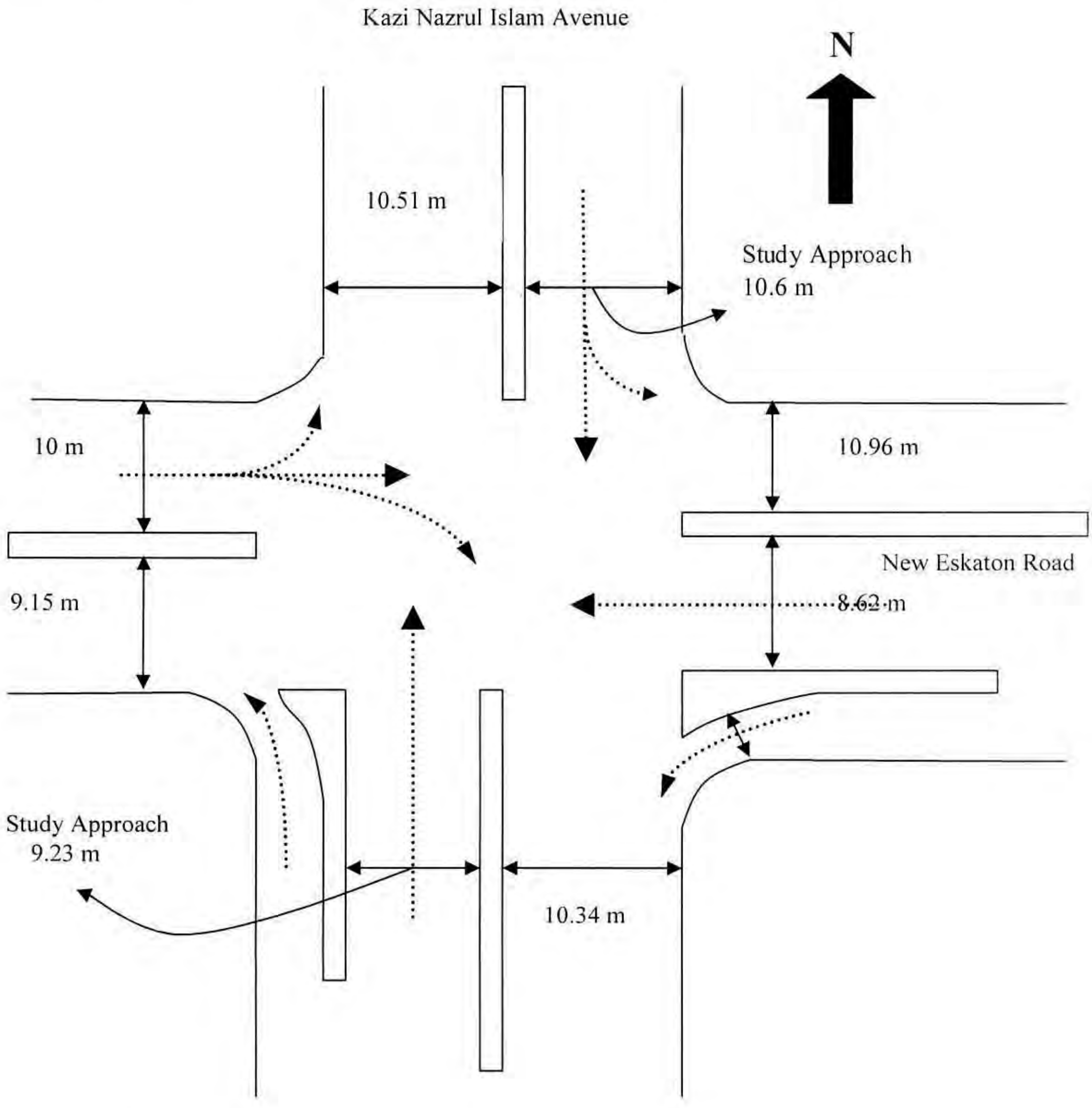

Figure 4.6 Geometric Features of Bangla Motor Intersection

4.3 Data Retrieval

The recorded film was replayed in the laboratory on a large screen TV monitor to extract the desired information. Different type's data were retrieved for saturation flow and delay calculations.

4.3.1 Saturation flow measurement

Saturation flow rate is the maximum discharge rate during green time. It is calculated either in PCU/hr or Vehicles/hr. In this study saturation period which is defined as the period when a stable moving queue has been crossing the stop line and movement wise classified traffic volume has been conducted for the whole approach as vehicle does not move in a disciplined way. The procedure for measuring prevailing saturation flow is summarized below. A sample worksheet used for recording retrieved information is included in Appendix A-1. One person can retrieve required data but more persons may help to reduce total retrieval time. Observation point was selected by playing video cassette. The observation point is normally stop line (desired position to stop). At some approaches stop line was missing. For those approaches a reference point has been considered as a desired position to stop. Start of the green was noted down from video camera timer. Video camera gives time with accuracy of one minute. Conventional stop watch was used to measure time in seconds. Stop watch is set to zero, by pausing the cassette at the moment signal turn to green. Now cassette is played until the last vehicle from the queue crossed the observation point. Saturation period was noted down from the VCP timer. The period of saturation flow begins when the green has been displayed for 3 seconds (Following ROAD NOTE 34 it was measured). Saturation flow ends when the rear axle of the last vehicle from a queue crosses the stop line. Now cassette is reversed to original position and replayed. This time different types of vehicles count is done for each movement (Left turn, through and right turn separately). Initial 3 seconds from the start of green are left to take into account start up loss time. It is not possible to count all types of vehicle count at a time for all movements. Therefore, cassette was replayed number of times and every time vehicle count of one or two types was done. The above procedure was repeated for each cycle of saturation period.

4.3.2 Delay measurement

Method suggested by HCM 2000 is based on direct observation of vehicles-in-queue counts at the intersection. This method does not directly measure delay during deceleration and during part of acceleration, which are very difficult to measure without sophisticated tracking instrument. However, this method has been shown to yield a reasonable estimate of control delay. The method includes an adjustment for error which may occur when this type of sampling technique is used, as well as an acceleration/deceleration delay correction factor.

A sample worksheet used for recording retrieved data is included in Appendix A-2. The survey period should begin at the start of the red phase of the study approach, ideally when there is no cycle failure (no overflow queue) from the previous green period. Recorded cassettes were replayed to retrieve data for delay calculations. The procedure adopted to retrieve data is summarized below.

- The moment signal turns to red, cassette is paused and VCP timer is set to zero. The overflow queue has been excluded from queue counts. The reason for this is the need for consistency with the analytical delay equation, which is based on delay to vehicles that arrive during the survey period. This time period may differ from analysis period which is typically considered as 15 minutes as per HCM 2000, because all the vehicles that join the queue within this analysis period should include in queue count until they cross the stop line.

- Cassette is played and number of vehicles in queue was recorded at regular interval of 10 to 20 seconds (As per HCM 2000). The regular interval should not be an integral divisor of the cycle length. Meanwhile it is necessary to keep track of end of standing queue by observing the last vehicle in that stops because of signal. This includes vehicles arriving when the signal is actually green, but stopped because vehicles in front have not yet started moving. The vehicles in queue counts often include some vehicles that have regained speed, but have not yet exited the intersection.

- Vehicles-in-queue count was done at regular interval for analysis period of about 15 minutes. End of the survey period must be clearly defined, since the last arriving vehicle (s) that stop in the period must be clearly defined and counted until they exit the intersection, per the next step. Stopping vehicles that arrive after the end of the analysis period are not included in the final vehicle-in-queue counts.

- Volume counts of total vehicles (V_{tot}) arrived during the survey period, and total vehicles arrived during the survey period that stops one or more times. A vehicles stopping multiple times was counted only once as a stopping vehicle (V_{stop}) as per HCM 2000 delay measurement guideline.

- The average time-in-queue per vehicle arriving in the survey period is estimated as:

$$\text{Time-in-queue per vehicle, } d_{Vq} = \left(I * \frac{\sum V_{iq}}{V_{tot}}\right) * 0.9 \qquad (4.1)$$

Where:

I = interval between vehicle-in-queue counts, s,

V_{iq} = sum of vehicle-in-queue counts, vehicle,

V_{tot} = total number of vehicles arriving during the survey period, vehicle, and

0.9 = an empirical adjustment factor accounts for the errors that may occur when this type of sampling technique is used to derive actual delay values, which normally results in an overestimate of delay (As per HCM 2000).

- Next, the fraction of vehicles stopping and the average number of vehicles stopping in a queue in each cycle are computed.

$$\text{FVS} = \text{Fraction of vehicles stopping} = V_{stop} / V_{tot}$$

- Correction factor given by HCM is selected based on average free flow speed (That was measured at the upstream of the selected approaches) and average number of vehicles stopping per queue in each cycle. The values of correction factor are given Table 4.1.

- The fraction of vehicles stopping is multiplied with correction factor and the product is added to the time-in-queue value to obtain the final estimate of control delay.

Acceleration and deceleration delay

$$d_{ad} = FVS * CF$$

$$\text{Control Delay/vehicle, } d = d_{V}q + d_{ad}$$

Control delay includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay.

Table 4.1 Acceleration/Deceleration Delay Correction Factor (CF) - seconds

Free-Flow Speed	≤7 vehicles	8-19 vehicles	20-30 vehicles
≤ 60km/h	+5	+2	-1
>60-71 km/h	+7	+4	+2
>71 km/h	+9	7	+5

Source: HCM 2000

4.4 Summary

This chapter gives insight into the study area, selection of intersections and data collection. Each intersection has been briefly described along with geometric configuration. Video camera was used for data collection. This method is much more superior to manual data collection. Methodology of measurement of saturation flow and delay in the field is discussed in detail. Subsequent chapters present development of model based on collected data. Next chapter has deals with analysis of development of regression model for saturation flow.

Chapter 5

SATURATION FLOW ANALYSIS

5.1 General

Saturation flow is the maximum rate of vehicular flow that can pass through a given intersection approach, during green. This is one of the important parameter in capacity analysis of signalized intersection. Saturation flow is essential parameter for signal design also. Saturation flow depends upon number of different factors. Traffic flow characteristics near intersections have been discussed in chapter two. This chapter primarily deals with field measurement of saturation flow, estimation of passenger car unit (PCU) values and development of saturation flow models.

5.2 Traffic Composition at Selected Intersection Approaches

Video recording was done at five different intersections in the city of Dhaka. Description of each intersection with geometric configuration is given in Chapter 4. Figure 5.1 through 5.8 gives graphical presentation of classified traffic volume at different approaches. Table 5.1 gives brief description of other features such as green period, percentage of right turn, cycle length at following intersections.

