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THE UNIVERSITY OF ALBERTA

A METHODOLOGY TO IMPROVE TRAFFIC REGULATIONS ON AN URBAN SIGNALIZED NETWORK

by

© EUGENE SACUTO

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

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THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled A METHODOLOGY TO IMPROVE TRAFFIC REGULATIONS ON AN URBAN SIGNALIZED NETWORK submitted by EUGENE SACUTO in partial fulfilment of the requirements for the degree of Master of Science.

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ABSTRACT

The aim of this study is to implement a methodology to improve traffic regulations on an urban signalized network. It is assumed that passengers use two different types of vehicles (the private automobile and the transit bus).

In a first part, a computer program is written to evaluate the total passenger delay at signalized intersections, the traffic and the timing plan being given.

The second part of this thesis investigates through specific cases the effect of banning a left-turn, changing the mode split and creating reserved bus lanes.

This research work has been made with the idea that passenger delay is a more significant indicator of the quality of a network than vehicular delay.

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CHAPTER I

INTRODUCTION

The early 1920's in North America saw many transportation developments. This was the time of the rise of the automobile industry when the transportation problems in our cities became more significant. This was also the time when, mainly for safety reasons, the first traffic signals were developed in the United States ; the entire idea was to give protection to pedestrians. Just a few years later the motorists started to worry about the time lost at signalized intersections and the idea of actuation or allowing for the actual presence of traffic did come into use in the latter part of the 1920's (12). Later, during the 1930's, with the growth of the automobile traffic, solutions had to be found to the ever-increasing traffic problem in the United States. The traffic light which was originally a safety device for pedestrians became at the same time a tool to increase the capacity of a network. Simultaneously, the first synchronized systems came out on the market in order to reduce the delays imposed on the drivers. Bridging the years, we reach the 1970's when saturation of the traffic networks in most of the North American and European cities is such that traffic lights just cannot solve the problem anymore. The only feasible answer which allows a more efficient movement of people is mass transportation. As a matter of fact, the idea is not very new. Saltzman (16) states that: "Prior to 1912, electric street cars and rapid transit railways were prosperous monopolies that carried almost all urban passenger trips.". Originally, these public transportation companies had been promoted to make profit; but for many reasons, namely: the increasing costs , the fixed fares, the lack of capital, some regulatory problems, and the antitrust legislation, transit ridership has, for the past fifty years, slowly and predictably declined, causing many bankruptcies. Today, we must redress the tendency to face the traffic problem in our cities. For this purpose, we have to attract the individual automobile driver in order to make him shift to public means of transportation. One way to do it, without imposing anything on him, is to increase the quality of service of the transit system and consequently of the network.

1. STATEMENT OF THE PROBLEM

The aim of this study is to develop a method which could help the traffic engineer to find out some regulations which would increase the quality of service of any medium-sized urban network. The method consists in a computer simulation program which allows us to evaluate for any particular traffic regulation the total delay of people at the signalized intersections. By traffic regulation we mean, for example, a particular timing of the signals, the prohibition of a left turn, the closing of a street, the creation of an exclusive bus lane or a similar regulation. In fact, this thesis investigates the relationship between the total delay of people at signalized intersections and the network and traffic characteristics.

2. LIMITATIONS

While this study could be used for any medium-sized urban area (up to 10 to 20 signalized intersections), at any time of the day, it has only been tested with Edmonton data and has been done with the peak hour in mind. Many parameters allow a calibration to almost any kind of problem, however, at off-peak hours, random variations of traffic are such that the simulation model can be considered as too rough to give usable results. Two other very basic assumptions which, in a way, restrict the study, should always be kept in mind. First of all, the traffic flow is supposed to be constituted only by two types of vehicles: the private automobile and the transit bus. All other vehicles are converted into one of these two types with regard to their size and to the number of people they are supposed to serve. Secondly, trying to increase the quality of service of the network, we are interested in reducing the total delay of people at signalized intersections (TDPSI). For this purpose, we assume that every minute someone waits is equally important whether he just arrived at the intersection or he has already been waiting for half-an-hour. Only the total delay interests us. In other words, a reduction of the TDPSI could eventually increase considerably the delay at one particular intersection. In this case, it is up to the traffic engineer to decide whether or not to improve the network in such a way that the community interest does not inflict too much on some particular people.

CHAPTER II

REVIEW OF PREVIOUS WORK

Traffic problems are generally very complicated because they involve a large number of parameters which all vary almost randomly. Therefore, any analytical study becomes rapidly too complex to be handled manually. Until 1956, most of the solutions which were proposed came out rather empirically. Even though not sophisticated, they were good enough to help a lot; the only thing was that the trial-and-error method was not the most elegant and was very time consuming. D.L. Gerlough (7) was about the first to realize that: "The electronic computer offers promise of becoming a powerful tool in studying the flow of traffic...". He built a very microscopic model where vehicles were represented by binary digits, and simulated, in this way, the traffic on a free-way. Although this approach is henceforth out of date, it presents a historical interest and thus, should be noted.

About 10 years later, the idea of using the computer had come its way and simulation became a more common practice. This was the time when F.A. Wagner Jr. and D.L. Gerlough (13) made a traffic network model called TRANS, for the purpose of evaluating the effect of traffic signal settings on traffic flow in a street network. TRANS had a more macroscopic character and, therefore, individual vehicles were not identifiable anymore; they were handled in groups and positions were accounted in terms of relatively coarse elements of the roadway. Although operational for research purposes and having been successfully utilized in preliminary studies, TRANS was not a practical tool for the traffic engineer. The following step was made at about the same time by S.B. Miller (10) who set up a criterion in order to measure the performance of signal settings. He simulated the traffic on a computer which calculated the number of stops and the trip delay as well. The criterion used was a weighted sum of stops and delays. When a specific number was required, he used one stop as equivalent to 15 seconds of trip delay. After him, some people began to use this criterion more and more extensively and two considerable programs of research were carried out simultaneously, in North America under the name SIGOP, and in Great Britain under the name TR[illegible]T. As we could expect it, we shall find many similarities in these two studies which we are going to examine in detail hereafter.

1. SIGOP

The SIGOP traffic SIGnal OPtimization program is a method for determining the cycle times, phase splits and offsets (see glossary in Appendix D) in a signalized roadway network (3) (9) (21).

METHOD: The cycle time, which must be the same for all the signals in the network, is given by the user. The phase splits are determined by distributing the available green to each stage in proportion of the total flow on that stage or, alternatively, in proportion to the flow ratio (average arrival to saturation flow) on that stage. The distribution of the green is subject to minimum green restraints. The calculation of the offsets is the major optimizing feature of SIGOP. First, the optimum differences of offsets for the signals at each end of each link are calculated for each link. Then, the optimization procedure minimizes a weighted sum over all links of square differences between the actual and optimum offsets. Usually the weighting factors are the flows on the links, but other arbitrary values of weights may be used. Since the procedure finds the local minimum adjacent to the starting settings, the calculation is carried out for a number of randomly chosen starting points and the best result is selected. This optimization procedure is supposed to provide the best offsets for a given cycle time. To compare different cycle times, a very simple simulation is carried out to estimate delays and stops in the network for every cycle time.

FIELD TESTING: The simulation used to find the best cycle time evaluates each link independently and under the assumption of unimpeded flow; therefore, the results are not well suited to before-and-after comparisons of settings. However, if one takes the point of view that the object is to produce signal settings, the performance of SIGOP in actual field testing has been reasonably good. Good results have been reported in Washington, D.C., Baltimore, New Haven, Louisville, Kansas City and in Glasgow, Scotland.

2. TRANSYT

TRANSYT (TRAffic Network StudY Tool) is a method for determining fixed-time traffic signal settings which has been developed by the Road Research Laboratory in England (14), (21) (19). The aim of this method is to minimize a linear function of delays and stops of the vehicles.

TRANSYT TRAFFIC MODEL: A computerized traffic model is used to evaluate the different plans. It calculates a linear function of delays and stops of the vehicles: the performance index. The data given to carry out this calculation refer to : firstly, the network, which is represented by nodes interconnected by links; secondly, the traffic, that is to say the flow which wants to get through the network and its behaviour through the nodes and the links; and thirdly, the timing plan of all the signalized intersections, which means the cycle length, the split and the offset for each node.

TRANSYT OPTIMIZATION PROCEDURE: The procedure used consists in altering the offsets of the signals in order to minimize the performance index which is computed at every iteration. It helps evaluating the improvement, if any, and finding the next trial offset by comparing results with the previous runs. One at a time, the offset of each signal in turn is adjusted in this way: the signals are dealt in an order which has to be specified. To reduce the possibility of obtaining a poor local optimum, both large and small step sizes are used in the alteration of the offset for the iterative procedure . The amount of calculation required to compute the performance index is reduced by noting that, at every iteration, the delays and the stops only change on the links which lead to the signalized intersection which is currently optimized. In order to simplify the optimization procedure and to reduce the computer time, it is possible to group some intersections by setting their offsets between each other.

TRANSYT COMPUTER PROGRAM: On the 16,384-word core store computer of the Road Research Laboratory, a network up to 165 links can be studied. For the normal sequence, in which both offsets and stage durations are optimized, the solution time is given by:

TS = 8*N*L*10exp(-3) minutes, where

N is the number of nodes and,

L the number of links.

ACCURACY OF TRANSYT: Although it is difficult to check results with field data, the TRANSYT traffic model has given accurate predictions (within 5%) on an experiment conducted by the Road Research Laboratory on Cromwell Road in London. On the other hand, the optimization procedure has been proved to be very little affected by the initial signal settings. The TRANSYT method has been used to prepare signal timing plans for Glasgow and West London.

3. CURRENT RESEARCH

Independently of these two extensive studies in the field of area traffic control, some recent work has been done also by N. Gartner (5) to reduce the vehicular delay on signal-controlled links. This microscopic analysis which takes into account the phenomenon of platoon dispersion on a link after a signalized intersection has been done with the idea of using the results in a dynamic process to regulate the various flows. Concerning the delays, we should also mention the studies carried out by F.F. Allsop (1) in the field of fixed-time traffic signals; his theoretical approach summarizes helpfully many practical results in this field.

Up to this point, most of the research efforts which have been reported concern the reduction of the vehicular delay through the timing plan of the signals. Other traffic regulations like reserving a bus lane or closing a street have been studied in many particular cases (2) (4) (11) but practically no method is available for the traffic engineer to measure their efficiency. Another point should be noted: among all the simulation studies which have been reported to this date, the only delay which is always used is the vehicular delay. That is to say that in the case of mixed traffic, the delay of a transit vehicle is weighted as 2.0 or 2.5 private cars, which holds true in terms of capacity of the roadway. In terms of delay, a transit vehicle should be weighted as 10 or 20 private cars since it carries generally 10 to 20 times as many people as an automobile. The only person who, firstly, used this notion of passenger delay instead of vehicular delay was D.I.Robertson in a fifth version of TRANSYT which was tested out in Glasgow, Scotland, by the end of 1972. No final report has been issued as yet about it, but, it has been predicted through TRANSYT version 5 which minimizes the total passenger delay, that the new timing plan would get a 17 percent reduction in bus delays achieved at the expense of a 2 to 4 percent increase in delay to the other traffic. Bus journey speeds have also been increased by about 6 percent (15).

CHAPTEP III

THEORY

APPROACH TO THE PROBLEM

Traffic signals commonly imply delay since they are used to share a carriageway between conflicting traffic streams. In the case of a heavy flow, this inconvenience is generally impossible to avoid, even with a sophisticated synchronization of the lights; volumes on the network are such that the only purpose of the signal is to stop some vehicles in order to let other ones get through the intersection. In this case, the vehicle behaviour is illustrated by G.F. Newell (13) on Figure III-1:

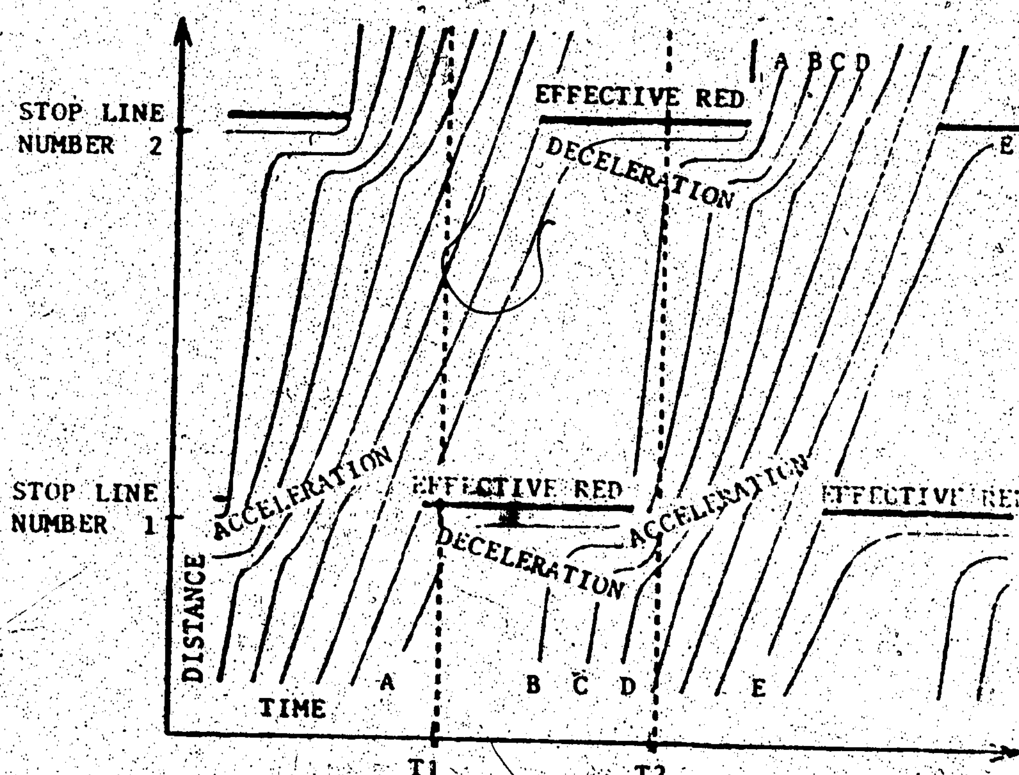

FIGURE III-1:

TRAFFIC FLOW PATTERNS ON HEAVY CONDITIONS

Every curve represents the flow of a bunch of cars through two consecutive traffic signals. Some current behaviours appearing on the plot (A,B,C,D and F) are described hereafter:

"A" stops first at the intersection number 1, then, leaves the queue to reach the intersection number 2 in the second position; therefore, "A" must stop behind the stop line number 2.

"B" and "C" must also stop and line up at the intersection number 1. They are respectively in second and third position behind "A". "C" has to wait less than "B" which has to wait less than "A". The first vehicles which are stopped by a red light, being delayed the most. When "B" reaches the second intersection, it must stop a short while and "C" can proceed without stopping. Yet, "C" is delayed since its speed must be reduced to not hit the first vehicles in the queue which are just starting and accelerating.

"D" is a vehicle which does not stop but is delayed at both intersections.

"F" proceeds at a constant speed through the first intersection and, then, must decelerate and start queuing up at the second one.

These straight-forward observations constitute the very basis of our study. Another secondary phenomenon should, however, be noted on the same plot; it is called <u>platoon dispersion</u> (17). At the time T1, the vehicles are leaving the intersection number 1: they are, one after the other, accelerating to reach a certain speed; but the speeds they are aiming for are different. The first vehicles discharging the queue are going faster (greater slope on the curve) than the next ones which respect the car-following laws (6). This is why the spaces between flowing vehicles at the time T1 are much smaller than they are at the time T2. This platoon dispersion affects the pattern of arrivals in the queue, but, for the departures, one should note on the plot, that the time intervals between the crossing of the stop line by successive vehicles are, approximately, equal. These results have been obtained in the field and experience has shown that they hold true in most cases.

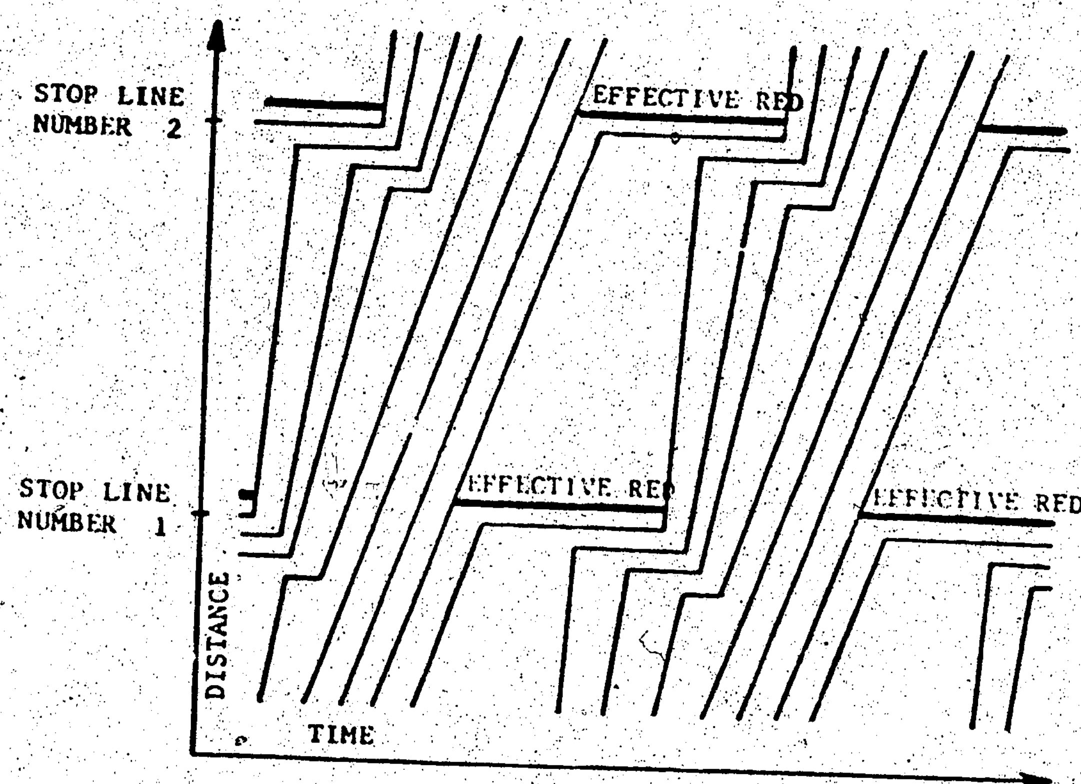

FIGURE III-2:

SIMPLIFIED FLOW PATTERNS IN HEAVY CONDITIONS

In our study, the flow patterns we use are som-how simplified. They sketch the actual vehicle behaviour without affecting the delays too much. The types of curves used are represented on Figure III-2; deceleration and acceleration times have been reduced to zero. Each vehicle is assumed to have a constant speed between two consecutive stops. However, all vehicles do not have the same speed; they travel slower and slower at the beginning of the discharge until part of the queue is filled up. Afterwards, the following vehicles travel at a constant average speed. The delays are simply represented by the horizontal sections of the curves. The total sum of their various lengths is a measure of the vehicular delay.

