A Thesis

on

A STUDY OF TRAFFIC SIGNAL PERFORMANCE IN METROPOLITAN DHAKA : THE CASE OF MAGHBAZAR INTERSECTION

Submitted to the Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, in partial fulfilment of the requirements for the degree of

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By

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ABSTRACT

A study has been made on the traffic signal performance in the Dhaka Metropolitan Area. The traffic signal at Maghbazar intersection was investigated as a particular case in this regard. The study included a detailed review of literature concerning various aspects of traffic signals. It also included collection of necessary data by field investigation and analyzing them to evaluate the overall performance of traffic signal. The field investigation involved vehicular volume count by type on all the approaches of the intersection during peak hours and the speed distribution of the approaching traffic stream. Based on the data collected from the field, traffic phasing and the optimum cycle length were determined. The calculated cycle length and the traffic phasing were compared with the existing phasing by considering various aspects, traffic delay in particular to appraise the designed signal timings. The optimum cycle length of three phase signal was determined to be of 73 seconds compared with existing length of 105 seconds. This reduced cycle length has the potential to reduce overall delays by about 54%. After detailed examination of the various aspects of the traffic signal along with other traffic control devices and subsequent field investigations, potential remedial measures were suggested to alleviate traffic conflicts and safety hazards. Their applications were recommended for overall improvements of the traffic signal performance at the intersection. It is also recommended that similar examination should be carried out at other major intersections towards better performance of the traffic.

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CHAPTER -1

INTRODUCTION

1.1 General

Traffic signals are an essential component of traffic control devices. The installation of traffic signals has a central role in attending traffic hazards at the critical points of road system such as intersection. An intersection is the focal point of conflicts and congestion. Since it is common to two or more roadways(Pignataro,1973)

In an uncontrolled intersection crossing, converging and diverging movements of traffic stream are found to occur (Hobbs,1979). The conflicting movements of traffic from different approaches of the intersection result in the reduction of speeds causing delay, increased congestion and greater possibility of accidents (Ogden and Bennett, 1981). As the frequency and severity of intersection conflicts increase, regulation and control of traffic become necessary which may be done by the installation of traffic control devices such as, signals, signs and pavement markings.

A traffic signal is a power operated traffic control device by which traffic is warned and directed to take some specific action. The primary aims of signal control at an intersection are:

i) to reduce conflicts and hence the potential for accidents,

ii) to better regulate and stabilize traffic movements, and

iii) hence reduce delays(Ogden and Bennett, 1981)

The installation of traffic signals provides a temporal split in vehicle rights-of-way to enable the efficient and safe movement of conflicting traffic streams. It is at the change of right-of-way that the accident potential is highest. Design standards provide strict guidelines to minimise these potential conflicts by defining i) geometric layout requirements and ii) operational performance requirements. A signal has the potential to reduce both the number and severity of accidents occurring at the intersection. However, to realise this potential, strict attention must be paid to proper signal design and installation.

1.2 Performance of Traffic Signal

According to Pignataro (1973) traffic signals are useful in releiving congestion and unsafe situations where no other control device is adequate. The alternating assignment of right-of-way to intersection legs can eliminate most or all conflicting movements in the intersection area.

When signals are used in STOP & GO operation, it is necessary to determine how much of the total time available at the intersection will be apportioned to each flow of traffic movement. At a given intersection the total traffic is broken up into phase of movement in which one or more flows will take place. Certin flows are ascribed in the right of entry to the intersection and are given a green (i.e.,GO) signal aspect, while all other flows are stopped and given a red (i.e.,STOP) signal aspect. The objective of phasing is to accommodate all traffic movement with increased safety and minimum delay. Safety urges a phasing which will reduce or eliminate all potential conflicts. On the otherhand consideration for minimum delay impels a phasing which will accept as many simultaneous flows as practicable, to achieve a high volume of accommodation. (Matson, et al,1955).

1.3 Objective of the Study.

For the proper design and installation of traffic control devices a study should be made regarding vehicular and pedestrian volumes, accident hazards, co-ordinated movement and traffic flow characteristics. The proposed research aims at the study of the existing system and performance of traffic signal at Maghbazar intersection. Essentially, this requires the examination of traffic volume, delay and accident characteristics.

The main objective of the study is to improve the control and regulation of traffic particularly at the intersections, in terms of safety, economy, and efficiency. The objective could be achieved only when the traffic are regulated and controlled in an effective manner based on proper traffic engineering practice.

1.4 Justification of the Study.

The operation and safety characteristics of the road system depends more on intersections than any other single parameter under the control of the road design engineer (Ogden & Bennett, 1981). The importance of intersections as major elements in the road system may be stressed by accident statistics which show that about 52% of all casualty accidents in the central Dhaka city occur at the intersections (Hoque, 1981).

The results of the study of traffic signal performance at Maghbazar intersection would necessiate the need of studying other similar important intersections in the city.

1.5 The Study Area.

Hazardous pattern of traffic flow is a common phenomenon at almost all the intersections of the Dhaka Metropolitan Area. The hazards are associated with lack of design and control of intersections and comprises the following contributory factors (Hoque, 1981).

i) Lack of intersection capacity

ii) The left lanes are constantly occupied by the slow moving vehicles causing hazards to fast moving vehicles.

iii) Blockage of through lanes by vehicles which intend to make right turn.

iv) The major portion (sometimes entire portion) of carriage-ways are occupied by dominant percentage of the slow moving vehicles.

v) Curbside stopping and parking near intersections are the regular affairs.

vi) Driveways at the intersections are creating extra hazards to flow of the traffic.

vii) The sequence of police control operations happens to be misleading more often and create traffic conflicts at the intersections.

viii) Frequent wrong side maneuvering at the intersection creating hazards both for the pedestrian crossing as well as the vehicles turning into, and passing through the intersections

ix) Lack of traffic signals exclusively for the pedestrian crossing at some of the major intersections.

x) Lack of willingness of the pedestrian to use over-passes wherever provided.

The intersection at Maghbazar may be considered as one of major intersections of the Dhaka city. Besides, the south and southeast bound commercial vehicles from both Aricha and Joydevpur pass through this intersection to reach their destinations beyond Dhaka city. As such this intersection is an important one and deserves urgent attention. This study aims at the investigation of the existing system and performance of traffic signal at Maghbazar intersection. Essentially this requires the examination of traffic volumes, delay and accident characteristics at the intersection.

1.6 Study Outline

This study consists mainly of analytical study to determine all the parameter of signal design for which vehiclular, and pedestrian study are required. This study involves some important steps which are outlined in the following steps.

1.6.1 Survey Phase

1) Traffic survey for determining peak hour volume, pattern of turning movements, composition of traffic (percentage of different types of vehicles, e.g., slow and fast moving vehicles).
2) Preliminary survey of the intersection area and its approaches in connection with traffic signs, traffic markings on pavements.
3) Measurement of widths of the carriage way of the approaches of the intersection.
4) Study of existing cycle time, phasing, amber time and all red time if any.
5) Vehicle speed study for determining allowable speed.
6) Study of pedestrians crossing the intersection legs.

1.6.2 Analysis of Data

1) Analizing the traffic data collected from the field study.
2) Designing optimum cycle time and its splits.
3) Comparing the results with the existing cycle time.

CHAPTER - 2

THEORETICAL ASPECTS OF TRAFFIC SIGNAL

2.1 Introduction

The chapter introduces the theoretical aspects of traffic signal by defining different terms and describing their relationship with the traffic signal performance.

2.2 Definitions

The study of traffic signal performance should include a number of useful definitions of several traffic engineering aspects which are somehow related to the traffic signal. The following definitions have implications to the study.

2.2.1 Intersection

An intersection is a node within a road system where roads meet or cross in order that vehicles may transfer from one route to another (Ogden & Bennett,1981)

The legal intersection with its relevant parameters may be represented as shown in Fig.2.1.

2.2.2 Traffic Control Devices

Traffic control devices are all signs, signals, markings and devices placed on or adjacent to a street or highway, by authority of a public body or official having jurisdiction to regulate, warn or guide traffic (Manual on uniform traffic control Devices,BPR,1961)

2.2.3 Traffic Signal

A traffic signal may be defined as a power operated device for regulating, directing or warning the motorists or the pedestrians in an efficient and effective manner. The traffic signal installed at an intersection alternately assigns the use of the intersection central area first to one stream of traffic, then to another. Right-of-way is allocated by time seperation of conflicts(Pignataro,1973).

2.2.4 Traffic Sign

A traffic sign is a device mounted on a fixed or portable support, whereby a specific message is conveyed by means of words or symbols officially errected for the purpose of regulating, warning or guiding traffic(Pignataro, 1973).

2.2.5 Traffic Marking

Traffic markings are all lines,patterns,words,colours, or other devices, except signs, set into the surface of, applied upon, or to objects within or adjacent to the roadway, officially placed for the purpose of regulating, warning or guiding traffic (Pignataro, 1973).

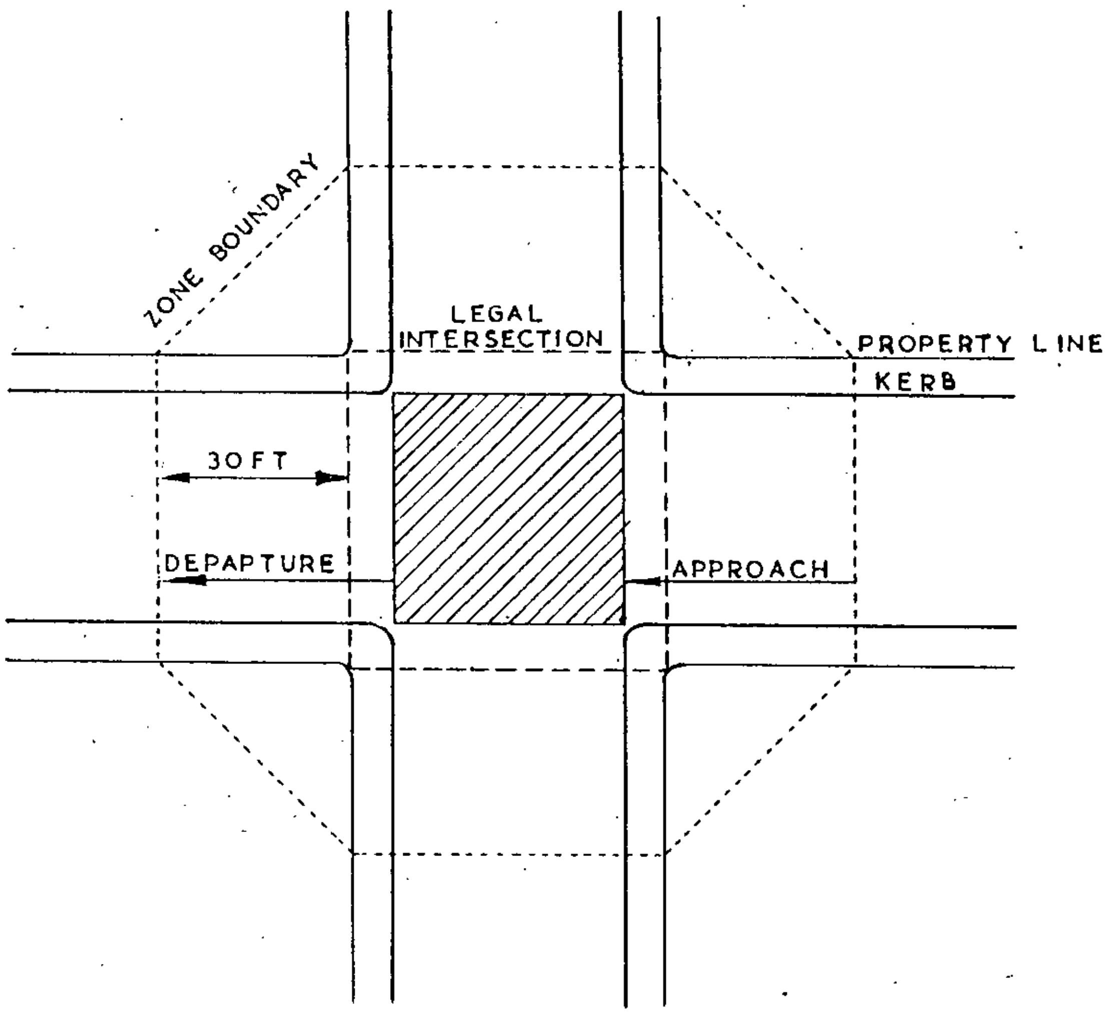


Fig. 2.1 Typical Cross Intersection.
Source : Hoque and Andreassen (1986)

2.2.6 Traffic

Vehicles, persons (pedestrians) or animals travelling on a road considered collectively, are traffic.

2.2.7 Traffic Volume

The number of vehicles passing a given point during a specified period of time or the number of vehicles that pass over a given section of a lane or a roadway during a specified period of time (Highway Capacity Manual, HRBSR 87, 1965)

2.2.8 Peak Hour Traffic

The highest number of vehicles found to be passing over a section of a lane or a roadway during 60 consecutive minutes. This term may be applied to a daily peak hour or a yearly peak hour. (Pignataro,1973)

2.2.9 Peak Hour Factor

A ratio of the volume occuring during the peak hour to the maximum rate of flow during a given time period within the peak hour. It is a measure of peaking characteristics, whose maximum attainable value is unity. The term must be qualified by a specified short period within the hours; this is usually 5 or 6 minutes for freeway operation, and 15 minutes for intersection operation (Pignataro,1973).

2.2.10 Passenger Car Equivalent (PCE)

Often in traffic engineering practice the number of vehicles is converted to equivalent passenger car units by multiplying the number of vehicles with appropriate(recommended)factors. The factor is termed as PCU factor(Sharma,1985).

2.2.11 Cycle Time

The number of seconds requied for one complete revolution of the timing dial or for one complete sequence of signal indications. (Pignataro,1973)

The cycle time may be measured as the time required for the complete sequence of events from the start of green of a particular phase until the start of green again for that phase(Fig.2.2).

2.2.12 The Traffic Phase

A part of the time cycle allocated to any traffic movement receiving the right-of-way, or to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals. A traffic movement can signify a vehicular movement alone, a pedestrian movement alone, or a combination of vehicular and pedestraian movements. The sum of all traffic phases is equal to the time cycle. (Pignataro,1973)

2.2.13 Clearance Interval/Amber Time

A yellow lighting of short interval immediately following the green to advise the motorists that the red interval is about to commence and to permit the motorists to come to a safe stop and to allow the vehciels that have entered the intersection legally sufficient time to clear the point of conflict prior to release the opposing vehicles(Hobbs,1979).

2.2.14 All-red Interval

The time of display of a red indication for all entering traffic. It is some times used following a clearance interval to permit vehicles or pedestrains to clear an excessively large intersection before opposing vehicles receive a green indication. It may also be used to create an exclusively pedestrain phase (Pignataro,1973)

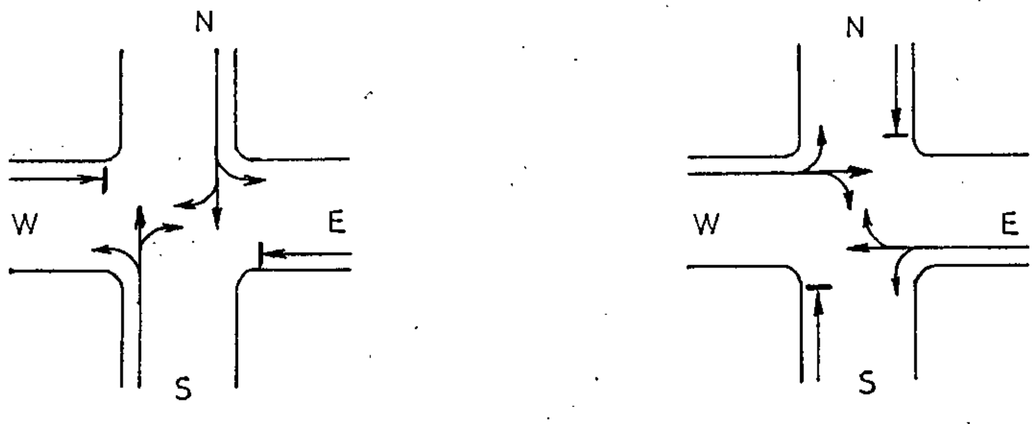
2.2.15 Intergreen Period

The period of time between the termination of the green indication for one phase(or group) and the begining of the green indication for the next successive phase(or group). Intergreen time comprises the closing amber and all-red period (Homburger & Kell,1981)

The signal aspect with a two phase system may be shown diagrammatically as out lined in Fig.2.2.

2.2.16 Pedestrian Phase

A phase of signal time allotted exclusively to pedestrain traffic.These are erected for the exclusive purpose of directing pedestrain traffic at signalized locations. They consist of WALK-DON'T WALK signals (Pignataro,1973).