1. New Market
2. Bangla Motor
3. Panthapath
4. Science Lab
5. Sheraton

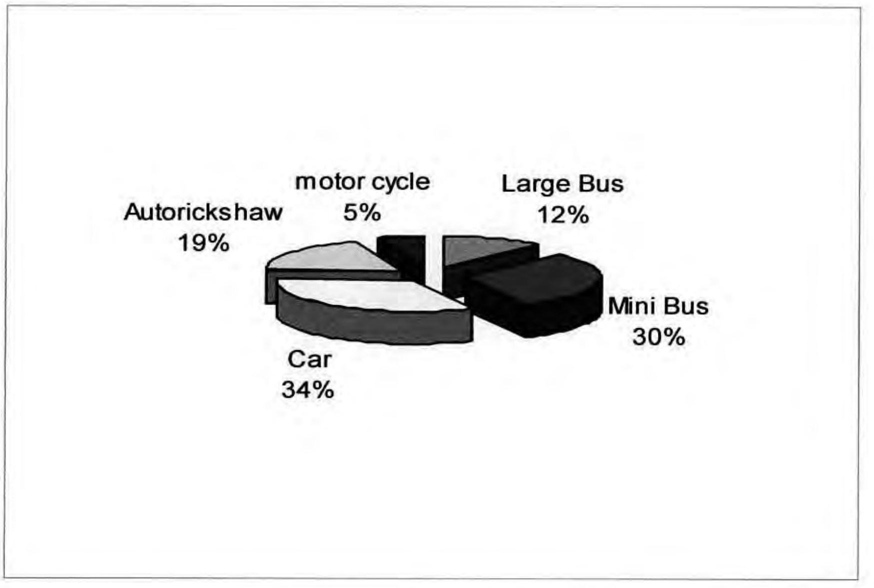

Figure 5.1 Vehicle Composition at New market South Approach

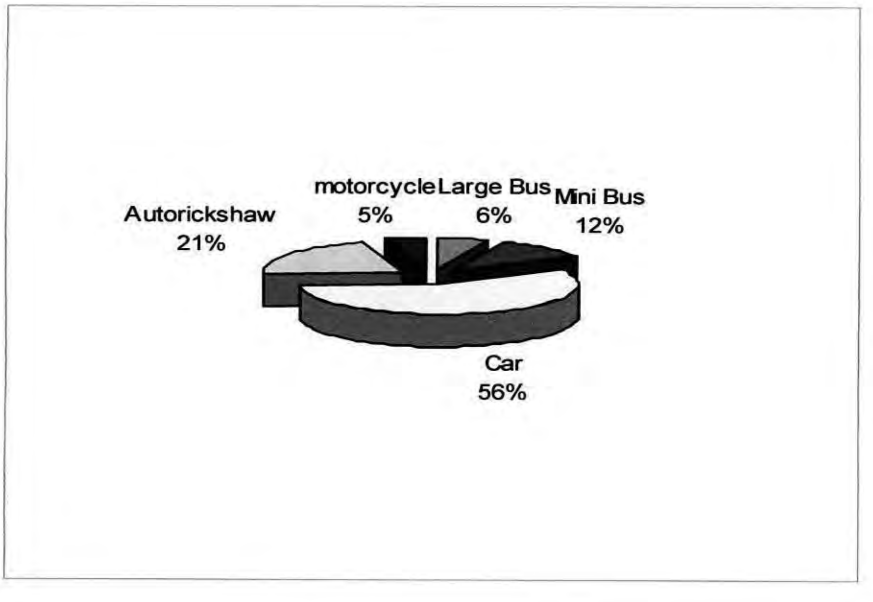

Figure 5.2 Vehicle Composition at New market North Approach

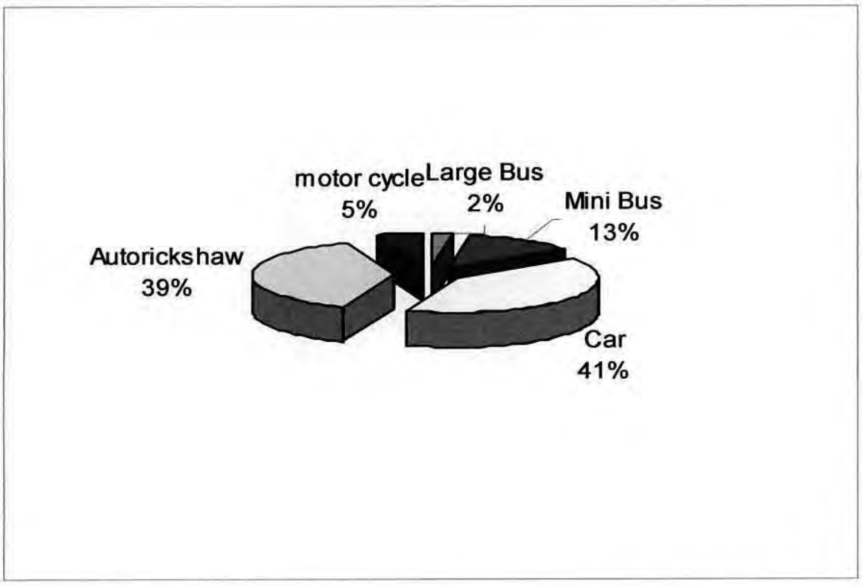

Figure 5.3 Vehicle Composition at Bangla Motor South Approach

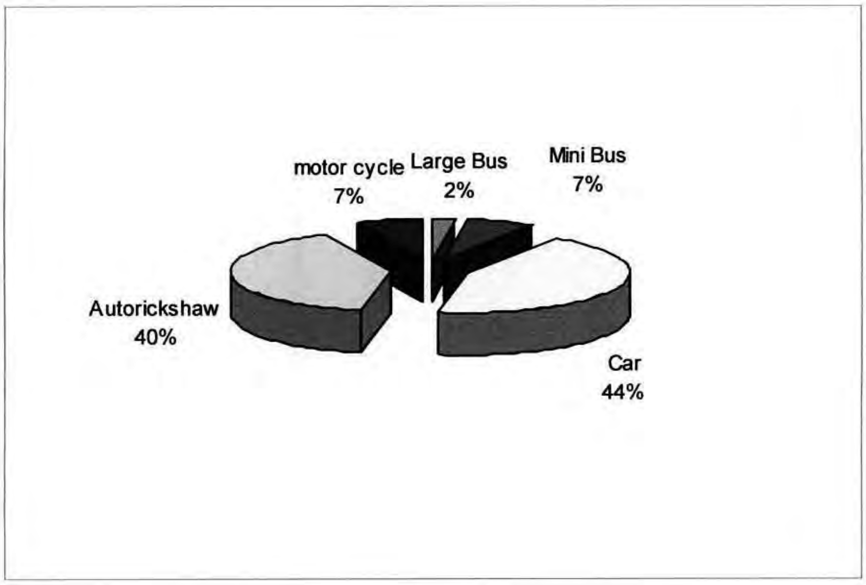

Figure 5.4 Vehicle Composition at Bangla Motor North Approach

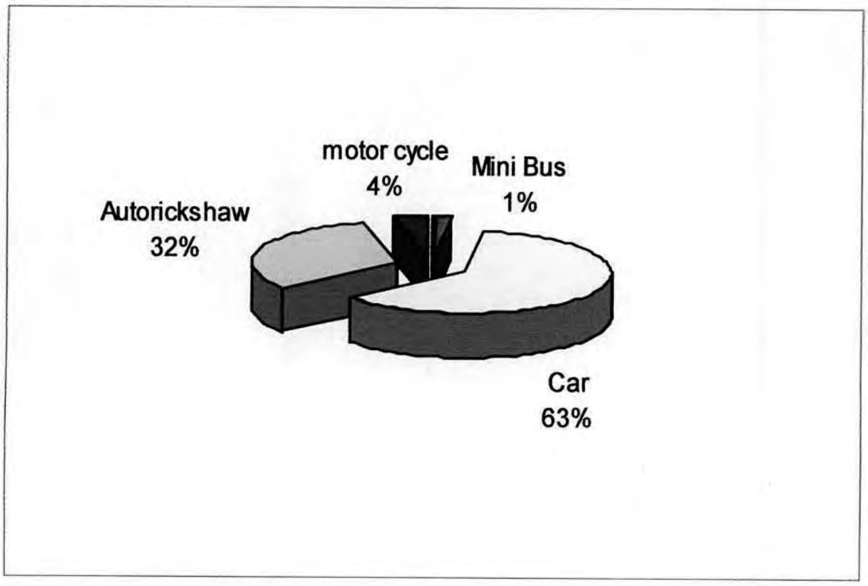

Figure 5.5 Vehicle Composition at Panthapath East Approach

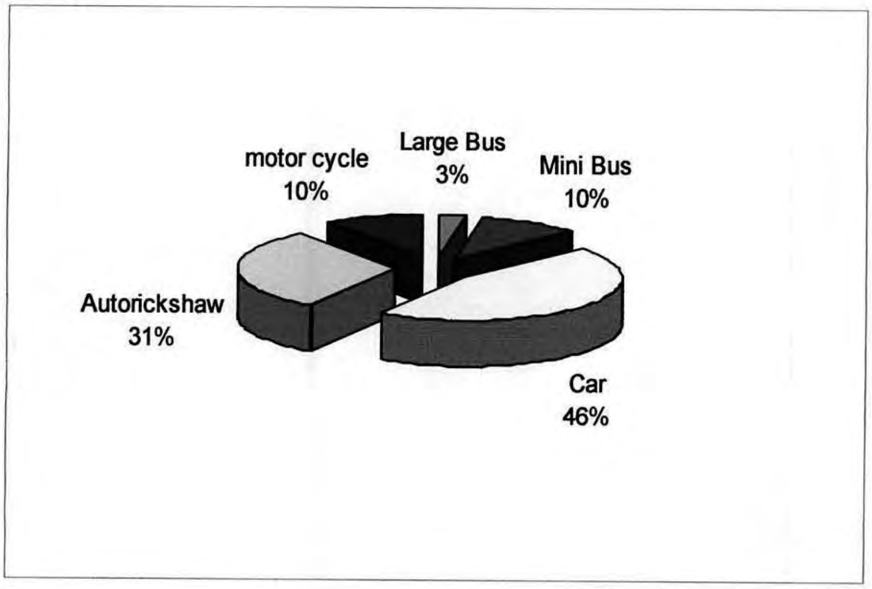

Figure 5.6 Vehicle Composition at Science Lab North Approach

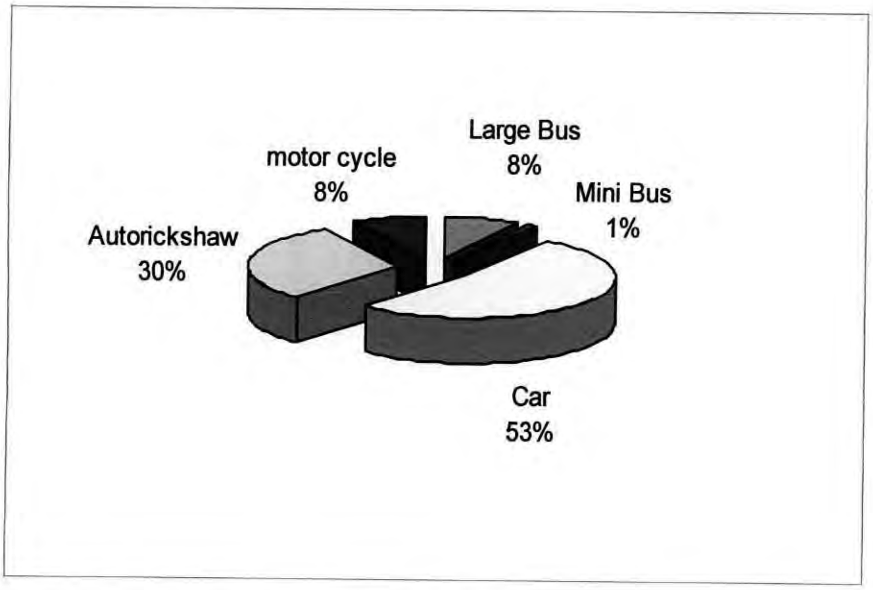

Figure 5.7 Vehicle Composition at Science Lab East Approach

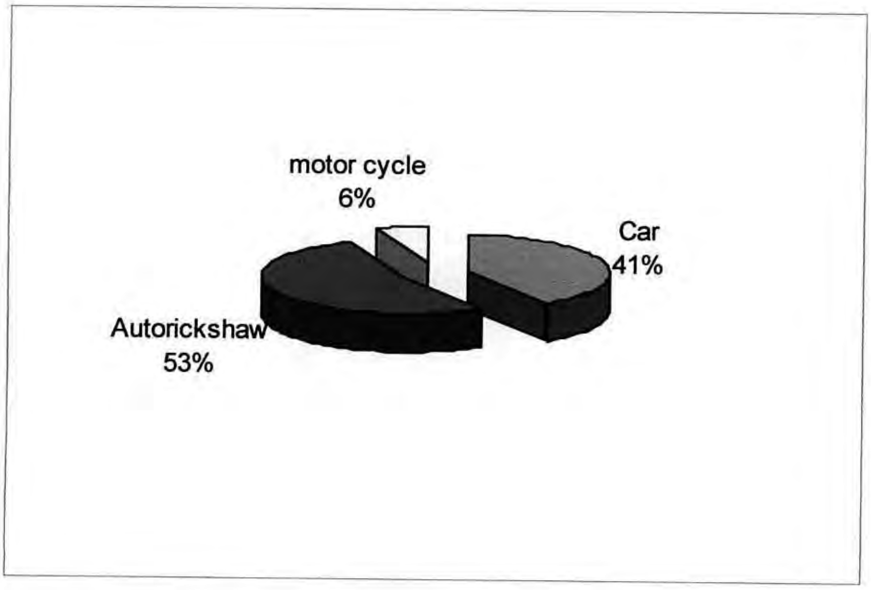

Figure 5.8 Vehicle Composition at Sheraton East Approach

Table 5.1 Description of Study Intersection Approach

Intersection	Approach Width (m)	Cycle Time (Sec)	Green time(Sec)	% of cars	% of Left turn traffic	% of Through traffic	% of Right turn traffic
New Market							
a) South Approach	11.57	219	32	34	3	88	9
b) North Approach	11.9	219	47	56	0	53	47
Bangla Motor							
a) South Approach	9.23	135	100	41	0	100	0
b) North Approach	10.6	135	100	44	0	100	0
Panthapath							
a) East Approach	12.97	190	47	63	10	83	7
Science Lab							
a) North Approach	6.78	167	107	46	0	100	0
b) East Approach	7.8	167	47	53	0	0	100
Sheraton							
a) East Approach	12.68	158	68	41	0	0	100

Note: The turning proportions those have been reported on study intersection approaches exclude exclusive left-turn.

From the above table, it is clear that most of the selected intersection comprise dominantly as through movement excluding New Market north approach which consist of a large number of right turning vehicle.

5.3 Field Measurement of Saturation Flow

The average headway method based on time headway of departing vehicles cannot be used for non lane based traffic condition, because in non lane based traffic flow, headways are difficult to observe, as vehicles do not move in lanes. Traffic is analyzed on the basis of total width of approach and hence, the option of vehicle counting is adopted. Saturation flow is calculated independently for each observed saturation period and then averaged over observed cycles. All counted vehicles are added and sum is divided by saturation period to get saturation flow in vehicles per hour. Methodology adapted to measure field saturation flow has been given in chapter 4. Table 5.2 gives average saturation flow in vehicles per hour at the eight lane groups.

Table 5.2 Field Measured Saturation Flow

Name of Intersection	Avg. Saturation Flow (Veh/hr)
New Market : South Approach	2463
New Market : North Approach	3575
Bangla Motor : South Approach	4574
Bangla Motor : North Approach	5568
Panthapath : East Approach	4734
Science Lab : North Approach	3029
Science Lab : East Approach	3540
Sheraton : East Approach	5257

From the above table, it is clear that saturation flow in terms of vehicle/hour is lowest at New Market south approaches because it comprises a large number of bigger sizes of vehicle (large bus and mini bus). Whereas, the highest flow has been observed at Bangla Motor north approach because it comprise smaller size of vehicle (auturickshaw and car) as a dominant proportion of total vehicle.

5.4 Determination of Passenger Car Unit (PCU)

The traffic operation at signalized intersection would be very much easier and simplified if all vehicles in the traffic stream were of an identical size and traveled straight ahead only. In practice, however, the operations are complicated because the traffic stream normally consists of an inseparable mixture of different types of vehicles performing different maneuvers at the traffic intersection. In respect of its road-capacity requirements each type of vehicle is equivalent to a number of passenger cars and this is called the 'passenger car unit' (PCU) equivalent. It is needed to remove the effects of traffic composition from saturation flow calculation.

In this study PCU values have been found out by synchronous regression method. The saturated green time is regressed against the number of each category of vehicles crossing the stop line during the saturated green time, assuming linear relationship between the variables. The vehicles are grouped into five categories viz, Large Bus, mini bus, car, auto rickshaw and motor cycle. Non-motorized vehicles are almost absent, hence not considered in the study. The general form of regression equation is as given in equation 5.1.

$$T = a_0 + a_1x_1 + a_2x_2 + a_3x_3 + a_4x_4 + a_5x_5 \qquad (5.1)$$

Where, T = saturated green time (sec),

a_0 = y-intercept,

a_3 = coefficient of car,

a_1, a_2, a_4, a_5 = coefficient for large bus, mini bus, autorickshaw and motor cycle respectively,

x_1, x_2, x_3, x_4, x_5 = number of vehicles of each category in time T.

Then, Passenger car unit of vehicle type i, $PCU_i = a_i / a_3$

Table 5.3 represents value of net regression coefficient for five types of vehicle along with y-intercept. The PCU values obtained for each category of vehicles at each approach, along with the percentage of vehicles and regression statistics have been presented in Table 5.4. From Table 5.4, it is observed that PCU values of a particular vehicle are not constant for the different intersections. This finding re-establishes the fact that unified passenger car unit concept for different vehicles do not hold good for the non-lane based traffic condition. The result also reveals that, the PCU by synchronous regression is not generally unbiased. Weighing coefficient for motor cycle at Panthapath intersection was found negative. As these vehicles are narrow and have high manoeuvre power, due to the lack of lane discipline they can easily avail any trapped gaps in the stream and come to the front of the queue by penetrating standing queue using inter vehicular gaps. Moreover, as their initial acceleration rates are higher than any types of vehicle, they can discharge quickly when green starts. As a result they have no effect on saturated green time rather they increase the flow more than the capacity of the approach in terms of PCU. Hence, to adjust the flow negative weighing factor has been obtained during regression and consequently negative PCU value has been measured. Again, PCU values for large bus is to some extent large for approach (Science Lab East) having right turning maneuvering only which is logical. Again for vehicles, whose proportion is very less, the method does not give proper values.

Table 5.3 Coefficient of Different Types of Vehicle from Synchronous Regression Analysis

Intersections	a_0	a_1	a_2	a_3	a_4	a_5
Bangla Motor North approach	21.690	1.585	0.924	0.704	0.238	0.319
Bangla Motor South approach	23.456	1.171	0.863	0.373	0.272	0.114
New Market South approach	20.470	0.840	0.749	0.342	0.316	0.169
New Market North approach	15.387	1.832	0.906	0.776	0.283	0.357
Panthapath East approach	12.583	2.775	2.225	0.829	0.282	-0.876
Science Lab North approach	13.280	2.968	1.987	1.146	0.246	1.032
Science Lab East approach	14.613	3.280	0.958	0.719	0.426	0.417
Sheraton East approach	10.236	_	_	0.470	0.329	0.051

Table 5.4 PCU Values and Regression Statistics

Intersections		Large Bus	Mini Bus	Car	Auto rickshaw	motor cycle	R^2	DOF	Table t-value (95% CL)
Bangla Motor North approach	PCE	2.252	1.313	1.000	0.338	0.453	0.909	13	1.77
	% Vehicle	2.273	6.993	43.794	40.253	6.687			
	t-value	1.573	2.037	2.223	1.083	0.703			
Bangla Motor South Approach	PCU	3.135	2.311	1.000	0.727	0.421	0.855	13	1.77
	% Vehicle	2.429	13.046	40.319	38.931	5.274			
	t-value	0.603	1.304	1.537	0.932	0.406			
New Market South Approach	PCE	2.454	2.188	1.000	0.922	0.493	0.490	13	1.34
	% Vehicle	12.195	30.488	32.622	19.207	5.488			
	t-value	1.237	1.678	0.852	0.595	0.217			
New Market North Approach	PCE	2.362	1.169	1.000	0.365	0.461	0.819	12	1.78
	% Vehicle	5.819	11.792	56.662	20.980	4.747			
	t-value	3.161	1.996	3.204	1.817	0.848			
Panthapath East Approach	PCE	3.346	2.682	1.000	0.340	-1.056	0.985	7	1.9
	% Vehicle	0.413	1.446	48.967	31.612	3.926			
	t-value	0.730	1.713	2.907	0.747	-1.233			
Science Lab North Approach	PCE	2.589	1.734	1.000	0.215	0.900	0.819	28	1.7
	% Vehicle	2.457	8.921	53.472	26.335	8.814			
	t-value	1.243	3.269	3.890	0.796	1.258			
Science Lab East Approach	PCE	4.564	1.333	1.000	0.593	0.579	0.840	12	1.78
	% Vehicle	6.769	1.077	59.538	25.692	6.923			
	t-value	2.339	0.306	1.805	0.622	0.605			
Sheraton East Approach	PCE	*	*	1.000	0.690	0.108	0.998	5	2.02
	% vehicle	*	*	48.136	46.515	5.348			
	t-value	*	*	3.517	5.149	0.120			

Note: Large bus and mini bus were not present at the time of video recording at east approach of Sheraton Intersection.

5.5 Estimation of Saturation Flow (in PCU/hr)

Saturation flow is estimated in PCU/hr using the PCU values obtained at each intersection and also using average PCU for all the intersections. In order to calculate average PCU, the vehicles with negative PCU values are neglected. Following conventional procedure is adopted to find out saturation flow value for each approach. First, the saturated green time (T sec) is divided by the number of different categories of vehicles that have been converted into passenger car unit to get the time headway. Inverse of headway gives the saturation flow. Thus the saturation flow in PCU/hr is obtained as:

$$S = \frac{(PCU_{LB} * x_1 + PCU_{MB} * x_2 + PCU_C * x_3 + PCU_{AR} * x_4 + PCU_{MC} * x_5)}{T} \text{ X } 3600$$

The PCU values of Table 5.4 and Table 5.5 have been used to estimate saturation flow for each cycle. The average saturation flows in PCU/hr and in veh/hr for a particular approach over the entire number of cycles have been reported in Table 5.6. Figure 5.9 shows the relationship between width of road and estimated saturation flow based on individual intersection PCU values and average for all the observed approach. From table 5.6 it is clear that saturation flow in terms of PCU/hr is higher than vehicle/hr for the both study approaches of New Market intersection. Whereas, for rest of the study approaches reverse scenario has been obtained. Because, for the both north and south approaches of New Market intersection proportion of larger vehicle (large bus and mini bus) was higher. Hence, saturation flow estimated in terms of PCU/hr was also higher.