2. DEFINITIONS AND NOTATIONS

Capacity is defined first. According to the Highway Capacity Manual, (8), it "is the maximum number of vehicles which has a reasonable expectation of passing over a given section of a lane or a roadway (...) during a given time period...". In the case of signal-controlled intersections, the capacity of an approach is also the maximum value of its saturation flow which is defined by F.V. Webster (20) as the "flow which could be obtained if there was a continuous queue of vehicles and they were given 100 per cent green time". This saturation flow (s) can be easily estimated by knowing the various characteristics of the approach and, essentially, its width and the proportions of large vehicles and turning traffic. Webster has shown

(20) that s is proportional to the width of the approach (see Appendix A). Usually expressed in terms of passenger car units (p.c.u.'s) per hour of green time, s is calculated with the following p.c.u. equivalents (20):

1 heavy or medium goods vehicle = 1.75 p.c.u.

1 bus = 2.25 p.c.u.

1 light goods vehicle = 1.00 p.c.u.

1 private car = 1.00 p.c.u.

Every left-turner is counted as 1.75 straight-ahead vehicle if there is an opposing flow, otherwise no conversion is needed. In the case of a heavy right-turn movement (more than 10% of the whole traffic), a correction is made counting every right-turner over the first 10% of the traffic, as 1.25 going-through vehicle. Capacity manual (8) uses different corrections.

Concerning the timing of the signals, every set of traffic lights at one intersection has a <u>cycle</u> which is divided in a number of phases during which, different approaches (generally with not very conflicting traffic streams) are discharged at the saturation flow. As a matter of fact, it takes a few seconds of green before the saturation flow is reached and, at the other end, a few more seconds of amber are needed for the flow to stop. "It is convenient to replace the green and amber periods by an <u>effective green</u> period" (20) during which flow is assumed to take place at the saturation rate. The cycle is, thus, roughly divided in effective green periods. Since, for safety reasons, and also for better operation (to let left-turners clear the intersection), there are some periods during which no approach is discharging, we define the lost time as the difference between the cycle time and the sum of the effective green periods. For every approach, we then define the effective red period as the difference between the cycle and the effective green. Note that these definitions of the lost time and the effective red are slightly different from Webster's. The following Figure (Fig. III-3) shows an example:

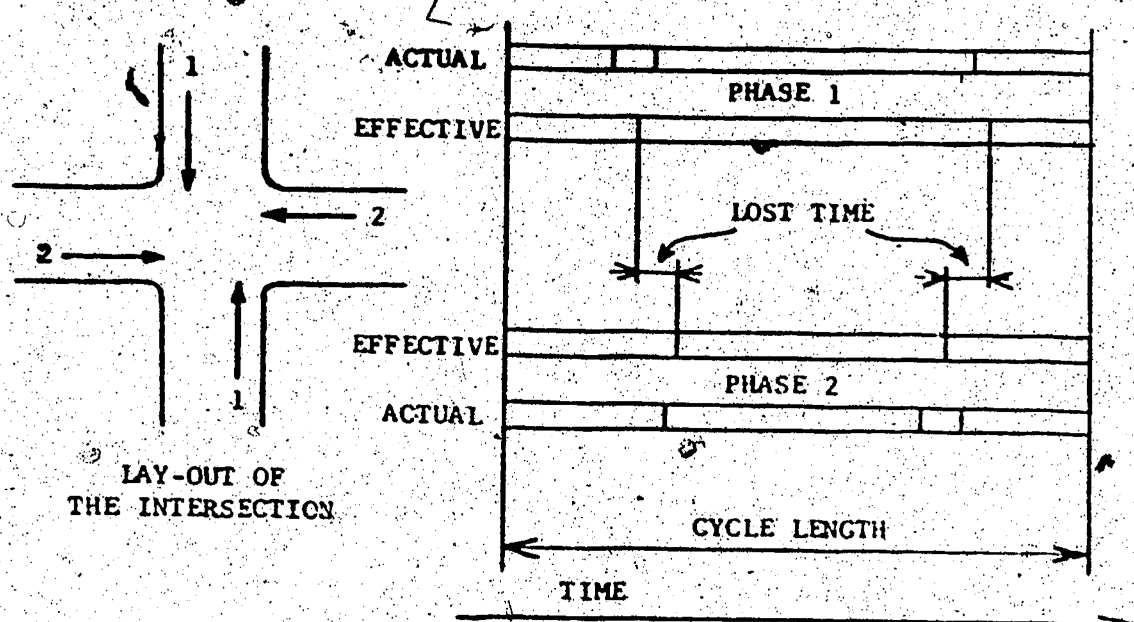

FIGURE III-3:

SCHEMATIC DEFINITION OF THE EFFECTIVE GREEN AND RED

The following notations will be used. Let:

c = the cycle time,

g = the effective green time,

r = the effective red time,

q = the average rate of arrivals on the approach in pcu's per unit of time,

s = the saturation flow on the approach in pcu's per unit of time,

d = the average delay to pcu's on the approach,

λ = g/c , i.e., the proportion of the cycle that is effectively green for a particular phase,

y = q/s , i.e. maximum ratio of flow to saturation flow for a given phase,

x = qc/gs = q/λs = degree of saturation, ratio of the flow to the maximum flow which can actually pass the intersection.

Concerning the network, four definitions specific to our study must also be given. First, an __external approach__ to an intersection is such that all the vehicles which are in it , always come from outside the network under study. As opposed to it, the other approaches are called __internal__ . As a consequence, links leading to such approaches are respectively called __external__ and __internal links__.

3. CALCULATION OF DELAYS

Even though quite helpful to understand the vehicle behaviour, the approach of the paragraph 1. is not suitable to an extensive calculation of delay over a whole network. The reason for this is, that one would have to draw curves, like on Figure III-2, to be able to evaluate the vehicular delays. This procedure is too long and even awkward in our particular case. We shall use the following method.

In an urban network with many synchronized traffic lights, the patterns of arrivals and departures of vehicles at any signal can be easily assessed in the case of an internal approach. The following diagram (Fig. III-4) shows an example:

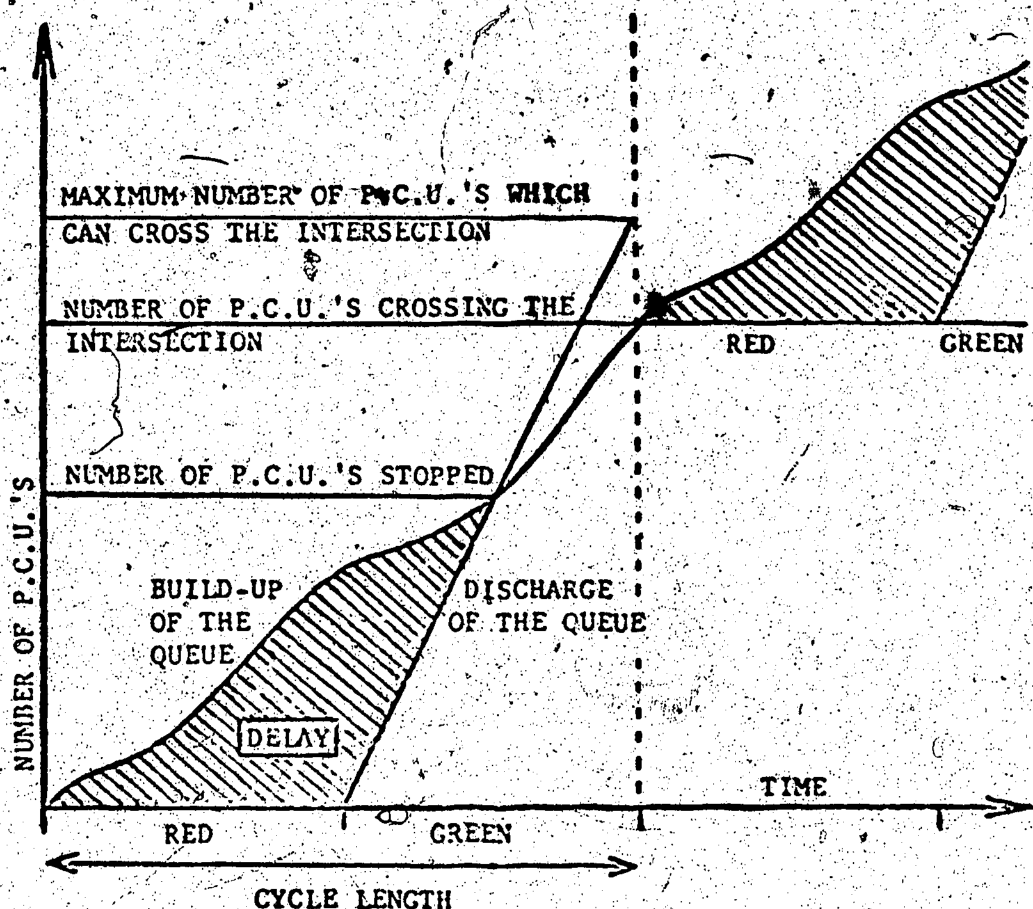

FIGURE III-4:

DELAY KNOWING ARRIVALS AND DEPARTURES IN THE QUEUE

By patterns of arrivals and departures, we mean the variation of the cumulative number of vehicles which have reached and left the queue over the cycle time. Arrivals are computed using the various counts and the signal timing of the various origins of the links leading to the approach we are interested in. Departures are computed assuming that the vehicles leave the queue at the saturation flow during the effective green time. The knowledge of these patterns allow us to draw up the two curves of Figure III-4 which show the number of vehicles which have joined and left the queue. Any horizontal strip of area between the arrival and departure curves has a length which is equal to the time a certain number of vehicles (the width of the strip) must wait. This can be integrated for all the vehicles crossing a signalized intersection during the cycle. However, since there is only delay when there is a stop, in other words, unstopped vehicles do not have a negative delay, only the area under the arrival curve and above the departure curve is a measure of the vehicular delay.

This is true for internal approaches. In the case of external approaches, since we do not know the pattern of arrivals of the vehicles in the queue, this method does not work. We therefore use an estimation of the vehicular delay which is given by the Webster's simplified expression (20):

$$d = \frac{9}{10}\left[\frac{c(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)}\right]$$

A test on the accuracy of this formula is reported in the Appendix B.

Finally, to get the passenger delay, an average occupancy per pcu is computed for every approach with the total number of pcu's and the total number of passengers. The passenger delay is, thus, equal to the vehicular delay weighted by the average occupancy of a pcu. The total delay of passengers at signalized intersections (TDPSI) is obtained by summing the results over all different approaches.

4. THE RANDOM COMPONENT

In the simulation model, the various flows at the intersections are supposed to be the same at every cycle. As a matter of fact, they are variations which always increase the average vehicular delay. This random component must be taken into account.

For this purpose, we assume that fluctuations of the flow around the mean follow the Poisson curves of distribution. In other words, any approach has arrivals such that t being the time and dt a small interval, we have:
probability (1 pcu arrives in (t, t+dt)) =qdt + o(dt) and,
prob. (more than 1 pcu arrive in (t,t+dt)) = o(dt)
In this case, it has been shown (14) that the extra delay is a function of x, degree of saturation on the approach. We have:

$$d = \frac{x^2}{4q(1-x)}$$

Note, by the way, that this random component appears in Webster's expression of delay which also assumes Poisson arrivals.

CHAPTER IV

THE SIMULATION MODEL

This chapter explains in some details the way the simulation model works. It is divided into three main parts which deal respectively with the data, the calculations and the results.

1. THE DATA

As we already mentioned, we use three kinds of data which concern the network, the traffic and the timing plan. The description of the network constitutes the least changeable information; however, there are some exceptions to this rule: temporary parking lanes at some hours of the day are typical examples. On the other hand, traffic is basically variable; so is the timing plan since the first aim of this model is to optimize it.

The description of the network is made on the approach basis. That is to say that each approach of a signalized intersection is given a number (IAP) for identification purposes. These numbers must be positive integers and must not be greater than the total number of approaches. The rest of the information is, then, put in the form of arrays described hereafter:

INTER(IAP) is the code number of the intersection to which belongs the approach IAP. It must be an integer not larger than the total number of intersections of the network.

IDENT(IAP) can only be equal to 1, 2, 3 or 4;

"1" means that the approach IAP is a northern approach,

"2" means that the approach IAP is an eastern approach,

"3" means that the approach IAP is a southern approach, and,

"4" means that the approach IAP is a western approach.

ICPPO(IAP) is either equal to 1 or 0;

"1" means that there is opposing traffic, and,

"0" means that there is no opposing traffic.

IWDTH(IAP) is the width of the approach IAP in feet.

ILNGT(IAP) is the length of the link leading to the approach IAP. It is also given in feet.

IWILI(IAP) is the width of the link in feet; this width is, in most cases, equal to IWDTH(IAP): it is used to compute the length of the actual queue knowing the number of vehicles and the area occupied by a vehicle in the queue.

TSPID(IAP) is the average speed of the vehicles on this link considered when they have to stop a few seconds at every intersection (see the parameters RA1 and RA2 at the end of this section); this speed is expressed in miles per hour. It is through this parameter that allowances can be made for gradients and low accelerations because of poor traction (icy streets) or any other reason.

IORGN(1,IAP), IORGN(2,IAP) and IORGN(3,IAP) are the code numbers of the approaches used as origins by the vehicles. When the origin approaches are outside the network under study, "99" indicates that it is an external approach. The presentation of the origin approaches is such that they cannot be more than three; it has been proved that this restriction does not limit the possibilities of the model.

ISTRT(IAP) is the number of the street if the approach is on a street. If the approach is not on a street, this parameter is equal to zero.

IAVNU(IAP) is the number of the avenue if the approach is on an avenue. If the approach is not on an avenue, this parameter is equal to zero. It is assumed that every approach is always either on a street or on an avenue. These two parameters are not used in the calculations; they are given to locate more easily the approaches and to allow further implementation of the model regarding the drawing of intersections.

IBSLA(IAP) is either equal to 0 or to 1.

"1" means that the approach IAP is a bus lane and,

"0" means that the approach IAP is ordinary.

PARKN(IAP) is a positive number always smaller than or equal to one. "1.0" means that all the vehicles coming from the origin approaches get to the approach under consideration. "0.8", for example, means that only 80 per cent of them reach the approach under study. The remaining 20 per cent getting parked before. This parameter makes allowance for the effect of large parking facilities on the network.

ILTPC(IAP), ITRPC(IAP) and IRTPC(IAP) are the volumes of passenger cars, respectively, turning left, going through and turning right during the hour under consideration.

ILTBS(IAP), ITRBS(IAP) and IRTBS(IAP) are the volumes of large vehicles (mainly buses), respectively, turning left, going through and turning right from this approach, during the hour under consideration.

TABLE IV-1: EXAMPLE OF PRINT-OUT OF THE DATA

Table IV-1 represents the print-out of this information from the computer. Each line of the array is entered in the form of a punched card. The end of the page shows also other types of information like the total number of intersections and approaches, the occupancies of cars and buses. In our example (see Table IV-1), we assume that E, the number of pcu's a large vehicle is equivalent to, is equal to 2.20. The area A that a pcu occupies in the queue is also given (300 sq.ft., in our example). This parameter is used to compute the actual length that vehicles can be driven before reaching the end of the queue. This length is a function of the number of vehicles in the queue, the area occupied by a vehicle, and the geometrics (width and length) of the link. The ratio between the saturation flow and the width of the approach is also entered as a parameter (92.0 in our example). The bigger the city, the larger this number is. Experiments carried out in London, England, have shown that 160 was an accurate figure for British traffic (20).

RA1 and RA2 are two ratios which describe the phenomenon of platoon dispersion (see Figure IV-1) in terms of variations of speed with the length of the queue to be reached. These parameters give a mathematical description of the simplified flow patterns seen in Chapter III. When there is no queue, the vehicles are supposed to travel at the highest speed; then, this speed decreases down to ISPID when a certain portion (RA2) of the link is filled up. Afterwards, it is assumed to stay constant. RA1 and RA2 are drawn from the observation of driving practices on the network under study.

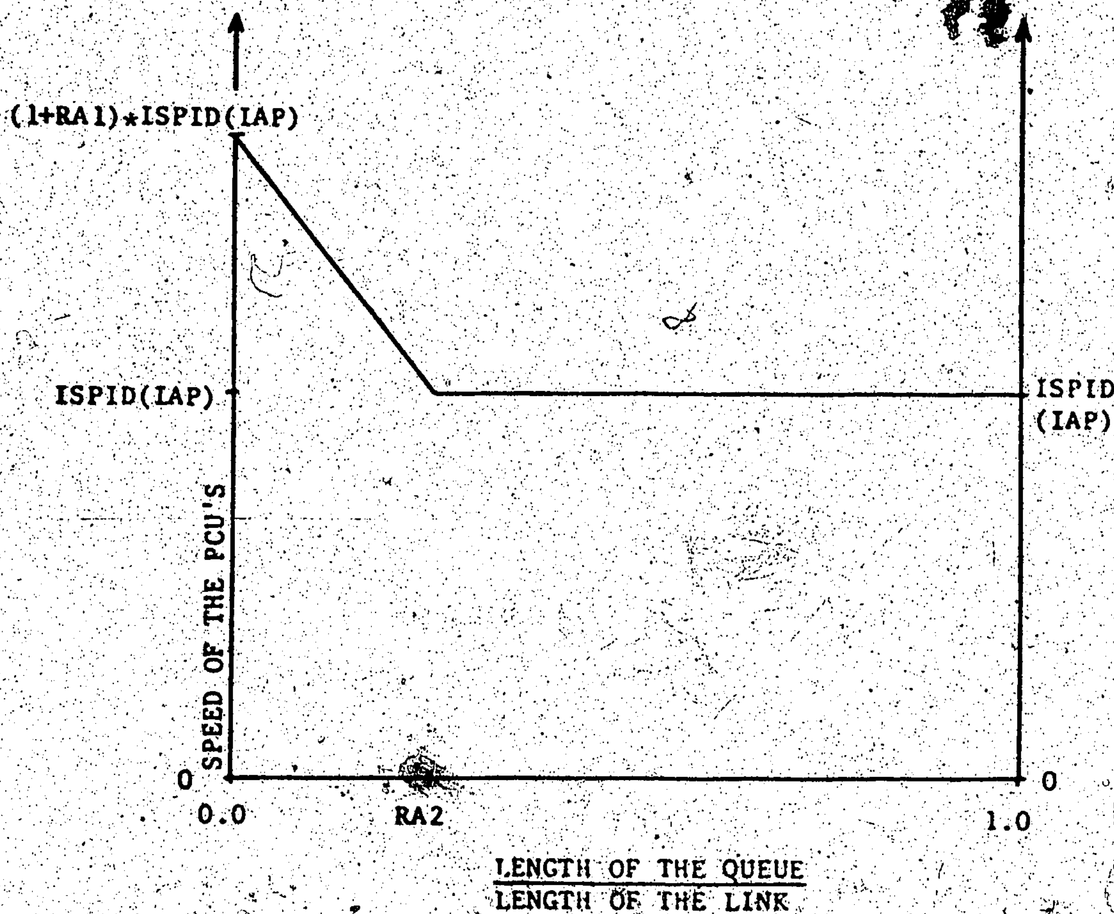

FIGURE IV-1:

DESRIPTION OF THE PLATOON DISPERSION THROUGH RA1 AND RA2

The timing plan constitutes the third part of the needed data for the simulation. We assume that the cycle, whatever its length, is divided in 20 equal sections of time. This convention is based on the fact that it is impossible to get more than 5 per cent accuracy due to the fluctuations of the traffic demand through the peak hour we are studying. We must keep in mind that our model considers the demand as being the same at every moment which is not true but compensated by the random component of delay. For each approach, the timing plan is described by two numbers smaller than or equal to 20 and also greater than or equal to 1. The first one gives the section of the cycle at which the effective green starts and the second one gives the section of the cycle at which the effective red starts. For each approach, by definition, the effective green finishes when the effective red starts and vice-versa. The lost time, if any, is the number of sections of cycle between the end of an effective green on an approach and the beginning of the effective green on a conflicting approach. The cycle length which is common for the whole network must also be given.