Phase 1 Phase 2

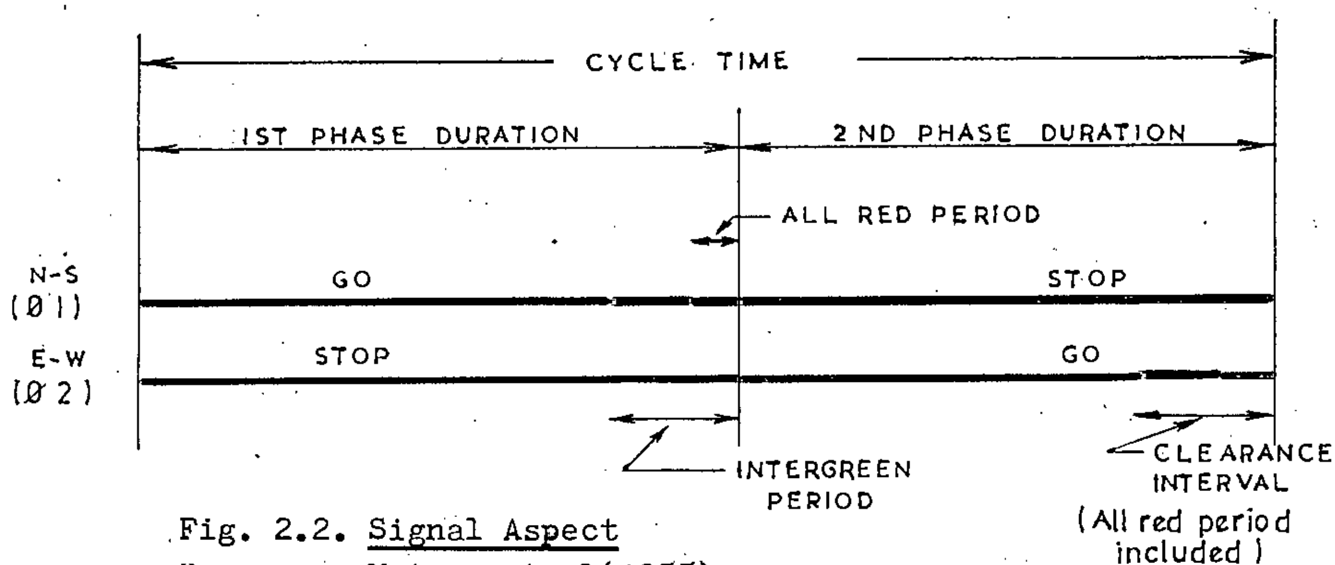


Fig. 2.2. Signal Aspect
Source: Matson et al(1955)

2.2.17 Design Hourly Volume or Arrival Flow

Traffic volumes are much heavier during certain hours of the day or year and it is for these peak hours that the traffic signal is designed (Pignataro, 1973).

2.2.18 Saturation Flow

It is the maximum flow of vehicle that can pass through an intersection from one approach without impedance by signal(Hobbs,1979).

The saturation flow along with the starting delay and the stopping delay intervals can be represented diagrammatically as shown in fig. 2.3.

2.2.19 Delay

This is the time lost while traffic is impeded by some element over which the driver has no control. Delay occurs mainly due to traffic friction and traffic control devices. Delay may be of four types, namely :-

i) Fixed Delay. The delay to which a vehicle is subjected regardless of the amount of traffic volume and interference present.

ii) Operational Delay. The delay caused by interference from other components of the traffic stream.

iii) Travel-time Delay. This delay is the difference between the actual time required to traverse a section of street or highway and the time corresponding to the average speed of traffic at a point of uncongested flow on the section.

iv) Stopped-time Delay. This is the time a vehicle is actually standing still because of any influencing factor.

(Hobbs, 1979)

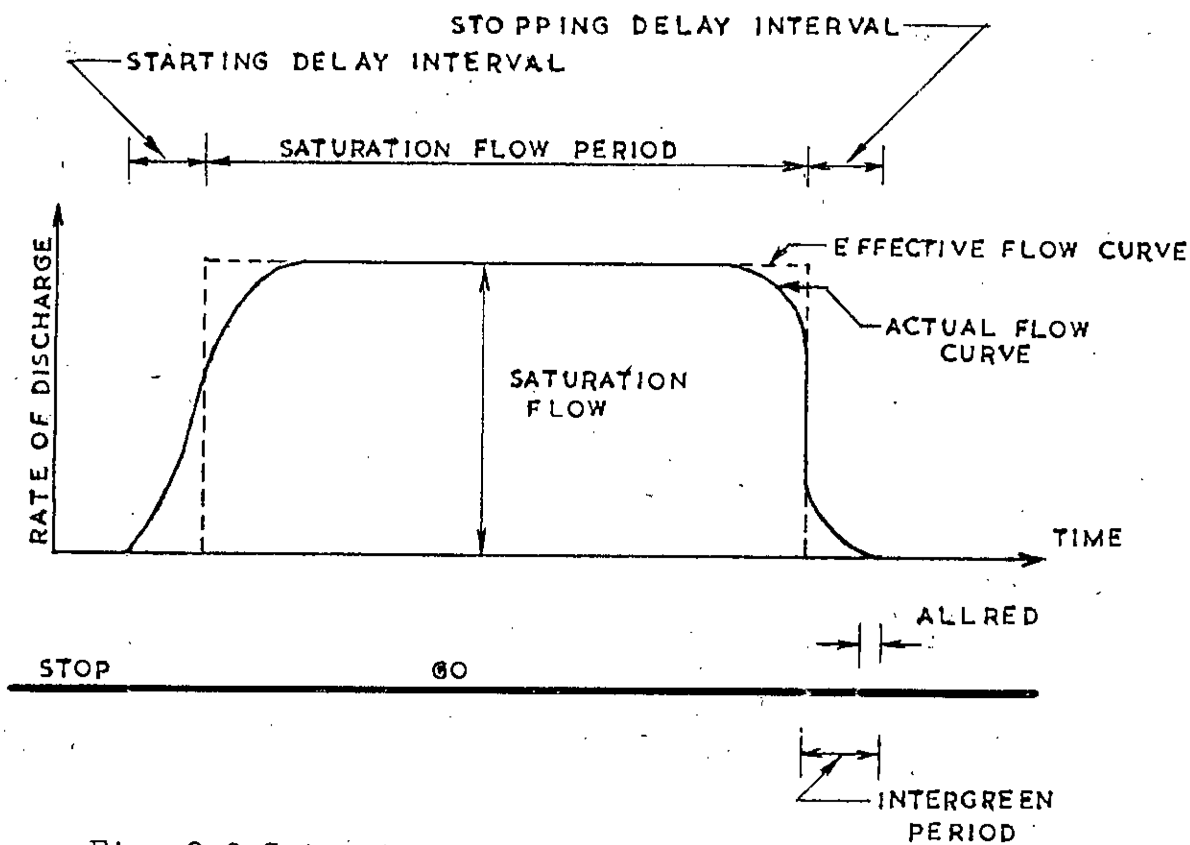


Fig. 2.3 Saturation Flow Diagram
Source : Ogden and Bennett(1981)

2.2.20 Spot Speed

A spot speed is the speed of traffic at one point or spot on a trafficway. The spot speed study consists of a series, or a sample, of observations of the individual speeds at which vehicles are approaching an intersection or passing a point at a nonintersection location. These observations are used to estimate the speed distribution of the entire traffic stream at that location, under the conditions prevailing at the time of the study(Pignataro, 1973).

2.2.21 85th Percentile Speed

That speed below which 85 percent of all traffic units travel, and above which 15 percent travel. This is used in establishing maximum speed limit and safe speed. The 85th percentile speed is sometimes referred to as critical speed (Pignataro,1973).

2.2.22 Design Speed

A speed determined for design as related to the physical features of a highway that might influence vehicle operation. It is the maximum safe speed that can be maintained over a specified section of highway when conditions are so favourable that the design features of the highway govern.

2.3. Conflicts at the Intersection

Conflicts at an intersection are the results of opposition of traffic movement associated with the interference and collision at the intersecting (common) points of two or more streams of traffic flow at that particular section (Homburger & Kell,1981).

The conflicts at an intersection may be classified as follows :- (Wohl & Martin,1967)

i) Crossing conflict : Two opposing stream of traffic flow intersecting at right angle.

ii) Merging conflict : Two seperate traffic stream, moving in the same general direction, combine or unite to form a single stream.

iii) Diverging conflict : The dividing of a single stream of traffic into seperate stream to different direction.

The selection of phasing pattern for the efficient and effective traffic control depends on the extent of permitted type of conflicts and the gap acceptance criterion in the opposing flow at the intersection. By providing a conservative phasing pattern, the potential conflicts can be reduced. But the phasing pattern and the phase length have a great influence on the delay of traffic at the intersection.

In a typical twoway cross intersection there may exist 24 points of major conflicts. These conflicting points are shown in Fig.2.4.

2.4 Reduction of Potential Conflicting Movements

The number of potential vehicular conflicts at intersection is considerably reduced when twoway operation is converted to oneway. This is primarily due to the removal of opposing turning movements (Table 2.1)

Table 2.1 Number of Conflicting Movements

Street A	Street B	Basic movements	Number of conflicts
two-lane, two-way	two-lane, two-way	12	24
two-lane, one-way	two-lane, two-way	7	11
two-lane, one-way	two-lane, one-way	4	6

The number of conflicts between lanes is reduced. Oneway streets eliminate the head on conflict between opposing traffic streams.

The number of pedestrian-vehicular conflicts is reduced. Pedestrains need look oneway, or at the most two ways, instead of four ways. Due to the greatly reduced number of turning movements at intersections when one or both of the streets are one way, there is much less interference with pedestrains when they are in the cross walk. (Pignataro, 1973).

2.5 Advantages of Traffic Signal

i) Provide for the orderly movement of traffic.

ii) Reduce the frequency of certain types of accidents e.g., right-angle collisions.

iii) Provide a means of interrupting heavy traffic to allow other traffic, both vehicular and pedestrains, to enter or cross.

iv) Promote driver confidence by assuring right-of-way.

(Sharma, 1985)

2.6 Signal Classification

Traffic signals, on the basis of their operating function may be classified into 3 categories, namely :-

i) Pretimed or fixed time signal

ii) Traffic actuated signal

iii) Traffic adjusted signal

(Pignataro, 1973)

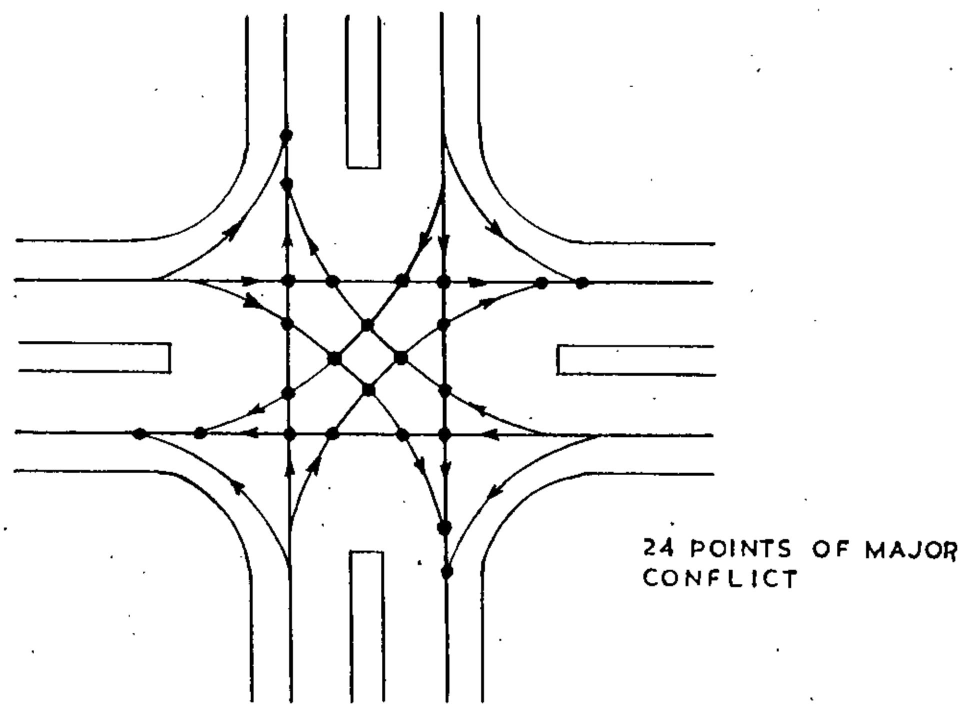


Fig. 2.4 Conflict Points at a Cross Intersection.

Source : Ogden and Bennett, (1981)

2.6.1 Pre-timed Signal

This type of traffic signal is set to repeat a given sequence of signal indications regularly. In this case signal directs traffic to stop and permits it to proceed in accordance with a single,predetermined time schedule or a series of such schedules.

A controller is used in the operation of traffic signal. The controller is an electrical mechanism for controlling the operation of traffic signals, including the timer and all necessary auxiliary apparatus mounted in a cabinet. The type of controller selected to operate a pretimed signal is primarily dependent upon whether or not co-ordination of signals is to be provided.(Pignataro,1973).

The detailed signal aspects of the pre - timed signal has been investigated in this study, which has been described analytically in chapter 4.

2.6.2 Traffic Actuated Signal

The operation of this type of signal is varied in accordance with the demands of traffic as registered by the actuation of vehicles or pedestrain detectors on one or more approaches. Traffic actuated signals may again be classified as follows :-

a) Semi-actuated signals : These devices provide means for traffic activation on one or more but not all intersection approaches. This type of control is applicable primarily to an intersection of a heavy volume, urban or sub-urban traffic artery with a relatively lightly traveled minor road or street.

This type of control is excellent for use where side-street volumes cannot safely cross major flows without signalization. Where both street volumes fluctuate widely, semi-actuated control should not be used, since there are no detectors on one or more legs.

b) Fully actuated signals : This device provides for actuation by vehicles on all legs of the intersections. It is applicable primarily to an isolated intersection of streets or roads that carry approximately equal traffic volumes, but where distribution between approaches varies and fluctuates. It then becomes necessary to take into consideration the demand on all approaches (Pignataro,1973) .

2.6.3 Traffic Adjusted Signals

These are centrally controlled, e.g., by a digital computer and have settings which are updated from measurements of the system through detectors. A combination of the advantages of a pre-timed, flexible, progressive signal system and traffic actuation can be realized by a system in which a traffic actuated master controller is used to supervise either pretimed or semi-actuated local controller. Detectors are placed at representative locations in the progressive system if an artery is to be controlled, or at typical locations throughout an area if a grid system is to be controlled. The detectors transmit information about traffic volume and direction to a computer in the master controller. The information is analized, and the master controller selects the cycle length and offset combination predetermined to best serve the directional distribution and volume characteristics existing at that time. The local controllers are inter-connected to the master controller and operate at any given instant upon the cycle and offset selected by the master controller. (Pignataro, 1973).

2.7 Signal Co-ordination

To achieve smooth flow on a street or highway system, it is not sufficient to ensure that each point of local conflict is efficiently controlled. Unless the controls at each of these locations are co-ordinated in some fashion, continuous, smooth flow on the system will be impossible. It is particularly important that signals in close proximity to each other be co-ordinated, to prevent inefficient stop-and-go flow from developing.

The National Manual on Uniform Traffic Control Devices (MUTCD) recommends that signals within one half mile of each other be co-ordinated on major streets and highways. With greater lengths between signals, vehicles tend to spread out of platoons, making co-ordination less effective. In these situations, intermediate spacer signals may be justified, for the sole purpose of maintaining platoons.(Pignataro, 1973).

2.8 Objectives of Signal Timing for Pretimed Signal

The major objectives of a signal timing plan are the minimization of delay and congestion at intersections and within block lengths and series of block lengths, and the increase in safety for all road users. Full utility of traffic control signals is realized only when they are operated so as to satisfy, as nearly as possible, actual traffic requirement.

In general, short cycle lengths are desirable, because the delay to standing vehicles is reduced. Cycle lengths ordinarily fall between 30 and 120 seconds. Heavier volumes require longer cycle lengths because the sum of the 'GO' or green intervals must be greater to gain sufficient capacity. The sum of 'GO' intervals is a higher percentage of a cycle length with longer cycles since clearance times remain essentially fixed (Pignataro, 1973).

Webster developed an approximate formula for determining the optimum cycle length in terms of minimum delay, which is used in this study to determine the optimum cycle time (Appendix-B).

2.9 Purpose of Vehicle Clearance Interval

The purpose of the amber signal indication following the green interval is to warn the moving traffic facing the signal to come to a stop and if possible, to do so with safety. It should provide enough time for vehicles to clear the intersection before cross traffic start to move. Theoretically, it should be long enough to permit a vehicle to travel, at normal intersection approach speed, a distance equal to the cross street width between curbs plus the safe stopping distance. This is necessary because if the approaching vehicle is a few feet less than the safe stopping distance back from the intersection when the signal changes, time should be allowed for the vehicle to travel the stopping distance plus the street width before the cross street traffic receive the green indication. (Pignataro, 1973).