Table 5.5 Average PCU Values

	Large Bus	Mini Bus	Car	Auto rickshaw	motor cycle
Average PCU	2.68	1.82	1	0.52	0.29
Std. Dev	0.40	0.58	0	0.24	0.108

Table 5.6 Comparison of Observed and Estimated Saturation Flow

Name of Intersections	Saturation flow based on average PCU	Saturation flow based on Individual PCU	Avg. Saturation Flow (Veh/hr)
New Market South Approach	3365	3125	2463
New Market North Approach	3805	3600	3575
Science Lab North Approach	2921	2735	3029
Panthapath East Approach	3895	3764	4734
Bangla Motor North Approach	4849	4600	5568
Bangla Motor South Approach	4309	4283	4574
Science Lab East Approach	3293	3498	3540
Sheraton East Approach	3909	4027	5257

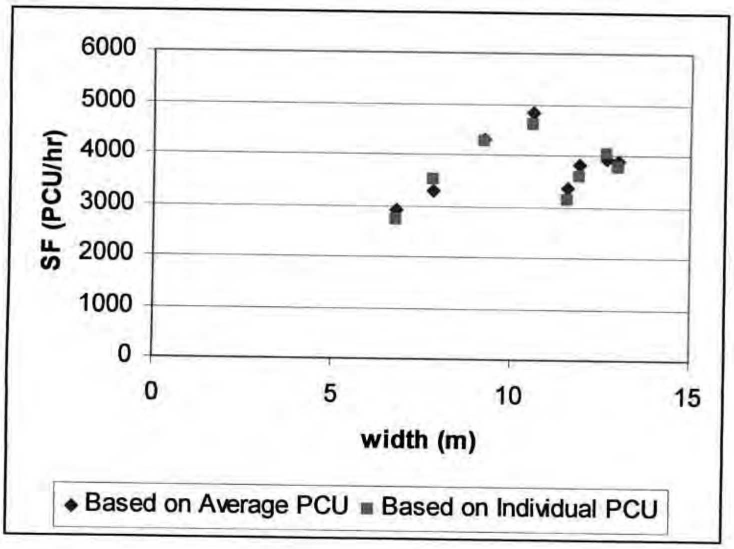

Figure 5.9 Relationships between Road width and Estimated Saturation Flow

5.6 Development of Saturation Flow Model

A study into the saturation flow of signalized intersections under non lane based traffic would require a large database from field over a range of traffic flow and geometric conditions (Turner et al 1993). Such a large database from the relevant situation is not available. In present study, field data have been collected at eight approaches. Using this data, multiple regression analysis has been made in order to estimate the saturation flow in passenger car unit per hour. From Figure 5.9 in can be seen that saturation flow based on individual PCU and that on the average PCU value for all the intersection are almost same. That's why saturation flow obtained based on average PCU are used for the regression analysis. The database that has developed from this study in order to conduct regression analysis is given in Appendix A-3. The independent variables used are listed in Table 5.7. Selection of these variables was largely on the basis of ease of collection and the experience of earlier studies (Turner et al 1993).

Table 5.7 Independent Variable for Saturation Flow

Variable	Variable Name	Description
A	WIDTH	Width of study approach at stop line being surveyed.
B	SIGSET	Length of signal green time for approach being surveyed.
C	L/TURN W	Width of intersection exit for left-turning traffic.
D	R/TURN W	Width of intersection exit for right-turning traffic.
E	STR-ON W	Width of intersection exit for traffic traveling straight on.
F	%RT	Percentage of right turning vehicle (Include all types of vehicle)

Three models have been developed from regression analysis (Excluding saturation flow for the approaches having right turn only) carried out by SPSS V11. These models have been given below along with their statistics.

Model 1 (width only-forced through origin)

S= 356 w

MODEL SUMMARY

Model	R	R Square[a]	Adjusted R Square	Std. Error of the Estimate
1	.975[b]	.951	.951	**********

ANOVA

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	1.19E+09	1	1193194403	1685.081	.000[a]
	Residual	64436483	91	708093.215		
	Total	1.26E+09[b]	92			

COEFFICIENT

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	WIDTH	355.635	7.591	.975	46.850	.000

Model 2 (width only)

S= 2217 + 140 w

MODEL SUMMARY

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.445[a]	.198	.191	**********

ANOVA

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	9420814	1	9420813.906	19.802	.000[a]
	Residual	42817025	90	475744.719		
	Total	52237839	91			

COEFFICIENTS

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	2217.104	270.262		8.204	.000
	WIDTH	140.790	26.871	.445	5.239	.000

Model 3 (Multi-variate)

S= 24 SIGSET + 337 w + 45.5 STR-ON-W – 3.61 %RT – 1662

MODEL SUMMARY

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.999[a]	.998	.988	**********

ANOVA

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	42900317	4	10725079.24	99.928	.000[a]
	Residual	9337522	87	107327.835		
	Total	52237839	91			

COEFFICIENTS

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-1662.415	279.440		-5.949	.106
	SIGSET	24.301	1.763	1.186	13.786	.046
	WIDTH	337.447	38.454	1.103	8.775	.072
	STR-ONW	45.375	45.463	.094	.998	.501
	%Rightturn	-3.609	2.030	-.098	-1.778	.326

5.7 Comparison of Regression Model Saturation Flow with the previous study

The simplest model is biased as it is forced through the origin. Similar model was developed by Webster et al. 1966 based on regression analysis of extensive field data collected from several intersections of London city. Though a higher degree of explanation has been achieved for model 1 but the accuracy of this model due to the inherent bias is unclear. Perhaps a better indication can be given by the model derived with an intercept term. Whilst reporting a reduced degree of explanation for its width variable compared to the model without an intercept. Similar model was also derived for non lane based traffic condition for Javanese (Indonesia) cities by Sutomo 1992 and Turner et al. 1993. Equation 5.2 and 5.3 represent the regression equation of the above two studies respectively.

$$S=-376 + 627w \tag{5.2}$$

$$S=964 + 349w \tag{5.3}$$

The degree to which model 2 compares with other studies can be seen in Fig. 5.10. From Fig. 5.10 it can be seen that all the models predict higher saturation flow as a function of the approach width than the present model. At the same time it is also clear that prediction of saturation flow depends not only upon approach width as considerable differences between saturation flow may be found for the same width from the previous studies and also among all the studies for the non lane based traffic including the present one.

For the above reason Model 3 was developed for which much higher degree of explanation is achieved as a function of length of signal green time for approach being surveyed, width of approach at stop line being surveyed, width of junction exit for traffic traveling straight on and Percentage of right turning vehicle. In the previous study most of the flow model was developed as a function of approach width only. So the multivariate model was unable to compare.

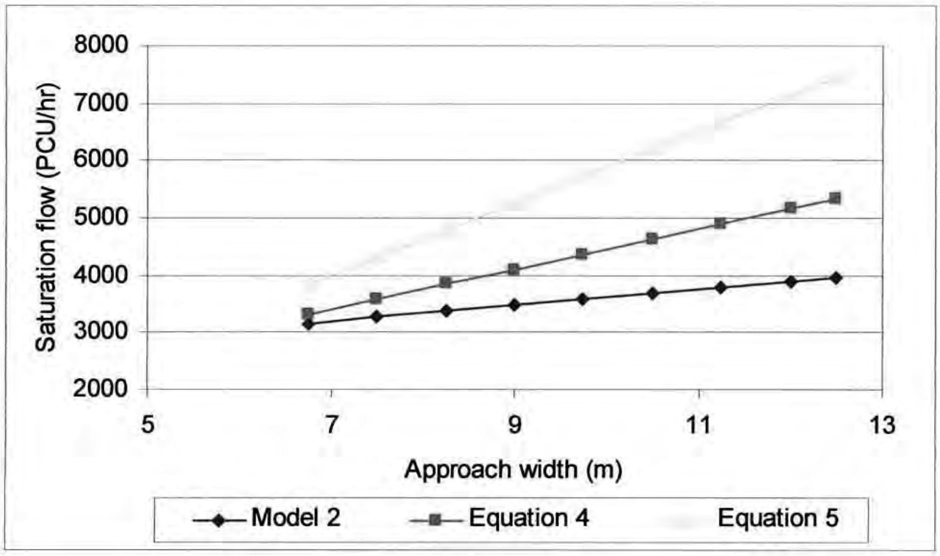

Figure 5.10 Comparisons of Model 2, Equation 5.2 and Equation 5.3

5.8 Comparison of Predicted Saturation Flow Model with Field Observed Saturation Flow

Saturation flow is calculated using regression Model 1, 2 and 3. Fig 5.11 to 5.13 compares observed flow with the three model of this study. Table 5.8 shows root mean square error (RMSE) and R^2 values. From Table 5.8 it is observed that Model 3 gives highest value of R^2 and lower RMSE than Model 1 and 2. Hence, Model 3 has proposed in this study to estimate saturation flow for non-lane based traffic condition.

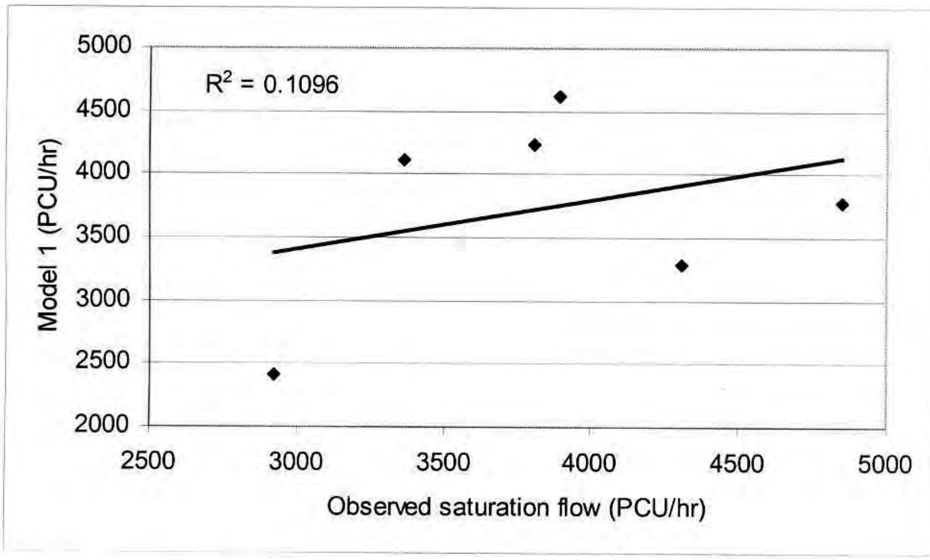

Figure 5.11 Correlation between Observed and Model 1 Saturation Flow

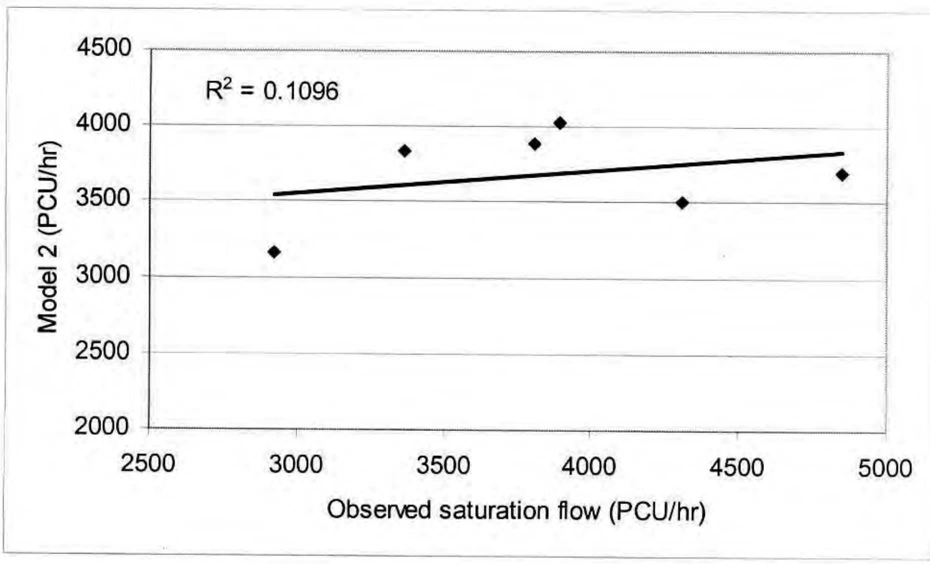

Figure 5.12 Correlation between Observed and Model 2 Saturation Flow

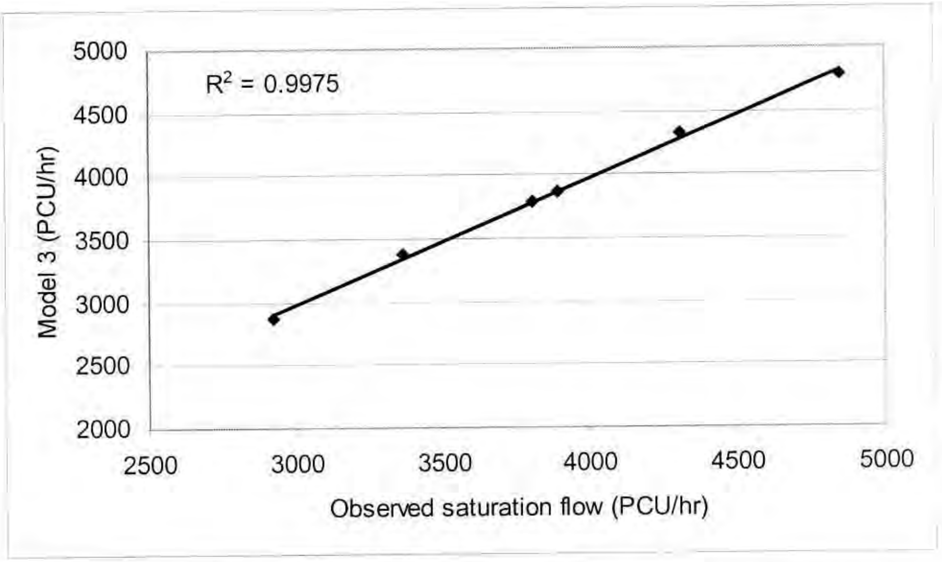

Figure 5.13 Correlation between Observed and Model 3 Saturation Flow

Table 5.8 RMSE and R^2 Values for the Saturation Regression Model

Model No.	RMSE	R^2
1	22.4	0.1096
2	15.7	0.1096
3	1.12	0.9975

5.9 Summary

Accurate saturation flow values are a fundamental building block in the management of efficient urban traffic signal control and intersection design. In this chapter, initially A ranges of site-specific PCU values were obtained using synchronous multiple linear regression. The saturation flow for each survey approach was calculated using the average PCU values and multiple linear regression techniques were then used to derive predictive saturation flow models. The saturation flow values for full approach were regressed against several intersection characteristics. The resulting models and statistical performance indicators were reported than. Chapter ends with comparing model values with field values. Next chapter basically deals with delay model for the study approaches.

Chapter 6

DELAY ANALYSIS

6.1 General

The next step after saturation flow in capacity analysis of signalized intersection is the determination of delay. HCM 2000 has defined six levels of services based on control delay. More information about types of delay and theoretical delay models have been given in chapter three. Field saturation flow value or that obtained by models given in previous chapters may be used in estimation of delay. This chapter includes determination of delay by various methods and development of delay models for non lane based traffic conditions.

6.2 Field Measurement of Delay

Field measurement of delay was done at five intersection approaches. Detailed description about intersections is given in chapter 4. Traffic recording was done at approaches covering the whole queue. Data retrieval was done in the laboratory and delay calculated for five intersection approaches. HCM 2000 procedure is followed to calculate field delay. Procedure for the field measurement of delay has been presented in chapter 4.

In this method, the number of vehicles in queue is recorded at regular interval of 10 to 20 seconds. The regular interval should not be an integral divisor of the cycle length, to eliminate potential survey bias caused by queue buildup in a regular cyclic pattern. This number is then multiplied by the interval length, resulting in total vehicle seconds of delay on the approach over the analysis period. This total is then divided by the total volumes of vehicles passed through the approach over the analysis period, resulting in the delay per vehicles at an approach. This value is then multiplied by the correction factor of 0.9 to account for the overestimation of delay by this method. The resultant number is time-in-queue per vehicle. Estimated acceleration/deceleration delay is added to time-in-queue. The resultant delay is control delay for particular approach, which is also Level of Service criterion for signalized intersections as per HCM 2000. Survey period of about 15-20 minutes is taken for delay measurement as per convenience. Table 6.1 gives the field measured delay.

Table 6.1 Field Measured Delay

Intersection	Arrival flow rate (veh/hr)	Saturation flow rate (veh/hr)	Capacity (veh/hr)	Degree of saturation	Time-in-queue (sec)	Acc/Dec Delay (sec)	Control Delay (sec)
New Market North Approach	940	3575	767	1.226	107.234	3.574	110.809
	1120	3575	767	1.460	117.000	3.785	120.786
	1152	3575	767	1.501	121.563	3.652	125.215
	1200	3575	767	1.564	133.980	3.733	137.713
	1160	3575	767	1.512	122.648	3.765	126.414
	1280	3575	767	1.668	148.668	1.950	150.619
Science Lab North Approach	1296	3029	1940	0.668	27.666	1.006	28.673
	1264	3029	1940	0.651	33.664	2.000	35.665
	1248	3029	1940	0.643	28.615	2.038	30.654
	1304	3029	1940	0.672	39.092	2.208	41.301
	1364	3029	1940	0.703	37.847	1.982	39.830
	1348	3029	1940	0.695	40.166	2.219	42.386
Science Lab East Approach	1104	3413	1263	0.874	41.673	2.811	44.486
	1048	3413	1263	0.83	49.534	2.854	52.389
	1052	3413	1263	0.833	54.889	3.117	58.008
Panthapath North Approach	988	4734	1171	0.844	62.161	1.514	63.676
	1000	4734	1171	0.854	74.664	1.616	76.280
	1224	4734	1171	1.045	93.352	1.803	95.157
	1164	4734	1171	0.994	85.298	1.759	87.058
	1116	4734	1171	0.953	89.032	1.770	90.803
Sheraton East Approach	1540	5257	2262	0.681	46.566	1.283	47.849

6.3 Comparison of Field Delay with Theoretical Delay

Delay is estimated using theoretical models. Delay estimated with theoretical model is compared with field measured delay values. Following theoretical delay models are considered for analysis.