Before ending this section, a particular problem must be evoked: How to treat turning lanes or bus lanes? The answer is rather simple. Any specific lane on any approach is considered as a particular approach having its own characteristics concerning geometrics, traffic and lights. A continuous bus lane which is for example 12 feet wide, only carries bus traffic and has its main origin in the preceding bus lane approach; the rest of the pavement carries the rest of the traffic on a restricted width.

2. TREATMENT OF INFORMATION

Once the data have been read, some preliminary computations are necessary to aggregate the different kinds of vehicles through different movements and to assess an average occupancy which will be used to convert vehicular delay into passenger delay. These calculations, made on the approach basis too, are carried out for both external and internal approaches.

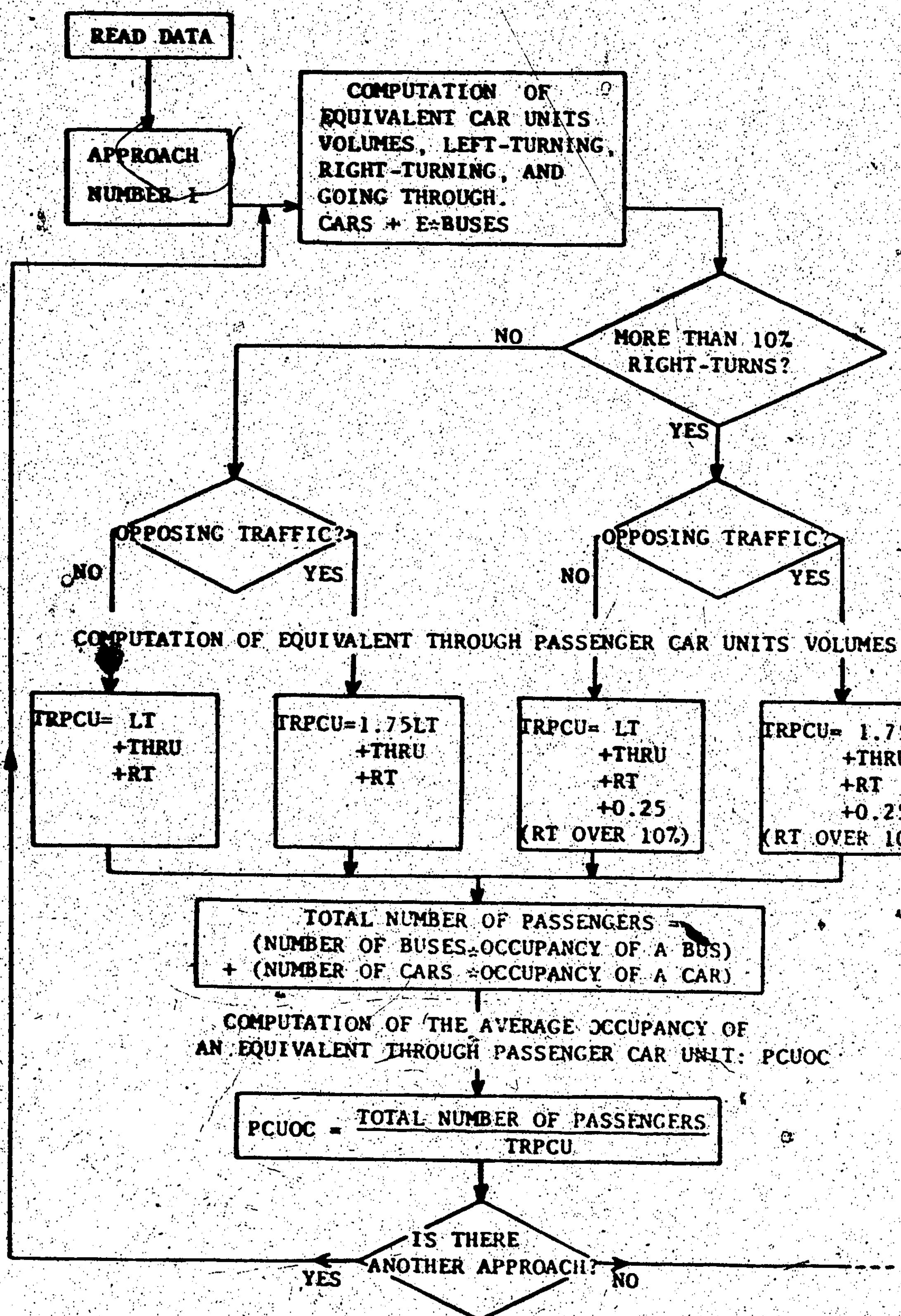

FIGURE IV-2: PRELIMINARY COMPUTATIONS

The flow-chart represented on Figure IV-2 explains to us in detail the process followed. We have, for every approach, the left-turning (LT), going through (THRU), and right-turning (RT) volumes of private cars and buses. For these three movements, we aggregate the volumes using the fact that every large vehicle is equivalent to E passenger car units. The next step consists in aggregating these three movements (LT, THRU and, RT) and converting them into <u>through passenger car units</u> volumes (TRPCU). For this purpose, we use the rules given by F.V. Webster (Webster, 1966). Four cases are distinguished depending on the percentage of right-turners and also on whether or not left-turners are opposed.

1 right-turner is counted as 1.25 through pcu for the excess over 10 per cent of the total volume. There is no conversion for less than 10 per cent righ-turners.

1 left-turner is counted as 1.75 through passenger car units if this movement is opposed by a conflicting flow with right of way (otherwise no conversion).

We now have TRPCU for every approach. A simple calculation gives us the <u>average occupancy</u> (PCUOC) of an equivalent through pcu. We only have to divide the total passenger volume by TRPCU. PCUOC is the factor we will use later to convert the vehicular delay into passenger delay.

The following part of calculations only concerns the internal approaches. The arrivals as well as the departures are computed for every internal approach.

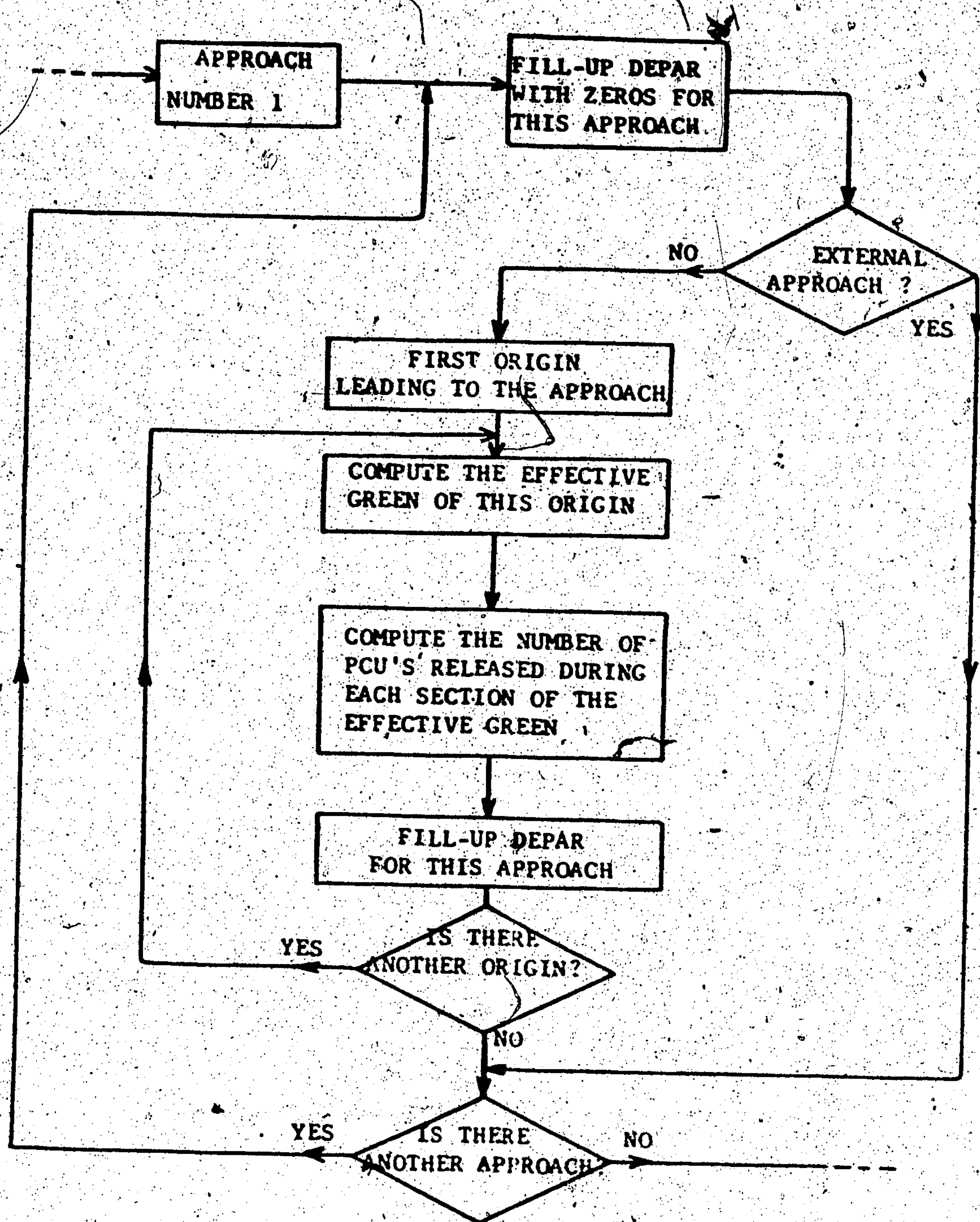

FIGURE IV-3: COMPUTATION OF ARRIVALS (Part 1)

First of all, we use a two dimension array DEPAR which elements are DEPAR(ITIME,IAP). Each element is the number of pcu's leaving the various origins during the time interval (ITIME-1,ITIME) towards the approach IAP which is under study (see Figure IV-3:). The calculations needed to fill-up this array are simple. For every origin approach, we divide the total number of pcus going to IAP by the number of effective green sections. The constitution of this array DEPAR is the primary step in figuring out the pattern of arrivals at the approach under consideration.

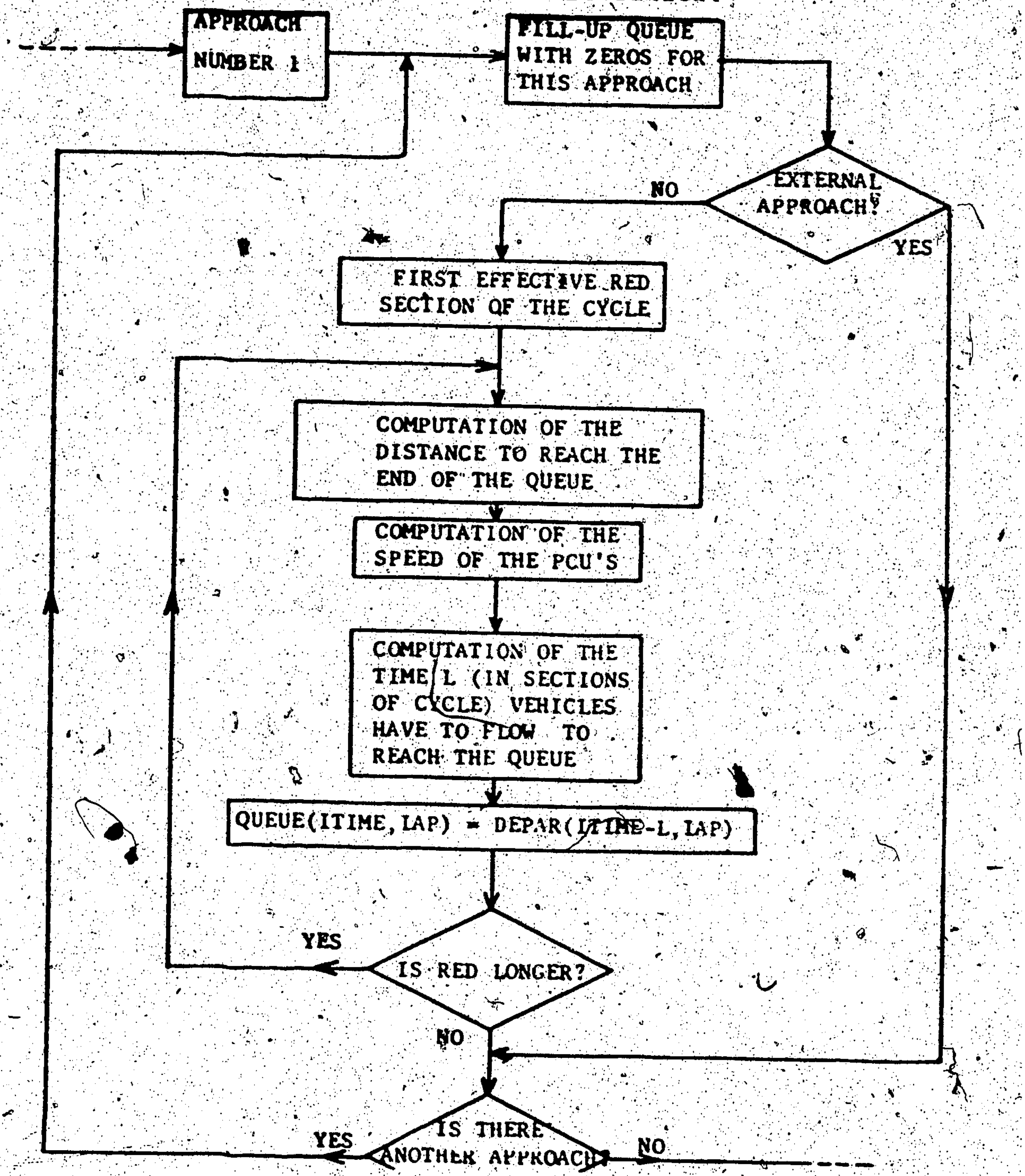

FIGURE IV-6: COMPUTATION OF ARRIVALS (Part 2)

QUEUE is a second two dimension array which elements QUEUE(ITIME,IAP) are the numbers of pcu's in the queue number IAP at the time ITIME of the cycle. In order to fill up this array (see Figure IV-4:) we use the facts that:

1. No vehicle is waiting when the effective red starts, and,
2. During one section of the cycle, the increase of the queue is equal to the number of vehicles which left the various origins L sections before; L is the number of sections of the cycle it takes for the pcu's to reach the end of the queue IAP. L is a function of the length of the link, the actual length of the queue at this time, and also the average speed used. Platoon dispersion is taken into account through RA1 and RA2.

The pattern of departures of the vehicles from the queue is described through the use of a two dimension array DISCH which elements DISCH(ITIME,IAP) are the maximum numbers of pcu's which can discharge the queue number IAP at the time ITIME of the cycle. For any internal approach, DISCH is equal to zero at the beginning of the effective green and then, for each subsequent green section of the cycle, DISCH is increased by the maximum flow which can occur at this approach during one section of the cycle. The saturation flow used for this calculation is assumed to be proportional to the width of the approach (see Appendix A). DISCH is equal to zero during the effective red sections of the cycle.

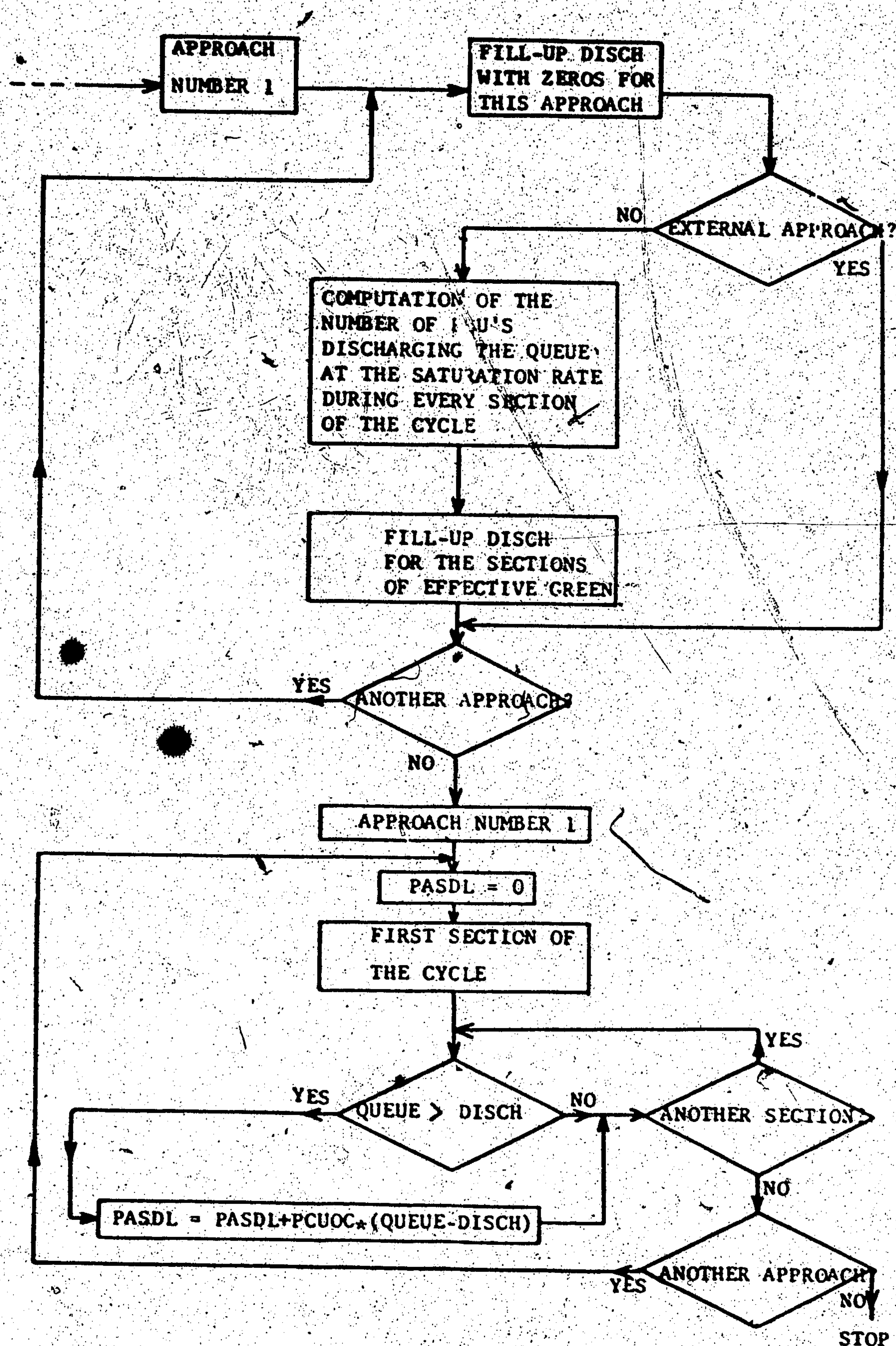

FIGURE IV-5: COMPUTATION OF DELAYS

By plotting for each approach the pattern of arrivals QUEUE and the pattern of departures DISCH through the cycle length, we get similar curves to those shown on Figure III-4. The area which is below the QUEUE curve and above the DISCH curve is a measure of vehicular delay. We get the passenger delay (PASDL) by multiplying it by PCUOC. These calculations appear more clearly on the last part of the flow-chart (Figure IV-5).