Too long clearance interval may be used as part of green interval and too short interval may constitute a hazard and increase rear-end collisions. This aspect of signal has practical and conceptual significance in reducing potential accidents. Matson et al(1955)cited this significance:

" Legal versus Natural Causes. In the search for accident causation there is a tendency to charge road users with violation of some preconceived notion of moral or statutory law and thus to establish the cause of accident. While the traffic engineer is vitally concerned with the system of traffic regulation and accepted conventions of society, it is his responsibility to search for the scientific facts which surround accidents and if possible find the laws of nature which influence or govern accident causation.

In one case, for example, right-angle collisions at a signalized intersection on a high-speed road were numerous. In the attempt to reduce accidents, many persons were charged with violation of signals. It was later found that the mere lengthening of the amber of clearance period practically eliminated all right-angle collisions and numerous read-end collisions. Here it is clear that violation of the natural laws of intra, momentum, and human PIEV time, rather than intended violation of legal statute governing the meaning of signal legend, was the cause of accidents".

At most urban intersections a clearance interval of 3 seconds produces good results (Pignataro,1973).

2.10 Design Approach of Signal Timing

The formula developed by Webster has been used in this study to determine the optimum cycle length. The following steps are considered.

1) Determination of amber period on the basis of approach speed.
2) Selection of traffic phases.
3) Estimation of saturation flow.
4) Computation of optimum cycle length.
5) Computation of effective green times.
6) Computation of actual green times.
7) Check for pedestrain requirements.
8) Drawing of timing diagram.

2.11 An Overview

This chapter reviewed the literature of all aspects related to traffic signal performance, and their interaction. Based on this literature the following chapters investigate the traffic signal under study.

CHAPTER - 3

COLLECTION AND ANALYSIS OF DATA

3.1 Introduction

This chapter deals with the collection of necessary data relating to the study. The data includes inventory of traffic signal, traffic survey, vehicle and pedestrian count, determination of vehicle composition, spot speed study and delay study. These data are then analyzed successively to achieve some results.

3.2 Data Collection

The study requires adequate traffic data for the analysis and examination of the traffic signal performance. The establishment, maintenance and operation management of the traffic control devices within Dhaka Metropolitan Area are under the jurisdiction of the Dhaka Municipal Corporation and the Dhaka Metropolitan Police. The former establishes the devices under guidance from the latter. Therefore, it was desired that necessary data in connection with the study would be collected from these two organizations. The author officially approached the competent authorities of both the organizations. It was found that the data as desired are not maintained by them. As a result field investigation was made as an alternate source of data collection.

3.3 Inventory of Traffic Signal

A preliminary survey was carried out throughout the Dhaka Metropolitan area to find out total number of major intersections and their characteristics. On the basis of i) characteristics of road, ii) traffic movement and iii) accident hazards (Hoque,1981) it was observed that there are 68 major intersections out of which 39 intersections are signal controlled, 14 intersections are controlled by round abouts and the remaining intersections are manually (by traffic policemen) controlled. The locations of the major intersections are shown in a map in Fig. 3.1.

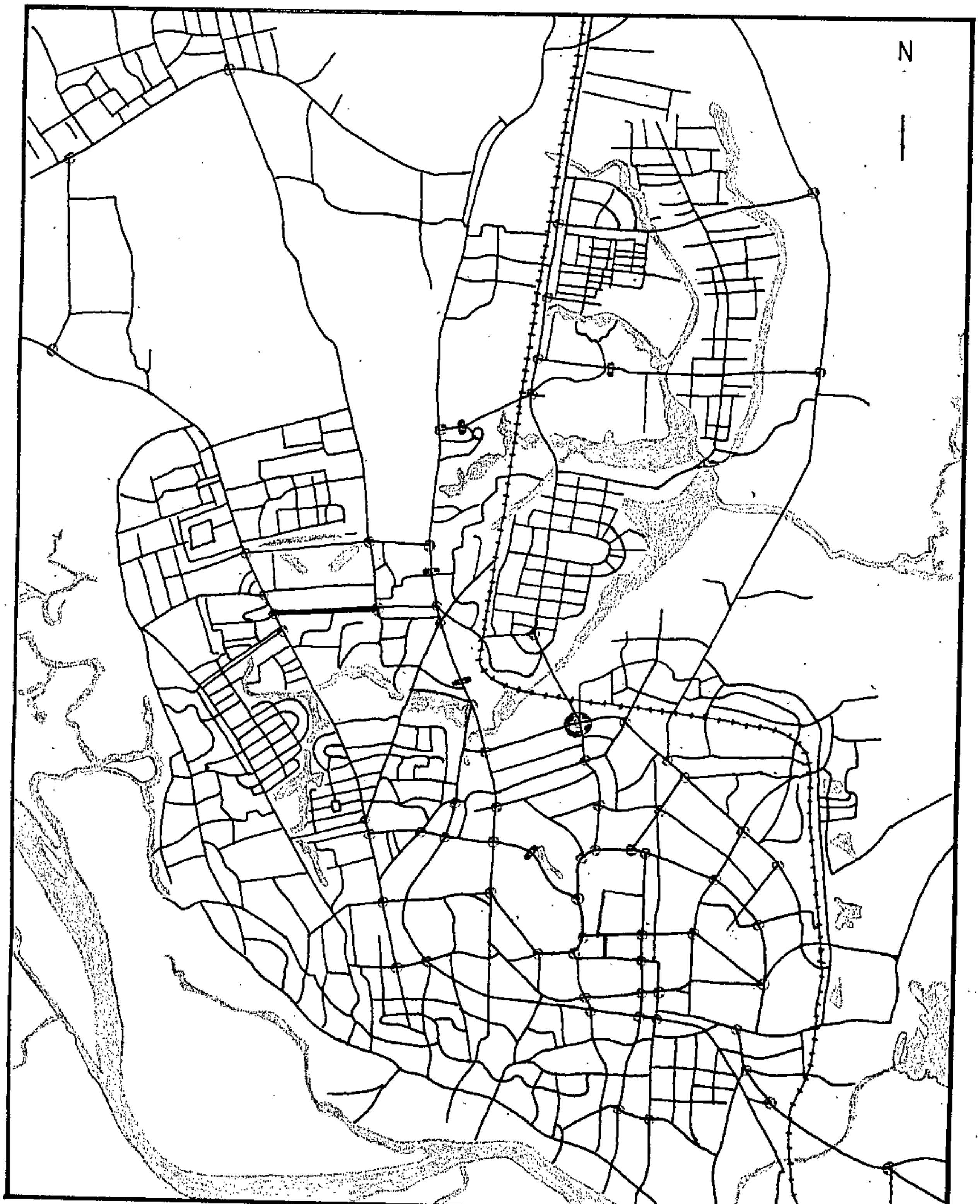


Fig. 3.1 Map of Metropolitan Dhaka Showing Major Intersections.

- Major Intersections.
- Signal for Exclusive Pedestrian Crossing.
- Study Area.

Hazardous and disorderly movement of traffic was found at almost all the major intersections particularly during peak hours. The list of most hazardous intersections in respect of accident hazards (Hoque,1981) may be referred to, which is given in Table 3.1.

During observation buses and mini-buses were found to create hazards at the intersections where the bus stops are very near to the intersections. The slow moving vehicles, particularly the rickshaws were also found to park near the intersections frequently. This sort of parking sometimes make hazards at the centre of the intersection when the departing vehicles cannot clear the intersection and the cross traffic start moving. Cross walks for the pedestrians were found inadequate. The traffic signs and the markings on pavement were found very poor and insufficient in almost all the roads. Only the VIP road (from Bangabhaban to New Airport via Farmgate) was found to have some traffic markings on pavement at several sections which is also not satisfactory in terms of traffic engineering practice. The paintings of pavement marking including pedestrian corss walks are not maintained regularly. Only 5 traffic signals were found for exclusively pedestrian crossing, all of which were at mid blocks, i.e., in between two adjacent intersections (Fig.3.1).

3.4 Traffic Survey at Maghbazar Intersection

The intersection at Maghbazar is a cross intersections. Thus it has four major legs. Besides four major legs a narrow lane originates from the junction point beside Tongi Diversion Road and runs into Maghbazar residential area. Out of four major legs Tongi Diversion Road runs towards Tejgaon Industrial Area (to the north), Maghbazar road on the south runs towards Bailey road, Outer Circular road on the east runs towards Malibag, and New Eskaton Road on the West runs towards Banglamotor. The geometric layout of the intersection is given in Fig 3.2.

Table 3.1 Most Hazardous Intersection Locations in order of Decreasing Frequency of Accidents.

(Source : Hoque, Traffic Accidents in Dhaka, 1981)

Intersection location	Police station	No of total accidents (all types)
1. Kawranbazar (Near Sonargaon Hotel)	Tejgaon	94
2. Shahabagh(Airport Rd. to Bhashani Rd.)	Ramna	88
3. Bangla Motr (T.intersection)	Ramna	71
4. Farmgate (with green road)	Tejgaon	62
5. Science Laboratory(with New Elephant Road) T-Intersection	Dhanmondi	54
6. Farmgate (near Police Box)	Tejgaon	49
7. Bejoynagar(Topkhana & Bejoynagar road)	Ramna	47
8. Hotel Intercon(Mymensigh Rd.&Minto Rd.)	Ramna	43
9. Gulistan(Infront of Bus terminal)	Motijheel	43
10. Science laboratory(Mirpur Rd.&Rd.No.2)	Dhanmondi	41
11. Asadgate (Mirpur road)	Mohammadpur	41
12. Airport Crossing(At Airport Road)	Tejgaon	37
13. Khejurbagan Intersection at S.B.Nagar	Tejgaon	37
14. Near Dainik Bangla office at DIT road	Motijheel	31
15. Farmgate (with Indira road)	Tejgaon	29
16. Palashi (near S.M. hall)	Lalbagh	28
17. Allenbari(at Airport Rd.near RHD office)	Tejgaon	27
18. Near Employment Exchange Centre(Santinagar Rd. with Inner circular road)	Ramna	27
19. Near Mirpur Tech.Centre(Aricha Rd.& Mirpur Road)	Mirpur	26
20. Maghbazar(Eskaton Rd.& Tongi Div.Road)	Ramna	26
21. Mohakhali(Airpurt road and Tongi road)	Gulshan	25
22. Fakira pool(Near Noorani Hotel)	Motijheel	25
23. Mauchak Market(Rampura Rd.& Outer Circular road)	Ramna	24

(Continued)

(Table 3.1 continued)

Intersection location	Police station	No.of total accidents (all types)
24. Highcourt (with A. Ghani road)	Ramna	24
25. Infront of Ittefaq office	Motijheel	22
26. Gulistan (Near Cinema Hall)	Motijheel	22
27. Awlad Hossain Market (at Airport road)	Tejgaon	21
28. New Elephant road and Elephant Road crossing	Dhanmondi	20
29. Tikatuli (Hatkhola road and Narayangonj road)	Sutrapur	20

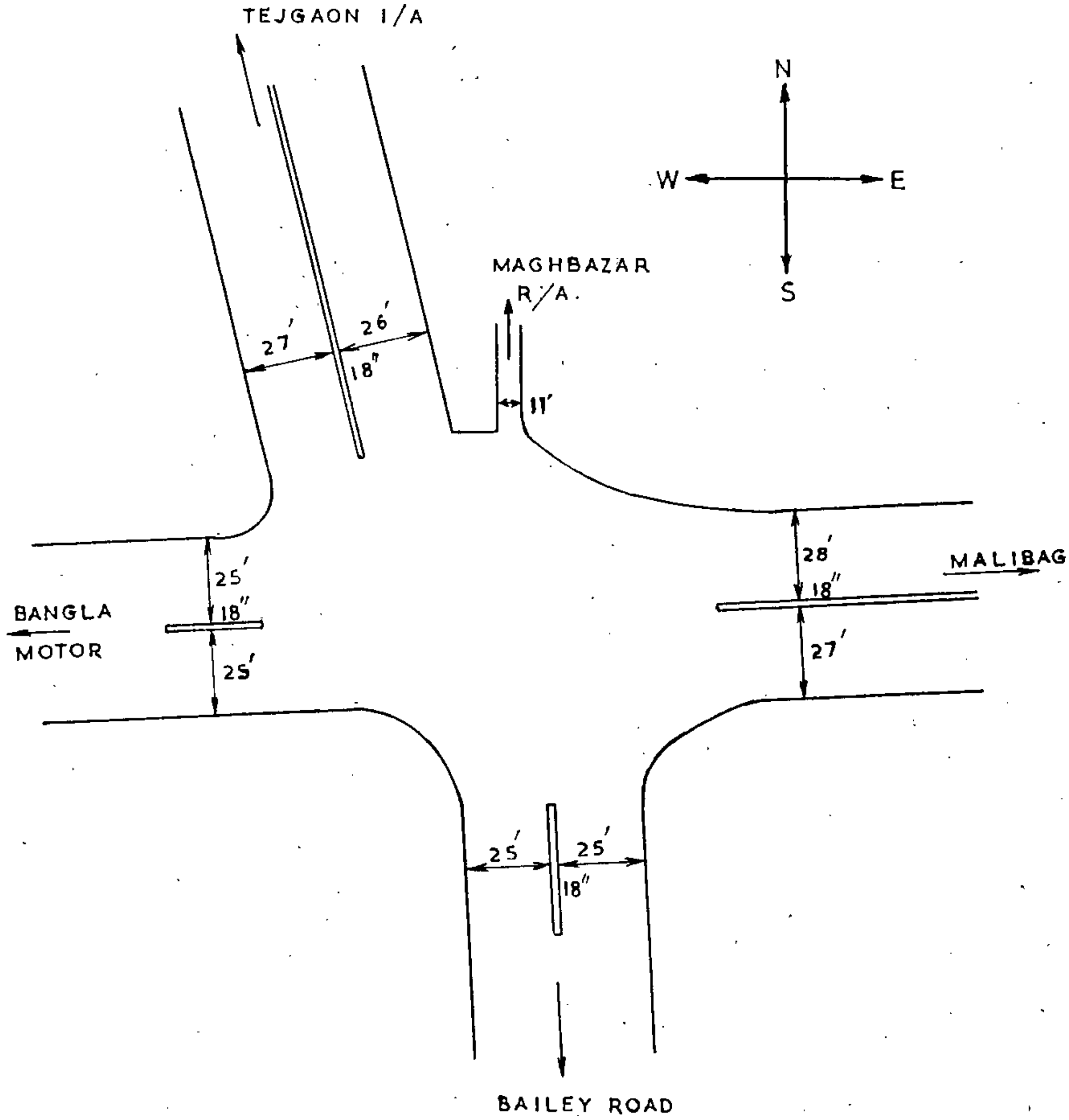


Fig. 3.2 Existing Geometric Lay-out of Maghbazar Intersection in Line Diagram Showing Pavement Widths (Carriage Way)

(Not to scale)

Only 3 traffic signs were discovered on three legs of the intersection, one of which warns the motorists not to blow horns, one restricts the entry of buses and trucks into the southersn leg and the third one is a bus stop sign. All these signs were found improperly placed and they do not acheive their purpose (discussed in details in Chapter V).

No traffic marking on pavement was found on three legs of the intersection. The only marking on pavement was found on the south leg. But these markings (very old paintings)have been erased and have become so illegible that they have lost their identity to guide the motorists and the pedestrians any more. No predestrian facility for crossing the roads was found at the intersection excepting the south leg. There was neither marking on pavement for pedestrian crosswalk nor WALK-DON'T WALK signal exclusively for the pedestrains. The pedestrians were found to cross the roads with their own risks.

3.5 Traffic Volume Count

Intersectional volume counts of vehicle were made to determine the following:

1) The total traffic entering the intersection on each leg individually.
2) The total traffic executing each of the possible turning movements and running straight.
3) Classification of vehicles by type.

The traffic volume counts were carried out manually for the entire study. There are four different counting techniques,(Transportation & Traffic Engineering Handbook,1982), namely manual, mechanical, photographic and moving car method. Due to lack of equipment and transport manual counting technique was adopted.

For proper signal design peak hourly volume is essential, which requires continuous traffic count throughout the day. But it was not possible due to the lack of man power and time limitation. For these limitations instead of continuous 24 hours traffic counts the arrangements for vehicle count were made during the probable hours of peak traffic flow. The probable hours of peak flow were taken as, from 8.00 AM to 11.00 AM, from 1.00 PM to 3.00 PM from 4.00 PM to 6.00 PM in different days. The observed traffic volumes are represented in Appendix-A.