1. HCM 2000 Model
2. Ackelik's Model
3. Reilly's Model
4. TRANSYT-6 Model
5. Webster's Model

Detailed calculation of delay for each model is given in Appendix A-5. Table 6.2 summaries delay values along with percentage difference with field values for each model. Figure 6.1 presents graphical comparison of theoretical models with field measured delay. Table 6.3 shows root mean square error (RMSE) and R^2 values. From Table 6.3 it is observed that Webster's model gives lower value of R^2 and higher value of RMSE. Again Webster's equations cannot use if v/c ratio (Degree of saturation) is greater than 1.0. This equation gives very high delay for v/c values close to 1.0. Reilly's model and Ackelik's Model gives slightly higher value of RMSE and satisfactory value of R^2. This equation gives very close results for isolated signalized intersection but do not take into account effect of signal coordination and uncoordinated nearby intersections. Both HCM 2000 delay model and TRANSYT-6 Model have satisfactory value of RMSE and R^2. Again among all of these models only HCM2000 takes into account effect of signal coordination and uncoordinated nearby intersections. Figure 6.2 through 6.6 shows correlation between theoretical delay model and observed delay.

Table 6.2 Comparison of Theoretical Delay with Field Measured Delay

Intersection	Theoritical Delay (sec/veh)					Control Delay (sec/veh)	Percentage Difference				
	HCM 2000 model	Akcelik Model	Reilly's Model	TRANSYT 6 Model	Webster Model		HCM 2000 model	Akcelik Model	Reilly's Model	TRANSYT 6 Model	Webster Model
New Market North Approach	216.980	224.405	158.02	222.431	_	110.809	48.932	50.621	29.877	50.183	_
	342.025	358.257	228.89	355.036	_	120.786	64.685	66.285	47.229	65.979	_
	358.741	376.406	238.62	373.066	_	125.215	65.096	66.734	47.525	66.436	_
	391.371	411.224	257.05	407.734	_	137.713	64.813	66.511	46.425	66.225	_
	364.172	382.198	241.68	378.832	_	126.414	65.287	66.924	47.694	66.631	_
	445.944	469.572	288.01	465.893	_	150.619	66.225	67.924	47.704	67.671	_
Science Lab North Approach	20.682	18.839	18.84	20.682	18.749	28.673	-38.639	-52.199	-52.199	-38.639	-52.923
	20.212	18.497	18.50	20.212	18.486	35.665	-76.451	-92.812	-92.812	-76.451	-92.929
	19.986	18.331	18.33	19.986	18.356	30.654	-53.378	-67.224	-67.224	-53.378	-66.995
	20.803	18.926	18.93	20.803	18.817	41.301	-98.533	-118.222	-118.222	-98.533	-119.486
	21.769	19.608	19.61	21.769	19.338	39.830	-82.965	-103.131	-103.131	-82.965	-105.964
	21.502	19.422	19.42	21.501	19.196	42.386	-97.129	-118.236	-118.236	-97.129	-120.807
Science Lab East Approach	43.123	41.373	39.31	45.916	37.962	44.486	-3.159	-7.521	-13.169	3.115	-17.184
	40.082	38.455	37.41	42.808	35.535	52.389	-30.706	-36.234	-40.045	-22.382	-47.429
	40.267	38.632	37.53	42.998	35.661	58.008	-44.056	-50.156	-54.572	-34.906	-62.666
Panthapath North Approach	77.173	70.035	69.02	75.532	66.974	63.676	17.489	9.080	7.744	15.697	4.925
	77.915	70.691	69.46	76.268	67.588	76.280	2.098	-7.906	-9.822	-0.015	-12.859
	113.895	110.271	91.42	113.247	0.000	95.157	16.452	13.706	-4.082	15.975	_
	98.497	91.872	81.62	96.775	0.000	87.058	11.613	5.239	-6.669	10.041	_
	89.419	81.932	76.17	87.719	88.818	90.803	-1.547	-10.827	-19.207	-3.514	-2.235
Sheraton East Approach	47.063	36.253	36.25	37.929	34.912	47.849	-1.671	-31.987	-31.987	-26.153	-37.055

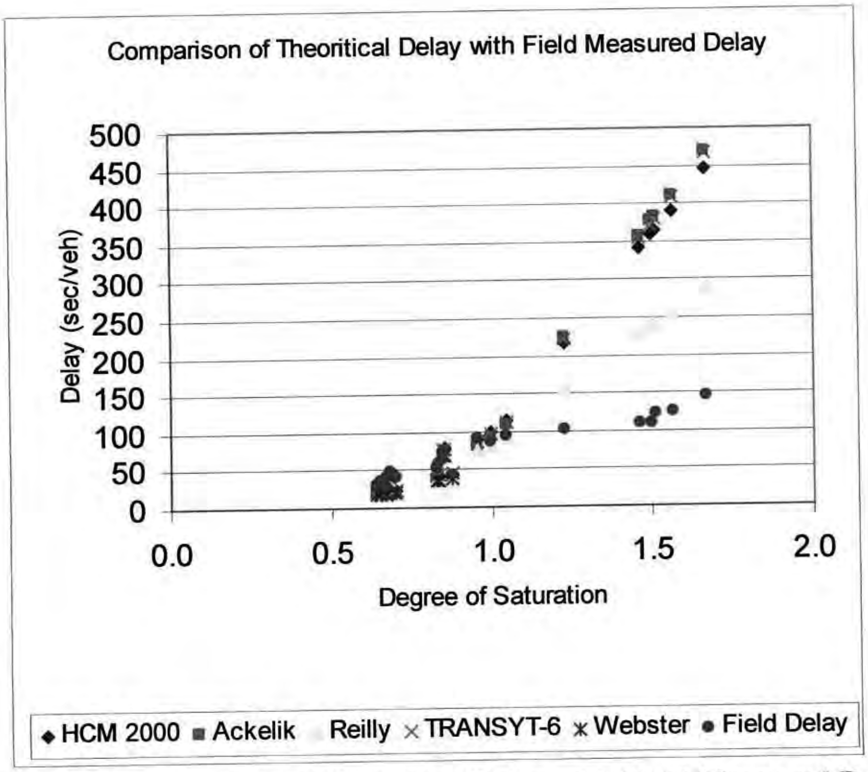

Figure 6.1 Comparison of Theoretical Delay with Field Measured Delay

Table 6.3 RMSE and R^2 Values between Observed Delay and Theoretical Delay Model

Delay Model	RMSE	R^2
HCM 2000 model	27.872	0.926
Akcelik Model	32.181	0.9279
Reilly's Model	34.343	0.926
TRANSYT 6 Model	27.407	0.9317
Webster Model	43.4365	0.9036

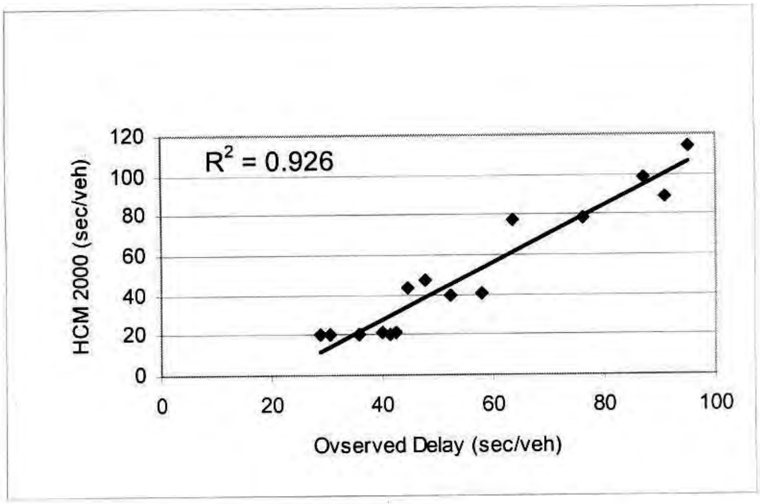

Figure 6.2 Correlation between HCM 2000 Model and Observed Delay

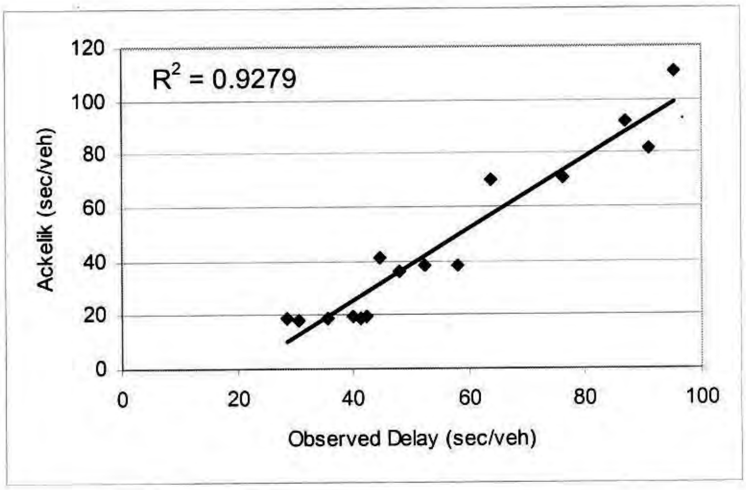

Figure 6.3 Correlation between Ackelik's Model and Observed Delay

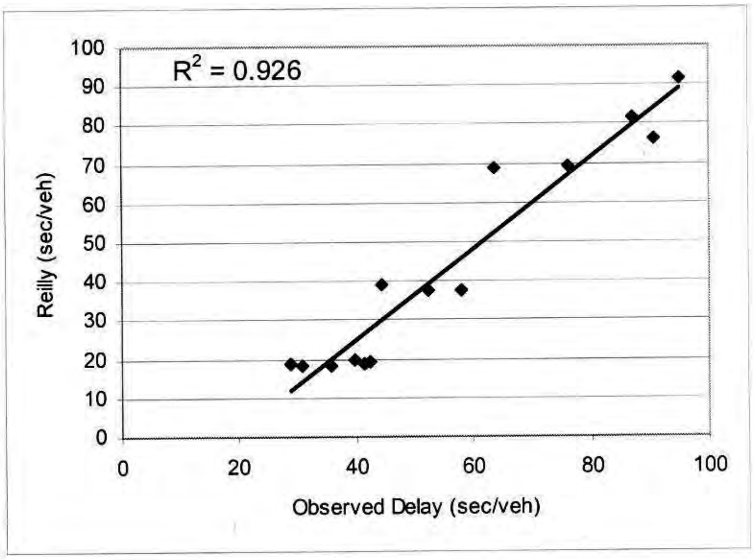

Figure 6.4 Correlation between Reilly's Model and Observed Delay

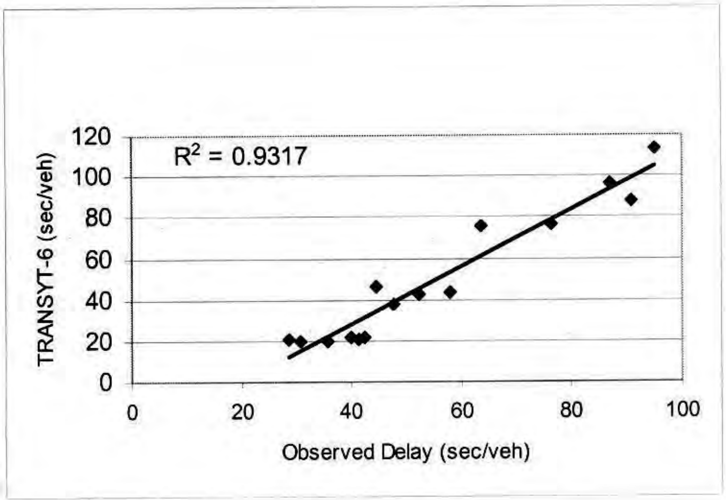

Figure 6.5 Correlation between TRANSYT-6 Model and Observed Delay

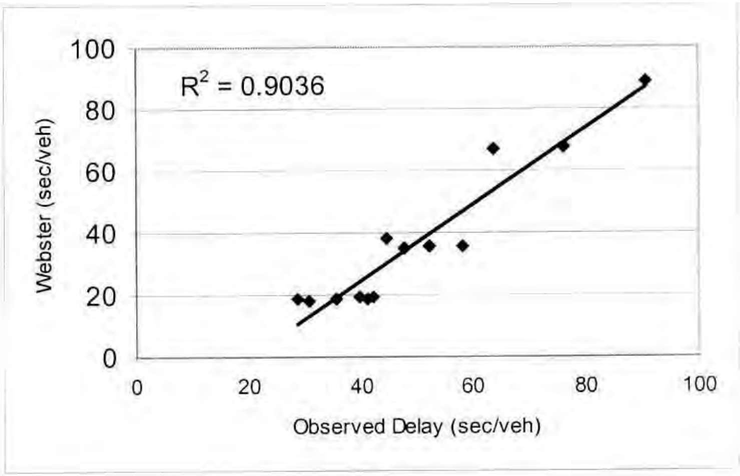

Figure 6.6 Correlation between Webster's Model and Observed Delay

6.4 Delay Model for Non Lane Based Traffic Condition

HCM 2000 delay model takes into account effect of signal coordination and uncoordinated surrounding intersections. Among all theoretical models HCM 2000 model is selected to modify in order to be able to estimate control delay for non lane based traffic condition. From Table 6.2 it is observed that HCM 2000 model consistently overestimates field-measured delay values at v/c ratio above 1.0 and underestimate delay at v/c ratio less than 1.0.

Details about HCM 2000 equation are available in chapter 3. Control delay which includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay is given by:

$$d = d_1 * PF + d_2 + d_3 \tag{6.1}$$

Where,

d_1 = uniform delay assuming uniform arrival

d_2 = incremental delay due to random arrival and over saturation queues

d_3 = residual delay.

In present study, starting time survey is selected in such a way that there is no residual delay and hence d_3 is zero. For the purpose of regression analysis above delay equation can be written as

$$d_f = ax_1 + bx_2 \tag{6.2}$$

Where,

d_f = field delay

$x_1 = d_1 * PF$

$x_2 = d_2/900$

a and b = calibration parameters.

Where:

$$d_1 = 0.5C \frac{(1-\frac{g}{C})^2}{(1-Min(1,X)\frac{g}{C})}$$

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

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

The main aim of the analysis is to find suitable value of constant in the equation for d_1 and d_2 which are currently 1 and 900 respectively. Value of saturation flow is one of the most important variables in finding out delay. For the purpose of regression analysis average saturation flow of a particular approach has been used that was observed over the entire survey period. And delay estimation equation has been proposed based on regression analysis carried out by SPSS V11. Proposed Delay model along with statistics are given in the next page.

$$d_f = 1.171\ x_1 + 264\ x_2 \tag{6.3}$$

MODEL SUMMARY

Model	R	R Square[a]	Adjusted R Square	Std. Error of the Estimate
1	.975[b]	.950	.942	13.556619

ANOVA

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	41992.740	2	20996.370	114.246	.000[a]
	Residual	2205.383	12	183.782		
	Total	44198.123[b]	14			

COEFFICIENTS

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	X1	1.171	.173	.931	6.780	.000
	X2	264.003	734.422	.049	.359	.725

The first term of equation 6.3 is for uniform delay and second term is for delay due to random arrival and over saturation queues. The first term of equation 6.3 suggests that uniform delay equation gives close estimate of field value which is 1.134. The second constant is 264. The value of this constant is 900 in HCM 2000 delay equation. In preparing models for the 1985 Highway Capacity Manual, Reilly et al. (1983) conducted extensive field studies to measure delay. They found that Ackelik's equation consistently overestimated field measured values, and recommended that the theoretical overflow delay results be reduced by 50% to better reflect field conditions. Present study establishes the similar fact.