External approaches receive a special treatment because the patterns of arrivals and departures are unknown. We therefore assume that they follow the Poisson curves of distribution; by this way, the use of the Webster's simplified expression of delay becomes possible. In this formula, the random component of delay is already included in the second part and does not need to be added up.

3. THE RESULTS

This computer simulation provides two items of results. The basic results are on the print-out (See Table IV-2) of the computer. For each approach, they show:

IRPCU = number of equivalent through passenger car units,

PCUOC = average occupancy of an equivalent through pcu,

PCUDL = average waiting time in seconds for each pcu at each approach under study (line IAP),

TPCUD = vehicular delay in pcu's*hours during one hour of traffic,

PASDL = Passenger delay in passengers*hours per hour,

FINAL RESULTS OF SIMULATION

IAP	TPCUD(IAP)	PCUD(IAP)	PCUDL(IAP)	TPCUST(IAP)	PASDL(IAP)	PCUST(IAP)	PASST(IAP)	INDLA(IAP)
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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14	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
15	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
16	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
17	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
18	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
19	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
20	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
21	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
22	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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26	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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30	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
31	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	PCU	PASSENGER PER PCU	SECONDS OF DELAY PER PCU PER CYCLE	PCU-HOURS PER HOUR OF TRAFFIC	PAS.-HOURS PER HOUR OF TRAFFIC	PCU STOPS PER CYCLE	PAS. STOPS PER HOUR OF TRAFFIC	

TOTAL PASSENGER DELAY AT SIGNALIZED INTERSECTIONS = 171.0 PASSENGER-HOURS PER HOUR OF TRAFFIC
TOTAL PASSENGER STOPS AT SIGNALIZED INTERSECTIONS = [illegible] PASSENGER STOPS PER HOUR OF TRAFFIC

TABLE IV-2: PRINT-OUT OF THE RESULTS

PCUST = average number of pcu stops during one cycle length at this approach,

PASTO = average number of passenger stops during one hour of traffic. These two results concerning the number of stops are not used in the optimization process. They are only given for an eventual further implementation of the model. Since their calculation was rather simple, it seemed to us interesting to do it and to print it out.

INDEX is a rough number which aggregates the results concerning stops and delays and, thus, allows us to point out rapidly where the problems are.

$$\text{INDEX} = \frac{\text{(PAS.DELAY + PAS. STOPS CONVERTED INTO DELAY) FOR THIS APPROACH}}{\text{(PAS.DELAY + PAS. STOPS CONVERTED INTO DELAY) FOR ALL APPROACHES}}$$

The total passenger delay which is considered as the main usable result of this simulation is given on the two last lines of the print-out. It is simply the sum of the column PASDL over all the approaches. The sum of PASTO is also executed and the result is given in the last line of the print-out.

The second kind of results is constituted by the curves drawn on the plotting machine by the computer (see Figure IV-6). They show the pattern of arrivals and departures for the internal approaches. The purpose of this plotting is to make the optimization of the timing plan easier by examining the shape of these curves.

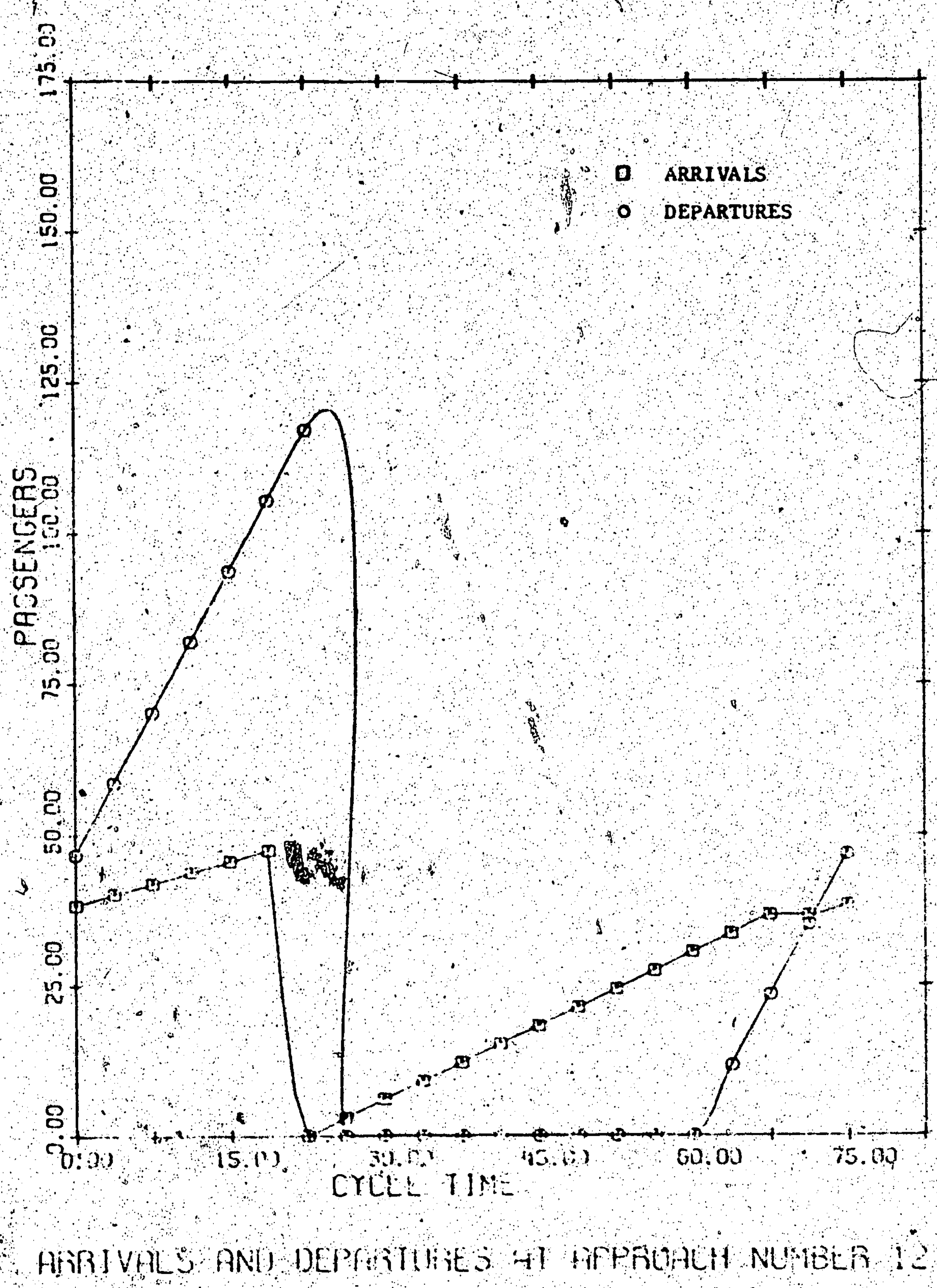


FIGURE IV-6: COMPUTER PLOTTING

CHAPTER V

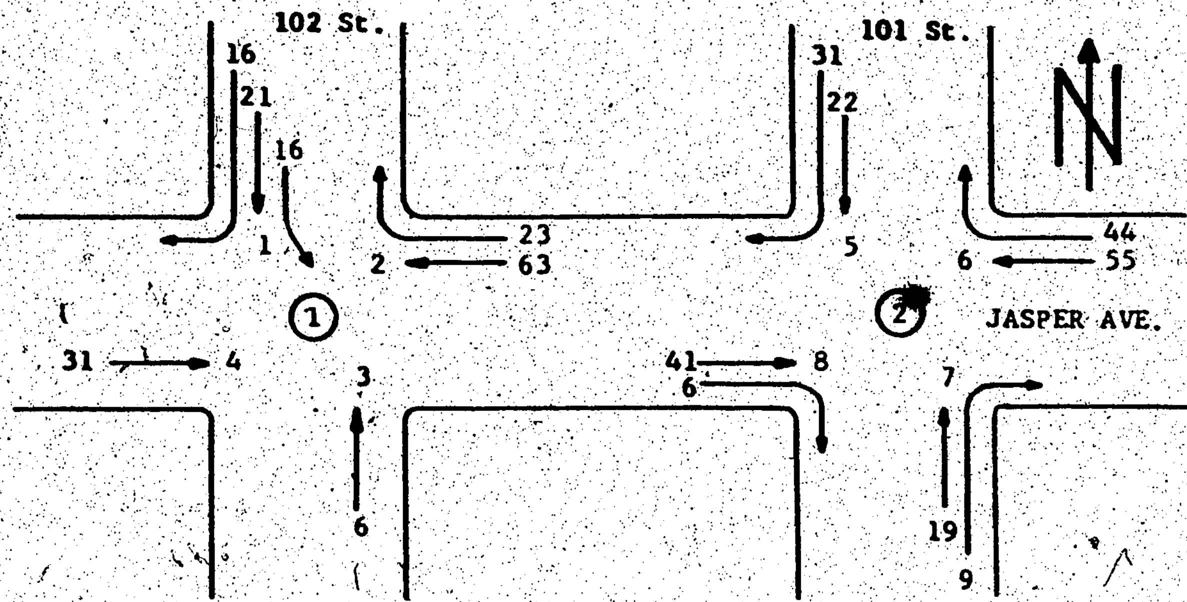
RESULTS

In this chapter, the main results obtained through the use of computer simulation are presented. They deal with a few specific problems like optimizing a timing plan, creating a bus lane, barning a left-turn or changing the mode split. In order to get acquainted with the type of results we get, a simple example constituted by an actual network of only two intersections is explained in some detail in the first section.

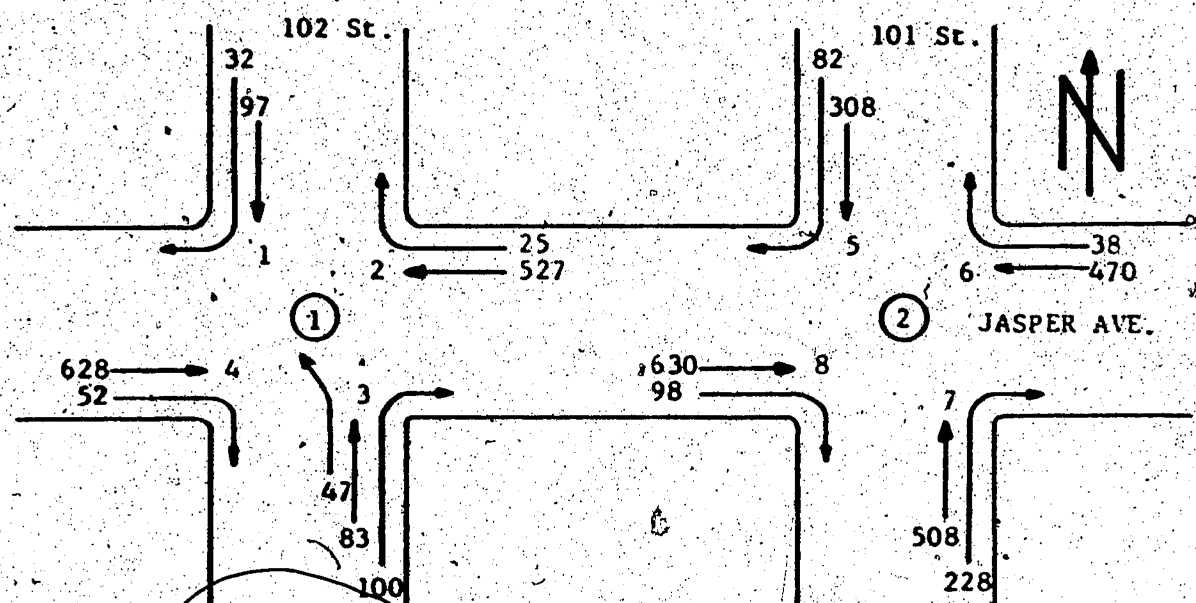
1. A SIMPLE EXAMPLE

Figure V-1 shows the network under study. There are two intersections numbered 1 and 2. Each intersection has four different approaches. This means that there are no special lanes for special movements or for the buses: all the vehicles, left-turning, going through and right-turning are mixed together at the approaches. There are 8 different approaches numbered 1 to 8. The traffic flows, expressed in vehicles per hour, are also shown on Figure V-1.

We are going to optimize the timing plan for two different sets of conditions. In the first case, the network and the traffic are those described by Fig. V-1. In the second case, we assume the existence of two bus lanes on both sides of Jasper avenue (approaches 2 and 6). These two bus lanes would carry an average of 3300 passengers at the peak hour, and the rest of the pavement (4 lanes) an average



BUS TRAFFIC ON THE TWO INTERSECTIONS



CAR TRAFFIC ON THE TWO INTERSECTIONS

FIGURE V-1: NETWORK UNDER STUDY

of 1700 passengers (these figures come from actual traffic counts done in June 1973 at the a.m. peak). We use a cycle length of 75 seconds and optimize the timing plan through variations of green splits and offsets. For the green splits, we consider independently the two intersections and we run the simulation program for all the possible distributions of green time. We have assumed a lost time of 1 section of cycle (75/20=3.75 seconds). Figures V-2 and V-3 show the plotting of the results. The curves present a minimum which happens for the optimum green split.

A detailed study of these two plots allows us to point out some first results concerning this particular problem.

* The minimum passenger delay is reached at the intersection number 1 (see Fig. V-2) when the effective green is around 25 percent of the cycle for the approach number 1 (North).
* For the intersection number 2 (see Fig. V-3), the minimum is reached with 45 percent of effective green at the approach number 5 (North).
* The curves are sharper for the intersection number 2 than for the intersection number 1. This is explained by the fact that the intersection number 2 is much more loaded by traffic and, therefore, a mistake in the percentage of green would create a much greater increase of delay than at the intersection number 1. In other words, the range of variation of green split which is possible without any saturation is smaller.