3.6 Determination of the Passenger Car Equivalents(PCE)

Traffic volume entering an intersection from different legs comprises various types of vehicles and it is required in Traffic Engineering studies to get this volume in terms of passenger car units (PCU) for the purpose of design of signal timing. The traffic volume count in the field was made by vehicle type. This volume was then converted to passenger car units by multiplying the number of each type of vehicles with suitable recommended passenger car unit(PCU) factors. The PCU factors used in the conversion were extracted from the Indian Roads Congress(IRC) recommended values, as listed in Table 3.2.

TABLE 3.2

Passenger Car Equivalent Factors

(Source:Sharma,1985)

Serial No.	Type of vehicle	PCE Factor
1	Passenger Car, Jeep, Microbus, Pickup, Three wheeler tampo and autorickshaw	1.0
2	Cycle rickshaw, rickshaw van	1.5
3	Cycle, Motor cycle and scooter	0.5
4	Hand pushed cart	3.0
5	Bus, Minibus, truck, tractor, tanker	3.0

3.7 Summary of Traffic Volume Counts

It was dicovered from the traffic volume count that about 45% of the total volume of traffic were slow moving vehicles(i.e., rickshaws), about 41% were passenger cars and 5% were commercial vehicles and the rest were motor cycles.

Among the fast moving vehicles(excluding rickshaws) about 72% were passenger cars inclusive of cars, taxies, jeeps, microbuses and three-wheeler tampoes. About 14% were commercial vehicles and the rest included motor cycles.

The street in the direction of north-south was considered as the major street in terms of maximum flow volume of traffic. The peak flow on this street was found during 9:00 AM to 10:00 AM. Hence the peak hour was taken as the hour from 9:00 AM to 10:00 PM as per design criterion (Pignataro,1973).

It was also found that there were three notable turning movement from three approaches, namely:-

i) About 30% of traffic volume on the north approach turn left.

ii) About 44% of traffic volume on the east approach turn right.

iii) About 34% of traffic volume on the west approach turn right.

The vehicle composition of the peak flows are shown in Table A.5.

3.8 Spot Speed Survey

A spot speed study was conducted to determine the instantaneous speeds of different types of vehicles on all major legs of the intersection. The determined instantaneous speeds were analyzed to achieve the approaching speed. The speed survey consisted of a series of observations of the individual speeds at which vehicles were approaching the intersection. These observations were used to estimate the speed distribution of the entire traffic stream at that location, under the conditions prevailing at the time of study.

3.8.1 Method Used in Conducting Spot Speed Survey

There are a number of ways to collect speed data depending upon the equipments available. There are two basic methods, (Pignataro, 1973) namely :

1) Involves the measurement of time and distance.

a) Time versus measured distance

b) Distance versus measured time

2) Involves the Doppler principle using rader and ultrasonic.

The method involving time versus measured distance was used in this study. This is the simplest and most common method of collecting speed data. In this method time required by a vehicle to traverse a measured course or ' trap ' is measured.

There are three methods (Pignataro, 1973) for measuring time over a meaured distance, namely-

1) stop watch method

   a) using transverse pavement markings at each end of the trap.

   b) using enoscope or flash boxes set at a fixed distance apart along the edges of the road.

2) Method using pneumatic tubes

   a) using timing unit known as speed watch.

   b) using graphic recorder (20-pen type)

   c) using electrically operated meter

   d) using electronic meter

3) Photographic technique

   a) using time lapse photography.

   b) using continuous strip photography.

In this study stop watch method using transverse pavement markings at each end of the trap was used. Trap length of 100 ft. was used. Thus two station 100 ft. apart were selected on each approach leg and time required by a car to enter and leave the trap end was recorded by a stop watch.

From the known distance (trap length) and recorded time the speeds were calculated using the formula :-

$$V = \frac{15\ S}{22\ t}$$

where, V= speed in mph

S= trap length in ft. (100 ft.)

t= elapsed time in second

For the operation of spot speed study two off-peak hours were selected as per recommendation (Manual of Traffic Engineering Studies, ITE,1964). First hour was selected between 11.00 AM and 12.00 noon and the second hour was between 3.00 PM and 4.00 PM. Sample speeds of 25 vehicles for fast moving vehicles and of 30 vehicles for mixed traffic were taken on each approach for the purpose.

3.8.2 Analysis and Presentation of Spot Speed Data

Because of the uncertainty involved in the characterization of an entire population by variables based upon a sample, and because vehicles in a traffic stream are not traveling at uniform speeds but follow a distribution of speeds within a comparatively wide range, the mathematics of statistics must be introduced into the analysis of spot speed data.

In analyzing speed data, the cumulative percentage of observation versus speed curve was drawn. From this curve the 85th percentile speed was determined. The arithmetic mean speed (time mean speed) and the standard deviations were also determined. Table 3.3 and Table 3.5 represent the elapsed time data and corresponding speeds, Table 3.4 and Table 3.6 represent the frequency distribution and Fig. 3.3 and Fig. 3.4 show the cumulative frequency distribution curves.

3.9 Summary of the Spot Speed Survey

For fast moving vehicles the spot speed survey was carried out on the passenger cars only. The passenger cars included cars, jeeps, three wheeler taxies and microbuses. The 85th percentile speed for passenger cars was found 45 kph which was used in the design of signal timing.

For the mixed traffic the speed survey was carried out on all types of vehicles. The 85th percentile speed for the mixed traffic stream was found 30.57 kph.

3.10 Delay Survey at the Intersection

Delay at intersection is a major problem in the analysis of congestion. Intersection delay studies at individual intersections are required in evaluating the efficiency or effectiveness of a traffic control method. Other factors include accidents, cost of operation, and motorists' desires (Wohl and Martin, 1967).

Investigation was made to measure traffic delay at the intersection. Stopped time delay method was adopted in which a sampling procedure was used which involved the counting of number of vehicle in the intersection approach at successive intervals.

Table 3.3 Elapsed Time and Corresponding Speed for Passenger Cars

Trap Length 100 Ft.
Travel Time in Second.

Speed in mph (1500/22t)
Data Collected on 24.07.89.

Time	Speed	Time	Speed	Time	Speed	Time	Speed	Time	Speed
2.64	25.83	2.47	27.60	3.37	20.23	2.63	25.92	2.78	24.53
2.89	23.59	1.89	36.08	2.61	26.09	2.90	23.50	2.62	26.02
2.83	24.09	2.62	26.02	2.85	23.92	2.11	32.31	2.14	31.86
3.21	21.25	2.90	23.52	3.13	21.75	2.53	26.91	2.85	23.95
2.62	26.03	3.12	21.85	2.63	25.93	2.83	24.07	2.57	26.53
2.69	25.35	2.84	24.00	2.39	28.50	2.47	27.62	3.23	21.09
2.57	26.52	2.67	25.50	2.62	26.01	3.10	22.00	2.84	24.00
2.91	23.40	2.44	27.95	2.83	24.10	2.74	24.87	2.60	26.20
2.28	29.90	2.43	28.00	1.79	38.05	2.68	25.42	1.99	34.30
2.90	23.52	2.89	23.62	2.44	27.95	2.39	28.50	2.86	23.80
2.58	26.43	2.60	26.22	2.85	23.90	2.79	24.40	2.64	25.83
2.85	23.89	2.35	29.03	2.57	26.57	2.59	26.35	2.27	30.05
2.84	24.02	2.90	23.52	2.23	30.57	2.25	30.30	2.88	23.67
2.63	25.89	2.74	24.84	3.06	22.30	2.45	27.85	2.62	26.01
3.54	19.28	2.63	25.92	2.91	23.45	2.57	26.50	3.09	22.05
2.47	27.61	2.85	23.90	2.45	27.85	2.89	23.63	2.77	24.62
2.59	26.35	3.10	22.00	2.84	24.02	3.13	21.80	2.62	25.99
3.10	22.01	2.45	27.80	1.91	35.70	2.59	26.32	2.13	31.95
2.54	26.82	2.61	26.17	2.60	26.23	2.43	28.10	2.50	27.30
2.43	28.05	2.02	33.80	2.79	24.40	1.94	35.20	3.15	21.63

Table 3.4 Frequency Distribution Table for Passenger Cars.

Speed group in mph	Mean speed of group X	Number of observation f	Percent of total observation	Cumulative percent of total observation.	fx	fx^2
19.00-20.99	20.00	2	2	2	40	800
21.00-22.99	22.00	11	11	13	242	5,324
23.00-24.99	24.00	29	29	42	696	16,704
25.00-26.99	26.00	30	30	72	780	20,280
27.00-28.99	28.00	14	14	86	392	10,976
29.00-30.99	30.00	5	5	91	150	4,500
31.00-32.99	32.00	3	3	94	96	3,072
33.00-34.99	34.00	2	2	96	68	2,312
35.00-36.99	36.00	3	3	99	108	3,888
37.00-38.99	38.00	1	1	100	38	1,444
TOTAL	-	100	-	-	2,610	69,300

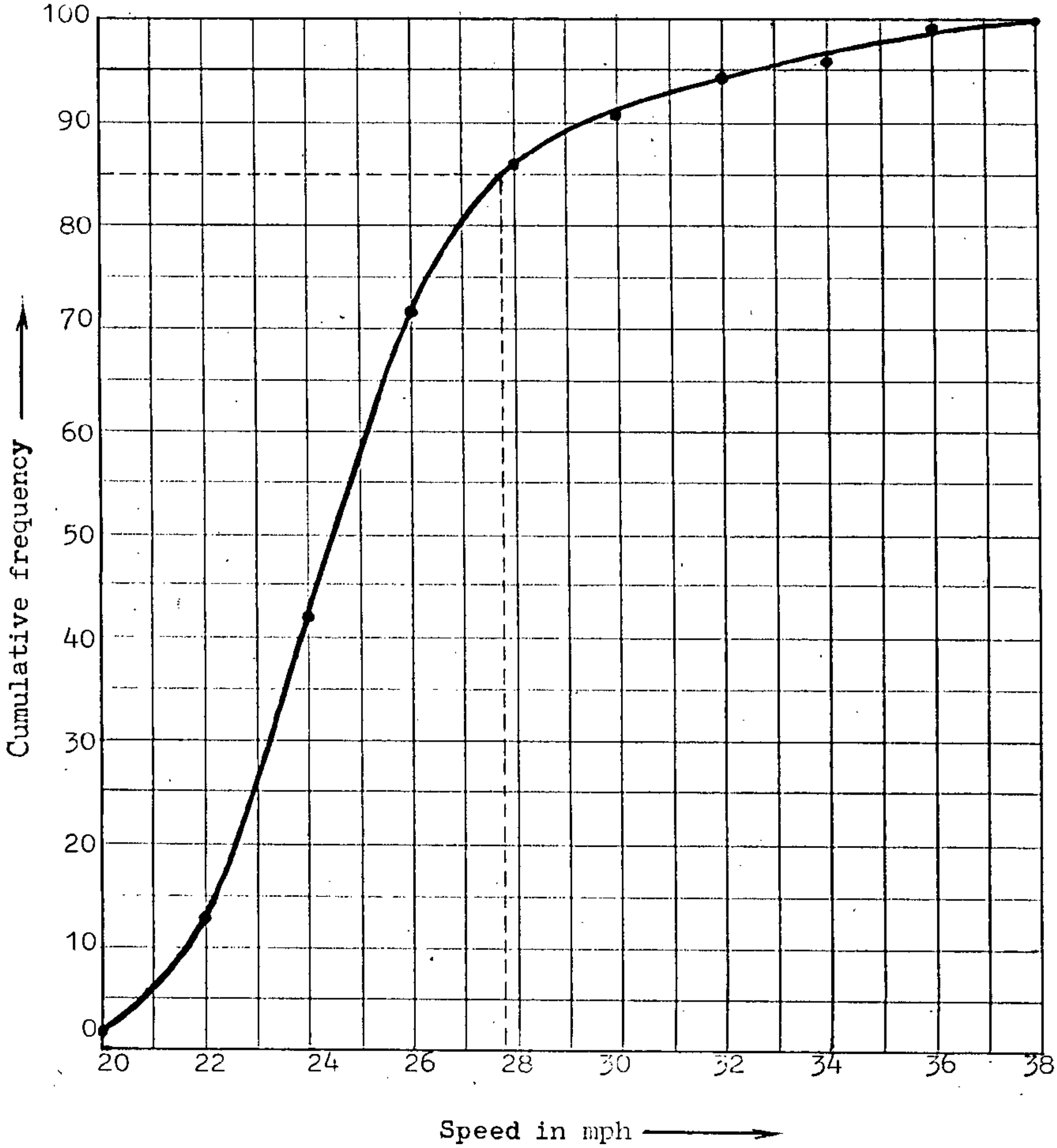
Calculation :

$$\text{mean speed}, \overline{X} = \frac{\Sigma fx}{n} = \frac{2610}{100} = 26.10 \text{ mph} = 42 \text{ kph.}$$

$$\text{standard deviation}, s = \sqrt{\left[\frac{\Sigma fx^2 - n\overline{x}^2}{n-1}\right]}$$

$$= \sqrt{\left[\frac{69{,}300 - 100X(26.10)^2}{100-1}\right]}$$

$$= 3.45$$



85th percentile speed = 27.90 mph=45 kph.

Fig. 3.3 Cumulative Speed Distribution Curve for Passenger Cars only.

TABLE 3.5 Elapsed Time and Corresponding Speed for the Mixed Traffic Stream

Trap length = 100 ft.
Travel time in second
Speed in mph (1500/22t)
Data collected on 27.07.1989.

Time	Speed	Time	Speed	Time	Speed	Time	Speed	Time	Speed
5.62	12.13	25.05	2.72	7.01	9.73	9.00	7.58	6.15	11.09
3.81	17.90	5.23	13.04	5.66	12.05	5.91	11.54	2.92	23.35
5.89	11.58	3.82	17.85	3.95	17.26	2.21	30.85	4.99	13.66
2.07	32.94	4.89	13.94	5.80	11.76	3.82	17.85	3.32	20.54
5.00	13.64	3.50	19.48	3.42	19.94	5.75	11.86	5.22	13.06
3.12	21.85	4.80	14.20	6.75	10.10	2.29	29.77	3.80	17.94
5.01	13.61	3.72	18.33	18.70	3.65	6.20	11.00	5.37	12.70
5.90	11.56	5.75	11.86	5.50	12.40	4.95	13.77	11.30	6.03
3.99	17.09	6.20	11.00	4.00	17.05	4.11	16.59	4.10	16.63
2.70	25.25	4.35	15.67	6.80	10.03	3.62	18.83	3.52	19.37
5.29	12.89	6.06	11.25	5.92	11.52	6.00	11.36	6.54	10.43
2.85	23.92	2.39	28.53	3.41	19.99	4.25	16.04	4.26	16.01
5.23	13.04	4.50	15.15	6.10	11.18	5.35	12.74	5.90	11.56
4.25	16.04	6.16	11.07	4.71	14.48	3.00	22.73	11.15	6.11
3.52	19.37	2.16	31.57	7.23	9.43	4.59	14.85	4.51	15.12
3.11	21.92	6.15	11.09	5.01	13.61	10.09	6.76	6.21	10.98
4.89	13.94	4.00	17.05	3.02	22.58	4.62	14.76	3.41	19.99
4.99	13.66	4.97	13.72	3.20	21.31	3.57	19.10	4.48	15.22
4.58	14.89	2.99	22.80	4.50	15.15	6.07	11.23	5.00	13.64
11.21	6.08	10.21	6.68	6.33	10.77	14.32	4.76	12.61	5.41
6.20	11.00	1.95	34.97	5.88	11.60	6.59	10.35	6.33	10.77
3.18	21.44	5.82	11.72	2.09	32.62	3.40	20.05	2.55	26.77
3.90	17.48	9.62	7.09	4.26	16.01	5.89	11.58	6.70	10.18
5.20	13.11	8.00	8.52	3.52	19.37	6.21	10.98	10.62	6.42

Table 3.6 Frequency Distribution Table for Mixed Traffic

Speed group in mph	Mean speed group X	Number of observation f	Percent of total observation	Cumulative percent of total observation	fx	fx^2
2.00 - 5.99	4.00	4	3	3	16	64
6.00 - 9.99	8.00	11	9	12	88	704
10.00-13.99	12.00	50	42	54	600	7200
14.00-17.99	16.00	25	21	75	400	6400
18.00-21.99	20.00	16	14	89	320	6400
22.00-25.99	24.00	5	4	93	120	2880
26.00-29.99	28.00	4	3	96	112	3136
30.00-33.99	32.00	4	3	99	128	4096
34.00-37.99	36.00	1	1	100	36	1296
Total	-	120		-	1820	32176