And from the equation 6.3, it is clear that theoretical incremental delay should be reduced by 70% for non lane based traffic condition. The suggested delay equation shows good correlation with field measured delay. Table 6.4 compares it with HCM 2000 and field values. Value of RMSE is 1.39 and R^2 =0.895 for the suggested model. Where as that was 27.87 and .926 respectively for HCM 2000 theoretical delay formula. Figure 6.8 presents bar chart of three delay values

Table 6.4 Comparison of Delay by Suggested Model with Field and HCM 2000 Delay

Intersection	Delay (sec/veh)			% Differences	
	Field Observed	HCM 2000	Suggested Model	HCM 2000	Suggested Model
New Market North Approach	110.8	217.0	139.1	48.93	20.35
	120.8	342.0	176.2	64.69	31.47
	125.2	358.7	181.2	65.10	30.88
	137.7	391.4	190.7	64.81	27.79
	126.4	364.2	182.7	65.29	30.83
	150.6	445.9	206.7	66.22	27.14
Science Lab North Approach	28.7	20.7	22.6	-38.64	-26.87
	35.7	20.2	22.2	-76.45	-60.92
	30.7	20.0	22.0	-53.38	-39.65
	41.3	20.8	22.7	-98.53	-81.84
	39.8	21.8	23.6	-82.96	-68.81
	42.4	21.5	23.4	-97.13	-81.50
Science Lab East Approach	44.5	43.1	42.9	-3.16	-3.73
	52.4	40.1	41.3	-30.71	-26.92
	58.0	40.3	41.4	-44.06	-40.17
Panthapath North Approach	63.7	77.2	83.8	17.49	23.98
	76.3	77.9	84.2	2.10	9.38
	95.2	113.9	97.7	16.45	2.58
	87.1	98.5	93.0	11.61	6.42
	90.8	89.4	89.5	-1.55	-1.43
Sheraton East Approach	47.8	47.1	53.6	-1.67	10.79

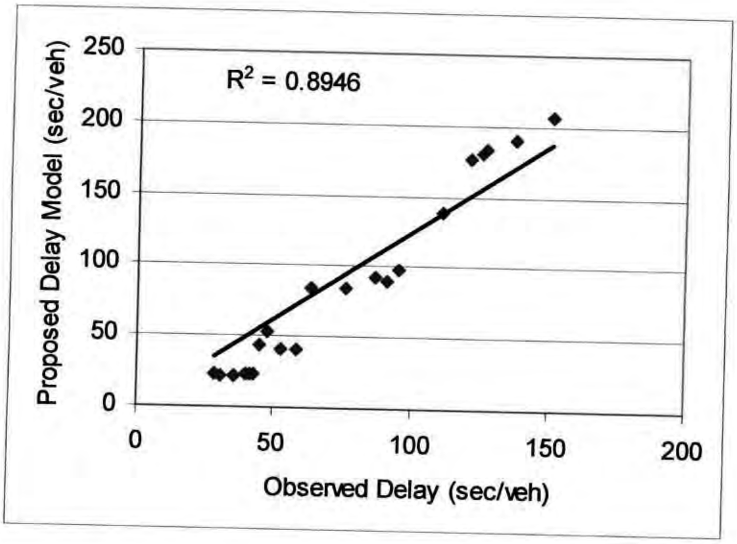

Figure 6.7 Correlation between Observed Delay and Suggested Delay Model

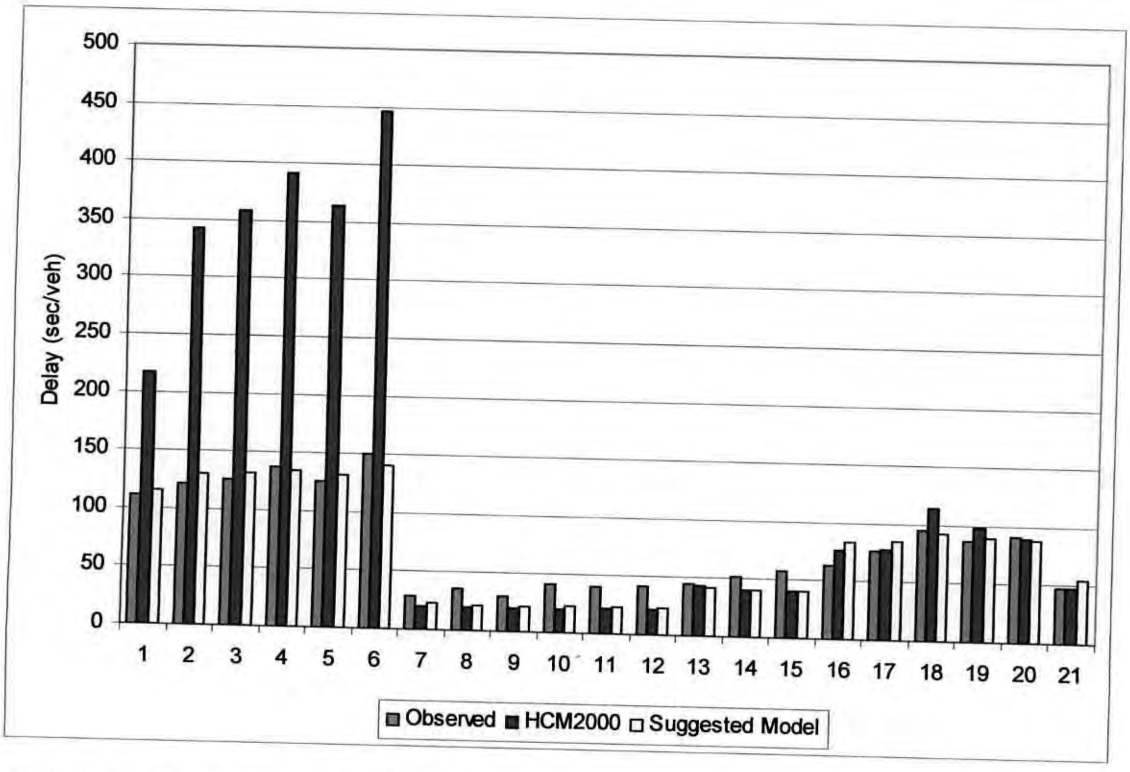

Figure 6.8 Comparison of Delay by Suggested Model with Field and HCM 2000 Delay

6.5 Summary

Delay is a very important parameter in capacity analysis of signalized intersections. HCM 2000 has defined six LOS based on control delay. This chapter gives insight into the field measurement of delay, theoretical estimation of delay and recommendation to HCM delay model to become applicable in non-lane based traffic condition. Field delay is calculated as per HCM guidelines. This delay is compared with various theoretical delay models. It is found that theoretical delay values are consistently higher than field values when v/c exceeds 1.0. It is suggested to reduce overflow delay by 70% to reflect better field conditions.

Chapter 7

ESTIMATING VEHICLE STOPS AT SIGNALIZED INTERSECTION

7.1 General

Micro-simulation of traffic as a tool for investigating traffic systems has increased in popularity over the last decades. The number of simulation models is very large and the simulation approach utilized in these models is to a large extent differentiated. This chapter presents an overview of the model building process of an AIMSUN model developed by Transportation Simulation Systems Ltd. (TSS). This has been done using the specific example of the north approach of New Market intersection located on Mirpur arterial (comprise only motorized vehicle). This microscopic simulation model has been applied to estimate vehicle stops for oversaturated condition. Finally, an analytical equation has been derived from the simulation result to compute vehicle stops for over-saturated approaches over a given analysis period.

7.2 Simulation Process in AIMSUN

The logic of the simulation process in AIMSUN (Advanced Interactive Microscopic Simulator for Urban and Non-Urban Network) is illustrated in Figure 7.1. It can be considered as a hybrid simulation process, combining an event scheduling approach with activity scanning. At each time interval (simulation step), the simulation cycle updates the unconditional events scheduling list (i.e. events such as traffic light changes which do not depend on the termination of other activities). The "Update Control" box in the flow chart represents this step. After this updating process a set of nested loops starts to update the states of the entities (road sections and junctions) and vehicles in the model. Once the last entity has been updated, the simulator performs the remaining operations such as inputting new vehicles, collecting new data, etc. Depending on the type of simulation, new vehicles are input into the network according to flow generation procedure (headway distributions for example) at input section.

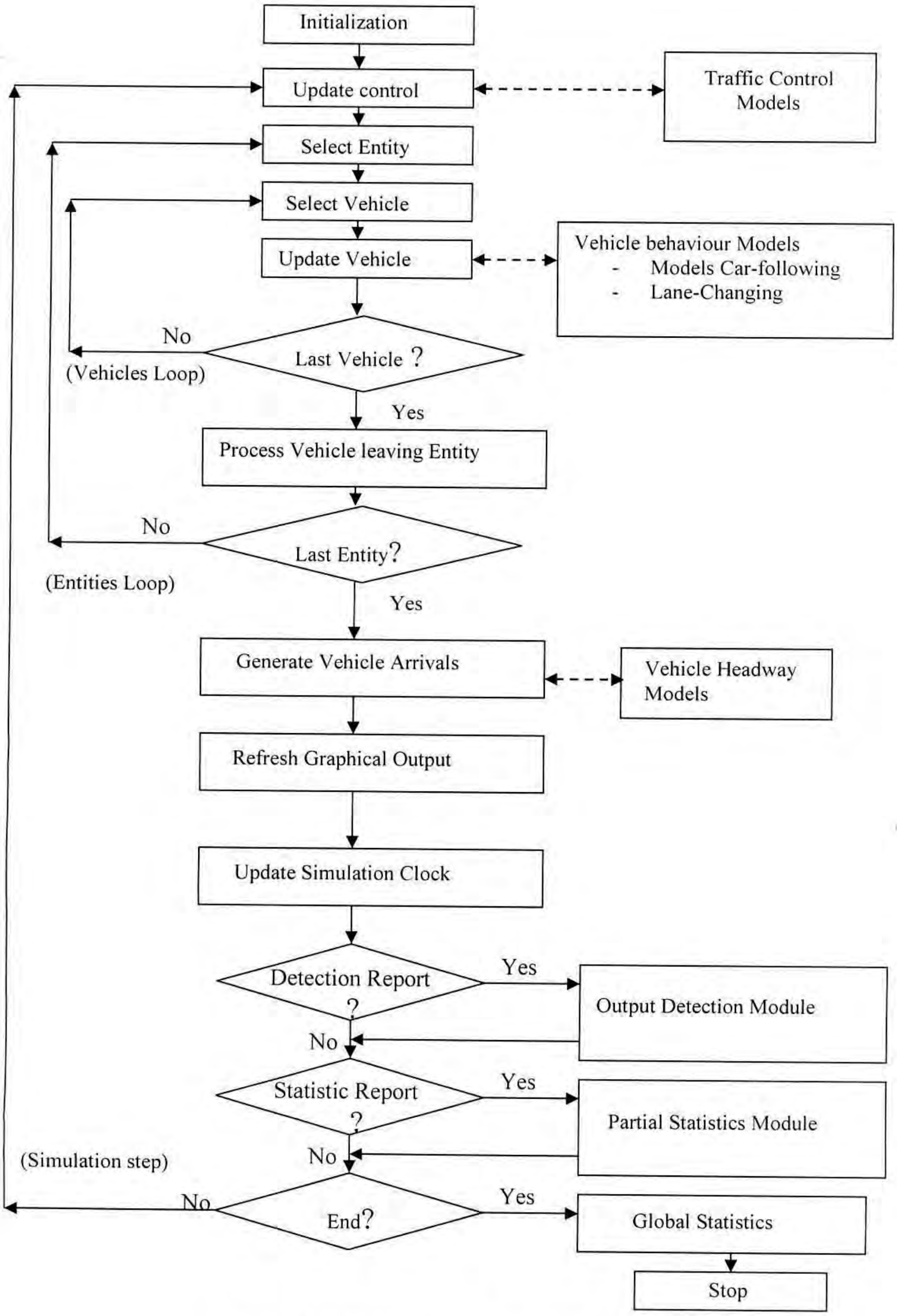

Figure 7.1 The Simulation Process in AIMSUN

7.3 Vehicle Modeling Parameters in AIMSUN

In AIMSUN vehicle maneuvers are modeled in detail using car following, lane changing and gap acceptance models. These vehicle behavior models are a function of several parameters that allow modeling of different types of vehicles: cars, buses, trucks, etc., and a variation of individual vehicles in each type. These parameters can be grouped into three categories according to the level at which they are defined: vehicle attributes, local section parameters and global network parameters. Setting appropriate values for these parameters is part of the model calibration process.

Vehicle Attributes

These parameters are defined at the level of vehicle type (e.g. car, bus, truck, etc.). It is possible to define not only the mean values for the attributes of each vehicle type, but also the deviation, minimum and maximum values. The particular characteristics for each vehicle are sampled from a truncated Normal distribution. The attributes that characterize a vehicle type are the following:

- Name
- Length
- Width
- Maximum desired speed
- Maximum acceleration
- Normal deceleration
- Maximum deceleration
- Speed acceptance
- Minimum distance between vehicle

Local Parameters

Certain parameters may affect vehicle behavior, although they are not defined at the level of vehicle type, but are related to sections. This means that these parameters are applied locally to the vehicles while they are driving along the section, but they change as the vehicle enters in a new section.

- Section Speed limit
- Lane Speed Limit
- Turning Speed
- Visibility Distance at Junctions
- Yellow Box Speed
- Section Slope

Global Modeling Parameters

This is a set of parameters related to vehicle behavior models that is valid throughout the whole network and which is defined neither at the level of vehicle type nor at the section level. They are used for all vehicles driving anywhere in the network in the network during the entire simulation experiment.

- Driver's reaction time
- Reaction time at stop
- Simulation step
- Queuing up speed
- Queue leaving speed

7.4 Model Development

Model development implies transforming the conceptual model into a site-specific model that can be run in the selected model code. A major task in model setup is the processing of data in order to prepare the input files necessary for executing the model.

Intersection Geometry

Graphic User Interface (TEDI) provides the graphical interface for entering intersection geometry which is shown in Figure 7.2. Possible turning movement (i.e. through, right turn, u-turn, left turn) for every approach, including details about the lanes from which each turning is allowed has been given in the intersection layout. Again speed limits of the target approach considering arterial road and turning speed for allowed turns has been given through section editor- main folder.

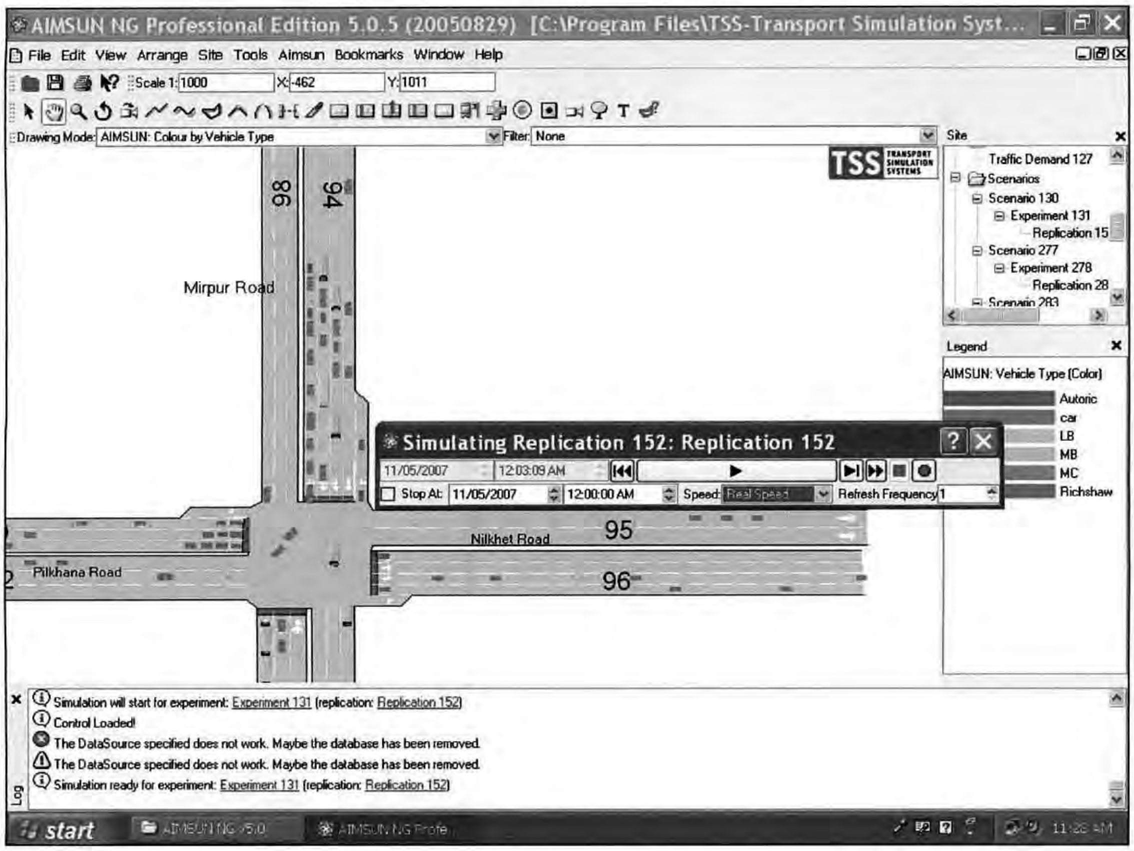

Figure 7.2 Screen Shot during a Simulation Run

Traffic Demand

AIMSUN allows two methods of traffic demand data input. One is by the traffic flows at the input sections and another is by an O/D matrix. For the present study traffic demand data has been entered by traffic flows on each approach. Five vehicle types such as large bus, mini bus, car, auto rickshaw, motor cycle have been considered. Flows at input section (200m upstream from the stop line) as a percentage of total flow for each vehicle type and subsequent turning percentages at each approach has been given as per video footage.

Traffic Control

For intersection control, a phase-based approach has been applied in which the cycle of the intersection is divided into four phases as observed from field, where each phase has a particular set of signal groups. Signal group consists of turning movements that was observed from field and are controlled by the same indication of traffic light. The units for defining the phases of the control plan are second.

Vehicle Characteristics

For each vehicle the traffic flows has been entered separately rather than as a percentage of the total flow. The amount of data to be entered therefore increases with the number of vehicle types. For the present model different vehicles have been grouped in to five classes: large bus, mini bus, car, auto rickshaw, motor cycle. Different vehicle type and their characteristics were entered into the model through TEDI.

7.5 Model Calibration

Calibration of the model ensures that the model simulations realistically replicate the existing traffic conditions. No standardized procedures exist that define when a model can be assumed to be calibrated. To calibrate the model different vehicle characteristics have been entered into the model through TEDI that was investigated by Hoque (1994) for Bangladesh traffic condition. Section parameter has been used as AIMSUN default value considering arterial road. Simulation step and driver's reaction time to react to speed changes in the preceding vehicle as global parameter has been considered as 1 sec which is used in car-following model (as per AASHTO driver's reaction time lies between 1-4 sec and the recommended value is 2.5 sec for rural highway). For the present study lower value has been considered due to the fact that drivers are usually more alert on urban road requiring less time to response. Again, driver's reaction time at stop has been also considered as 1 sec initially. Then considering exponential vehicle arrival as per Sutomo (1992) who studied non-lane based traffic behavior for Javanese (Indonesia) cities, the input section of the study approach has been feed by higher traffic demand and subsequently the discharge has been observed. At first the simulated flow was found more than the field observed capacity of the approach. To adjust the flow at every stage reaction time at stop has been increased by few seconds and subsequently flow has been observed. It was found that at 2 sec reaction time the model perfectly simulated the field observed capacity (767 veh/hr). According to AASHTO (1994) for signal controlled intersection the recommended value is 2.0. Then the input section was feed by different demand both at capacity and more than capacity as a trial. And all times it was found that model could simulate very close to the field observed capacity. Figure 7.3 is the output folder showing the flow that was observed at v/c ratio 1.0.