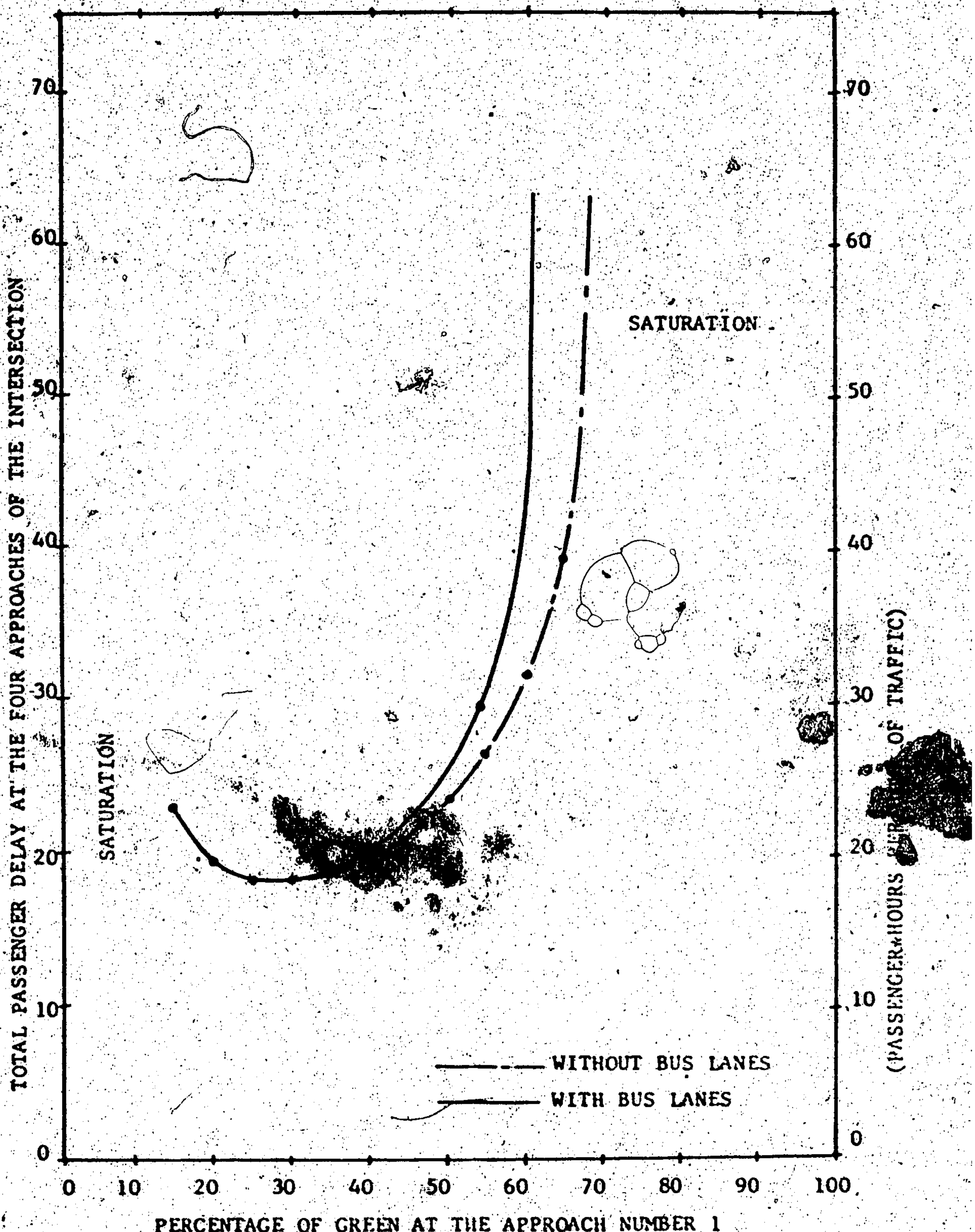


FIGURE V-2: TOTAL PASSENGER DELAY AT THE INTERSECTION NUMBER 1 VERSUS THE GREEN SPLIT

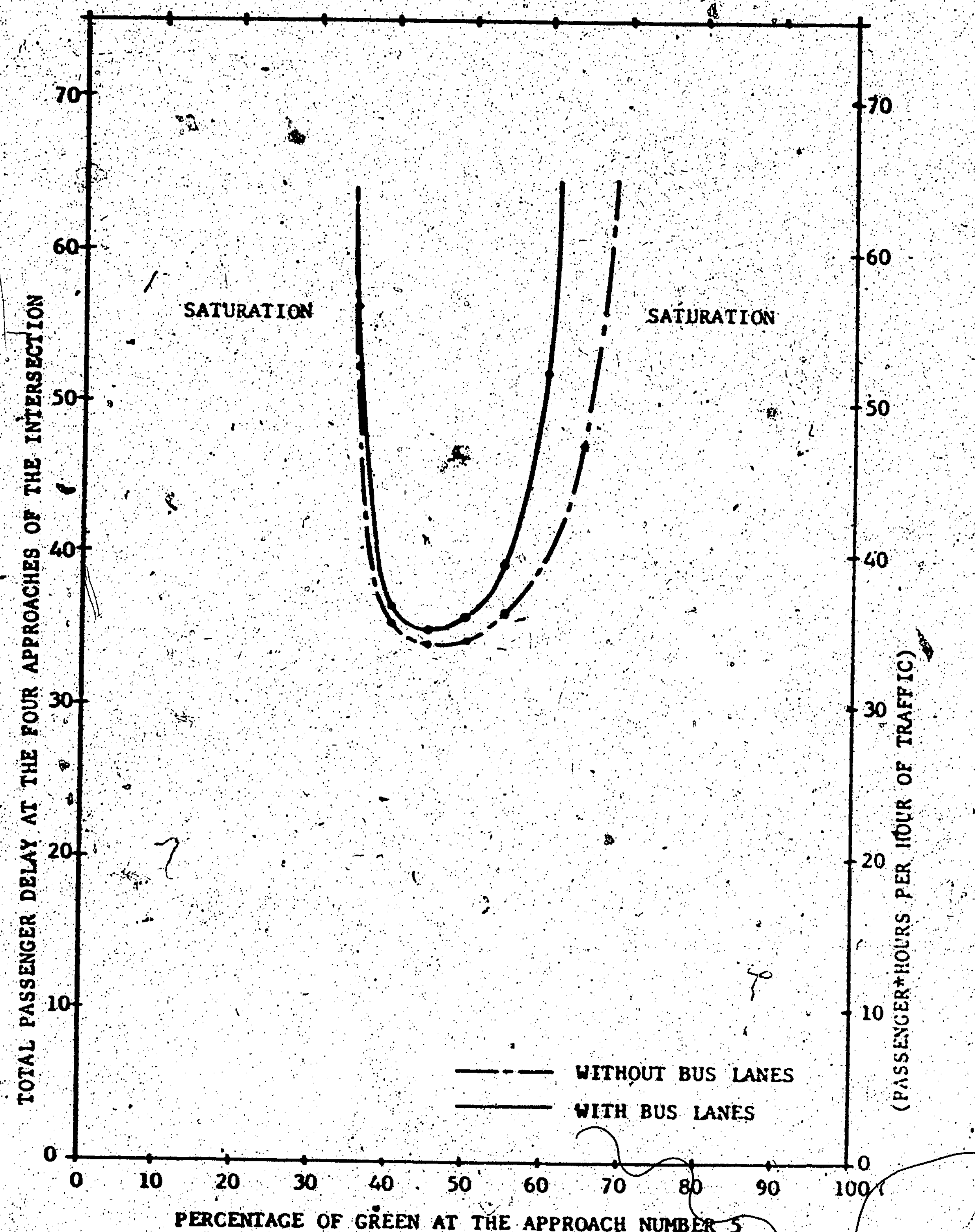


FIGURE V-3: TOTAL PASSENGER DELAY
AT THE INTERSECTION NUMBER 2 VERSUS THE GREEN SPLIT

Some other conclusions can also be drawn concerning the creation of reserved bus lanes on both sides of Jasper Avenue between 101 and 102 streets.

* The left branches of the curves (see Fig. V-2 and V-3) are not very affected by the bus lanes because we are close to saturation on 102 and 101 streets and, as a consequence, there is no problem on the avenue. One could state that in undersaturated conditions, the restriction of some lanes for transit buses does not affect the delay at signals.
* On the contrary, the right branches of the curves are displaced by the creation of bus lanes. This means that saturation is reached more rapidly on the avenue. This is due to the fact that, on a restricted width, with a low percentage of effective green, the bus lanes create a substantial increase of delay to the other vehicles. The simulation has shown that, with Edmonton conditions, saturation was always starting on the restricted width of the approach rather than on the proposed bus lanes.
* Simulation also shows (see Fig. V-2 and V-3) that the minimum total passenger delay, as well as the percentage of green for which this minimum happens, stays practically unaffected by the creation of bus lanes. The shift of this lowest point on the plots is almost not significant at the intersection number 2 and non-existent at the intersection number 1.

One should keep in mind that, in this first step of the optimization process, these two intersections are considered as separate problems. To complete the study, one must link the two intersections together and make the bus lanes continuous from one intersection to the other. This will constitute the second step of this process through which the proper offset between the two intersections will be found. We define the offset, in this example, as being the time difference between the beginning of the effective green at the approaches 6 and 2. As a consequence of the linkage of the two intersections, the approaches 2 and 8 become internal in the network; the six other approaches (1, 3, 4, 5, 6 and 7) stay external to the network and, therefore, since the cycle split remains optimum at the two intersections, the passenger delay does not vary at these approaches. The offset is the only variable in the timing plan and, hence, only the internal approaches (2 and 8) will be affected. Figures V-4 and V-5 show the variation of the passenger delay with the offset. The analysis of these two curves allows us to draw the following conclusions:

* For both approaches, the variation of offset creates a very important variation of delay; a proper offset can even reduce the delay to a value very close to zero (Fig. V-4).
* The creation of continuous bus lanes on both sides of Jasper avenue between 101 and 102 st. makes the maxima higher and the minima lower on the delay curves. In other words, the curves representing the situation with bus lanes are much sharper. This indicates that the optimum offsets should be set with more care when there are reserved lanes.

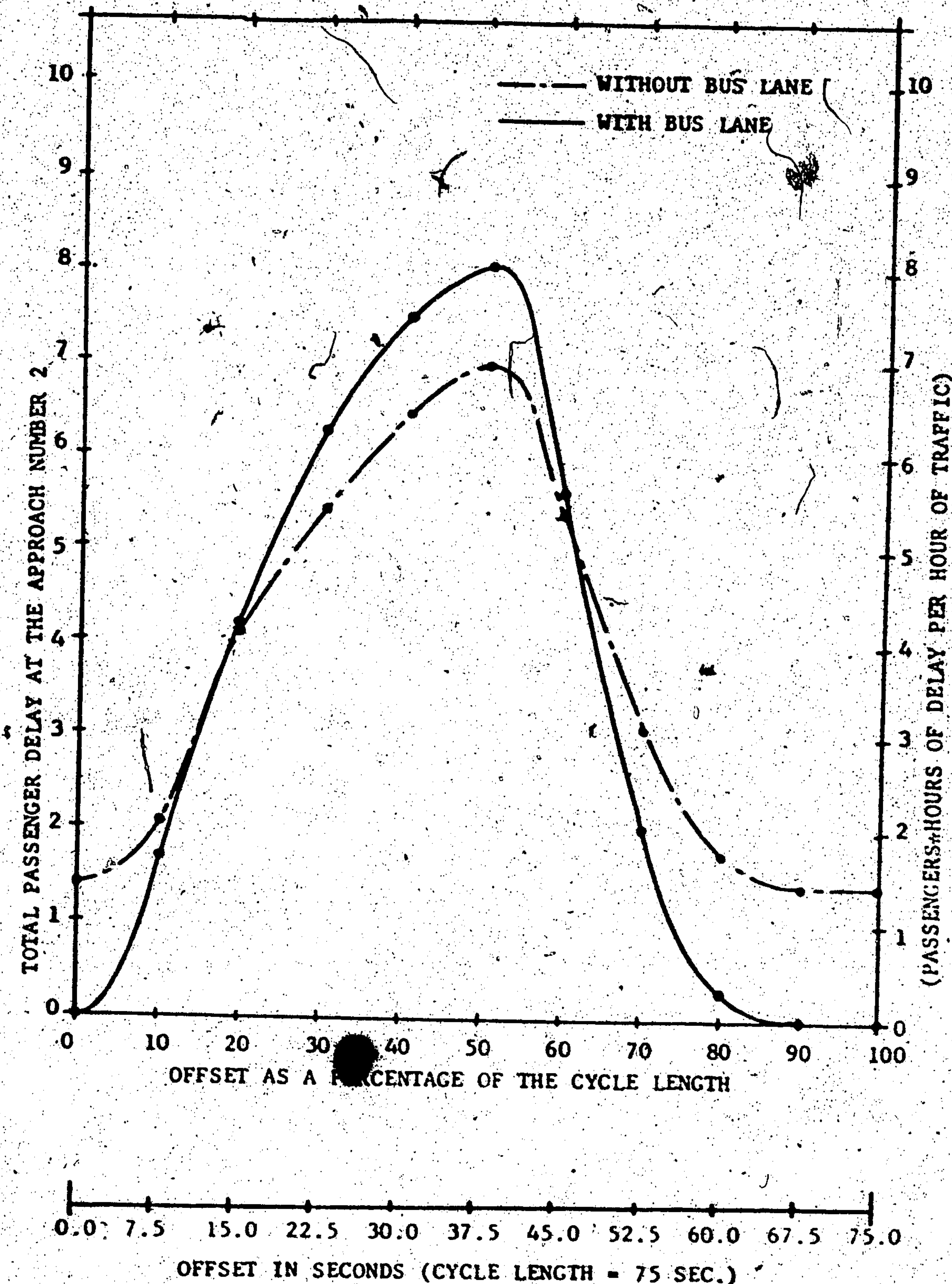


FIGURE V-4: TOTAL PASSENGER DELAY
AT THE APPROACH NUMBER 2 VERSUS THE OFFSET

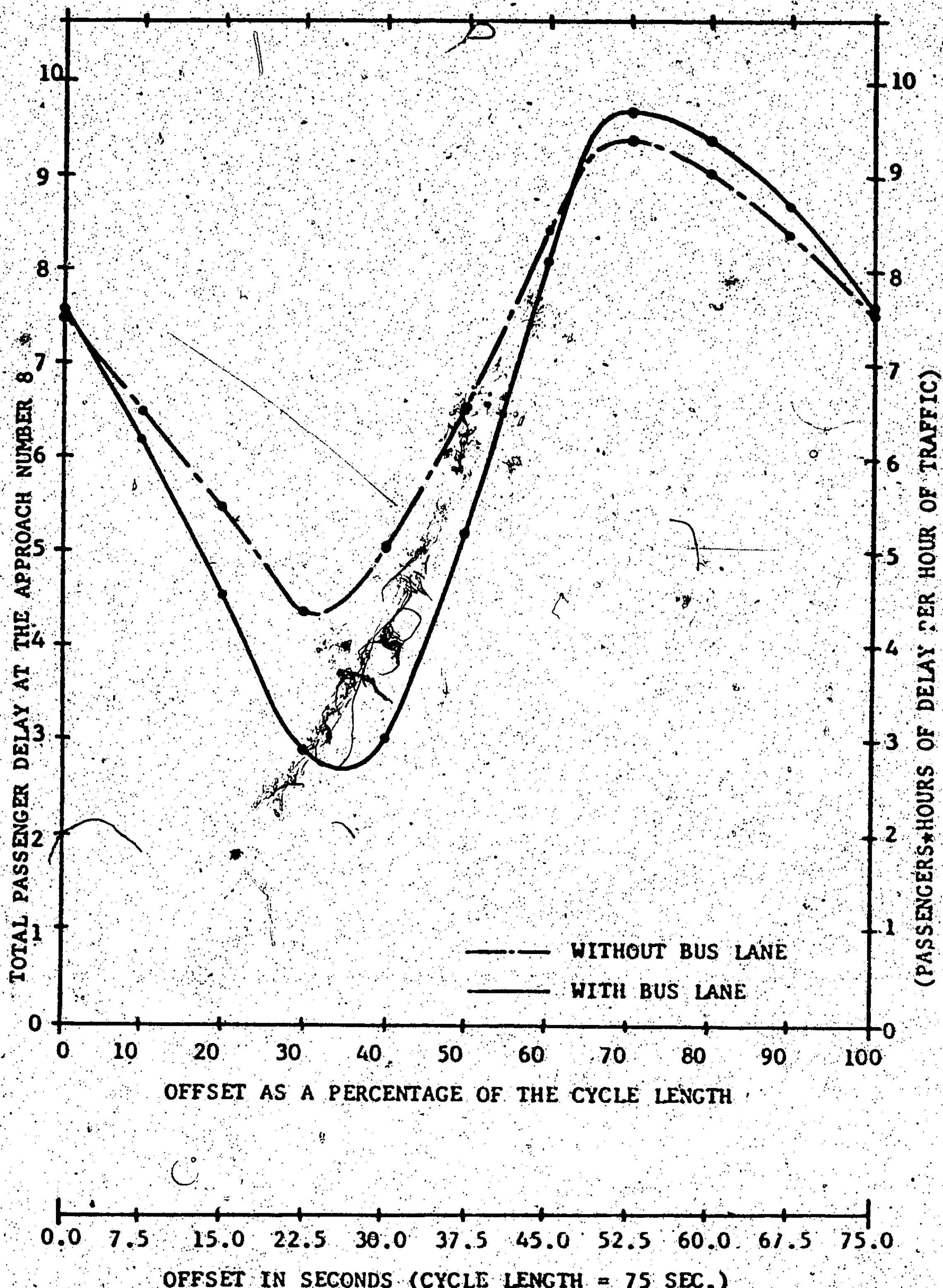


FIGURE V-5: TOTAL PASSENGER DELAY AT THE APPROACH NUMBER 8 VERSUS THE OFFSET

If it is the case, we can reach reductions of delay of about 1.5 passengers*hours by creating bus lanes.

* Maxima and minima on these curves are reached for about the same values of the offset whether or not there are bus lanes. This means that the implementation of bus lanes can generally be achieved without altering the offsets, if they are already optimum.

* The offsets which give the minimum passenger delay for the approaches 2 and 8 are not equal. Yet, in actual fact, the offsets must be the same for the approaches 2 and 8 since there are on the two ends of the same section of avenue. Hence the best offset will be the one which gives the minimum total passenger delay for both approaches 2 and 8. Figure V-6 shows the plotting of this total passenger delay. The curves have very similar shapes; the creation of bus lanes only makes them a bit sharper. One can also notice that the creation of two bus lanes on both sides reduces the passenger delay by 1.5 to 15 percent depending on the offset.

In conclusion to this particular restricted study, assuming these network and traffic conditions, one could state that, with a common cycle of 75 seconds:

a) 19 seconds (25 percent of 75, see Fig. V-2) is the best effective green time for the approaches number 1 and 3.

b) 52 seconds (70 percent of 75) is the best effective green time for the approaches 2 and 4.

c) 33 seconds (45 percent of 75, see Fig. V-3) is the best effective green time for the approaches 5 and 7.

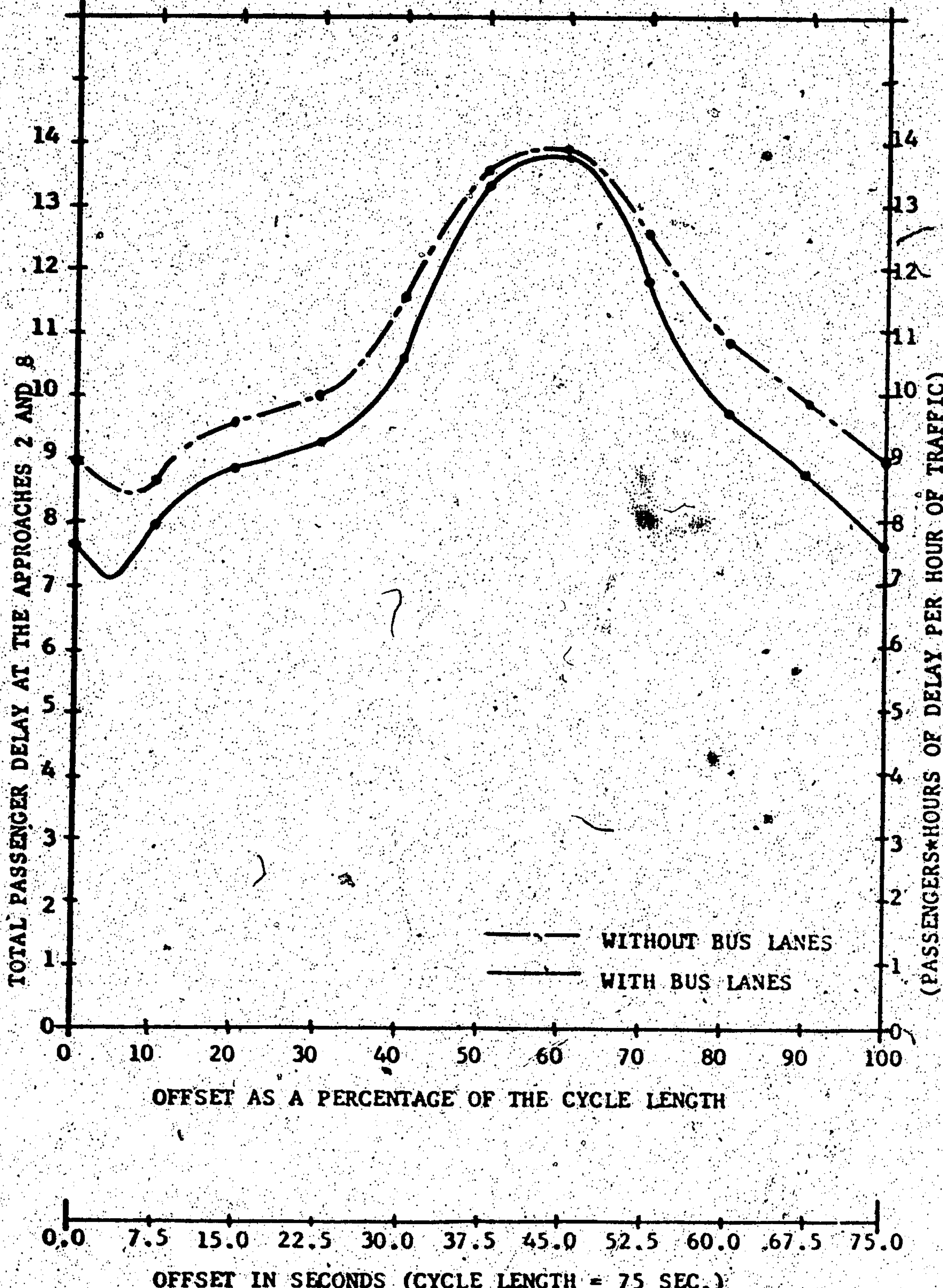


FIGURE V-6: TOTAL PASSENGER DELAY AT THE TWO ENDS OF THE LINK (APPROACHES 2 AND 8)

d) 38 seconds (50 percent of 75) is the best effective green time for the approaches 6 and 8.

e) 4 seconds (5 percent of 75) is the best offset between the starts of the effective green at the approaches 6 and 2.

f) With this timing plan, the creation of two continuous bus lanes on both sides of Jasper Avenue would reduce the passenger delay by about 15 percent on the concerned approaches. This result holds true when a third intersection (Jasper Avenue and 100 Street) is added to the network.