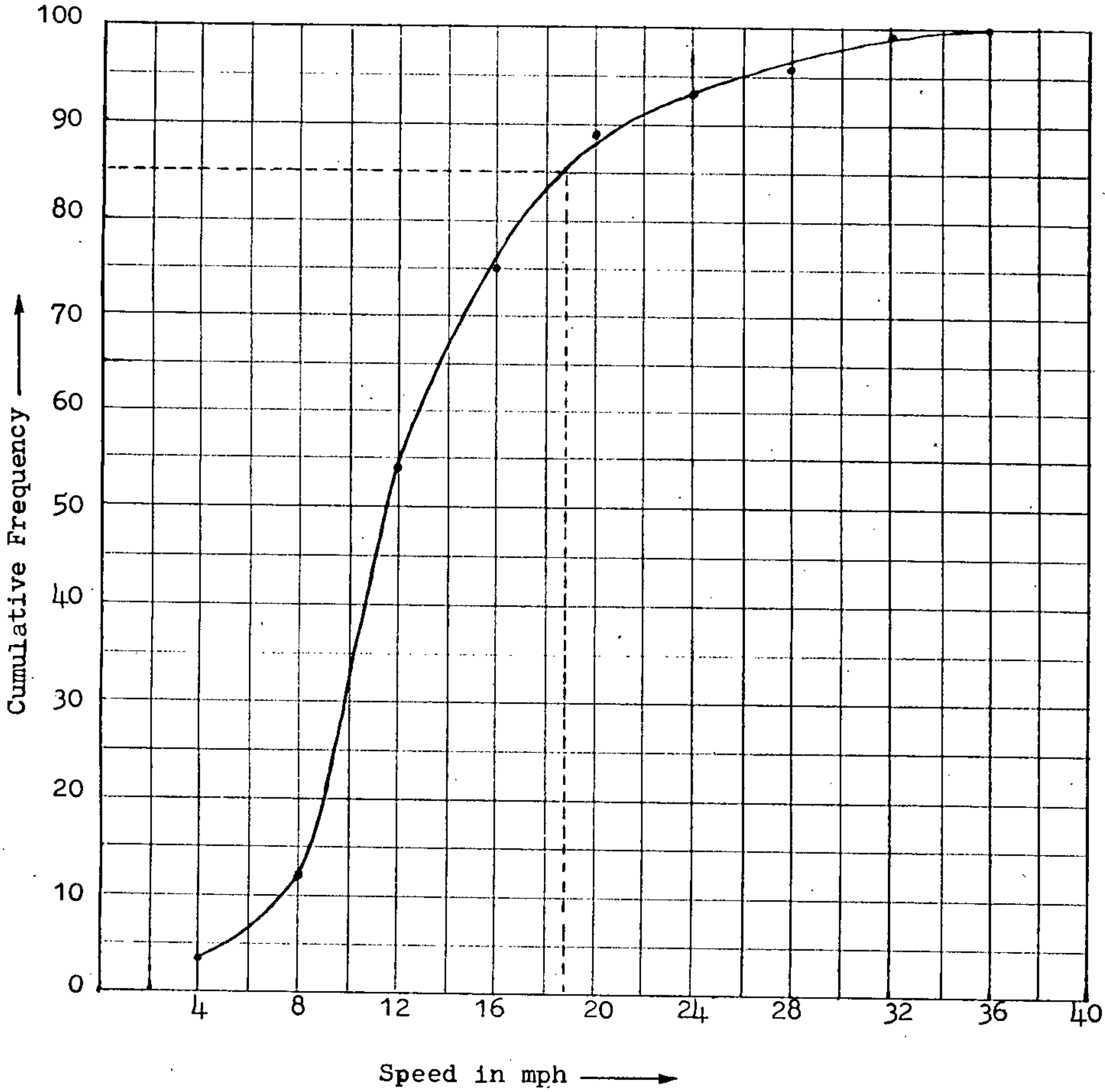
Calculation :

Mean Speed, $\bar{X} = \frac{\Sigma fx}{n} = \frac{1820}{120} = 15.17$ mph = 24.40 **kph.**

Standard deviation, $s = \sqrt{\left[\frac{\Sigma fx^2 - n\bar{x}^2}{n-1}\right]}$

$= \sqrt{\left[\frac{32176 - 120(15.17)^2}{120-1}\right]}$

$= 6.19$



85th percentile speed = 19 mph = 30.57 kph

Fig.3.4 Cumulative Speed Distribution Curve for Mixed Traffic.

It was observed that the presence of the slow moving vehicles in a high volume makes the whole traffic situation in the field a very complex one. The pattern of movement of the slow moving vehicles interferes with the normal flow of the fast moving vehicles, as a result it was very difficult to apply the method of measurement of delay in the field effectively. However, a trial was made to determine the delay by stopped time delay method on the north approach of the intersection under the mixed vehicle condition.

The sampling, along with a volume count during the same time observations were made. It estimated the vehicle-seconds of the stopped time delay. The sampling interval was selected 20 seconds, to make it an uneven sub-division of the existing cycle length of 105 seconds.(Pignataro,1973).

The sampling procedure containing 10 minutes of data is furnished in Appendix-D.

The investigation made in the field showed that the average delay per approach volume (only fast moving vehicles were considered) on the north approach was found 52 sec.and the average delay per stopped vehicles was 83 sec. It was also found that 62% of the approach vehicles stopped at the intersection.

3.11 An Overview

Collection of data pertaining to the investigation has been discussed in this chapter. Field ivestigation was the only source of data collection. Inventory of traffic signal, traffic volume count, spot speed study and delay study were made to collect data. These data are analyzed and some results were obtained subsequently.

CHAPTER - 4

DESIGN OF TRAFFIC SIGNAL AND DELAY STUDIES

4.1 Introduction

This chapter deals with the design of signal aspects based on the data collected from the field as described in the previous chapter. The design includes determination of a satisfactory cycle length and its splits defining traffic phases, amber time, all-red time, etc. The design also analyzes the delay per vehicle on the basis of formula using simulation techniques.

4.2 Design of Traffic Signal

4.2.1 Optimum Cycle Time

As out lined in chapter - 3, the design of traffic signal at an intersection involves the activities of ascertaining the number of traffic phases, phasing pattern and establishing the cycle length with its splits for the assignment of right-of-way alternately to different approaches, associated with other physical criteria of power operated illuminating device. The principal aspect of signal design is the determination of a satisfactory cycle length, especially for the pre-timed signal operation (Wohl & Martin, 1967).

The design of signal is greatly influenced by the concerned vehicular and pedestrian volume, accident hazards, co-ordinated movement and interruption to continuous traffic. Consideration were given to these influencing factors for the design of signal. Accordingly traffic volume study and spot speed study were conducted in the field.

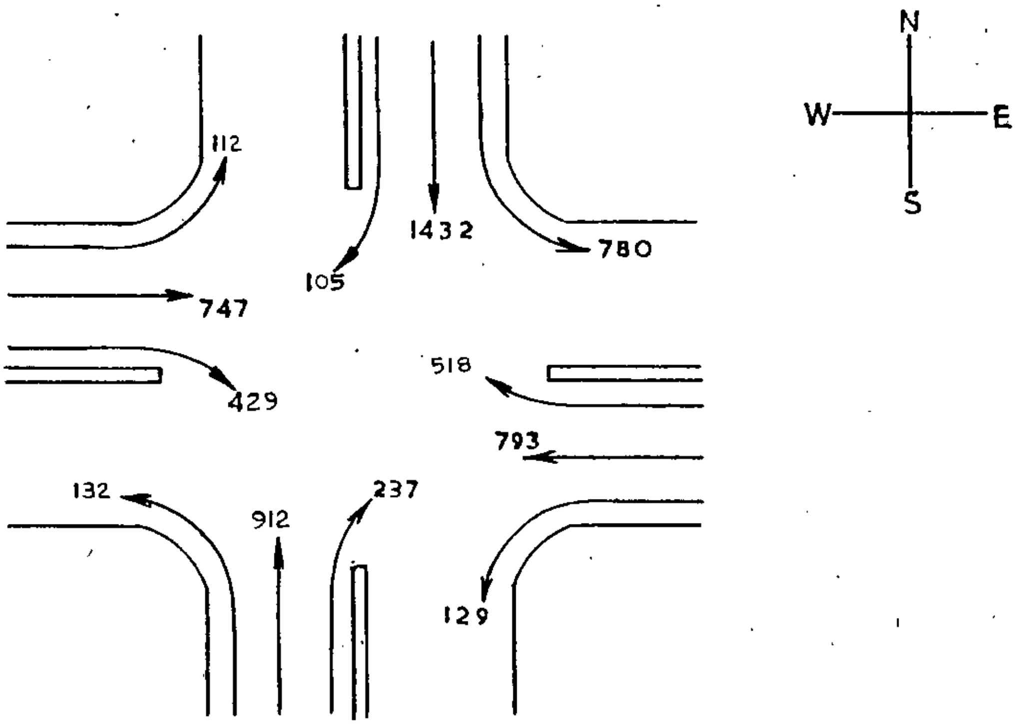
The traffic data obtained from the field reveals that the intersection is over-saturated because queues develop and exist for long periods within peak periods on almost all rights-of-way of the intersection and during this time it is not possible to clear a vehicle within the green time following his arrival(Pignataro, 1973).

As described in Chapter 3 the hourly volumes of traffic on all the approaches during 9:00 AM to 10:00 Am were taken as the peak hour volume which has been used as the approach flow for the purpose of designing signal timing. The peak hour volume data are extracted from Appendix A and are shown in Table 4.1. The corresponding peak flow diagram is shown in Fig. 4.1.

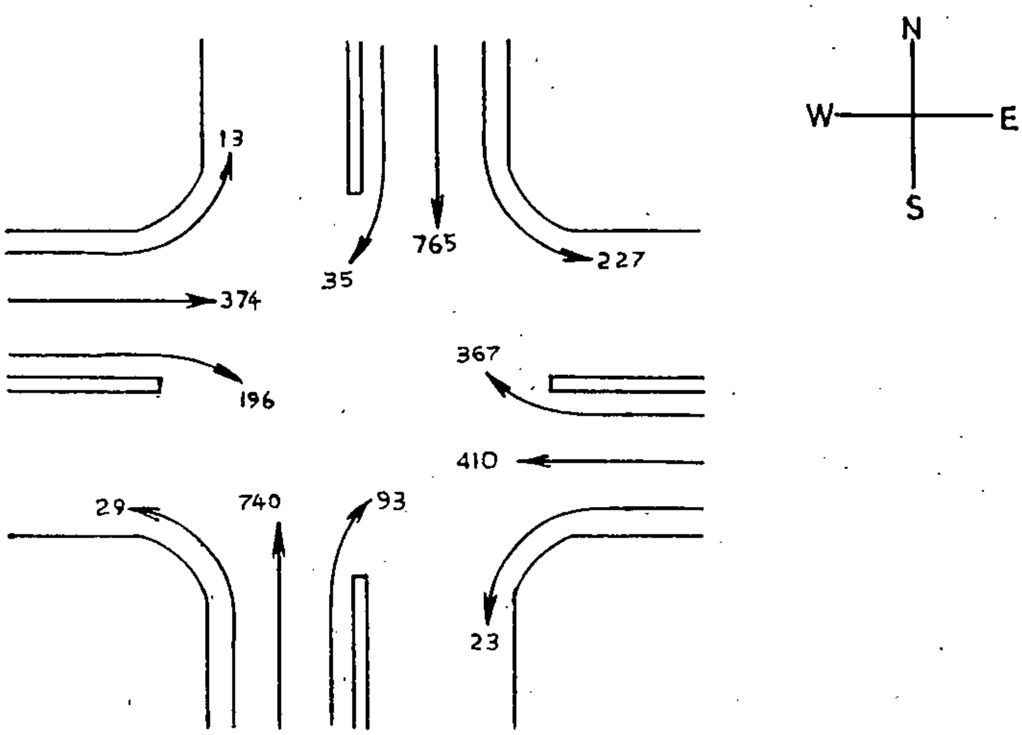
The traffic flow on the north-east narrow laned approach was found very less (only 5% of the volume on the north approach, and are mostly the rickshaws) compared to the other major approaches. Hence, consideration of this lane as a separate approach is omitted in the signal design. But the traffic on this approach should abide by the STOP-GO signal as provided for the north approach like present practice. But there should be adequate arrangement in this regard because the STOP-GO signal now placed on the north approach is not visible from the fifth approach.

Table 4.1 Observed Peak Hour Volume (Design Flow) in PCU at Maghbazar Intersection (Data Collected from Field Investigation).

Direction	Total Volume of all Types of Vehicles in PCU			Total Volume of Only Fast Moving Vehicles in PCU		
	Left	Straight	Right	Left	Straight	Right
N - S	780	1432	105	227	765	35
S - N	132	912	237	29	740	93
E - W	129	793	518	23	410	367
W - E	112	747	429	13	374	196



Mixed traffic.



Fast moving vehicles.

Fig. 4.1 Peak Flow Configuration

The huge volume of slow moving vehicles has made the intersection over saturated. The presence of the slow moving vehicles has necessiated the computation of signal timing based on only fast moving vehicles as well as on the mixed vehicles.

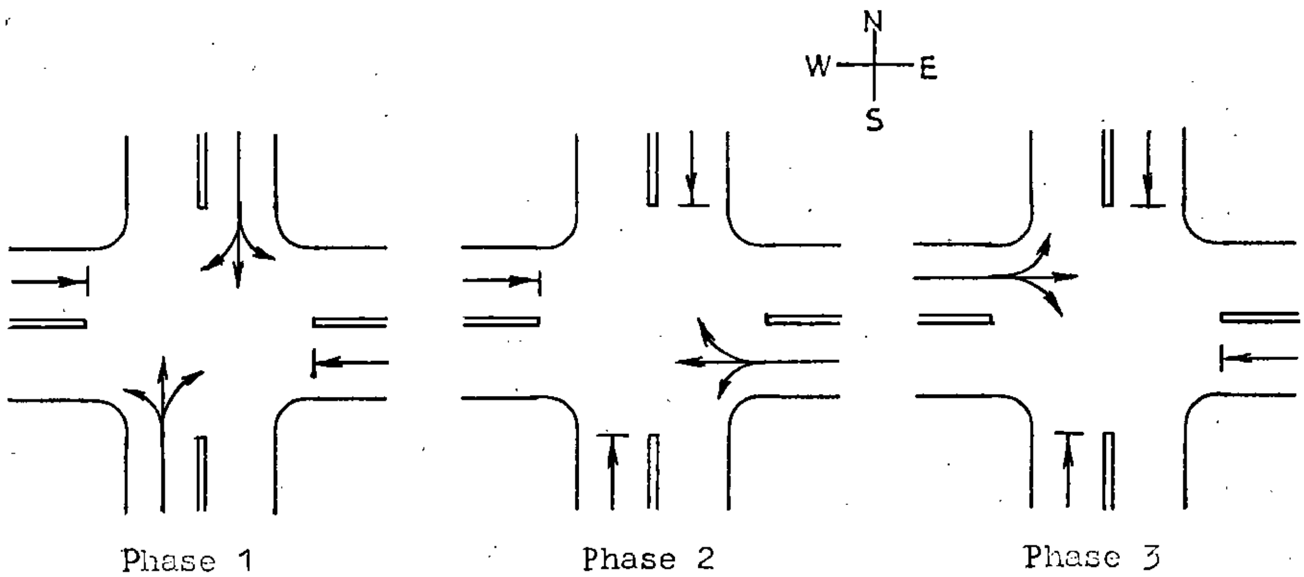
In computing cycle length Webster's (1958) approximate formula was used. This approximate formula determines the optimum cycle length in term of minimum delay. In fact optimum cycle lengths should permit traffic to pass through the intersection with a minimim of dealy. The singal timing calculations based on peak flow for fast moving vehicles and for mixed vehicles are shown in Appendices B & C respectively. The results obtained from calculation are furnished in Fig. 4.3 and Fig. 4.4 and the existing cycle length with its splits and traffic phases is shown in Fig.4.2.