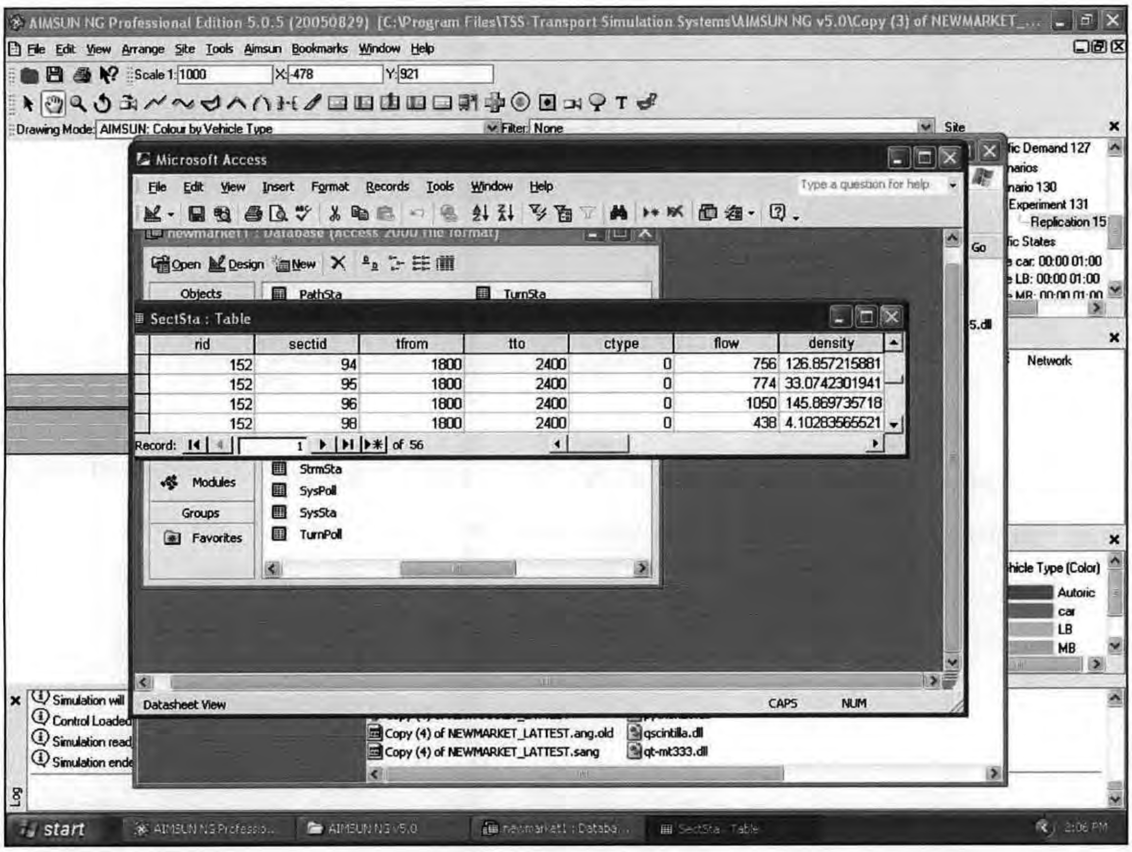

Figure 7.3 Simulated Flow at v/c=1.0

7.6 Model Validation

Validation establishes that the model behavior accurately and reliably represents the real world system being simulated, over the range of conditions anticipated (i.e., the model's "domain"). The validation procedure of AIMSUN model involved comparison of the simulated delay with field data, observed at north approach of New market intersection. Details of the data collection and field delay measurement are given in chapter 4. Table 7.1 shows both the observed simulated delay and field delay. A high coefficient of determination ($R^2 = 0.958$) indicating that the model is able to reproduce the observed traffic delay accurately.

7.7 Model Application

A validated simulation model is useful tools that will be enabling many aspects of a system to be studied for which road data are scarce. Current state-of-the-art analytical stop estimation models assume that all vehicles joining a queue during a red interval are able to cross the intersection before the next red interval. However, this assumption is not necessarily valid in heavy flow conditions, as vehicles may be forced to stop more than once before being able to cross an intersection. When such conditions exist, it becomes almost impossible to find general equations for estimating the number of vehicle stops on intersection approaches using traditional queuing analysis approaches, particularly when considering stochastic arrivals.

This section presents the results of a research effort that lead to the generation of a stop estimation model for over-saturated conditions through the establishment of an upper bound for the number of vehicle stops in oversaturated conditions using observed simulated data from the developed model. The section also briefly describes both the derivation of the equation for estimating the upper bound number of vehicle stops and the process that was utilized to generate an analytical model for estimating vehicle stops in over-saturated conditions using the upper bound stop estimates.

Stop Estimation by AIMSUN Model

To estimate the number of stops incurred by each vehicle the threshold value for queuing up speed has been considered as 1 meter/sec. That is a vehicle whose speed decreases the threshold value (1 m/s) has been considered to be stopped and consequently join a queue. Then the input section has been feed by different traffic demand for v/c 1.0 to 2.0 at an increment of .1, keeping the percentage of average number of different types of vehicle and their turning proportion fixed that was observed over the entire survey period. A simulation emulates the behavior of a complex system in which randomness is involved. Each simulation run produces one possible behavior of the system. To account stochasticity of the simulation results, given the same input data 3 replications has been considered and twenty runs were carried out for each of the different scenario. Figure 7.5 and 7.6 shows one example of tabular and graphical output of AIMSUN model for stop estimation at v/c ratio 1.8

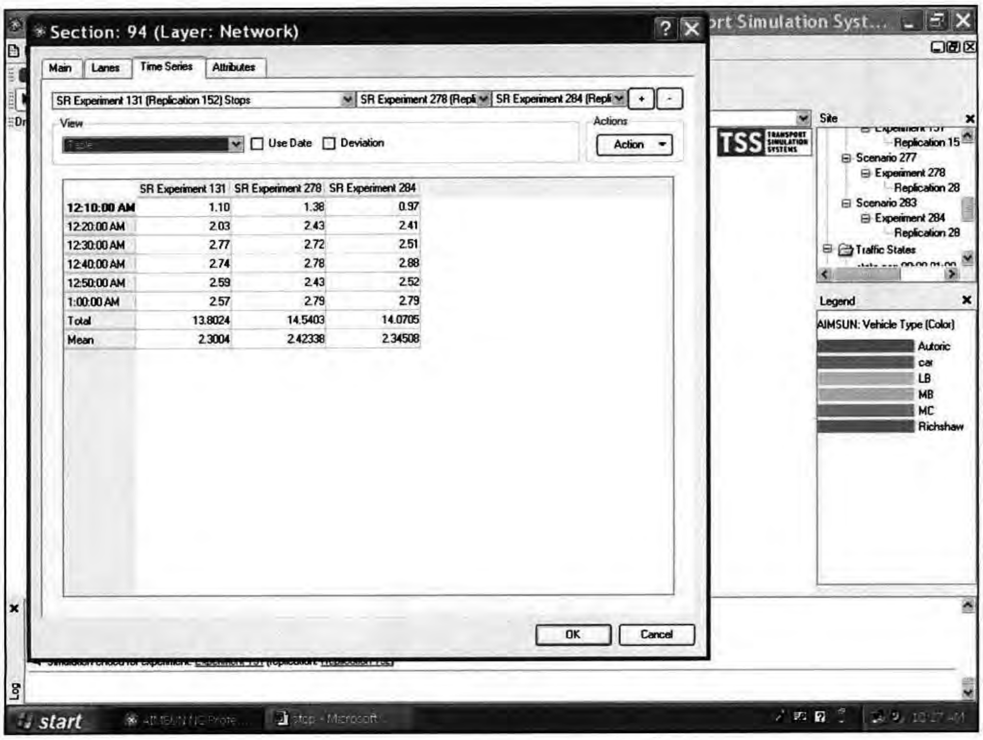

Figure 7.5 Tabular Output of Stop Estimation at v/c=1.8

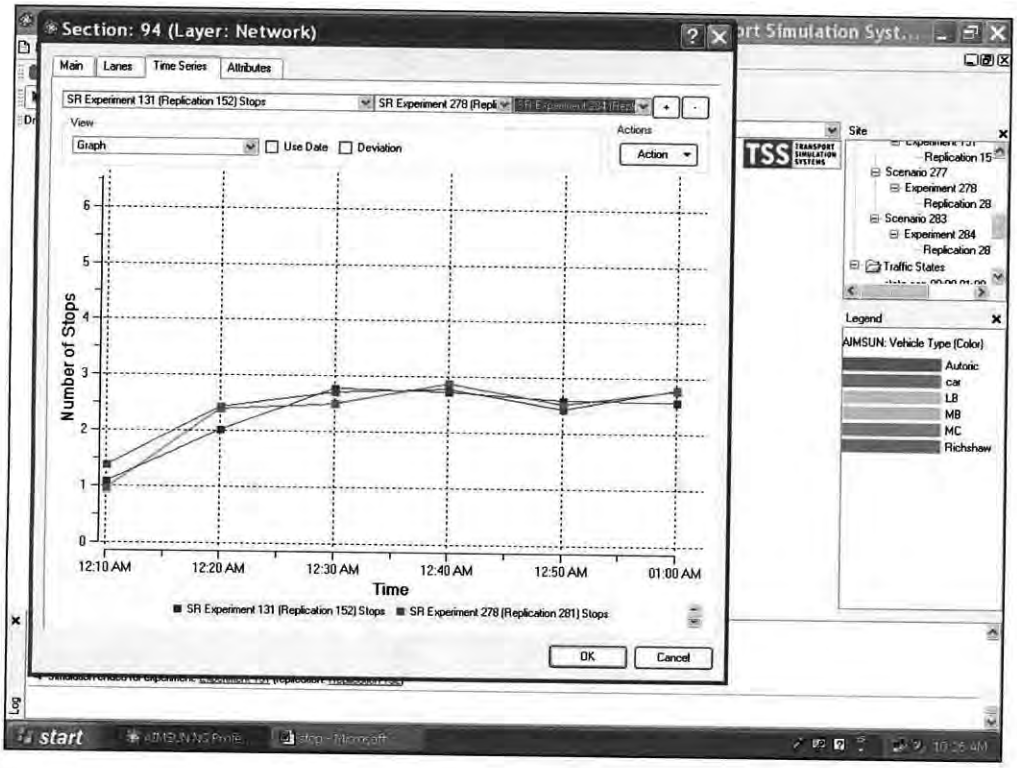

Figure 7.6 Graphical Output of Stop Estimation at v/c=1.8

Upper Bound Number of Stops

If vehicles were assumed to incur a single stop at an over-saturated signalized approach then the total number of stops would be equal to the product of the arrival rate and the analysis period ($q.t_e$). This assumption of a single stop per vehicle represents a lower bound for the number of vehicle stops. On the other end, an upper bound for vehicle stops can be derived by assuming that vehicles caught in a queue incur an additional full stop for each cycle that the vehicle is not served. The derivation of this upper bound for the number of vehicle stops at signalized intersections can then be computed using queue length estimation equations. Specifically, the maximum number of vehicle stops can be computed as the number of vehicle arrivals while a queue exists at the intersection plus the overflow of vehicles that are not served during previous cycles assuming that vehicles always incur a full stop.

To illustrate the calculations, Figure 7.7 shows the first two cycles of a traffic signal operation at an oversaturated signalized approach with uniform arrivals. In this diagram, the maximum number of stops for the second cycle is equal to the sum of all arrivals during the second cycle, plus the demand that did not discharge during the previous cycle. The volume that is not served in the first cycle is computed as the difference between the cumulative arrivals and departures. Similarly, the maximum number of stops for the third cycle is computed using the cumulative arrivals during the analysis period plus the number of vehicles that remain to be served at the end of the second cycle. By generalizing the calculation process, Equation 7.1 is finally derived for computing an upper bound for the maximum number of stops at over-saturated signalized approaches over a given evaluation period (t_e).

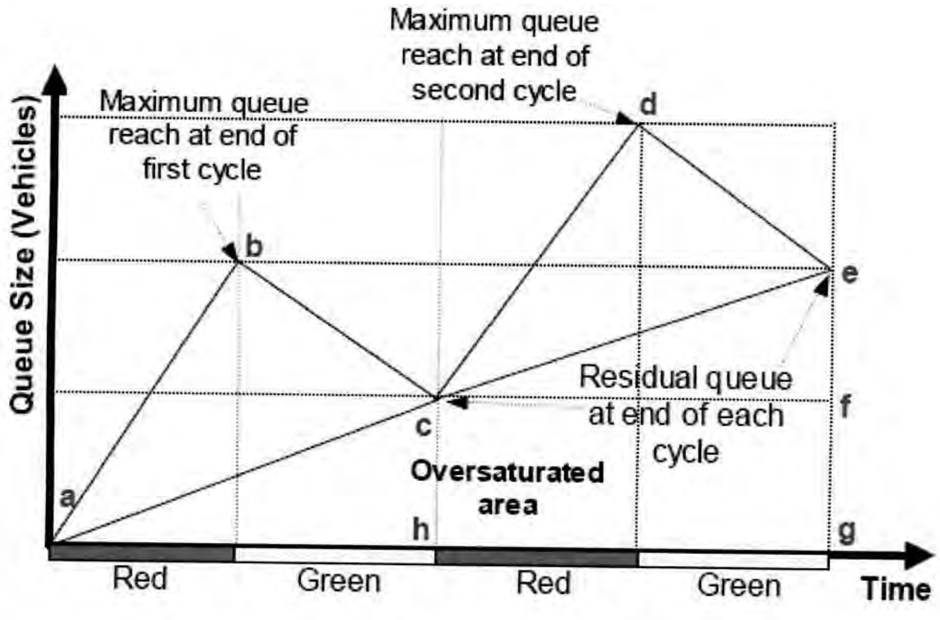

Figure 7.7 Queuing Diagram Analysis for Over-saturated Condition

$$N_{ub} = \frac{qt_e + \sum_{i=1}^{n-1} i.(vC - sg)}{q.t_e} \qquad (7.1)$$

Where:

N_{ub} = Upper bound average number of vehicle stops (stops/cycle),

v = Average arrival rate (vehicles/second),

s = Saturation flow rate (vehicles/second),

C = Cycle time (seconds),

g = Effective green time (seconds),

t_e = Evaluation period (seconds), and

n = Number of cycle lengths in analysis period (t_e/C).

7.8 Proposed Model for Over-saturated Conditions

Using stop estimates produced by the microscopic model, an adjustment factor as a function of the volume-to-capacity (v/c) ratio was developed to scale the upper bound vehicle stop estimates of Equation 7.1 to an actual estimate of stops. Specifically, the adjustment factor was developed using regression with the v/c ratio as the model's independent variable, as demonstrated in Equation 7.2.

Figure 7.8 illustrates the fit of the adjustment factor to the ratio of the simulated number of vehicle stops to the upper bound stop estimates (Equation 7.1) for v/c ratios that range from 1.0 to 2.0. The figure clearly demonstrates a good agreement between the adjustment factor regression curve and the estimated stop ratios as confirmed by the high coefficient of determination ($R^2 = 0.985$). The results also demonstrate that all the chosen variables: v/c ratio, squared v/c ratio and cubic v/c ratio are significant at a 95% confidence level. The figure further indicates that the adjustment model produces a correction factor that ranges from 0.83 at a v/c ratio of 1.0 to 2.37 at a v/c ratio of 2.0.