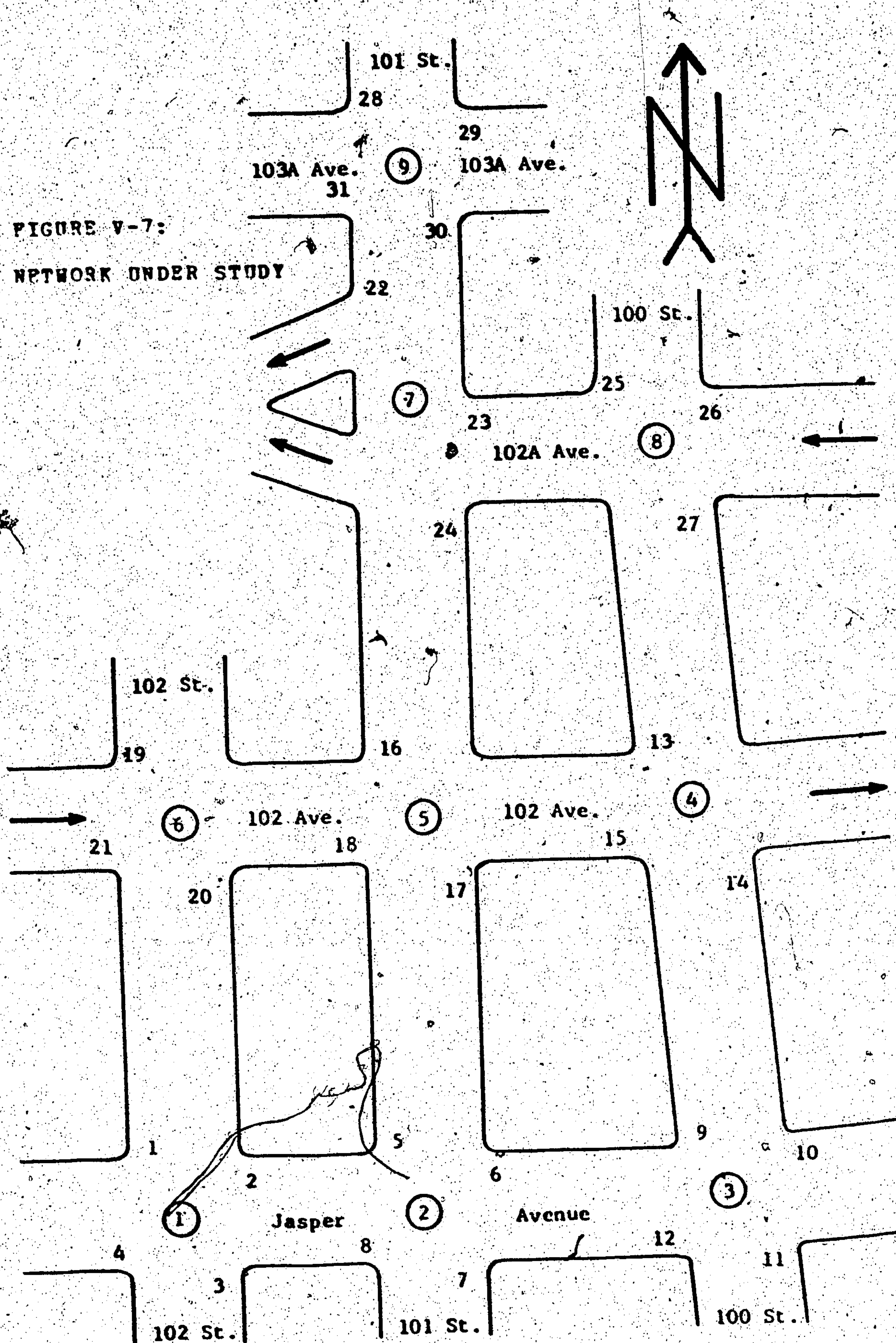
2.OPTIMIZING A TIMING PLAN

This section deals with the main use of the simulation model, since the passenger delay at the signalized intersections is the most important result of our calculations. Actually, this model was designed with the original idea of being used to optimize dynamically the timing plan on a signalized network. A further research project could eventually (see Chapter VI) lead to such implementation.

In the following pages, we study the case of an actual network in downtown Edmonton (see Figure V-7) on which the computer simulation has been applied. This network is constituted 4 by 9 intersections and 31 different approaches. Figure V-7 shows the numbering of the intersections and approaches. The study is based on the a.m. peak traffic counts in early summer 1973. The only variables in the optimization process are related to the timing plan. All the rest of the data is not supposed to vary.

FIGURE V-7:

NETWORK UNDER STUDY



In a first step, we are interested in the common cycle length. On the two-way streets, where the main problem is to find the offsets for both directions, one should use cycle lengths of about two times the best offsets. In other words, the signals should be roughly alternate on all consecutive intersections on the two-way streets. This is only possible when the distances between intersections are the same over the network. In our particular case, we use the average size of a block and the average speed to determine the best cycle length: 55 seconds (twice the average best offset between two consecutive intersections).

In a second step, offsets are chosen on all the links. Quite often the rule of alternate green cannot be followed because of too many inter-relations created by various next intersections on the network. In such a case, we take a guess and use an average value for initialization. At this stage, the first run of the model can be made on the computer. We get in our example a total passenger delay of 189.0 passengers*hours.

The last step of the optimization process only involves computer calculations. Various runs are made using the same cycle length and altering the offsets at critical intersections where the passenger delay is especially high. The use of computer plotting (see Figure IV-6) of the pattern of arrivals and departures at these critical approaches is particularly useful in determining the alterations of the timing plan. Using such a procedure, in our example, we get, after 10 runs of the simulation, a total passenger delay of 118.5 passengers*hours. A reduction of 37 percent has been implemented.

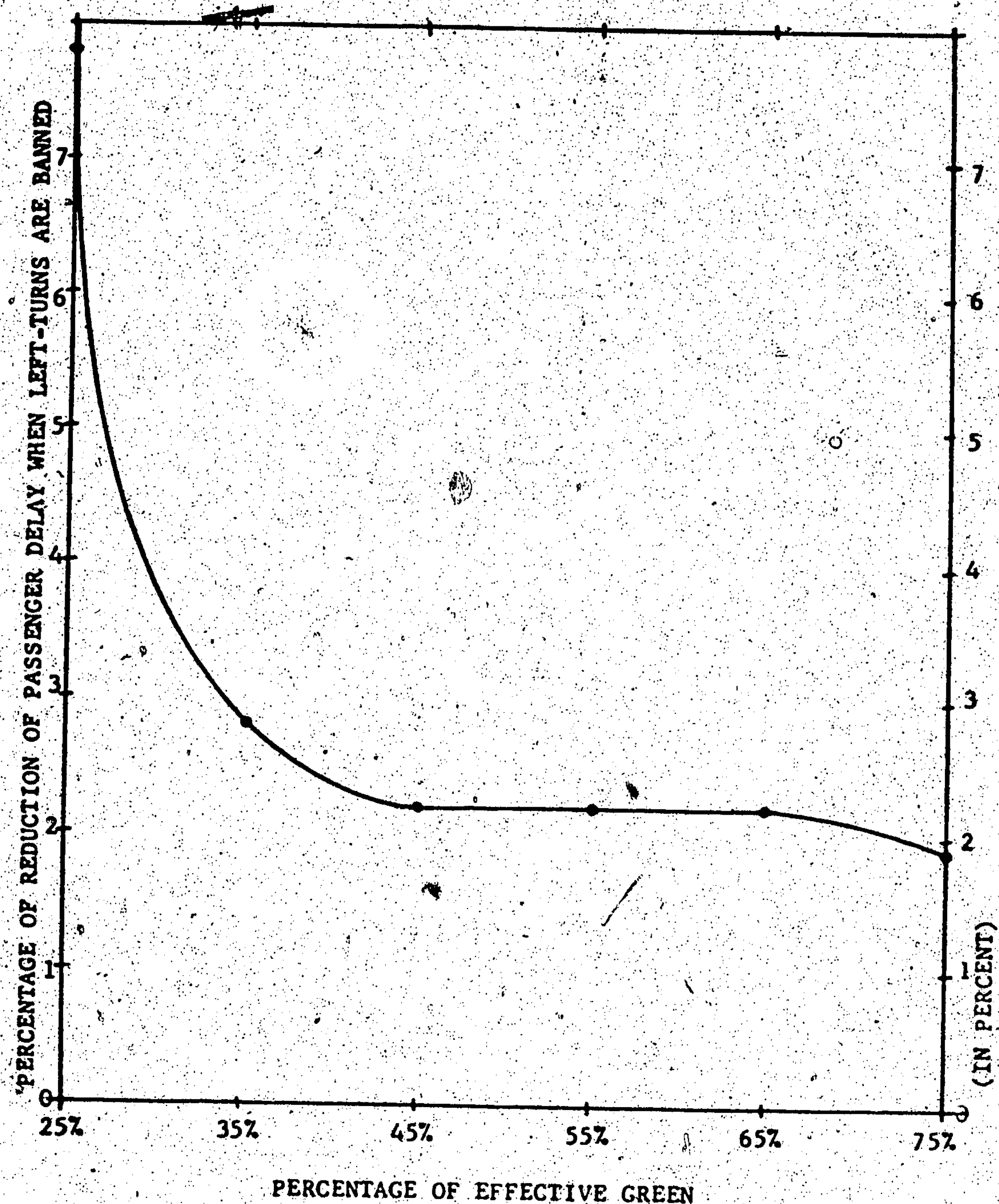
In summary, this section shows how the simulation model can be used to refine an initial timing plan on a particular example. Some precautions must however be taken. First, it cannot be used easily to determine the best cycle length. This is due mainly to the external approaches, which delays are roughly proportional to the cycle length according to Webster's formula. This is also due to the fact that only optima can be compared and, therefore, one must go through the optimization process before being able to compare the results for two different cycles. Another remark must be mentioned in this regard: in our model, the average speed and the cycle length are considered as independent variables. This is only roughly true; to get more reliable results, one should study their relation and use different speeds for various cycle lengths.

3. BANNING A LEFT-TURN

The effects of banning a left-turn can also be studied through simulation. Our model gives us results regarding an eventual reduction of passenger delay. In this section, we are going to study this phenomenon at a particular single intersection (Jasper Avenue and 1?? Street).

At this location, the left-turns are presently forbidden on the street and allowed at the two other approaches on Jasper Avenue. The eastern approach and the western approach carry respectively 18% and 6% left-turners. The use of the simulation model for this particular intersection has shown that the suppression of left-turning movements would reduce the passenger delay by about 2 percent at the two concerned approaches. The calculations were carried out with the actual setting for the signals. Such a result (2%) is not significant because of the roughness of our calculations. We must keep in mind that it has been assumed that 5 percent was the best accuracy which we could expect. We shall therefore conclude that in this particular case the fact of banning these movements does not sensibly affect the passenger delay.

This result has been obtained by using the actual timing plan for the signals. The use of different percentages of effective green will cause different degrees of saturation. The results from various computer runs with a large range of green periods show (see Figure V-8) that: when the approach is close to saturation, the banning of a left-turn can considerably reduce the passenger delay. In our case, an effective green time of 25 percent corresponding to a degree of saturation of 0.92, gives a delay which can be reduced by almost 8 percent if the left-turns are banned. Taken to an extreme, when the degree of saturation reaches 1.00, the delay becomes infinite and the suppression of left-turners reduces it to a finite value.



PERCENTAGE OF EFFECTIVE GREEN	25	35	45	55	65	75
PERCENTAGE OF REDUCTION OF PASSENGER DELAY WHEN LEFT-TURNS ARE BANNED	7.8	2.8	2.2	2.2	2.2	1.9

-8-

THE EFFECT OF BANNING A LEFT-TURN

4. CHANGING THE MODE SPLIT

Here is another example of the use of the simulation model. It is possible to evaluate through computer calculations, the effect of the change of the mode split on the passenger delay at signals. Such figures are very difficult to get through field testing, and, simulation is one of the few means which are available to us. In this paragraph we consider the network of 9 intersections which has been studied in the optimization of the timing plan. We assume constant occupancies of 1.3 and 25 for, respectively, private cars and transit buses. The total number of passengers over the whole network is not supposed to vary. We analyse, first of all, three different changes in the mode split:

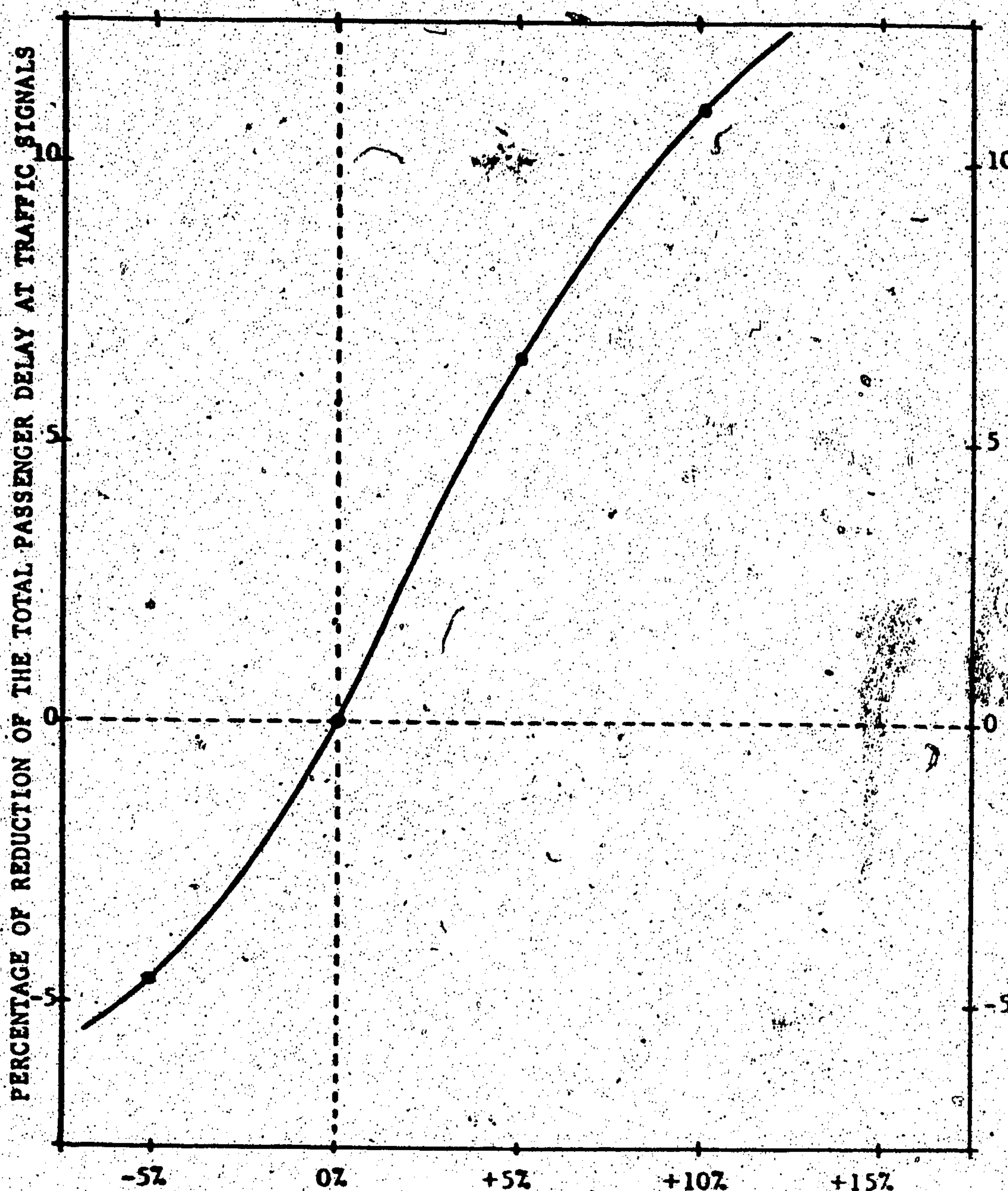
a) 5% passengers shift from buses to private cars,

b) 5% passengers shift from private cars to buses, and,

c) 10% passengers shift from private cars to buses.

The results of the simulation on these three cases are represented by Figure V-9. It is shown that the percentage of reduction of passenger delay is approximately equal to the percentage of passengers shifting from private to transit vehicles. This means, for example, that when 10 percent passengers shift from cars to buses, there is a reduction of about 10 percent (11% exactly) in the total passenger delay at signals.

Two other cases, which are very extreme, have also been studied:



PERCENTAGE OF PASSENGERS SHIFTING FROM CARS TO BUSES	-5	0	+5	+10
PERCENTAGE OF REDUCTION OF THE TOTAL PASSENGER DELAY	-4.6	0.0	+6.6	+11

FIGURE V-9:

THE EFFECT OF CHANGING THE MODE SPLIT

d) No buses (bus drivers' strike, for example): all the transit passengers shift to the automobile. In this case, since people are pooling together more easily, we have increased the average occupancy of a car to an assumed value of 1.8. Calculations carried out on this basis show complete saturation on some approaches. This means that, for such extreme situations, the timing plan cannot be kept the same. It has to be altered to handle this amount of vehicles.

e) no private cars: all the passenger flow is handled by transit vehicles. In this case, we assume that the bus occupancy increases to 35 instead of being 25. There is no saturation problem since the number of pcu's over the network is reduced to its lowest value. The delay reduction reaches 25 percent if the timing plan is kept the same.