4.2.2 Signal Phasings

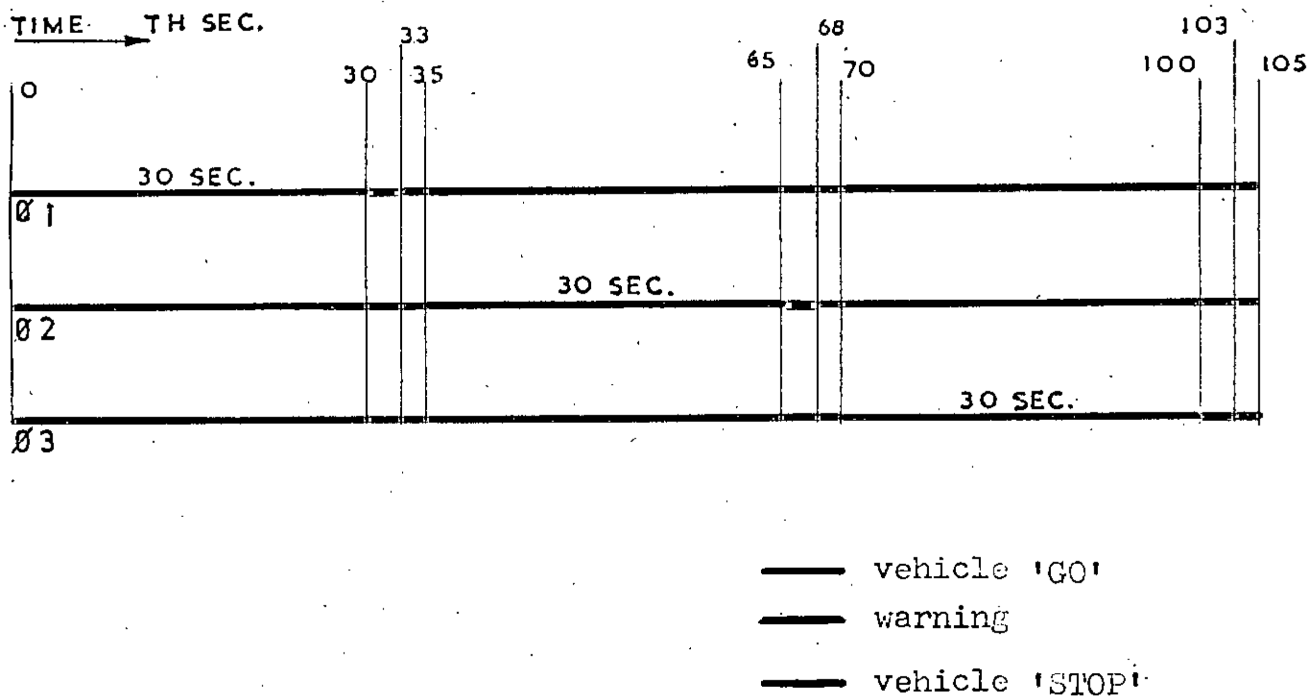
Based on the flow demand of the fast moving vehicles the cycle length has been designed to be consisting of 73 sec. with a 3 phase system (Fig.4.3). To minimize critical conflicts due to turning movements 3 phase system has been selected for satisfactory performance.

The existing cycle length now in operation, consists of 105 sec. with a 3 phase system (Fig.4.2). It is notable that the designed cycle length implies a reduction of time of 30% of the existing cycle length.

The cycle length based on all types of vehicles, inclusive of rickshaws, has been designed to be consisting of 96 sec. with a 3 phase system under proposed improvement of the intersection (Fig.4.4).



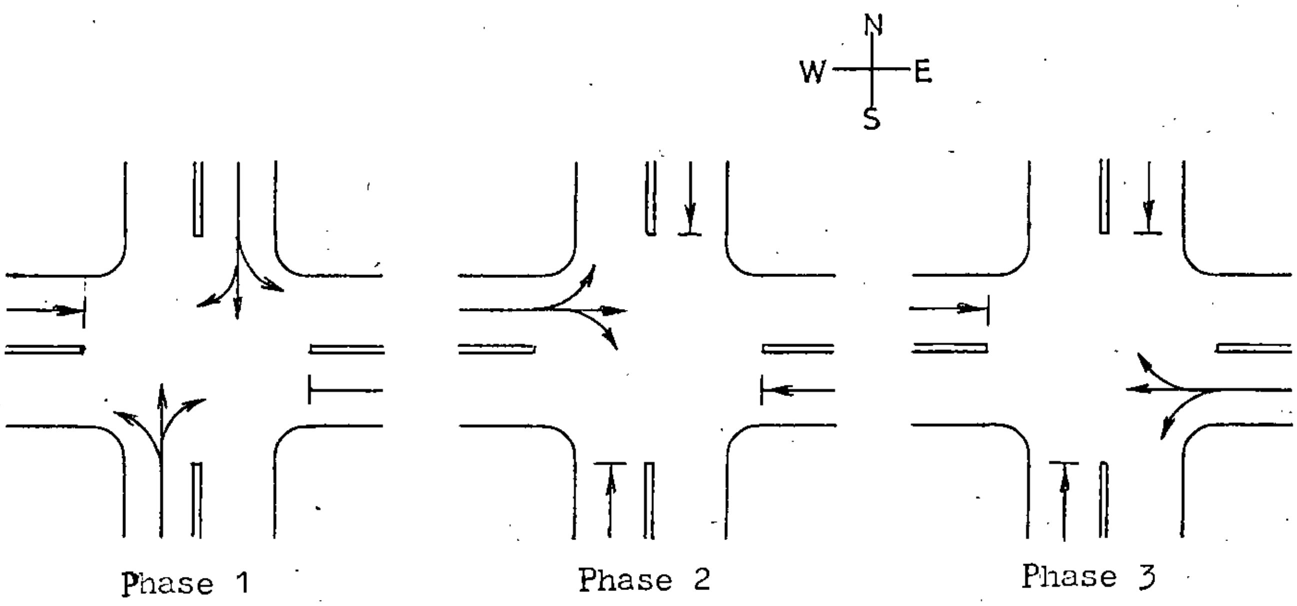
a) Traffic phasing.



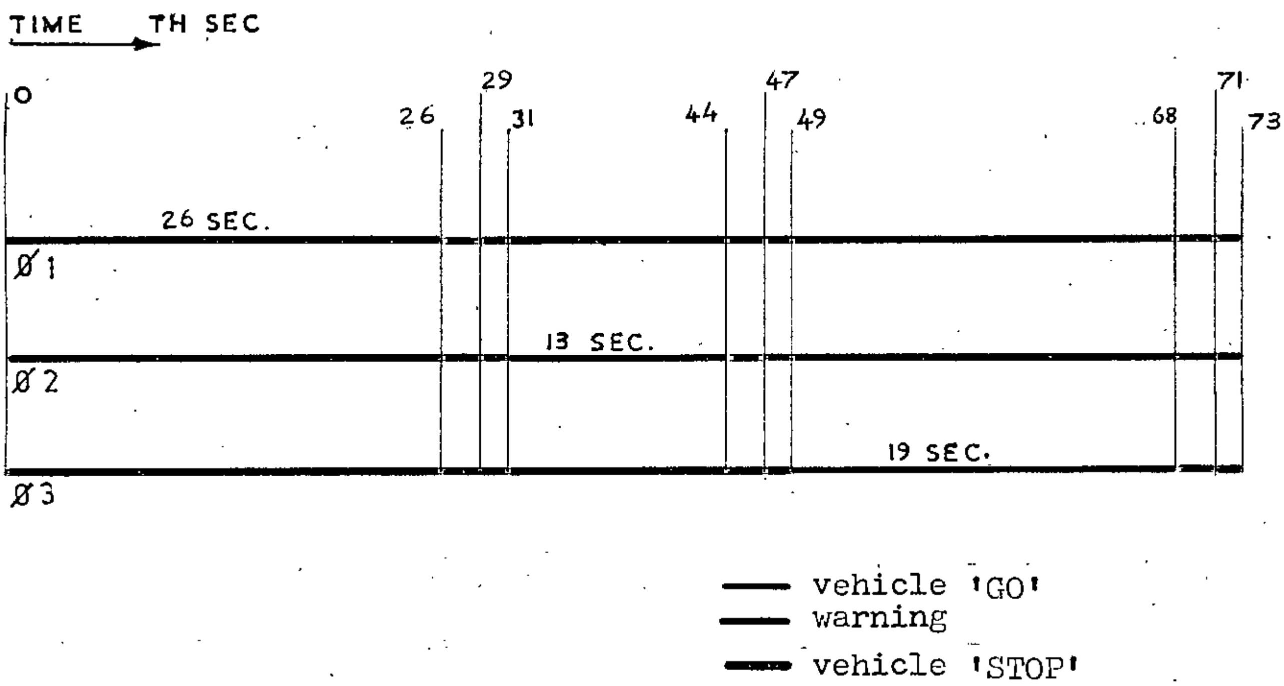
b) Time split of cycle.

Fig. 4.2 Existing Timing Diagram of Traffic Signal with Phasing Pattern.

Source : Dhaka Metropolitan Police.

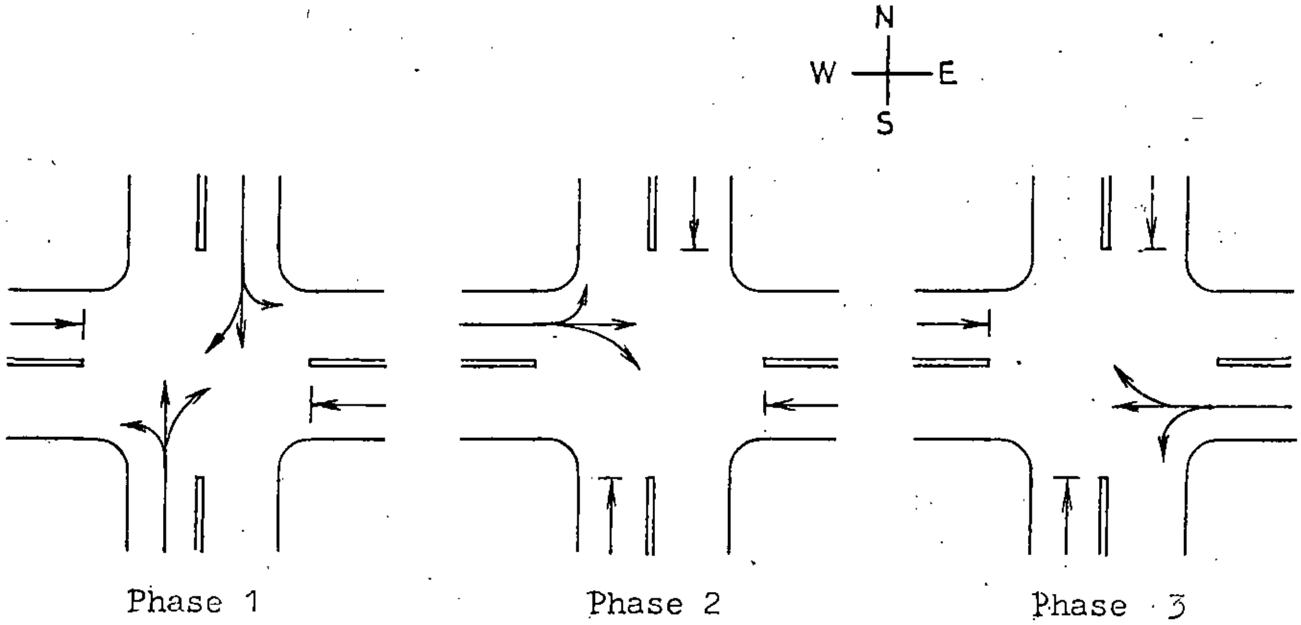


a) Traffic phasing.



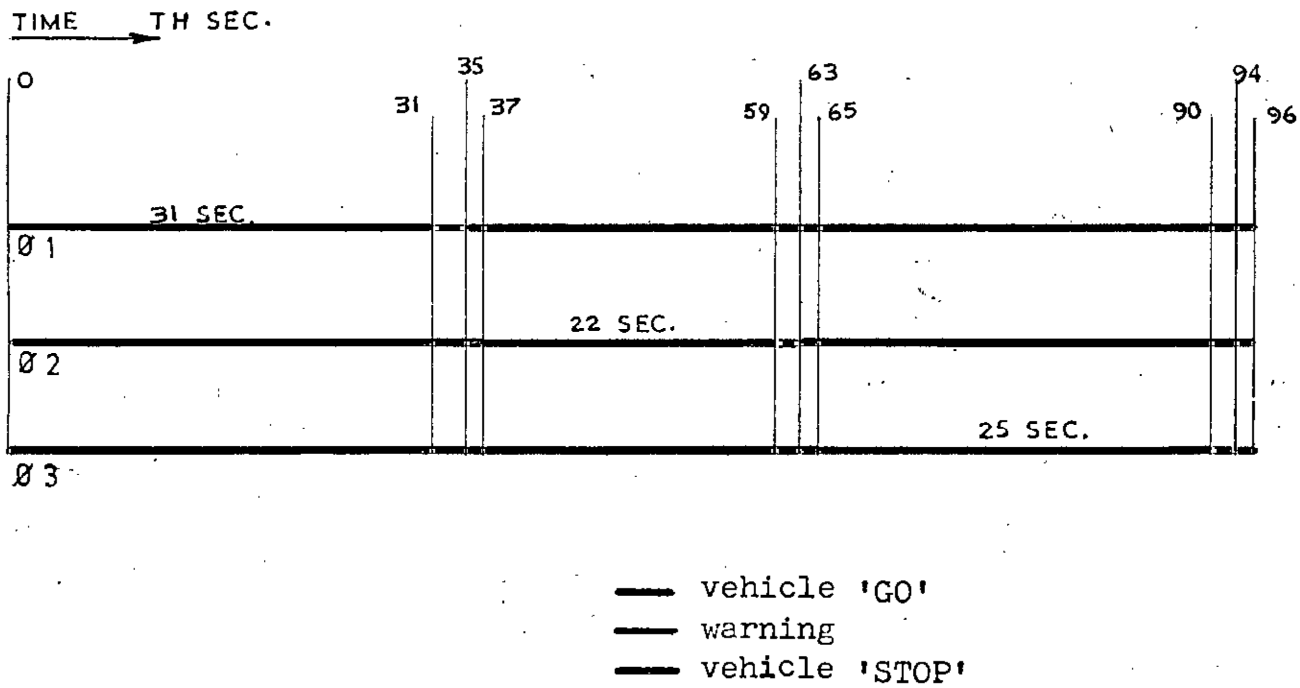
b) Time split of cycle

Fig. 4.3 Designed Timing Digagram of Traffic Signal with Phasing Pattern, Based on Fast Moving Vehicles with Existing Road Width.



Movement of left turners free through left lane.

a) Traffic phasing.



b) Time split of cycle.

Fig. 4.4 Designed Timing Diagram of Traffic Signal with Phasing Pattern, Based on All Types of Vehicles with Proposed Approach of Three Lanes & Exclusive Left Turning Lane.

4.3 Determination of Delay at the Intersection

Delay at the intersection is a major factor in the analysis of congestion. There are several factors which should be considered in evaluating the efficiency and effectiveness of the intersection traffic control, but intersection delay is of primary importance.

The analysis of traffic delay at the intersection is made in Appendix D, where an average delay per approach volume is determined on the basis of recommended formula for the designed signal timing. A sampling procedure for measuring delay in the field under existing traffic and roadway condition is also shown in Appendix-D.

The average delay per approach vehicle was found out to be 24 sec., 21 sec., 37 sec., and 29 sec. for north, south, west and east approaches respectively, considering the fast moving vehicles only.

The stopped delay per approach vehicle was investigated in the field. The delay on the north approach was measured to be 52 sec.

It is observed that the designed cycle length reduces the average delay per vehicle by about 54% as found on the north approach. Therefore, the designed cycle length has some good effect.

A comparison of the delays found out by computation is shown in Table 4.2. The possible queue that can be produced on the approches as a result of the stopping of the approaching vehicles during red phases and awaiting green phase is analyzed and determined on the basis of formula in Appendix-D.

4.4 An Overview

This chapter has dealt with the designing of an optimum cycle time on the basis of flow demand measured on different approaches by field investigations. The design procedure adopted the approximate formula given by Webster. The delay analysis in respect of average delay per approach vehicles has also been made in this chapter, which also included consideration of possible queue lengths in terms of number of vehicles. The designed cycle time suggests a time reduction of 30% of the existing cycle length and shows a reduction of 54% in average delay per vehicle

Table 4.2 Average Delay at the Intersection for Different Traffic Conditions

Approach	Average delay per approach volume, sec.		
	For only fast moving vehicles with existing approach widths	For the mixed vehicles with proposed improvement of approaches	For fast moving vehicles.
North	24	33	52
South	21	28	Not measured
West	37	40	Not measured
East	29	37	Not measured
Remarks	determined on the basis of formula and designed cycle.	determined on the basis of formula and designed cycle.	measured in the field *

* The delay measured in the field is stopped time delay only and does not include time losses due to deceleration and acceleration.

CHAPTER - 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The aim of this study was to investigate the traffic signal performance in the Metropolitan City of Dhaka, Moghbazar intersection in particular. The study also aimed at looking into other features regarding controlling and regulating the traffic. Accordingly, the study involved the technique of collecting traffic data and investigates other factors influencing traffic control. All the data were collected by field investigation.

In the study it was found that about 45% of the total traffic volume were rickshaws. This huge volume has a great implication in making the existing approaches of the intersection over saturated for which widening of roads has become essential.

It is notable that the design formula is valid for only fast moving vehicles, yet in this study it was used for the mixed vehicles (including the slow moving vehicles) because there is no appropriate design for the mixed vehicles.

During study it was found that there is lack of adequate traffic signs and markings and pedestrian facilities for crossisng roads.

The cycle length determined on the basis of this study was found shorter (30%) than the existing one and the lengths of trafic phases are based on flow demand which should be more effective in controlling the traffic with reduced delay. The average delay calculated out on the basis of the proposed cycle time was also found to be shorter(54%) than the average delay measured in the field.