Using the adjustment factor model of Equation 7.2, the model of Equation 7.3 was finally proposed to compute the average number of vehicle stops at over-saturated signalized approaches.

$$\mathbf{AF = 8.98 - 18.394x + 12.945x^2 + 2.7x^3} \quad (7.2)$$

$$\mathbf{N_s = N_{ub} \ X \ AF} \quad (7.3)$$

Table 7.2 Stops Estimate by Analytical Approach and Micro-simulation Model

X=v/c	Nub (stops/veh)	Simulation output			
		Replication-1 (stops/veh)	Replication-2 (stops/veh)	Replication-3 (stops/veh)	Average (stops/veh)
1	1	0.787	0.838	0.785	0.803333
1.1	1.022	0.855	0.912	0.877	0.881333
1.2	1.041	1.017	0.906	0.892	0.938333
1.3	1.056	1.094	1.024	1.101	1.073
1.4	1.07	1.201	1.145	1.166	1.170667
1.5	1.081	1.619	1.704	1.26	1.527667
1.6	1.091	2.096	1.702	1.986	1.928
1.7	1.1	1.919	2.518	1.279	1.905333
1.8	1.108	2.3	2.423	2.345	2.356
1.9	1.115	2.427	2.529	2.509	2.488333
2.0	1.122	2.658	2.643	2.688	2.663

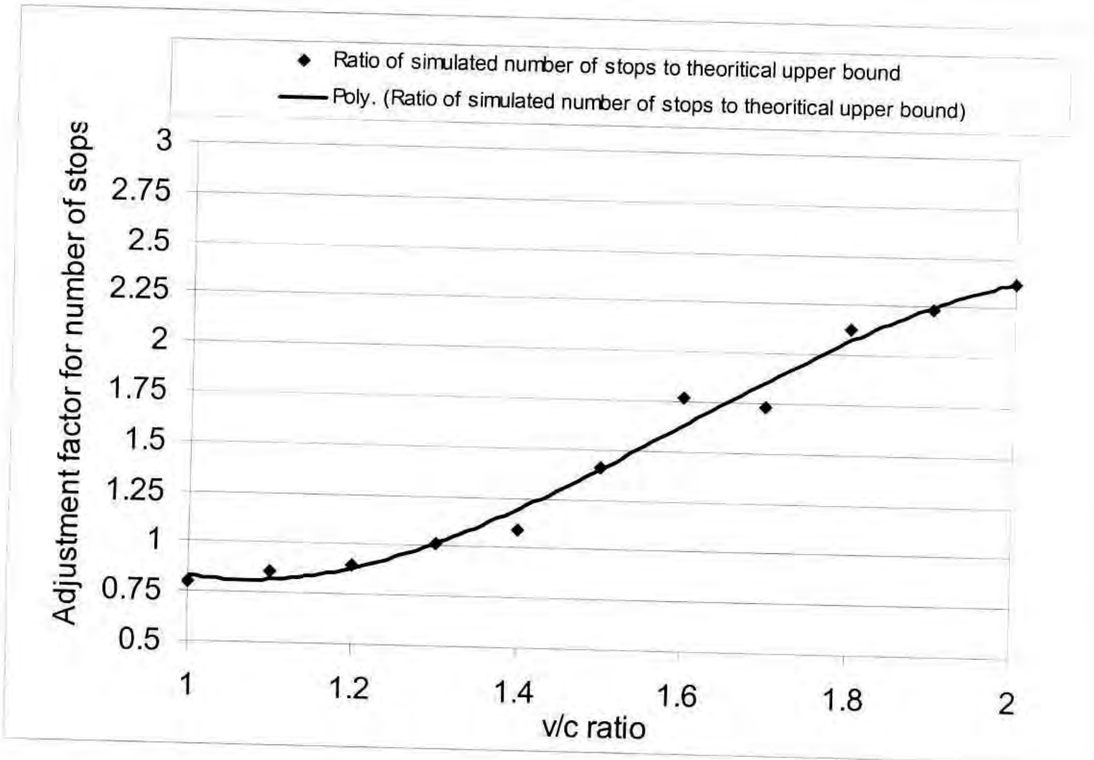

Figure 7.8 Comparison of Simulated Stop Ratio and Stop Adjustment Factor

COEFFICIENTS

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	8.980	3.428831639	2.619051	0.034457	0.872394	17.08818
X Variable 1	-18.394	7.177643244	-2.56269	0.037405	-35.3665	-1.42165
X Variable 2	12.945	4.886591613	2.649199	0.03298	1.39061	24.5005
X Variable 3	2.70	1.083983875	-2.49082	0.041547	-5.26322	-0.13679

MODEL SUMMARY

Regression Statistics	
Multiple R	0.993
R Square	0.985
Adjusted R^2	0.979
Standard Error	0.085
Observations	11

7.9 Summary

The measurement of the level of performance of signalized intersection has been an area of interest for traffic engineers since the birth of the profession. For many years, this interest has primarily focused solely on vehicle delay. Besides delay, other performance measures such as the numbers of vehicle stops and the spatial extent of queues on intersection approaches have also been found to play an important in the evaluation of signalized intersections. This chapter introduces two approaches for computing vehicle stops at signalized intersection. And finally an analytical equation has been proposed to compute stops that will play an important role in determining fuel consumption and emissions on intersection approach.

Chapter 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

The lane group capacity is the maximum hourly rate at which vehicles can reasonably be expected to pass through the intersections under prevailing traffic, roadway, and signal conditions. In HCM 2000 level of service for signalized intersection is defined in terms of control delay, which is a measure of driver discomfort, frustration, fuel consumption, and increased travel time. HCM have defined six level of service for each type facility. Saturation flow, delay, and signal timings are the important parameter in the analysis of signalized intersection.

Traffic flow in developing countries consists of different types of vehicles. Vehicles do not follow any lane discipline. To study behavior of different types of traffic and their effect on the design and capacity analysis of signalized intersection, field data were collected at five selected intersections in Dhaka city. Video camera was used for data collection and data retrieval was done in laboratory on large screen. The non-uniformity in the static and dynamic characteristics of the vehicles is normally taken into account by converting all vehicles in terms of common unit. In the present study a range of site-specific PCU values were obtained using synchronous multiple linear regression. The saturation flow for each study approach was calculated using the average PCU values and multiple linear regression techniques were then used to derive predictive saturation flow models. The saturation flow values for selected approaches were regressed against several intersection characteristics. This model shows good correlation with field observed saturation flow and can be used to estimate saturation flow in non-lane based traffic condition.

There are number of theoretical models including HCM, available for the estimation of delay at intersection. But all these procedures are best suited for lane disciplined and more or less uniform traffic flow. Hence all these procedures cannot be applied directly in Bangladesh. Field delay was measured at selected intersections and compared with theoretical delay models. New delay model for non-lane based traffic condition is developed based on HCM 2000 model.

Micro simulation technique has proven to be an important development in a variety of problems solving areas. At the same time simulation technique has lots of scope in the field of transportation. In the present study AIMSUN model is developed to estimate number of stops at oversaturated condition. Calibration and validation of the model was carried out based on field observed flow and delay. The study also proposes an analytical model for estimating vehicle stops at intersections for oversaturated condition. The model uses the signal timings, approach arrival rate, approach saturation flow rate, and analysis period as independent variables for estimating vehicle stops. Such a model is ideal for estimating stops in the field when data are not available on vehicle speed profiles.

8.2 Conclusions

Following are the important conclusions that are drawn from the present study.

1. Video recorded data collection is much more superior to manual data collection. It requires very less manpower. It produces permanent, complete record of the traffic scene, but data extraction is tedious.

2. HCM 2000 suggests measurement of saturation flow should start after 10 second of green initiation which is considered as start up lost time. From the present study, it was found that auto rickshaws and motor cycle find way in between heavy vehicles and try to come near stop line. Most of the times these vehicles cross stop line before green starts. During red period large number of vehicles accumulates near stop line. This causes to discharge large amount of traffic during initial 10 seconds. Following ROAD NOTE 34, it was observed that for all the selected intersection start up lost time varies between 2-4 second. And hence it is

suggested that count for measurement of saturation flow should start after 3 seconds of green initiation for non lane based traffic condition.

3. Unified Passenger car unit (PCU) concept does not hold good for non-lane based traffic situation. From the present study, it was found that PCU value for particular types of vehicle does not remain constant for all the selected intersection approaches. This indicates that analysis should be site specific for non lane based traffic condition. Hence, PCU values those have been measured from the present study can be used for similar traffic characteristics.

4. Regression model developed to estimate saturation flow shows good correlation with field values. This model can used to estimate saturation flow at any intersections knowing signal timing, % right turn, and approach width and exit width. This model may not be suitable for extreme range of input variable.

5. Delay estimate by HCM 2000 model gives higher estimation of field delay. Other delay model such as Ackelek's, Reilly's etc do not take into effect of signal coordination and uncoordinated nearby signals. From the analysis it is recommended that the theoretical incremental delay (due to random arrival and over saturated queues) in HCM 2000 delay model be reduced by 70 % to better reflect field conditions.

6. HCM 2000 defines LOS F, if control delay is more than 80 seconds per vehicle. LOS F is designated for breakdown traffic flow. In this study, it was found that many times operational conditions were stable even though delay was more than 80 sec/veh. Chandra et al (1996) suggested degree of saturation (DOS) and proportion of vehicles required to stop as criteria to define LOS for non-lane based traffic condition. The above parameters do not take into account driver's perceptions in deciding LOS. Therefore, it is suggested to consider control delay as LOS criteria but increase higher limit of control delay to about 100 sec/veh and redefine LOS delay range.

7. Besides delay, the number of vehicle stops on intersection approach must be considered in the performance evaluation of signalized intersection. This measure not only relates to the level of service that is provided to the drivers, but also to the level of fuel consumption and air pollution that is generated by the vehicles traversing the signalized intersection.

8.3 Limitations of this study

1. Though the present research aimed at optimization of traffic signals of Dhaka city but the analysis does not consider NMV, which consist of significant proportion of vehicle composition prevailing in Dhaka metropolitan.

2. Saturation flow model and recommended modification of delay have been developed on only eight intersection approaches. Hence, the data point consider for these analysis were very small.

3. Saturation flow model that has been developed in this study for non lane based traffic condition was developed based on database where right turning proportion was very small. Hence, the effect of right turn that has been reflected in this model may not be accurate.

4. Impact of intersection layout configuration on saturation flow has not considered in this study.

5. The variation of saturation flow and degree of saturation with time and day was not considered in the present study.

6. Most of the intersections those were considered have very low pedestrian concentration. Saturation flow gets affected by pedestrian movement and also by parking activities near intersection. In order to incorporate these issues it was required to select the intersection more carefully.

7. Finally, simulation approach that was adopted to estimate number of stops by AIMSUN micro simulator basically based on lane based traffic condition where lane changing takes place from its own lane to the adjacent lane completely in one operation not in between the lanes or partially. Hence, lane changing decision is very sensitive to the gap acceptance criteria. But in non lane based operation, lane changing and overtaking is gradual and continuous process. For that case it is not as sensitive as the case with the lane based operation. So, the stop those were measured by this software may not be correct.

8.4 Future Scope

1. The regression model developed for saturation flow is based on traffic condition of Dhaka city, which is assumed to be similar to other parts of Bangladesh. This developed model may be applied in other cities of Bangladesh and checked for its usefulness.

2. Saturation flow depends on various factors. In present study all intersections were selected having almost flat surface. Saturation flow also gets affected by parking facility near intersection. All these factors needs to studied and develop new model taking into account maximum possible variables.

3. Recommended modification to the HCM 2000 delay model must be verified by applying it in other cities. In present study analysis has been carried out for five intersection approaches only. Similar analysis should be carried out for large number of intersection approaches.

4. LOS defined in HCM 2000 is based on perception of drivers in North America. Frustration level of drivers in Bangladesh may be different than that in North America. And hence LOS might be different for same amount of delay. Survey must be conducted to know the drivers perception for the delay and new criteria defined.

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APPEENDIX A-1

WORK SHEET FOR SATURATION FLOW DATA COLLECTION

INTERSECTION SATURATION FLOW DATA WIRKSHEET

Day & Date:
Time :
Weather :

Intersection Location :
Cycle Time :
Name of Approach :
Phase and Phase Time :

	No. of vehicle crossing the stop line during saturation period																				
Saturation Period (Seconds)	Large Bus			Mini Bus			Micro			Car			Auto Rickshaw			Utility			Two-Wheeler		
	L	T	R	L	T	R	L	T	R	L	T	R	L	T	R	L	T	R	L	T	R

APPEENDIX A-2

WORKSHEET FOR FIELD MEASUREMENT OF DELAY

Intersection Control Delay Worksheet											
Analyst: Agency or Company: Date Performed: Analysis Time Period:						Intersection: Area Type: Analysis Year:					
Input Initial Parameters											
Number of lanes, L: Free-flow speed, FFS(Km/hr): Survey count interval(s):							Total vehicle arriving, V_{Tot} = Stopped vehicle count, V_{stop} =				
Input Field Data											
Clock Time	Cycle Number	Count interval									
		1	2	3	4	5	6	7	8	9	10
Total											
Computations											
Total vehicles in queue, $\sum V_{iq}$ =							No of cycles survryed, N_c=				
Time-in-queue per vehicle,d_{vq} = (I x $\sum V_{iq} / V_{Tot}$) x .9							Fraction of vehicle stopping,FVS= $\frac{V_{stop}}{V_{Tot}}$				
Accel/Decel correction factor,CF=							Accel/ Decel delay,d_{ad} = FVS X CF				
No. of vehicles stopping per lane each Cycle= $\frac{V_{stop}}{N_c x L}$							Control delay = $d_{ad} + d_{vq}$				

Source: HCM 2000

Sample calculation for delay

Intersection Control Delay Worksheet	
Analyst: Md. Hadiuzaman Agency or Company: BUET Date Performed: 19/04/2007 Analysis Time Period:15 min	Intersection: Science Lab Area Type: CBD Analysis Year: 2007
Input Initial Parameters	
Number of lanes, L: 2 Free-flow speed, FFS(Km/hr): 34.85 Km/hr Survey count interval (s): 20	Total vehicle arriving, V_{Tot} = 316 Stopped vehicle count, V_{stop} = 158

Input Field Data

Clock Time	Cycle Number	Count interval									
		1	2	3	4	5	6	7	8	9	10
12:48	1	12	20	24	15	6	4	3	0	0	
	2	7	15	23	19	14	2	2	0	0	
	3	3	7	11	15	8	2	0	0	0	
	4	25	37	42	28	18	18	12	4	0	
	5	18	17	32	23	0	5	3	0	0	
	6	7	20	22	23*	21	11	0**			
Total		72	126	154	123	67	42	20	4	0	

Computations

* End of Analysis period (15 min)

** End of Survey period (16 min)

Total vehicles in queue, $\sum V_{iq} = 608$

No of cycles survryed, $N_c = 5.8$

Time-in-queue per vehicle, $d_{vq} = (I \times \sum V_{iq} / V_{Tot}) \times .9$

$= 34.66$ sec/veh

Fraction of vehicle stopping, $FVS = \dfrac{V_{stop}}{V_{Tot}}$

$= 0.5$

Accel/Decel correction factor, CF= +2

Accel/ Decel delay, d_{ad} = FVS X CF

= 1 sec/veh

No. of vehicles stopping per lane each Cycle= $\dfrac{V_{stop}}{N_c x L}$

= 13.62

Control delay = $d_{ad} + d_{vq}$

= 35.66 sec/veh

APPEENDIX A-3

Database used for the Development of Regression Model for Saturation Flow

Saturation Flow (PCU/hr)	sigset	WIDTH	STR-ON W	%Right turn
4936	100	10.6	10.34	0
5125	100	10.6	10.34	0
5171	100	10.6	10.34	0
5102	100	10.6	10.34	0
4521	100	10.6	10.34	0
4998	100	10.6	10.34	0
5071	100	10.6	10.34	0
4746	100	10.6	10.34	0
4745	100	10.6	10.34	0
4699	100	10.6	10.34	0
4733	100	10.6	10.34	0
4885	100	10.6	10.34	0
5307	100	10.6	10.34	0
4257	100	10.6	10.34	0
4438	100	10.6	10.34	0
4249	100	9.23	10.51	0
4796	100	9.23	10.51	0
4071	100	9.23	10.51	0
4082	100	9.23	10.51	0
4100	100	9.23	10.51	0
3826	100	9.23	10.51	0
4160	100	9.23	10.51	0
4041	100	9.23	10.51	0
3774	100	9.23	10.51	0
3915	100	9.23	10.51	0
4079	100	9.23	10.51	0
3969	100	9.23	10.51	0
3083	32	11.57	8.9	8.39
3503	32	11.57	8.9	8.39
3994	32	11.57	8.9	8.39
3593	32	11.57	8.9	8.39
3043	32	11.57	8.9	8.39
4056	32	11.57	8.9	8.39
3364	32	11.57	8.9	8.39
3497	32	11.57	8.9	8.39
3122	32	11.57	8.9	8.39
3147	32	11.57	8.9	8.39
3096	32	11.57	8.9	8.39
2964	32	11.57	8.9	8.39
2616	32	11.57	8.9	8.39
3755	32	11.57	8.9	8.39

Saturation Flow (PCU/hr)	sigset	WIDTH	STR-ON W	%Right turn
3181	32	11.57	8.9	8.39
3678	47	11.9	10.57	47.59
4152	47	11.9	10.57	47.59
3740	47	11.9	10.57	47.59
3913	47	11.9	10.57	47.59
3943	47	11.9	10.57	47.59
3757	47	11.9	10.57	47.59
3908	47	11.9	10.57	47.59
3467	47	11.9	10.57	47.59
4224	47	11.9	10.57	47.59
4098	47	11.9	10.57	47.59
3568	47	11.9	10.57	47.59
3560	47	11.9	10.57	47.59
3633	47	11.9	10.57	47.59
3634	47	11.9	10.57	47.59
2704	79	6.78	7.79	0.00
2522	79	6.78	7.79	0.00
2818	79	6.78	7.79	0.00
2531	79	6.78	7.79	0.00
2947	79	6.78	7.79	0.00
2012	79	6.78	7.79	0.00
3628	79	6.78	7.79	0.00
2357	79	6.78	7.79	0.00
2617	79	6.78	7.79	0.00
2678	79	6.78	7.79	0.00
2424	79	6.78	7.79	0.00
2866	79	6.78	7.79	0.00
2741	79	6.78	7.79	0.00
3313	79	6.78	7.79	0.00
2843	79	6.78	7.79	0.00
3024	79	6.78	7.79	0.00
2530	79	6.78	7.79	0.00
2774	79	6.78	7.79	0.00
3281	79	6.78	7.79	0.00
3265	79	6.78	7.79	0.00
3060	79	6.78	7.79	0.00
2610	79	6.78	7.79	0.00
3424	79	6.78	7.79	0.00
3076	79	6.78	7.79	0.00
2627	79	6.78	7.79	0.00
3362	79	6.78	7.79	0.00
3141	79	6.78	7.79	0.00
3405	27	12.97	11.74	7.38
3482	27	12.97	11.74	7.38
2786	27	12.97	11.74	7.38

Saturation Flow (PCU/hr)	sigset	WIDTH	STR-ON W	%Right turn
3271	27	12.97	11.74	7.38
3485	27	12.97	11.74	7.38
3764	27	12.97	11.74	7.38
3651	27	12.97	11.74	7.38
3333	27	12.97	11.74	7.38
3570	27	12.97	11.74	7.38
3478	47	7.8	10.79	100
3640	47	7.8	10.79	100
3074	47	7.8	10.79	100
3599	47	7.8	10.79	100
3093	47	7.8	10.79	100
2726	47	7.8	10.79	100
4007	47	7.8	10.79	100
3591	47	7.8	10.79	100
2744	47	7.8	10.79	100
2960	47	7.8	10.79	100
3555	47	7.8	10.79	100
3506	47	7.8	10.79	100
3143	47	7.8	10.79	100
2993	47	7.8	10.79	100
3821	68	12.68	14.17	100
4303	68	12.68	14.17	100
3343	68	12.68	14.17	100
4471	68	12.68	14.17	100
3488	68	12.68	14.17	100
3695	68	12.68	14.17	100
4241	68	12.68	14.17	100