All these results should be looked at with much care. Two major reasons can mislead us in our analysis. Firstly, we have to stay aware of the fact that we are only studying delays at signalized intersections and, therefore, other delays which are also very important in any evaluation are not taken into account. Waiting times at the bus stops are some of them; they vary with the frequency of service, i.e. with the number of buses. Secondly, in these calculations, the mode split is the only variable. Other variables which we use in our computations are in fact related to it but our model considers them as independent. This does not affect the final results very much when shifts of 5 or 10 percent are studied. For more extreme conditions, results are probably biased. Some more relations concerning, for example:

the variations of occupancies with the mode split,

the variation of the average speed versus the degree of saturation (i.e. the mode split),

are needed to represent better the actual situation.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

1. CONCLUSIONS

This thesis investigated, through the use of a computer simulation, the various relations between passenger delay at signalized intersections and the traffic conditions.

In a first part (Chapters III and IV), the simulation is explained extensively. Calculations are carried out on the approach basis. For this purpose a comprehensive description of the network, the traffic and the timing plan is needed. The main results, given for each approach, deal essentially with passenger delay. Other figures, concerning occupancies, vehicular delay and number of stops are also printed out.

The second part of this thesis (Chapter V) is an application of the simulation model to some specific traffic problems in Edmonton. Hence, the conclusions which are drawn are not general and only apply to the situation under study. The purpose of this second part is to give examples of real life applications of our model. Yet, through the analysis of the results that we get, one can state that, for similar conditions:

1. The use of the model to optimize the timing plan is particularly useful. Starting with an initial signal setting calculated by hand, and, using the computer simulation to

guide us through the trial and error procedure, reductions of passenger delay of more than 35 percent can be reached. This significant reduction is essentially due to the facts that: firstly, most of the methods used to calculate by hand timing plans are designed to reduce vehicular delay (and not passenger delay); secondly, for simplicity, the turning movements are usually neglected when computing signal progression by hand.

2. Banning a left-turn at one approach is very effective when the approach is close to saturation. For undersaturated approaches (degree of saturation of less than 0.5), usually, the prohibition of an average left-turn movement (5% to 20% of the vehicles) does not produce a significant reduction of delay.

3. A change in the mode split also affects the passenger delay at the signals. Approximately, the percentage of reduction of passenger delay can be expected to be equal to the percentage of passengers shifting from private cars to transit buses.

4. The creation of a bus lane can reduce the passenger delay at signals by about 15 percent on busy, wide streets at the peak hour when more than 60 percent of the passengers are travelling in transit vehicles. However, one must be very careful in choosing a green progression when there are bus lanes. Passenger delay curves are much sharper and a mistake in choosing offsets, or green splits could even produce an increase of delay in some cases.

RECOMMENDATIONS

The main purpose of this research work was, originally, to use the passenger delay at signals as a criterion to optimize dynamically the timing plan and regulate the traffic. In this thesis, a computer program has been written (see Appendix C) to evaluate this criterion, the other ~~data (traffic~~, network, timing plan) being given. Two further steps could be made:

1. The optimization of the timing plan which is now done manually, using the results of simulation as a guide, could be performed by the computer. For this purpose, a program should be developed to calculate the alterations which affect green splits and offsets. At this stage, only the data concerning the traffic and the network would have to be given. The best timing plan would be found out by the optimization program.
2. The last step could be implemented by using dynamic traffic counts to feed the computer. For example, every third minute, the counts of the past three minutes could be used to determine the best signal settings for the following three minutes. One must be careful not to do this step unless a lot of field testing had been made beforehand. Also, the regularity of the variations of the traffic flows throughout the peak hours, or some other specific condition, could not justify this last step. Such a use of the computer for dynamic regulation involves a lot of money and cautious economic studies must be done.

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APPENDIX A

SATURATION FLOW

According to F.V. Webster and B.M. Cobbe, the saturation flow (s) expressed in through passenger car units per hour is supposed to be proportional to the width (w) of the approach. The Road Research technical paper No. 56 (Webster and Cobbe, 1966) uses, for approaches more than 17 feet wide, the relation s=160*w where w is expressed in feet. The effect of gradients, composition of traffic, turning movements, pedestrians, parked vehicles and site characteristics are then studied. Yet, the relation above written does not represent properly Edmonton traffic. It probably fits better European conditions for which it has been established.

In Canada, the use of larger cars and the smoother flow due to less loaded networks make the former relation unapplicable. Also, icy streets during the winter reduces considerably the traction and, therefore, the saturation flow is much smaller. However, the type of formula does not seem to change and a proportional relation still works well. For these reasons, in our simulation model, the ratio s/w (=SATUR) is set according to field measurements made in the city under study.

In Edmonton, with summer conditions, saturated approaches in the downtown area are properly represented at the peak hours by 78<SATUR<114. In our model we usually use s=92*w. This ratio should probably be reduced by about 30 percent in the case of icy streets.

APPENDIX B

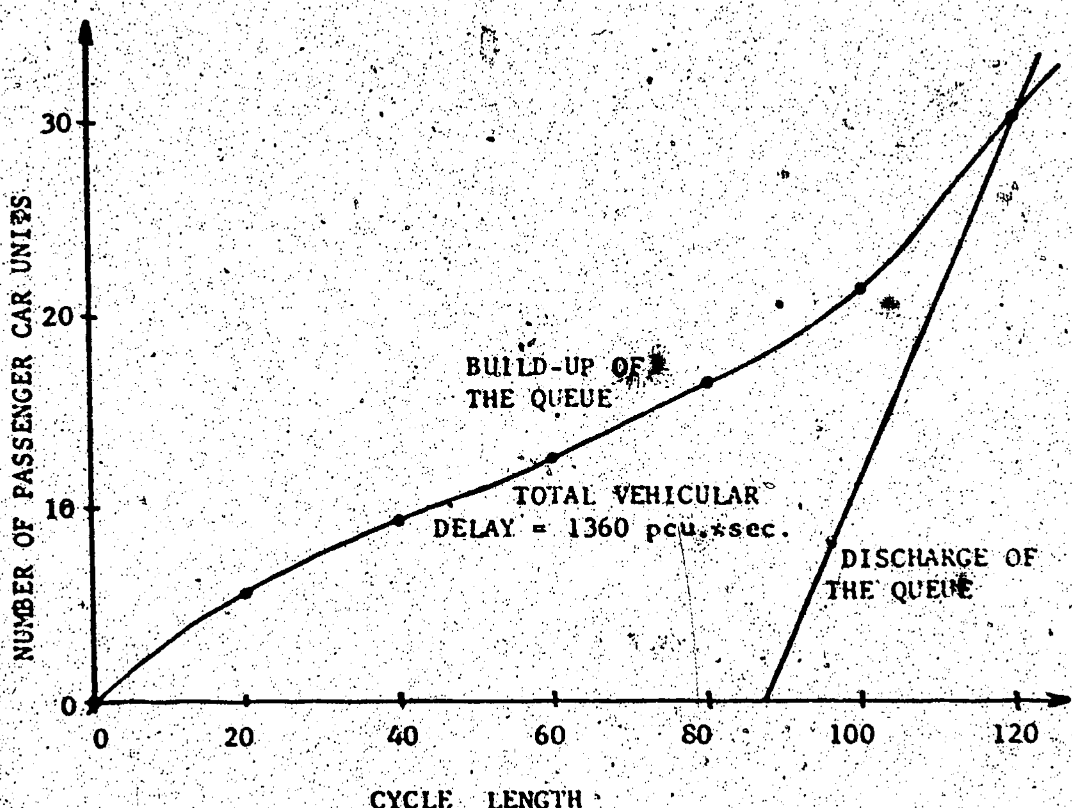
TEST OF WEBSTER'S EXPRESSION OF DELAY

Webster's simplified expression of delay has been derived from simulation at the Road Research Laboratory in Great Britain. It seems useful to check its accuracy before using it with Western Canadian traffic conditions (i.e. larger cars, smoother flow than in England).

For this purpose, measurements were carried out on one approach of an intersection in Edmonton: 82nd Avenue and 109th Street. Seven sets of cycle lengths with different green times and different flow patterns were tried out.

The pattern of arrivals is plotted below; departures from the stop line are supposed to occur at the saturation rate. The area in between the two curves represent the total vehicular delay.

average delay per pcu = 1360/30 = 45.5 seconds



Then, using the Webster simplified expression we compute the average delay per passenger car unit. We have:

g = green time = 32 seconds,

c = cycle time = 128 seconds,

$\lambda = g/c = 32/128 = 0.25$ pcu/sec.,

q = actual flow = 30/128 = 0.23 pcu/sec.,

s = saturation flow = 1.23 pcu/sec.,

x = degree of saturation = $q/\lambda s$ = 0.74, and,

d =

d = <u>42.6 seconds</u>

In conclusion, one can state that Webster's formula is accurate enough and applies fairly well (within 5%) to Edmonton traffic conditions.

APPENDIX C

THE COMPUTER PROGRAM

```
      DIMENSION INTER(99),IDENT(99),IOPPO(99),IWDTH(99),ILNGT(99)
      DIMENSION IWILI(99),ISPID(99),IORGN(3,99),ISTRT(99),IAVNU(99)
      DIMENSION IBSLA(99),PARKN(99)
      DIMENSION ILTPC(99),ITRPC(99),IRTPC(99)
      DIMENSION ILTBS(99),ITRBS(99),IRTBS(99)
      DIMENSION TRPCU(99),PCUDC(99),PCUDL(99),TPCUD(99),PASDL(99)
      DIMENSION DEPAW(20,99),QUEUE(20,99),DISCH(20,99)
      DIMENSION PQUEU(20,99),PDISH(20,99)
      DIMENSION CALIB(99)
      DIMENSION IRAFF(3),II(3)
      DIMENSION IBEGR(99),IBERD(99)
      DIMENSION OOOOO(21),DDDDD(21),XXXXX(21)
      DIMENSION TITLE(20,99)
      DIMENSION ALPHA(20)
      DIMENSION PCUST(99),PASTO(99)
      DIMENSION INDEX(99)
```

READ DATA

```
      READ (5,13) APLOT
   13 FORMAT (1X,F4.2)
      READ (5,10) CAROC,BUSOC,EBSPC,IAREA,SATUR,RA1,RA2,ESD
   10 FORMAT (1X,F3.1,1X,F3.0,1X,F3.1,1X,I3,F5.1,1X,F4.2,1X,F4.2,F3.0)
      READ (5,11) MXNTR,MXIAP
   11 FORMAT (1X,I2,1X,I2)
      DO 300 I=1,MXIAP,1
      READ (5,12) IAP,INTER(IAP),IDENT(IAP),IOPPO(IAP),IWDTH(IAP),ILNGT(
     1IAP),IWILI(IAP),ISPID(IAP),IORGN(1,IAP),IORGN(2,IAP),IORGN(3,IAP),
     2ISTRT(IAP),IAVNU(IAP),IBSLA(IAP),PARKN(IAP),ILTPC(IAP),ITRPC(IAP),
     3IRTPC(IAP),ILTBS(IAP),ITRBS(IAP),IRTBS(IAP)
   12 FORMAT (1X,I2,1X,I2,1X,I1,1X,I1,1X,I2,1X,I3,5(1X,I2),1X,I3,1X,I3,1
     1X,I1,1X,F3.1,4X,I4,1X,I4,1X,I4,3(1X,I3))
  300 CONTINUE
      READ (5,24) ICYCL
   24 FORMAT (1X,I3)
      CYCLE=ICYCL
      DO 301 I=1,MXIAP,1
      READ (5,27) IAP,IBEGR(IAP),IBERD(IAP)
   27 FORMAT (3(1X,I2))
  301 CONTINUE
      DO 311 IAP=1,MXIAP,1
      READ (5,310) (TITLE(I,IAP),I=1,20,1)
  310 FORMAT (20A4)
  311 CONTINUE
```

WRITE DATA

```
      WRITE (6,25)
   25 FORMAT(1H1,6(/),59X,'DATA CHECK',6(/),1X,'IAP   INTER IDENT IOPPO
     1IWDTH ILNGT IWILI ISPID IORGN IORGN IORGN ISTRT IAVNU IBSLA PARKN
     2ILTPC ITRPC IRTPC ILTBS ITRBS IRTBS',/,7X,7('(IAP) '),'1,IAP 2,IAP
     3 3,IAP',10(1X,'(IAP)'))
      DO 500 IAP=1,MXIAP,1
      WRITE (6,26)IAP,INTER(IAP),IDENT(IAP),IOPPO(IAP),IWDTH(IAP),ILNGT(
     1IAP),IWILI(IAP),ISPID(IAP),IORGN(1,IAP),IORGN(2,IAP),IORGN(3,IAP),
     2ISTRT(IAP),IAVNU(IAP),IBSLA(IAP),PARKN(IAP),ILTPC(IAP),ITRPC(IAP),
     3IRTPC(IAP),ILTBS(IAP),ITRBS(IAP),IRTBS(IAP)
   26 FORMAT (1H ,I2,3X,13(2X,I3,1X),2X,F3.1,6(2X,I4))
  500 CONTINUE
      WRITE (6,14) MXNTR,MXIAP,CAROC,BUSOC,EBSPC,IAREA,SATUR,RA1,RA2,ESD
   14 FORMAT (1H ,3(/),' WE  HAVE : ',I2,' INTERSECTIONS',/,12X,I2,' DIF
```

```
    1FERENT APPROACHES',/,12X,F3.1,' PASSENGERS PER CAR',/,12X,F3.0,' P
    2ASSENGERS PER BUS',/,' WE ASSUME: ','1 LARGE VEHICLE =',F5.2,' P.C
    3.U.S',/,12X,'1 PCU IN A QUEUE TAKES ',I3,' SQU. FT.  ',/,12X,'SATU
    4RATION FLOW = ',F4.0,' TIMES THE APPROACH WIDTH IN FEET',/,12X,'RA
    51 = ',F4.2,/,12X,'RA2 = ',F4.2,/,12X,'1 STOP EQUIVALENT TO ',F3.0,
    6' SECONDS OF DELAY')

                          CALCULATION OF TRPCU(IAP) AND PCUOC(IAP)
      DO 100 IAP=1,MXIAP,1
      E=ILTPC(IAP)
      F=ILTBS(IAP)
      A=E+EBSPC*F
      E=ITRPC(IAP)
      F=ITRBS(IAP)
      B=E+EBSPC*F
      E=IRTPC(IAP)
      F=IRTBS(IAP)
      C=E+EBSPC*F
      D=A+B+C
      IF (D-10*C) 15,15,16
   15 IF (ICPPO(IAP)) 17,17,18
   17 TRPCU(IAP) = A+B+(D/10.0)+1.25*(C-D/10.0)
      GO TO 21
   18 TRPCU(IAP) = (1.75*A)+B+(D/10.0)+1.25*(C-D/10.0)
      GO TO 21
   16 IF (IOPPO(IAP)) 19,19,20
   19 TRPCU(IAP) = D
      GO TO 21
   20 TRPCU(IAP) = (1.75*A)+B+C
      GO TO 21
   21 A = ILTPC(IAP)+ITRPC(IAP)+IRTPC(IAP)
      B = ILTBS(IAP)+ITRBS(IAP)+IRTBS(IAP)
      PCUOC(IAP) = (CAROC*A+BUSOC*B)/TRPCU(IAP)
  100 CONTINUE

                          CALCULATION OF DEPAR(ITIME,IAP)
      DO 200 IAP=1,MXIAP,1
      DO 204 ITIME=1,20,1
      DEPAR(ITIME,IAP) = 0.0
  204 CONTINUE
      IF (IORGN(1,IAP)-99) 31,30,31
   30 CALIB(IAP) = 0.0
      GO TO 200
   31 DO 201 J=1,3,1
      TRAFF(J) =0.0
      IF (IORGN(J,IAP)) 32,201,32
   32 II(J) = IORGN(J,IAP)
      K = (4+IDENT(IAP)-IDENT(II(J)))
      GO TO (34,35,36,37,34,35,36),K
   34 A = IRTPC(II(J))
      B = IRTBS(II(J))
```

A=NUMBER OF LEFT-TURNING PCU'S
B=NUMBER OF THROUGH PCU'S
C=NUMBER OF RIGHT-TURNING PCU'S

CAROC=CAR OCCUPANCY
BUSOC=BUS OCCUPANCY
SEE FIGURE IV-2 FOR PCUOC

SEE FIGURE IV-3

II(1), II(2) AND II(3)
ARE THE ORIGIN APPROACHES

```
      TRAFF(J) = A+(EBSPC*B)
      GO TO 201
   35 WRITE (6,38) IAP
   38 FORMAT (1H1,6(/),1X,'MISTAKE IN DATA OF APPROACH NUMBER  ',I2)
      GO TO 200
   36 A = ILTPC(II(J))
      B = ILTBS(II(J))
      TRAFF(J) = A+(EBSPC*B)
      GO TO 201
   37 A = ITRPC(II(J))
      B = ITRBS(II(J))
      TRAFF(J) = A+(EBSPC*B)
      GO TO 201
  201 CONTINUE
      A = 0.0
      DO 202 J=1,3,1
      A = A+TRAFF(J)
  202 CONTINUE
      CALIB(IAP) = (TRPCU(IAP))/(A*PARKN(IAP))
      DO 203 J=1,3,1
      TRAFF(J) = TRAFF(J)*CALIB(IAP)
  203 CONTINUE
      DO 205 J=1,3,1
      IF (IORGN(J,IAP)) 39,205,39
   39 EFFGR = IBERD(II(J))-IBEGR(II(J))
      IF (EFFGR) 40,41,42
   40 EFFGR = EFFGR+20.0
      GO TO 42
   41 WRITE (6,43) II(J)
   43 FORMAT (1H1,6(/),1X,'MISTAKE IN TIMING PLAN OF APPROACH NUMBER
     1I2)
      GO TO 200
   42 A = (TRAFF(J)*CYCLE)/(3600.0*EFFGR)
      IF (IBERD(II(J))-IBEGR(II(J))) 44,41,45
   44 I = IBEGR(II(J))+1
      IF (I-20) 215,215,216
  215 DO 206 ITIME=I,20,1
      DEPAR(ITIME,IAP) = A
  206 CONTINUE
  216 I = IBERD(II(J))
      DO 207 ITIME=1,I,1
      DEPAR(ITIME,IAP) = A
  207 CONTINUE
      GO TO 205
   45 I = IBEGR(II(J))+1
      K = IBERD(II(J))
      DO 208 ITIME=I,K,1
      DEPAR(ITIME,IAP) = A
  208 CONTINUE
      GO TO 205
  205 CONTINUE
  200 CONTINUE
```

A=NUMBER OF PCU'S RELEASED DURING EACH SECTION OF THE EFFECTIVE GREEN (SEE FIGURE IV-3)

CALCULATION OF QUEUE(ITIME,IAP)