The study infers that the traffic control devices should be set on proper traffic engineering ground to achieve their goal.

5.2 Recommendations

The preceding observations and findings lead to a clear view of the performance of traffic signal at Maghbazar intersection of Dhaka Metropolitan area, associated with other relevant traffic engineering applications. This study also brings into light some important problems and drawbacks of the existing traffic control devices at the intersection, which needs careful and effective manipulation. However, the following recommendations are made based of the findings of the study.

1) The approaches of the intersection should be widened immediately to accommodate the approach volumes because most of the time the approch volumes exceed the saturation flows of the approaches.

2) Separate left turning lanes should be provided on all the legs (preferably on the north and the south approaches) so that turners may not interfere the other directional traffic. Similarly right turning lanes may be provided particularly on the east and west approches.

3) Providing separate left turning lanes the left turners may be allowed to move all the time freely irrespective of whatever signal phase is alloted to an approach.

4) The disorderly movement of the slow moving vehicles is an inconsistency to the movement of the fast moving vehicles. Hence, adequate measure should be taken fairly to remove the slow moving vehicles, certainly by some sort of replacement, and minor streets may be kept open to the slow moving vehicles.

5) There should be proper traffic marking on the pavement for defining the lanes and to guide, warn and regulate the traffic.

6) Traffic marking should be made on all four legs of the intersection to provide cross walks for the pedestrians. Traffic signs supporting the pedestrian cross walk should also be installed at proper places as safety measure.

7) The fifth leg of narrow street on the north corner of the intersection should be eliminated and the street may be adjoined to its convenient major street after proper survey.

8) A traffic sign indicating bus stop is found placed on the east approach within about 100 ft. from the intersection without any parking lane. No bus stop should be made in the vicinity of the intersection. Bus stops should be far beyond the intersection with proper traffic sign and placed properly.

9) The rickshaw pullers have the trend to park their vehciels hither and thither at random especially near the intersection. It should be restrained extensively to increase the capacity.

10) The designed cycle length based on fast moving vehicles, consisting of 73 seconds with 3 traffic phases is recommended for use with the traffic signal at the intersection (Referred to fig.4.3).

11) One traffic sign indicating the restriction of entry of the commercial vehicles into the south leg (placed at the origin of the departing approach) is found placed improperly. There should be similar signs on each of the other three approaches to regulate those vehicles in advance.

12) The disregarding attitude of the rickshaw pullers and most of the drivers of the commercial vehicles, to the traffic control devices should be corrected thoroughly to attain overall success of the control devices and safety measures.

13) The study further recommends to carry out comprehensive investigation of the traffic control devices at other intersections as well as road network of Dhaka city and make signal co-ordination to increase capacity of overall road net-wrok.

APPENDIX - A

Table A.1 Observed Directional Traffic Volume (in PCU) for Total Vehicles (All Types of Vehicles) at Maghbazar Intersection on 23-07-1989.

Direction	Time of observation		Traffic volume in PCU			
	From	To	Total	Left	Straight	Right
N-S	8:00 AM	9:00 AM	2325	721	1415	189*
	9:00 AM	10:00 AM	2317	780*	1432*	105
	10:00 AM	11:00 AM	1926	698	1139	89
	1:00 PM	2:00 PM	1425	412	938	75
	2:00 PM	3:00 PM	1962	602	1239	121
	4:00 PM	5:00 PM	1495	509	887	99
	5:00 PM	6:00 PM	1183	487	603	93
S-N	8:00 AM	9:00 AM	1141	112	828	201
	9:00 AM	10:00 AM	1281	132	912	237
	10:00 AM	11:00 AM	1355	205	993	157
	1:00 PM	2:00 PM	1577	209	1153	215
	2:00 PM	3:00 PM	1829	293*	1289	247*
	4:00 PM	5:00 PM	1322	235	892	195
	5:00 PM	6:00 PM	1118	202	777	137
E-W	8:00 AM	9:00 AM	1402	98	689	615
	9:00 AM	10:00 AM	1440	129*	793*	518
	10:00 AM	11:00 AM	1528	111	790	627
	1:00 PM	2:00 PM	1591	112	699	780
	2:00 PM	3:00 PM	1691	121	775	795
	4:00 PM	5:00 PM	1558	89	680	789
	5:00 PM	6:00 PM	1481	78	691	712
W-E	8:00 AM	9:00 AM	1227	104	689	434*
	9:00 AM	10:00 AM	1288	112	747*	429
	10:00 Am	11:00 AM	1238	97	739	402
	1:00 PM	2:00 PM	1054	90	653	311
	2:00 PM	3:00 PM	1230	127	703	400
	4:00 PM	5:00 PM	756	92	453	211
	5:00 PM	6:00 PM	885	65	470	350

* Directional peak flow.

Table A-2 Observed Directional Traffic Volume(in PCE) for only Fast Moving Vehicles at Moghbazar Intersection on 23-07-1989.

Direction	Time of observation From	To	Traffic volume in PCU Total	Left	Straight	Right
N-S	8:00 AM	9:00 AM	1028	203	780*	45
	9:00 AM	10:00 AM	1032	227	765	35
	10:00 AM	11:00 AM	889	232*	633	29
	1:00 PM	2:00 PM	586	189	379	18
	2:00 PM	3:00 PM	798	175	535	38
	4:00 PM	5:00 PM	921	169	733	19
	5:00 PM	6:00 PM	897	187	690	20
S-N	8:00 AM	9:00 AM	734	37	600	97
	9:00 AM	10:00 AM	862	29	740	93
	10:00 AM	11:00 AM	777	43	653	81
	1:00 PM	2:00 PM	946	90	791	65
	2:00 PM	3:00 PM	1017	96	813*	101
	4:00 PM	5:00 PM	718	72	589	57
	5:00 PM	6:00 PM	689	25	637	27
E-W	8:00 AM	9:00 AM	688	17	359	312
	9:00 AM	10:00 AM	800	23	410	367
	10:00 AM	11:00 AM	894	8	433*	453
	1:00 PM	2:00 PM	769	17	395	357
	2:00 PM	3:00 PM	965	21	430	514*
	4:00 PM	5:00 PM	926	12	403	511
	5:00 PM	6:00 PM	798	6	312	480
W-E	8:00 AM	9:00 AM	515	14	301	200
	9:00 AM	10:00 AM	583	13	374	196
	10:00 AM	11:00 AM	537	9	375*	153
	1:00 PM	2:00 PM	311	8	253	50
	2:00 PM	3:00 PM	434	15*	301	118
	4:00 PM	5:00 PM	365	11	265	89
	5:00 PM	6:00 PM	380	9	265	106

* Directional peak flows.

Table A.3 Observed Directional Traffic Volume(in PCU)for the Total Vehicles(All Types of Vehicles)at Maghbazar Intersection on 26-07-1989.

Direction	Time of observation		Traffic volume in PCU			
	From	to	Total	Left	Straight	Right
N-S	8:00 AM	9:00 AM	2230	753	1290	187
	2:00 PM	3:00 PM	2051	702	1169	180
	5:00 PM	6:00 PM	1793	690	1008	95
S-N	8:00 AM	9:00 AM	1617	200	1205	212
	2:00 PM	3:00 PM	1828	287	1301*	240
	5:00 PM	6:00 PM	1533	219	1112	203
E-W	8:00 AM	9:00 AM	1513	111	605	797
	2:00 PM	3:00 PM	1599	120	689	790
	5:00 PM	6:00 PM	1476	95	580	801*
W-E	8:00 AM	9:00 AM	1168	129	612	427
	2:00 PM	3:00 PM	1232	139*	680	413
	5:00 PM	6:00 PM	920	117	453	350

* Directional peak flow.

Table A.4 Observed Directional Traffic Volume (in PCU) for only Fast Moving Vehicles at Maghbazar Intersection on 26.7.89.

Direction	Time of observation		Traffic Volume in PCU			
	From	to	Total	Left	Straight	Right
N-S	8:00 AM	9:00 AM	1033	215	771	47*
	2:00 PM	3:00 PM	729	190	493	46
	5:00 PM	6:00 PM	709	173	506	25
S-N	8:00 AM	9:00 AM	730	15	656	59
	2:00 PM	3:00 PM	1001	99*	801	108*
	5:00 PM	6:00 PM	600	60	453	87
E-W	8:00 AM	9:00 AM	770	18	402	350
	2:00 PM	3:00 PM	935	25*	421	489
	5:00 PM	6:00 PM	798	8	310	480
W-E	8:00 AM	9:00 AM	557	6	350	201*
	2:00 PM	3:00 PM	566	11	360	195
	5:00 PM	6:00 PM	378	66	321	51

* Directional peak flow.

Table A.5 Vehicle Composition of the Peak Flow.

	Direction of flow		Total number of vehicles.	Vehicle groups: Bus, minibus.	Truck	Passenger car, Jeep, Taxi, Microbus	Auto-rickshaw Three wheeler Tampo	Motor cycle Bi-cycle	Rickshaw Van	Hand pushed cart	Passenger car unit (PCU)
All Types of Vehicles	N-S	L	519	50	4	48	63	24	330	-	780
		S	1245	2	-	416	199	131	497	-	1432
		R	135	1	3	15	5	10	101	-	189
	S-N	L	256	-	1	64	42	40	109	-	293
		S	1145	6	1	342	279	117	400	-	1301
		R	175	1	-	53	43	28	89	-	247
	E-W	L	43	-	-	17	15	5	63	-	129
		S	609	31	11	79	101	97	288	2	793
		R	551	56	29	96	106	52	212	-	801
	W-E	L	104	-	2	6	13	12	70	1	139
		S	585	25	7	85	142	65	261	-	747
		R	351	3	-	101	39	27	181	-	434
Only Fast Moving Vehicle	N-S	L	151	42	6	33	40	30	-	-	232
		S	849	3	-	493	203	150	-	-	780
		R	43	2	3	19	7	12	-	-	47
	S-N	L	109	-	-	58	31	20	-	-	99
		S	891	5	1	412	293	180	-	-	813
		R	116	-	-	61	39	16	-	-	108
	E-W	L	25	-	1	13	7	4	-	-	25
		S	357	50	17	67	107	116	-	-	433
		R	409	47	31	105	124	102	-	-	514
	W-E	L	15	-	2	2	3	8	-	-	15
		S	352	29	7	90	128	98	-	-	375
		R	215	1	-	117	65	32	-	-	201

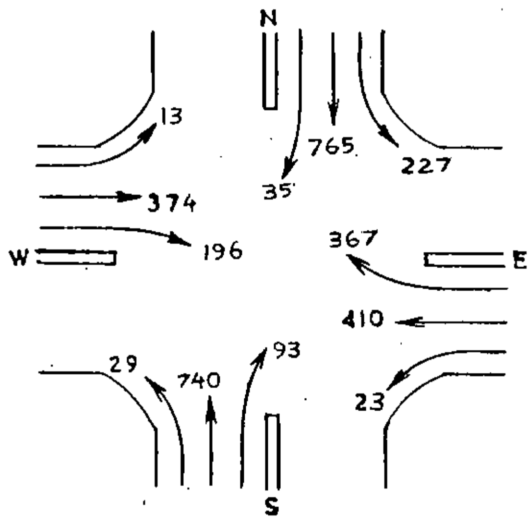
Appendix-B

Determination of Signal Timing Based on Fast Moving Vehicles

In the determination of signal timing the following steps were carried out.

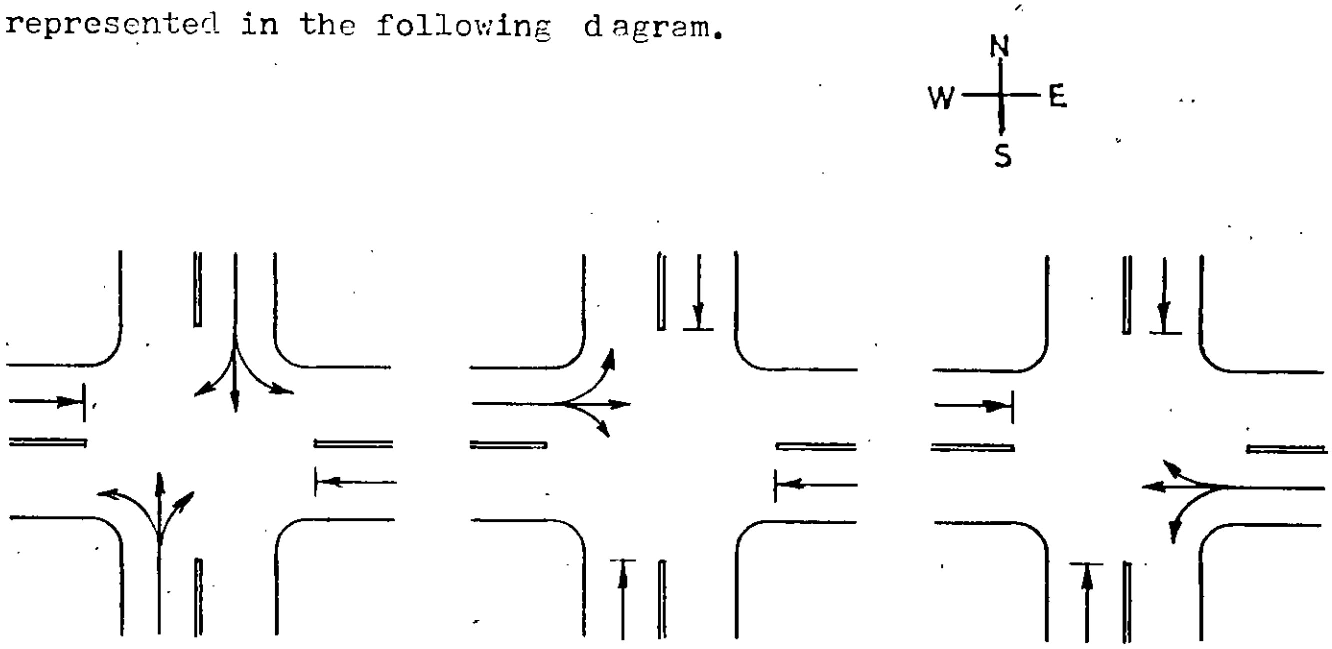
B.1 Peak Flow Diagram

On the basis of peak hour flow the following peak flow digagram was established.



B.2 Selection of Traffic Phase

A three traffic phase system of signal control was assumed suitable for the intersection. The assumed traffic phases are represented in the following dagram.



The assumed three phase system was considered suitable on the basis of the following reasons.

i) Three phases instead of two phases reduce more conflicts.

ii) The numbers of right turning traffic from the east and the west legs are appreciably high.

iii) In absence of separate right turning lane no exclusively right turning phase has been taken into consideration.

iv) Pedestrain crossing is more facilitated in three phase system than a two phase system.

B.3 Determination of Vehicle Clearance Interval

As per recommendation of the 'Traffic Engineering Handbook (1965) the following formula was used to determine the clearance interval which incorporates a safe stopping distance.

$$A = t + \frac{1}{2}\frac{v}{a} + \frac{w+l}{v}$$

Where A= Clearance interval, sec.

t= Perception-reaction time, sec.

v= Approach speed of clearing vehicle in fps.

a= Deceleration rate of clearing vehicle in fps.

w= Intersection width, ft.

l= Length of vehicle, ft.

From field investigation and obtaining suggested data the following values were used,

t= 1 sec

v= 45 kph (obtained from spot speed study)

= 41.07 fps.

a= 15 fps^2

w= 80ft (for N-S direction)

= 65ft (for E-W direction)

l= 20 ft

A_{N-S} = 4.80 sec. A_{E-W} = 4.44 sec.

5 sec. 5 sec.

B.4 Consideration for Pedestrian Crossings

Pedestrians, crossing roads, were not found immoderate. Hence no exclusive phase for pedestrian crossing was assumed and the pedestrian crossing time was allowed to run concurrently with the vehicle 'GO' period which is usual practice. Thus, pedestrain crossing times determine the minimum red phase for the street being crossed.

For phase 1, minimum red time, $R_1 = 5 + \frac{52}{3.5} = 19.86$ sec. where 5 sec. is the assumed pedestrian start up time, the E-W street is 52 ft wide and pedestrains are assumed to walk at 3.5 fps.

For phase-II, min. red time, $R_2 = 19.86$ sec.

For phase-III, min. red time, $R_3 = 5 + \frac{48}{3.5} = 18.71$ sec.

B.5 Determination of Optimum Cycle Length

Using Webster (1958) method optimum cycle time, $C_o = \frac{1.5L+5}{1-Y}$

Where, L= total lost time

$= nl + R$

$= \Sigma(I-A) + \Sigma l$

Where, n= no. of phases

l= average lost time per phase(usually starting delay only), excluding any all-red intervals.

R= total time during each cycle when all signals display red simultaneously.