APPEENDIX A-4

Database used for the Determining PCU values

Bangla motor North Approach

No. of Cycle Surveyed	Saturation Period (sec)	Large bus	Mini bus	Micro bus	Car	Auto rickshaw	Utility	Motor cycle
1	105	5	14	7	50	61	11	10
2	102	3	12	6	52	70	11	20
3	94	3	8	13	47	74	12	5
4	103	6	11	9	48	73	10	9
5	99	2	7	11	50	48	16	10
6	100	1	11	7	56	79	5	15
7	102	2	14	11	53	54	16	11
8	103	4	18	6	44	53	10	9
9	99	1	3	13	56	71	12	11
10	102	7	14	2	44	56	8	9
11	102	4	9	6	49	63	14	10
12	99	4	12	10	41	72	6	15
13	104	4	6	13	61	79	12	10
14	97	3	11	3	41	43	17	6
15	60	3	10	4	21	25	8	3

Bangla motor south Approach

No. of Cycle Surveyed	Saturation Period (sec)	Large Bus	Medium Bus	Micro bus	Car	Auto rickshaw	Utility	Motor cycle
1	89	2	10	10	35	62	10	8
2	97	4	16	3	37	63	16	6
3	96	5	18	4	38	25	7	7
4	90	2	19	6	29	42	8	6
5	98	1	16	6	40	60	8	1
6	90	3	13	6	32	46	6	3
7	98	2	16	6	41	50	7	9
8	98	2	17	3	36	48	7	11
9	94	2	17	3	29	46	8	2
10	98	2	17	7	33	50	7	8
11	88	7	11	6	29	32	11	6
12	98	3	18	4	40	37	3	9

Newmarket South Approach

No. of Cycle Surveyed	Saturation Period (sec)	Large bus	Mini bus	Micro bus	Car	Auto rickshaw	Utility	Motor cycle
1	32	2	5	1	6	7	2	0
2	33	4	7	2	3	2	1	2
3	37	6	7	1	5	5	2	1
4	32	2	9	3	3	5	1	1
5	27	2	5	1	6	2	0	0
6	32	2	10	0	7	5	2	1
7	29	3	4	0	6	8	1	0
8	35	3	8	3	5	4	1	0
9	33	2	6	2	5	7	1	1
10	32	4	6	0	5	0	0	1
11	34	2	9	0	3	5	1	1
12	33	1	4	1	12	6	0	2
13	33	1	8	0	3	3	1	2
14	29	3	8	0	5	2	0	2
15	28	3	4	2	3	2	1	4

Newmarket North Approach

No. of Cycle Surveyed	Saturation Period (sec)	Large bus	Mini bus	Micro bus	Car	Auto rickshaw	Utility	Motor cycle
1	44	2	5	5	13	8	8	1
2	50	4	7	7	12	9	10	1
3	48	1	5	8	17	13	6	2
4	45	3	7	2	16	12	3	1
5	49	3	6	2	16	18	6	2
6	45	1	5	6	23	6	2	3
7	43	3	3	6	15	11	6	1
8	49	4	1	8	19	8	2	3
9	52	3	9	1	18	10	11	2
10	51	4	8	1	17	7	10	1
11	48	2	6	7	19	5	3	0
12	47	2	5	1	20	13	2	4
13	45	4	4	1	16	12	0	7
14	45	2	6	3	18	5	4	3

Panthapath East Apprpoach

No. of Cycle Surveyed	Saturation Period (sec)	Large bus	Mini bus	Micro bus	Car	Auto rickshaw	Utility	Motor cycle
1	43	0	0	5	25	13	9	1
2	37	0	0	8	27	15	1	1
3	36	0	2	6	16	12	2	0
4	36	1	0	5	19	17	2	2
5	49	0	0	7	36	22	0	1
6	37	0	1	0	26	19	1	6
7	59	1	4	6	31	34	1	1
8	38	0	0	2	28	10	2	2
9	39	0	0	5	29	11	4	5

Science Lab North Approach

No. of Cycle Surveyed	Saturation Period (sec)	Large bus	Mini bus	Micro bus	Car	Auto rickshaw	Utility	Motor cycle
1	81	1	10	9	16	13	6	6
2	79	1	6	2	23	18	3	9
3	76	1	6	4	28	10	7	4
4	82	1	8	4	25	10	4	5
5	79	2	5	3	30	12	8	6
6	82	2	6	0	15	12	6	4
7	73	2	2	7	32	26	8	9
8	75	1	0	5	24	16	8	3
9	74	1	9	2	22	14	2	3
10	79	2	6	3	22	22	4	4
11	79	0	5	2	29	18	1	6
12	84	2	5	3	26	22	9	6
13	75	1	5	2	28	19	4	3
14	80	2	8	2	34	27	2	3
15	83	2	5	5	28	15	7	7
16	75	2	3	8	30	11	3	12
17	84	3	1	5	30	9	6	7
18	83	0	11	3	26	17	3	7
19	75	3	4	1	26	31	8	3
20	83	3	9	5	31	16	4	5
21	85	2	4	3	20	35	15	7
22	83	2	3	4	25	23	4	9
23	76	0	10	3	27	27	6	9
24	81	1	10	6	26	17	4	8
25	76	1	2	3	33	15	4	3
26	84	2	9	4	34	15	10	2
27	83	3	7	1	28	15	10	9
28	79	3	8	5	15	8	8	6

Science Lab East Approach

No. of Cycle Surveyed	Saturation Period (sec)	Large bus	Mini bus	Micro bus	Car	Auto rickshaw	Utility	Motor cycle
1	46	5	0	0	21	4	5	4
2	44	4	0	7	18	9	3	2
3	46	1	0	3	26	11	1	2
4	63	3	0	9	31	16	3	8
5	42	2	0	5	16	16	1	1
6	48	4	1	1	16	5	3	1
7	40	1	1	5	22	18	3	2
8	65	4	2	6	27	18	3	10
9	51	4	0	3	18	8	1	3
10	48	2	0	5	16	13	6	1
11	66	6	2	6	26	17	3	2
12	38	1	0	6	20	13	0	4
13	44	3	1	2	19	8	3	0
14	45	4	0	3	13	11	2	5

Sheraton East Approach

No. of Cycle Surveyed	Saturation Period (sec)	Large bus	Mini bus	Micro bus	Car	Auto rickshaw	Utility	Motor cycle
1	60	0	0	6	28	44	6	3
2	64	0	0	7	38	41	9	4
3	46	0	0	2	19	26	7	3
4	66	0	0	5	35	57	9	8
5	53	0	0	2	26	32	5	4
6	64	0	0	9	37	32	2	4
7	65	0	0	4	35	55	6	7

APPEENDIX A-5
CALCULATION OF DELAY

Calculation of Delay following HCM 2000 delay model

Intersection	C(s)	g(sec)	T (h)	g/C	v veh/hr	s veh/hr	c=s(g/C)	X=v/c	Min(X,1)	Control delay (d) by HCM 2000 (sec/veh)						
										d1	d2	Rp	P=(Rp*g/c)	fpa	PF	d
New Market North Approach	219	47	0.294	0.21	940	3575	767	1.226	1.000	86.00	130.98	1	0.215	1	1	216.98
	219	47	0.3	0.21	1120	3575	767	1.460	1.000	86.51	255.52	1	0.210	1	1	342.03
	219	47	0.294	0.21	1152	3575	767	1.501	1.000	86.51	272.24	1	0.210	1	1	358.74
	219	47	0.294	0.21	1200	3575	767	1.564	1.000	86.51	304.87	1	0.210	1	1	391.37
	219	47	0.294	0.21	1160	3575	767	1.512	1.000	86.51	277.67	1	0.210	1	1	364.17
	219	47	0.294	0.21	1280	3575	767	1.668	1.000	86.51	359.44	1	0.210	1	1	445.94
Science Lab North Approach	167	107	0.261	0.641	1296	3029	1941	0.668	0.668	18.84	1.84	1	0.641	1	1	20.68
	167	107	0.267	0.641	1264	3029	1941	0.651	0.651	18.50	1.71	1	0.641	1	1	20.21
	167	107	0.261	0.641	1248	3029	1941	0.643	0.643	18.33	1.65	1	0.641	1	1	19.99
	167	107	0.261	0.641	1304	3029	1941	0.672	0.672	18.93	1.88	1	0.641	1	1	20.80
	167	107	0.267	0.641	1364	3029	1941	0.703	0.703	19.61	2.16	1	0.641	1	1	21.77
	167	107	0.267	0.641	1348	3029	1941	0.695	0.695	19.42	2.08	1	0.641	1	1	21.50
Science Lab East Approach	127	47	0.272	0.37	1104	3413	1263	0.874	0.874	37.24	8.67	1.333	0.493	1.15	0.925	43.12
	127	47	0.272	0.37	1048	3413	1263	0.83	0.830	36.36	6.45	1.333	0.493	1.15	0.925	40.08
	127	47	0.272	0.37	1052	3413	1263	0.833	0.833	36.42	6.57	1.333	0.493	1.15	0.925	40.27
Panthapath North Approach	190	47	0.261	0.247	988	4734	1171	0.844	0.844	68.01	7.53	1.333	0.330	1.15	1.0241	77.17
	190	47	0.261	0.247	1000	4734	1171	0.854	0.854	68.22	8.04	1.333	0.330	1.15	1.0241	77.92
	190	47	0.267	0.247	1224	4734	1171	1.045	1.000	71.50	40.67	1.333	0.330	1.15	1.0241	113.89
	190	47	0.261	0.247	1164	4734	1171	0.994	0.994	71.36	25.42	1.333	0.330	1.15	1.0241	98.50
	190	47	0.256	0.247	1116	4734	1171	0.953	0.953	70.41	17.31	1.333	0.330	1.15	1.0241	89.42
Sheraton East Approach	158	68	0.256	0.43	1540	5257	2263	0.681	0.681	36.26	1.68	0.667	0.287	1	1.2516	47.06

Calculation of theoretical and field delay

Intersection	C(s)	g(sec)	T (h)	g/C	v veh/hr	s veh/hr	c=s(g/C)	X=v/c	Min(X,1)	V_0/c	Akcelik Model	Reilly's Model	TRANSYT 6 Model	Webster Model	Field Observed Delay		
															Time-in-queue	Acc/Dec Delay	Cont. Delay
New Market North Approach	219	47	0.294	0.21	940	3575	767	1.226	1.000	0.75	224.41	158.02	222.43	_	107.23	3.57	110.81
	219	47	0.3	0.21	1120	3575	767	1.460	1.000	0.75	358.26	228.89	355.04	_	117.00	3.79	120.79
	219	47	0.294	0.21	1152	3575	767	1.501	1.000	0.75	376.41	238.62	373.07	_	121.56	3.65	125.22
	219	47	0.294	0.21	1200	3575	767	1.564	1.000	0.75	411.22	257.05	407.73	_	133.98	3.73	137.71
	219	47	0.294	0.21	1160	3575	767	1.512	1.000	0.75	382.20	241.68	378.83	_	122.65	3.77	126.41
	219	47	0.294	0.21	1280	3575	767	1.668	1.000	0.75	469.57	288.01	465.89	_	148.67	1.95	150.62
Science Lab North Approach	167	107	0.261	0.641	1296	3029	1941	0.668	0.668	0.82	18.84	18.84	20.68	18.75	27.67	1.01	28.67
	167	107	0.267	0.641	1264	3029	1941	0.651	0.651	0.82	18.50	18.50	20.21	18.49	33.66	2.00	35.66
	167	107	0.261	0.641	1248	3029	1941	0.643	0.643	0.82	18.33	18.33	19.99	18.36	28.62	2.04	30.65
	167	107	0.261	0.641	1304	3029	1941	0.672	0.672	0.82	18.93	18.93	20.80	18.82	39.09	2.21	41.30
	167	107	0.267	0.641	1364	3029	1941	0.703	0.703	0.82	19.61	19.61	21.77	19.34	37.85	1.98	39.83
	167	107	0.267	0.641	1348	3029	1941	0.695	0.695	0.82	19.42	19.42	21.50	19.20	40.17	2.22	42.39
Science Lab East Approach	127	47	0.272	0.37	1104	3413	1263	0.874	0.874	0.74	41.37	39.31	45.92	37.96	41.67	2.81	44.49
	127	47	0.272	0.37	1048	3413	1263	0.83	0.830	0.74	38.46	37.41	42.81	35.54	49.53	2.85	52.39
	127	47	0.272	0.37	1052	3413	1263	0.833	0.833	0.74	38.63	37.53	43.00	35.66	54.89	3.12	58.01
Panthapath North Approach	190	47	0.261	0.247	988	4734	1171	0.844	0.844	0.77	70.04	69.02	75.53	66.97	62.16	1.51	63.68
	190	47	0.261	0.247	1000	4734	1171	0.854	0.854	0.77	70.69	69.46	76.27	67.59	74.66	1.62	76.28
	190	47	0.267	0.247	1224	4734	1171	1.045	1.000	0.77	110.27	91.42	113.25	-	93.35	1.80	95.16
	190	47	0.261	0.247	1164	4734	1171	0.994	0.994	0.77	91.87	81.62	96.78	-	85.30	1.76	87.06
	190	47	0.256	0.247	1116	4734	1171	0.953	0.953	0.77	81.93	76.17	87.72	88.82	89.03	1.77	90.80
Sheraton East Approach	158	68	0.256	0.43	1540	5257	2263	0.681	0.681	0.84	35.09	35.67	37.93	34.91	46.57	1.28	47.85

APPEENDIX A-6

VEHICULAR CHARACTERISTICS USED FOR SIMULATION

Length

Vehicle type	Sample size	Mean(m)	Std.Dev	Minimum(m)	Maximum(m)
Motorcycle	17	1.77	0.042	1.68	1.85
Autorickshaw	20	2.56	0.083	2.38	2.81
Car	73	3.75	0.51	2.74	4.77
Mini-bus	53	5.62	0.37	4.57	6.73
Large bus	51	9.02	1.082	7.01	11.03

(Source: Hoque 1994)

Width

Vehicle type	Sample size	Mean(m)	Std.Dev	Minimum(m)	Maximum(m)
Motorcycle	17	0.66	0.019	0.64	0.73
Autorickshaw	20	1.11	0.004	1.08	1.11
Car	73	1.61	0.069	1.46	1.73
Mini-bus	53	1.88	0.086	1.71	2.05
Large bus	51	2.41	0.115	2.14	2.74

(Source: Hoque 1994)

Stop gap

Vehicle type	Sample size	Mean(m)	Std.Dev	Minimum(m)	Maximum(m)
Motorcycle	21	0.68	0.272	0.45	1.15
Autorickshaw	21	0.68	0.272	0.45	1.15
Car	33	1.52	0.192	0.67	3.27
Mini-bus	14	1.98	0.37	0.69	3.73
Large bus	11	1.95	0.472	0.91	3.43

(Source: Hoque 1994)

Desired speed

Vehicle type	Sample size	Mean km/hr	Std.Dev	Minimum km/hr	Maximum km/hr
Motorcycle	43	56.21	5.012	45.34	65.34
Autorickshaw	75	51.98	4.872	43.19	67.86
Car	70	63.48	5.123	45.12	81.23
Mini-bus	45	57.23	5.783	41.09	71.21
Large bus	36	56.23	6.123	43.79	67.34

(Source: Hoque 1994)

Maximum turning speed

Vehicle type	Sample size	Right turning speed			
		Mean km/hr	Std.Dev	Minimum km/hr	Maximum km/hr
Motorcycle	10	29.21	3.127	23.45	35.45
Autorickshaw	15	28.79	2.453	24.81	31.23
Car	20	27.48	3.045	19.23	33.24
Mini-bus	10	25.56	2.569	19.67	28.12
Large bus	10	23.45	2.045	18.89	26.23

(Source: Hoque 1994)

Acceleration

Vehicle type	Sample size	Mean km/hr	Std.Dev	Minimum km/hr	Maximum km/hr
Motorcycle	8	2.32	0.38	1.25	3.21
Autorickshaw	10	1.61	0.326	1.22	2.19
Car	20	1.87	0.578	1.31	2.68
Mini-bus	10	1.46	0.379	1.19	1.98
Large bus	10	1.13	0.277	0.76	1.81

(Source: Hoque 1994)

Normal Deceleration

Vehicle type	Sample size	Mean km/hr	Std.Dev	Minimum km/hr	Maximum km/hr
Motorcycle	11	2.16	0.283	1.79	2.55
Autorickshaw	10	2.18	0.346	1.29	2.58
Car	20	1.88	0.421	1.25	2.39
Mini-bus	10	2.01	0.331	1.45	2.77
Large bus	12	1.68	0.255	1.16	2.12

(Source: Hoque 1994)

Maximum decelaration

Vehicle type	Sample size	Mean km/hr	Std.Dev	Minimum km/hr	Maximum km/hr
Motorcycle	5	4.56	0.47	3.42	4.83
Autorickshaw	5	5.01	0.31	4.35	5.92
Car	10	4.44	0.364	3.97	4.89
Mini-bus	5	4.89	0.41	3.51	5.62
Large bus	5	4.23	0.661	3.33	5.47

(Source: Hoque 1994)