SEE FIGURE IV-4

L=NUMBER OF SECTIONS OF THE CYCLE VEHICLES HAVE TO FLOW TO REACH THE QUEUE

```
C
      DO 217 IAP=1,MXIAP,1
      DO 244 ITIME=1,20,1
      QUEUE(ITIME,IAP) = 0.0
  244 CONTINUE
      IF (IORGN(1,IAP)-99) 243,217,243
  243 I = IBERD(IAP)-1
      J = IBERD(IAP)
      K = IBERD(IAP)+1
      IF (J-1) 218,218,213
  218 DO 221 ITIME=K,20,1
      M = ITIME-L(ILNGT(IAP),I=ILI(IAP),QUEUE((ITIME-1),IAP),IAREA,ISPID
     1(IAP),ICYCL,RA1,RA2)
  225 IF (M) 226,226,227
  226 M = M+20
      GO TO 225
  227 QUEUE(ITIME,IAP) = QUEUE((ITIME-1),IAP)+DEPAR(M,IAP)
  221 CONTINUE
      GO TO 217
  213 IF (J-20) 220,219,220
  219 M = 1-L(ILNGT(IAP),I=ILI(IAP),QUEUE(20,IAP),IAREA,ISPID(IAP),ICYCL
     1,RA1,RA2)
  240 IF (M) 241,241,242
  241 M = M+20
      GO TO 240
  242 QUEUE(1,IAP) = DEPAR(M,IAP)
      DO 222 ITIME=2,19,1
      M = ITIME-L(ILNGT(IAP),I=ILI(IAP),QUEUE((ITIME-1),IAP),IAREA,ISPID
     1(IAP),ICYCL,RA1,RA2)
  228 IF (M) 229,229,230
  229 M = M+20
      GO TO 228
  230 QUEUE(ITIME,IAP) = QUEUE((ITIME-1),IAP)+DEPAR(M,IAP)
  222 CONTINUE
      GO TO 217
  220 DO 223 ITIME=K,20,1
      M = ITIME-L(ILNGT(IAP),I=ILI(IAP),QUEUE((ITIME-1),IAP),IAREA,ISPID
     1(IAP),ICYCL,RA1,RA2)
  231 IF (M) 232,232,233
  232 M= M+20
      GO TO 231
  233 QUEUE(ITIME,IAP) = QUEUE((ITIME-1),IAP)+DEPAR(M,IAP).
  223 CONTINUE
      M = 1-L(ILNGT(IAP),I=ILI(IAP),QUEUE(20,IAP),IAREA,ISPID(IAP),ICYCL
     1,RA1,RA2)
  234 IF (M) 235,235,236
  235 M = M+20
      GO TO 234
  236 QUEUE(1,IAP) = QUEUE(20,IAP)+DEPAR(M,IAP)
      DO 224 ITIME=2,J,1
      M = ITIME-L(ILNGT(IAP),I=ILI(IAP),QUEUE((ITIME-1),IAP),IAREA,ISPID
     1(IAP),ICYCL,RA1,RA2)
  237 IF (M) 238,238,239
  238 M = M+20
      GO TO 237
```

```
239 QUEUE(ITIME,IAP) = QUEUE((ITIME-1),IAP)+DEPAR(M,IAP)
224 CONTINUE
    QUEUE(J,IAP) = 0.0
    GO TO 217
217 CONTINUE
    DO 290 IAP=1,MXIAP,1
    DO 291 ITIME=1,20,1
    PQUEU(ITIME,IAP)=QUEUE(ITIME,IAP)*PCUOC(IAP)
291 CONTINUE
290 CONTINUE
    WRITE (6,48)
 48 FORMAT (1H1,6(/),56X,'QUEUE(ITIME,IAP)',6(/),1X,'ITIME =      1
   1 2       3       4       5       6       7       8       9      10      11      12
   213      14      15      16      17      18      19      20',/,1X,'IAP    =',/,1X,
   3127('*'))
    DO 502 IAP=1,MXIAP,1
    WRITE (6,47) IAP,(QUEUE(ITIME,IAP),ITIME=1,20,1)
 47 FORMAT (1H ,1X,I2,3X,'*',20(1X,F5.1))
502 CONTINUE
    WRITE (6,312)
312 FORMAT (1H1,6(/),56X,'PQUEU(ITIME,IAP)',6(/),1X,'ITIME =      1
   1 2       3       4       5       6       7       8       9      10      11      12
   213      14      15      16      17      18      19      20',/,1X,'IAP    =',/,1X,
   3127('*'))
    DO 313 IAP=1,MXIAP,1
    WRITE (6,314) IAP,(PQUEU(ITIME,IAP),ITIME=1,20,1)
314 FORMAT (1H ,1X,I2,3X,'*',20(1X,F5.1))
313 CONTINUE
```

PQUEU=NUMBER OF PASSENGERS IN THE QUEUE

```
                              CALCULATION OF DISCH(ITIME,IAP)
    DO 250 IAP=1,MXIAP,1
    DO 251 ITIME=1,20,1
    DISCH(ITIME,IAP) = 0.0
251 CONTINUE
    IF (IORGN(1,IAP)-99) 257,250,257
257 A = INDTH(IAP)
    B = SATUR*A
    C = (B*CYCLE)/(3600.0*20.0)
    I = IBEGR(IAP)
    J = IBERO(IAP)
    K = J-1
    IF (K) 252,252,253
252 D = IBEGR(IAP)
    DO 254 ITIME=1,20,1
    E = ITIME
    DISCH(ITIME,IAP) = C*(E-D)
254 CONTINUE
    DO 255 ITIME=1,J,1
    E = ITIME
    DISCH(ITIME,IAP) = C*(E+20.0-D)
255 CONTINUE
    GO TO 250
253 D = IBEGR(IAP)
```

SEE FIGURE 1V-5

B=SATURATION FLOW IN PCU'S PER HOUR
C=NUMBER OF PCU'S DISCHARGING THE QUEUE AT EVERY SECTION OF THE CYCLE

```
      DO 256 ITIME=I,J,1
      E = ITIME
      DISCH(ITIME,IAP) = C*(E*D)
  256 CONTINUE
      GO TO 250
  250 CONTINUE
      DO 292 IAP=1,MXIAP,1
      DO 293 ITIME=1,20,1
      PDISH(ITIME,IAP)=DISCH(ITIME,IAP)*PCUOC(IAP)
  293 CONTINUE
  292 CONTINUE
      WRITE (6,50)
   50 FORMAT (1H1,6(/),56X,'DISCH(ITIME,IAP)',6(/),1X,'ITIME =      1
     1 2        3        4        5        6        7        8        9       10       11       12
     213       14       15       16       17       18       19       20',/,1X,'IAP    =',/,1X,
     3127('*'))
      DO 503 IAP=1,MXIAP,1
      WRITE (6,47) IAP,(DISCH(ITIME,IAP),ITIME=1,20,1)
  503 CONTINUE
      WRITE (6,315)
  315 FORMAT (1H1,6(/),56X,'PDISH(ITIME,IAP)',6(/),1X,'ITIME =      1
     1 2        3        4        5        6        7        8        9       10       11       12
     213       14       15       16       17       18       19       20',/,1X,'IAP    =',/,1X,
     3127('*'))
      DO 316 IAP=1,MXIAP,1
      WRITE (6,317) IAP,(PDISH(ITIME,IAP),ITIME=1,20,1)
  317 FORMAT (1H ,1X,I2,3X,'*',20(1X,F5,1))
  316 CONTINUE
```

PDISH=NUMBER OF PASSENGERS DISCHARGING THE QUEUE

CALCULATION OF STOPS

```
      NSTOT=0
      DO 330 IAP=1,MXIAP,1
      IF (IORGN(1,IAP)-99) 331,330,331
  331 I=IBEGR(IAP)
      J=IBERD(IAP)
      IF (I-J) 332,333,333
  332 DO 334 ITIME=I,J,1
      IF (QUEUE(ITIME,IAP)-DISCH(ITIME,IAP)) 335,335,334
  334 CONTINUE
      GO TO 335
  333 DO 336 ITIME=I,20,1
      IF (QUEUE(ITIME,IAP)-DISCH(ITIME,IAP)) 335,335,336
  336 CONTINUE
      DO 337 ITIME=1,J,1
      IF (QUEUE(ITIME,IAP)-DISCH(ITIME,IAP)) 335,335,337
  337 CONTINUE
  335 PCUST(IAP)= DISCH(ITIME,IAP)
      PASTO(IAP)=PCUST(IAP)*PCUOC(IAP)*3600.0/CYCLE
      NSTOT=FLOAT(NSTOT)+PASTO(IAP)
  330 CONTINUE
```

THIS PARAGRAPH ONLY CONCERNS INTERNAL APPROACHES
THE INTERSECTION OF THE TWO CURVES PQUEU AND PDISH GIVES THE NUMBER OF PASSENGER STOPS.
(SEE FIGURE III-4)

```
C                                   CALCULATION OF DELAYS
      TPDSI = 0.0
      DO 260 IAP=1,MXIAP,1
      PCUDL(IAP) = 0.0
      TPCUD(IAP) = 0.0
      PASDL(IAP) = 0.0
      IF (IORGN(1,IAP)-99) 261,340,261
  261 DO 262 ITIME=1,20,1
      IF (QUEUE(ITIME,IAP)-DISCH(ITIME,IAP)) 262,262,263
  263 TPCUD(IAP) = TPCUD(IAP)+(QUEUE(ITIME,IAP)-DISCH(ITIME,IAP))/20.0
  262 CONTINUE
      GREEN=IBERD(IAP)-IBEGR(IAP)
      IF (GREEN) 410,411,411
  410 GREEN=GREEN+20.0
  411 ALABD=GREEN/20.0
      XXX=(TRPCU(IAP))/(SATUR*FLOAT(I*DTH(IAP))*ALABD)
      DDD=XXX*XXX/(4.0*(1.0-XXX))
      TPCUD(IAP)=TPCUD(IAP)+DDD
      PCUDL(IAP) = TPCUD(IAP)*3600.0/TRPCU(IAP)
      PASDL(IAP) = TPCUD(IAP)*PCUOC(IAP)
      TPDSI = TPDSI+PASDL(IAP)
      GO TO 260
  340 GREEN = IBERD(IAP)-IBEGR(IAP)
  343 IF (GREEN) 341,341,342
  341 GREEN = GREEN+20.0
      GO TO 343
  342 GREEN=GREEN*CYCLE/20.0
      QQQ = TRPCU(IAP)
      ALABD = GREEN/CYCLE
      XXX = QQQ/(SATUR*FLOAT(I*DTH(IAP))*ALABD)
      QQQ = TRPCU(IAP)/3600.0
      PCUDL(IAP)=(0.9*CYCLE*(1.0-ALABD)**2.0)/(2.0*(1.0-ALABD*XXX))+(0.
     1*XXX**2.0)/(2.0*QQQ*(1.0-XXX))
      TPCUD(IAP) = (PCUDL(IAP)*TRPCU(IAP))/3600.0
      PASDL(IAP)=TPCUD(IAP)*PCUOC(IAP)
      YYY=QQQ/(SATUR*FLOAT(I*DTH(IAP)))
      EEE =(1.0-ALABD)/(1.0-YYY)
      PASTO(IAP) = EEE*TRPCU(IAP)*PCUOC(IAP)
      PCUST(IAP) = EEE*TRPCU(IAP)*CYCLE/3600.0
      NSTOT=NSTOT+PASTO(IAP)
      TPDSI = TPDSI+PASDL(IAP)
  260 CONTINUE
C
C
C                                   CALCULATION OF INDEX(IAP)
      TINDX=0.0
      DO 350 IAP=1,MXIAP,1
      A=PASDL(IAP)+(FSD*PASTO(IAP))/3600.0
      INDEX(IAP)=A
      TINDX=TINDX+A
  350 CONTINUE
      DO 351 IAP=1,MXIAP,1
      A=INDEX(IAP)
      B=A/(TINDX/MXIAP)
```

SEE FIGURE IV-5

USE OF WEBSTER'S SIMPLIFIED EXPRESSION OF DELAY FOR EXTERNAL APPROACHES

CALCULATION OF PASSENGER STOPS FOR EXTERNAL APPROACHES

```
      I=B
      C=I
      IF (B-C-0.5) 352,353,353
  353 INDEX(IAP)=C+1
      GO TO 351
  352 INDEX(IAP)=I
  351 CONTINUE
      WRITE (6,51)
   51 FORMAT (1H1,6(/),50X,'FINAL RESULTS OF SIMULATION',6(/),' IAP
     1TRPCU(IAP)    PCUOC(IAP)    PCUDL(IAP)    TPCUD(IAP)    PASOL(
     2IAP)    PCUST(IAP)    PASTO(IAP)    INDEX(IAP)')
      DO 504 IAP=1,MXIAP,1
      WRITE (6,52) IAP,TRPCU(IAP),PCUOC(IAP),PCUDL(IAP),TPCUD(IAP),PASOL
     1(IAP),PCUST(IAP),PASTO(IAP),INDEX(IAP)
   52 FORMAT (1H ,1X,I2,7X,F6.1,10X,F4.1,10X,F5.1,11X,F5.2,10X,F5.2,10X,
     1F5.1,10X,F5.0,10X,I3)
  504 CONTINUE
      WRITE (6,53)
   53 FORMAT (1H ,8X,'   PCUS        PASSENGERS      SECONDS OF      PCUS
     1HOURS      PAS.*HOURS     PCU STOPS      PAS. STOPS',/,24X,' PER
     2CU      DELAY   PER     PER  HOUR       PER HOUR       PER CYCLE
     3   PER HOUR',/,39X,' PCU  PER       OF TRAFFIC     OF TRAFFIC
     4                OF TRAFFIC',/,39X,'   CYCLE')
      WRITE(6,54) TPDSI,NSTOT
   54 FORMAT (1H ,3(/),1X,'TOTAL PASSENGER DELAY AT SIGNALIZED INTERSECT
     1IONS = ',F7.1,' PASSENGERS*HOURS PER HOUR OF TRAFFIC',/,' TOTAL PA
     2SSENGER STOPS AT SIGNALIZED INTERSECTIONS =',I9,' PASSENGER STOPS
     3PER HOUR OF TRAFFIC',3(/))
      DO 401 IAP=1,MXIAP,1
      G=IBERD(IAP)-IBEGR(IAP)
      IF (G) 404,404,405
  404 G=G+20.0
  405 W=IWDTH(IAP)
      IF (TRPCU(IAP)-SATUR*W*G/20.0) 401,401,402
  402 WRITE (6,403) IAP
  403 FORMAT (1H ,'SATURATION ON THE APPROACH NUMBER ',I2)
  401 CONTINUE
      WRITE (6,28) ICYCL
   28 FORMAT (1H ,6(/),28X,'TIMING PLAN',3(/),' CYCLE LENGTH = ',I3,' SE
     1CONDS',2(/),16X,'IAP',26X,'IBEGR',24X,'IBERD',/,45X,'(IAP)',24X,'(
     2IAP)')
      DO 302 IAP=1,MXIAP,1
      WRITE (6,29) IAP,IBEGR(IAP),IBERD(IAP)
   29 FORMAT (1H ,'APPROACH NUMBER ',I2,'   EFFECTIVE GREEN STARTS AT ',
     1I2,'   EFFECTIVE RED STARTS AT ',I2)
  302 CONTINUE
      WRITE (6,55)
   55 FORMAT (1H1)

C
C
C
C
C                            PLOT   PLOT   PLOT   PLOT   PLOT   PLOT
      DO 270 IAP=1,MXIAP,1
      IF ((APLOT*TPDSI)-PASOL(IAP)) 274,274,270
  274 DO 272 I=1,20,1
```

WRITE RESULTS

APLOT=PARAMETER CONTROLLING THE PLOTTING PROCEDURE

```
      ALPHA(I)=TITLE(I,IAP)
  272 CONTINUE
      IF (IORGN(1,IAP)-99) 273,270,273
  273 DO 271 ITIME=2,21,1
      QQQQQ(ITIME)=PQUEU((ITIME-1),IAP)
      DDDDD(ITIME)=PDISH((ITIME-1),IAP)
      XXXXX(ITIME) = (FLOAT(ITIME)-1.0)*CYCLE/20.0
  271 CONTINUE
      QQQQQ(1)=QQQQQ(21)
      DDDDD(1)=DDDDD(21)
      XXXXX(1)=0.0
      ND=21
      NF=1
      KA=1
      KB=1
      KC=3
      HA=0.0
      HB=CYCLE/5.0
      HC=5.5
      VA=0.0
      VB=25.0
      VC=8.0
      IOU=6
      CALL CGPL(XXXXX,QQQQQ,Z,ND,NF,KA,KB,KC,KD,HA,HB,HC,VA,VB,VC,ALPHA,
     1IOU)
      NF=2
      CALL CGPL(XXXXX,DDDDD,Z,ND,NF,KA,KB,KC,KD,HA,HB,HC,VA,VB,VC,ALPHA,
     1IOU)
  270 CONTINUE
      NF=0
      CALL CGPL(XXXXX,DDDDD,Z,ND,NF,KA,KB,KC,KD,HA,HB,HC,VA,VB,VC,ALPHA,
     1IOU)
      STOP
      END
```

CGPL IS A GENERAL PLOTTING
ROUTINE FOR THE CALCOMP PLOTTER
ORIGIN/AUTHOR: R.H.COOPER AND R.F.HOWELLS,
UNIVERSITY OF ALBERTA

```
      FUNCTION L(ILN,I=I,QUE,IAR,ISP,ICY,RA1,RA2)
      ALN=(FLOAT(ILN))-(QUE*FLOAT(IAR))/FLOAT(I=I)
      RAT = 1.0-ALN/FLOAT(ILN)-RA1
      IF (RAT) 319,318,318
  318 SPI=ISP
      GO TO 320
  319 SPI=(FLOAT(ISP))*(1.0-RA2*RAT/RA1)
  320 A=(ALN*3600.0*20.0)/(SPI*5280.0*FLOAT(ICY))
      L=A
      B=L
      IF (A-B-0.5) 321,321,322
  322 L=L+1
  321 RETURN
      END
```

APPENDIX D

GLOSSARY OF TERMS

CAPACITY: "maximum number of vehicles which has a reasonable expectation of passing over a given section of a lane or a roadway (...) during a given time period..." (8). It is equal to the saturation flow multiplied by the percentage of effective green.

CYCLE or CYCLE TIME or CYCLE LENGTH: the necessary length of time for the signals to come back to the same situation.

EFFECTIVE GREEN: period of the cycle time during which the vehicles (of a particular approach) are crossing the intersection.

EFFECTIVE RED: period of the cycle time during which the vehicles (of a particular approach) are not crossing the intersection.

EXTERNAL APPROACH: approach where all the vehicles are coming from outside the network under study.

EXTERNAL LINK: section of the roadway leading to an external approach.

INTERNAL APPROACH: an approach which is not external.

INTERNAL LINK: section of the roadway leading to an internal approach.

LOST TIME: period of the cycle time during which no vehicle (at a particular intersection) is crossing the intersection.

OFFSET: time difference between the beginning of the green light on two consecutive signals.

SATURATION FLOW: "flow which could be obtained if there was a continuous queue of vehicles and they were given 100 per cent green time." (20).

SPLIT or GREEN SPLIT or PHASE SPLIT: the way the cycle is divided into green, amber and red periods (for a particular traffic signal).