I= Intergreen period (closing amber+all red time)

A= amber time

$Y = \sum_{1}^{n} y_i$

Where y_i= ratio of approach flow to saturation flow at ith phase.

$= \frac{q_i}{s_i}$

To minimize the impatience among drivers awaiting the signal change, which may result in starting through the intersection before the green indication appears, it was supposed to be satisfactory to supplement the normal amber interval with a short all-red interval to permit clearance of the intersection before cross traffic is released.

Thus, the vehicle clearance interval of 5 sec. was subdivided into an amber time of 3 sec. and an all-red time of 2 sec. thereafter.

Average lost time per phase was assumed to be 2 sec. as suggested.

Saturation flow was assumed 3600 pcu per leg considering two laned roadway on each approach.

In calculating approach volumes the effects of turning traffic (Transportation & Traffic Engineering Handbook, 1982) were considered as follows.

1) Effect of right turning traffic

a) for the case without separate right turning phase

1 right turn=1.75 times straight ahead vehicle.

b) for case with separate right turning phase no correction is required.

II) Effect of left turning traffic:- If left turning vehicles form appreciably more than 10% of the traffic a correction would be made for the excess over 10% by assuming 1 left turner=1.25 straight ahead vehicles.

Thus, the approach flow & the saturation flow were calculated as follows :

$q_{1(N)} = 765+1.75(35)+103+1.25(124)=1084$ pcu.

$q_{1(S)} = 740+1.75(93)+29=932$ pcu.

$q_{2(W)} = 374+196+13=583$ pcu

$q_{3(E)} = 410+367+23=800$ pcu

$S_1 = S_2 = S_3 = 3600$ pcu.

$y_{1(N)} = \frac{1084}{3600} = 0.301$ $y_{1(S)} = \frac{932}{3600} = 0.259$

$y_{2(W)} = 0.162$ $y_{3(E)} = 0.222$

$Y = \Sigma y_i = 0.301+0.162+0.222=0.685$

$L = nl+R = 3x2+3x2 = 12$ sec.

$$C_o = \frac{1.5X12+5}{1-0.685} = 73 \text{ sec.}$$

B.6 Splitting the Cycle:

Effective green time for each phase was computed as follows:-

$$g_{ei} = \frac{y_i}{Y}(C_o - L)$$

where g_{ei} = effective green time for ith phase.

$$g_{e1} = \frac{0.301}{0.685}(73-12) = 27 \text{ sec.}$$

$$g_{e2} = \frac{0.16}{0.685}(73-12) = 14 \text{ sec.}$$

$$g_{e3} = \frac{0.222}{0.685}(73-12) = 20 \text{ sec.}$$

Actual green time for each phase was computed as follows:

$$G_{ai} = g_{ei} + 1 - A$$

where G_{ai} = actual green time for ith phase.

$G_{a1} = 27+2-3 = 26$ sec.

$G_{a2} = 13$ sec.

$G_{a3} = 19$ sec.

B.7 Check for Pedestrian Requirement

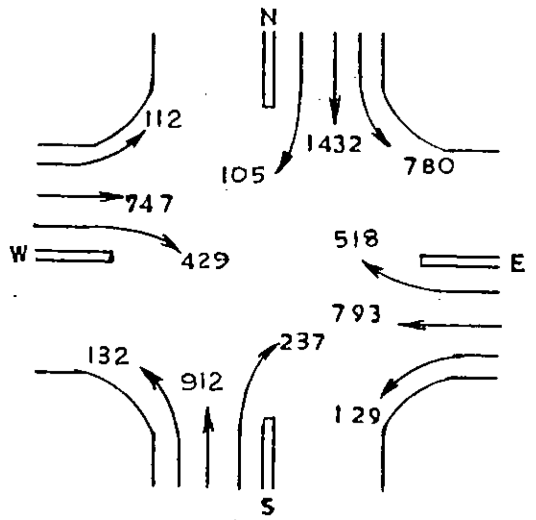
From the actual green times the minimum red time(of all the three phases)

$=(13+19+2X3+3X2)$ sec $= 44$ sec. $>$ 18.71 sec.

$\therefore$ the optimum cycle time and its splits are o.k.

APPENDIX-C

Determination of Signal Timing Based on Mixed Vehicles



Peak flow configuration

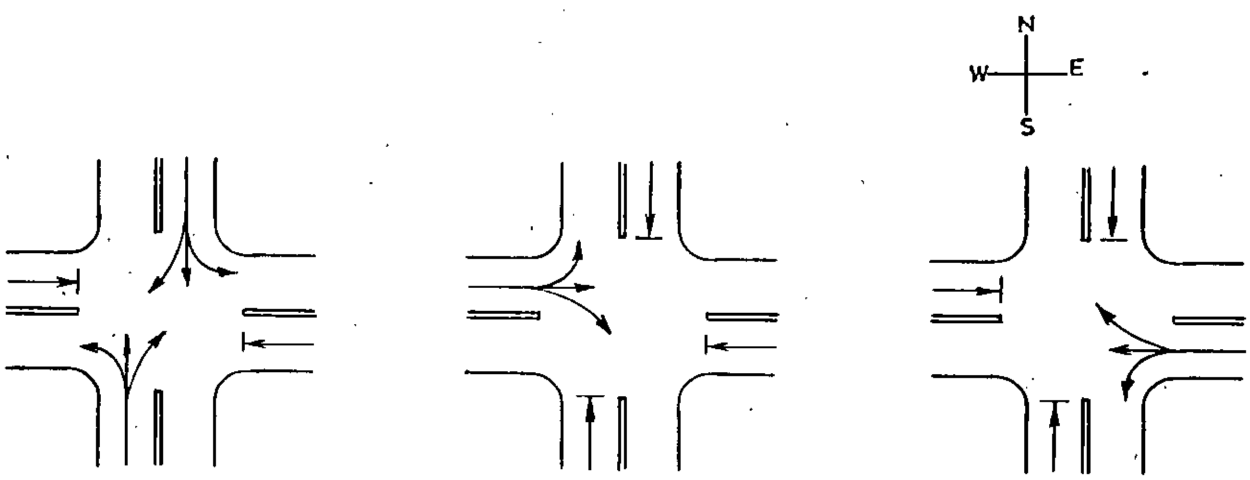
In determining the signal timing for mixed traffic the following improvement is proposed first.

1) There will be at least three lanes on each approach.
2) There will be seperate left turning lane excluding the above mentioned three lanes.
3) Left turners will be allowed to move all the time.

On the basis of the proposed improvement the following calculation is made.

C.I Traffic Phase :

The following three phase system is selected suitable.



Phase 1 Phase 2 Phase 3

C.2 Vehicle Clearance Interval

$$A = t + \tfrac{1}{2}\,\frac{v}{a} + \frac{w+l}{v}$$

From field investigation and obtained suggested data (Traffic Engineering Hand Book, 1965),

t= 1 sec.

v= 30.57 kph = 27.87 fps (From Fig.3.4)

a= 15* fps^2

w= 90 ft (for N-S direction)

= 75 ft (for E-W direction)

l= 20*ft

$$A_{N-S} = 1 + \tfrac{1}{2} X \frac{27.87}{15} + \frac{90+20}{27.87} = 5.88 \text{ sec} = 6 \text{ sec.}$$

$$A_{E-W} = 1 + \tfrac{1}{2} X \frac{27.87}{15} + \frac{75+20}{27.87} = 5.34 \text{ sec} = 6 \text{ sec.}$$

The vehicle clearance interval of 6 second is divided into an amber of 4 second and an all-red interval of 2 second thereafter.

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* These values are suggested for only fast moving vehicles and not for the mixed traffic conditions. Therefore, the actual vehicle clearance time may deviate slightly from the calculated value, which has been ignored in this study due to the lack of such data for the mixed traffic condition.

C.3 Pedestrian Crossing

Minimum red time for pedestrian crossing,

$R = 5+\frac{76}{3.5} = 27$ sec.

C.4 Optimum Cycle Time

$y_{1(N)} = \frac{q_1}{s_1} = \frac{1432+1.75x105}{5400} = 0.299$

$y_{1(S)} = \frac{912+1.75X237}{5400} = 0.246$

$y_{2(W)} = 0.218 \qquad y_{3(E)} = 0.243$

$Y=\Sigma y_i = 0.760$

L= 3x2+3x2=12 sec.

Optimum cycle time,

$C_o = \frac{1.5x12+5}{1-0.760} = 96$ sec.

C.5 Cycle Split

effective green time, $g_{e1} = 33$ sec.
$g_{e2} = 24$ sec.
$g_{e3} = 27$ sec.

actual green time, $g_{a1} = 31$ sec.
$g_{a2} = 22$ sec.
$g_{a3} = 25$ sec.

C.6 Check for Pedestrian Requirement

Minimum red time = 22 +25+2x4+3x2 = 61 sec. > 27 sec. so, o.k.

APPENDIX - D

ANALYSIS OF DELAY AT THE INTERSECTION

D.I Determination of Delay

Delay at the intersection was determined on the basis of Webster's delay analysis technique using the designed signal timing. Delay analysis was made for different approaches. The computation for determining the delay and queue lengths are shown hereinafter.

The following formula(Wohl & Martin, 1967) derived by Webster (1962) was used.

$$d = \frac{c(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)} - 0.65\left(\frac{c}{q^2}\right)^{\frac{1}{3}} x^{(2+5\lambda)}$$

where, d = average delay per vehicle on the particular approach.

c = cycle time.

λ = proportion of the cycle which is effectively green for the phase under consideration.

$= \frac{g_e}{c}$

g_e = effective green time

q = flow on the particular approach under consideration in vehicle/sec.

x = degree of saturation

$= \frac{q}{s}$

s = saturation flow

D.2 Delay Computation for only Fast Moving Vehicles under Existing Approach Widths.

D.2.1 Delay on the North Approach

$c = 73$ sec.

$\lambda = \frac{27}{73} = 0.370$

$q = 1084/3600$ v/s $= 0.301$ v/s

$x = \frac{1084}{0.370 \times 3600} = 0.814$

$$d = \frac{73(1-0.370)^2}{2(1-0.370\times 0.814)} + \frac{(0.814)^2}{2\times 0.301(1-0.814)}$$

$$-0.65\left(\frac{73}{0.301^2}\right)^{\frac{1}{3}} (0.814)^{(2+5\times 0.370)}$$

$= 24$ sec.

D.2.2 Delay on the South Approach

c= 73 sec. λ = 0.370 q= 0.259 v/s. x= 0.700

$$d=\frac{73(1-0.370)^2}{2(1-0.370X0.700)}+\frac{(0.700)^2}{2(0.259)(1-0.700)}-0.65\left(\frac{73}{0.30^2}\right)^{\frac{1}{3}}(0.700)^{(2+5x0.370)}$$

= 21 sec.

D.2.3 Delay on the West Approach

c= 73 sec. λ = 0.192 q= 0.162 v/s x=0.844

$$d=\frac{73(1-0.192)^2}{2(1-0.192x0.844)}+\frac{(0.844)^2}{2x0.162(1-0.844)}-0.65\left(\frac{73}{0.162^2}\right)^{\frac{1}{3}}(0.844)^{(2+5x0.192)}$$

= 37 sec.

D.2.4 Delay on the East Approach

c= 73 sec. λ = 0.274 q=0.222 v/s x=0.811

$$d=\frac{73(1-0.274)^2}{2(1-0.274x0.811)}+\frac{(0.811)^2}{2x0.222(1-0.811)}-0.65\left(\frac{73}{0.222^2}\right)^{\frac{1}{3}}(0.811)^{(2+5x0.274)}$$

= 29 sec.

D.3 Delay Computation for the Mixed Vehicle with Proposed Improvement of Approaches.

D.3.1 Delay on the North Approach

c= 96 sec.

$\lambda=\frac{33}{96}$= 0.344 q= 0.449 v/s x=0.870

$$d=\frac{96(1-0.344)^2}{2(1-0.344x0.870)}+\frac{(0.870)^2}{2x0.449(1-0.870)}-0.65\left(\frac{96}{0.449^2}\right)^{\frac{1}{3}}(0.870)^{(2+5x0.344)}$$

= 33 sec.

D.3.2 Delay on the South Approach

$c = 96$ sec. $\lambda = 0.344$ $q = 0.369$ v/s $x = 0.714$

$$d = \frac{96(1-0.344)^2}{2(1-0.344\times0.714)} + \frac{(0.714)^2}{2(0.369)(1-0.714)} - 0.65\left(\frac{96}{0.369^2}\right)^{\frac{1}{3}}(0.714)^{(2+5\times0.344)}$$

$= 28$ sec.

D.3.3 Delay on the West Approach

$c = 96$ sec. $\lambda = 0.250$ $q = 0.327$ v/s $x = 0.871$

$$d = \frac{96(1-0.250)^2}{2(1-0.250\times0.871)} + \frac{(0.871)^2}{2\times0.327(1-0.871)} - 0.65\left(\frac{96}{0.327^2}\right)^{\frac{1}{3}}(0.871)^{(2+5\times0.250)}$$

$= 40$ sec.

D.3.4 Delay on the East Approach

$c = 96$ sec. $\lambda = 0.281$ $q = 0.364$ v/s $x = 0.864$

$$d = \frac{96(1-0.281)^2}{2(1-0.281\times0.864)} + \frac{(0.864)^2}{2\times0.364(1-0.864)} - 0.65\left(\frac{96}{0.364^2}\right)^{\frac{1}{3}}(0.864)^{(2+5\times0.281)}$$

$= 37$ sec.

D.4 Delay of the Vehicles, Investigated in the Field

TABLE D.1 Sampling Procedure for Intersection Delay (10 Minutes of Data on the North Approach Collected on 9-8-89) (For Fast Moving Vehicles)

Time(minute starting at)	Total number of vehicles stopped in the approach at time.			Approach volume	
	+0 sec.	+20 sec.	+40 sec.	Number stopping	Number not stopping
8:00 AM	0	3	9	7	5
8:01 "	18	19	3	15	2
8:02 "	15	16	0	3	0
8:03 "	8	10	18	7	15
8:04 "	8	12	18	15	2
8:05 "	10	4	6	2	4
8:06 "	10	12	14	7	5
8:07 "	19	10	4	3	7
8:08 "	16	19	0	11	0
8:09 "	13	17	15	10	9
8:10 "	18	0	14	6	3
Sub-total	135	122	101	86	52
TOTAL	358			138	

D.4.1 Computation of Delay

Average delay per stopped vehicle $= \dfrac{\text{total delay}}{\text{number of stopping vehicles}}$

$$= \frac{358X20}{86} \text{ sec.} = 83.26 \text{ sec.}$$

Average delay per approach vehicle $= \dfrac{358X20}{138}$ sec. $= 51.88$ sec.

Percent of vehicles stopped $= \dfrac{86}{138}$ X100= 62% .

D.5 Computation of Queue Lengths

The following approximate formula for determining the average queue length interms of number of vehicles at the begining of the green period for the approach, as developed by Webster(1958) was used for the purpose of computation of queue length.

$$n = \frac{qR}{2} + qd \quad \text{or,} \quad n = qR \quad \text{whichever is the larger.}$$

where, n= average number of vehicles per lane using waiting in queue at the start of green period.

q= actual rate of flow, vehicle/Sec.

R= red phase in sec.

d= average delay per vehicle, in sec.

The above formula only permits determination of the average or that queue which will be exceeded during approximately 5 cycles out of every 10 (Wohl & Martin,1967).

The queue lengths for all approach are analized below .

D.6 Computation of Queue Lengths for only Fast Moving Vehicles.

D.6.1 Queue Length on the North Approach

q= 0.151 v/s

R= 44 sec.

d=12 sec.

$$n = \frac{0.151 \times 44}{2} + 0.151 \times 12 = 5,$$

or, $n = 0.151 \times 44 = 7$, larger

so 7 vehicles.

D.6.2 Queue Length on the South Approach

q= 0.130 v/s R= 44 sec. d= 10.5 sec.

n= 6 vehicles.

D.6.3 Queue Length on the West Approach

q= 0.081 v/s   R= 57 sec.   d= 18.5 sec.

n= 5 vehicles.

D.6.4 Queue Length on the East Approach

q= 0.111 v/s   R=51 sec.   d= 14.5 sec.

n= 6 vehicles.

D.7 Computation of Queue Lengths for the Mixed Vehicles

D.7.1 Queue Length on the North Approach

q= 0.150 v/s   R=61 sec.   d= 11 sec.

n= 10 vehicles.

D.7.2 Queue Length on the South Approach

q=0.123 v/s   R= 61 sec.   d= 93.3 sec.

n= 8 vehicles.

D.7.3 Queue Length on the West Approach

q=0.109 v/s   R=70 sec.   d= 13.3 sec.

n= 8 vehicles.

D.7.4 Queue Length on the East Approach

q=0.121 v/s   R=67 sec.   d=12.33 sec.

n= 9 vehicles.

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