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DISSERTATION

Presented in Partial Fulfillment of the Requirements for the Degree Doctor of Philosophy in the Graduate School of The Ohio State University

By

Prapon Vongvichien, B.C.E., M.Eng.

* * * * *

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CHAPTER ONE

INTRODUCTION

1.1 Statement of the Research Problem

It has long been recognized that the traffic signal offers the most effective and economical means for the positive control of safe and efficient traffic operation at an intersection. If traffic signals on an arterial are not properly coordinated into a progressive system, signals at adjacent intersections can cause unduly large delays and increases of overall travel times, consequently, causing considerable annoyance to drivers as well as increasing vehicle operating costs. In such a case, the effectiveness of the overall signal system is substantially reduced in spite of the "optimum" designs at individual intersections.

Accordingly, signal coordination on a street or highway is highly desirable because, if designed properly, it provides uninterrupted flow for the majority of traffic, thus tending to minimize overall travel time. To be effective, however, the timing of the signals must take into account specific site characteristics, traffic volume and composition, the dispersion of traffic, and other factors affecting vehicle movement.

One of the major constraints upon the design of a well-coordinated system of signals is the spacing between signalized intersections. In urban areas where signal spacings are generally small, rather efficient progressive
systems can be designed. However, in rural and suburban areas, larger spacings between signals are normally found and the question arises as to whether an efficient signal progression system can be designed.

At present there are no criteria available to aid the traffic engineer in determining the conditions under which it would be beneficial to coordinate signals which are separated by rather large distances.

1.2 Research Objectives and Methodology

The primary objective of this study is to develop design criteria for the introduction of signal progression on major signalized arterials in suburban-to-rural areas. The secondary objective is to determine the maximum signal spacing at which the provision of signal progression is still beneficial.

In achieving the research objectives, three main approaches are available. The three main approaches are:

1. Empirical data collection techniques for studying the behavior of platoons as they travel downstream from a traffic signal.

2. Utilizing computer simulation techniques.

3. Developing complex analytical models.

Each of these approaches has its advantages and disadvantages. Reviews and brief discussions of the literature pertaining to these techniques are found in Chapter Two. In this study the computer simulation technique is chosen because of its feasibility and direct applicability to the research problem. The feasibility and applicability of this technique are discussed in Chapter Four.
1.3 Structure of the Report

This report contains five chapters in addition to this introductory chapter. Chapter Two contains the results and some discussions of a literature review which is composed of three parts. The first part summarizes the results of a review of previous studies concerned with platoon dispersion, while the second part presents the results of a review of some relevant studies pertaining to the simulation of traffic, and the final segment reviews some pertinent studies employing the analytical approach.

Chapter Three is composed of discussions of the acquisition and analysis of ground-based data. Three input parameters needed for the development of a two-intersection simulation model are upstream inter-arrival time or headway distribution, starting headways, and running time distribution.

Four 'best' regression models are developed to indirectly relate means and standard deviations of running time data to signal spacings. The four models permit many studies of system performance under different signal spacings to be conducted.

Chapter Four is devoted to a detailed discussion of the design and the development of a two-intersection simulation model which is quite different from other simulation models found in the literature reviewed. Since the influences of many factors are to be investigated, they are incorporated into the developed simulation model. The incorporation of percentage of trucks necessitates the computer modelling of semi-microscopic behavior of traffic at each intersection.
The word 'semi-microscopic' is used to reflect the simulation of each separate vehicle, instead of a group of vehicles, and also partial car following behavior only at each intersection when the traffic is released after being delayed by the red signal. Otherwise, the car following behavior is ignored and macroscopic behavior of each vehicle is assumed. In addition to the detail of the design of the simulation model, this chapter also presents the results of the validation of the simulation model.

In Chapter Five, the results of the implementation of the validated simulation model are presented. Five factors are extensively investigated. The effect of different means and standard deviations of running time on three selected performance measures (mean delay per vehicle, mean traveltime and mean queue length) are discussed.

Utilizing the simulated data, three predictive models and subsequently three guideline models are developed. Two sample problem solutions are included to illustrate potential applications of the predictive and guideline models.

Chapter Six, constituting the last chapter of this report, summarizes the findings and presents conclusions of the study and recommendations for any further study.
CHAPTER TWO

REVIEW OF THE LITERATURE

2.1 Introduction

To accomplish both the primary and secondary objectives of this study, a variety of different traffic and operating conditions must be investigated. Such an investigation can be conducted in a number of methods. These methods can, however, be categorized into three main techniques, namely, empirical data collection techniques, computer simulation techniques and analytical techniques. Therefore, this review of the literature is limited to pertinent studies within the framework of the three main approaches. For each study that is reviewed, both the methodology and the significant findings are documented.

2.2 Empirical Data Collection Techniques

All the studies utilizing this approach concentrated on investigating platoon behavior after the platoon was released from a traffic signal. More precisely, researchers studied how much platoons of vehicles disperse as they travel further downstream from a signal, so that the desired offsets could eventually be determined. Some studies formulated mathematical models from which the dispersion of platoons at various downstream distances could be
predicted, while other studies analyzed platoon behavior based purely on the empirical data collected.

In 1955, Lighthill and Whitham introduced the kinematic wave theory of the flow of traffic on long crowded roads (Reference 1). Their theory assumed that traffic flow on a long crowded roadway is analogous to the water flow in a long river and that there is a certain functional relationship between traffic flow and density over a distance along the road. It also implicitly assumed that such a functional relationship holds constant over a given length of road at a given time. Although the theory is acceptable, it has a very limited application in the prediction of platoon dispersion because of three drawbacks. These drawbacks, as pointed out by Seddon (2), are:

1) The accuracy of the prediction of platoon dispersion depends on the accuracy of the functional relationship between traffic flow and density.

2) The predicted platoon time length does not increase in length at further downstream distances.

3) It is a time consuming process.

In an unpublished report written in 1956, Pacey attempted to apply the kinematic wave theory and also introduced a probabilistic theory of a diffusion process of vehicular platoons (3). Pacey's theory assumed that speeds of vehicles are independently and normally distributed and each vehicle travels at its independent and constant speed without any interaction, due to overtaking, between other vehicles. As such, the distribution of 'journey' time can be
directly obtained. At this point, it is worth noting that 'journey' time is exactly the same as running time, not travel time. The precise difference between running time and travel time is that running time is an absolute time for a vehicle to move from one point to another without any stopping delays at the two points, while travel time includes running time and any stopping delays that may occur to vehicle. By knowing the flow pattern at an initial point and the distribution of running time between two points, Pacey predicted the flow pattern at a downstream point by means of the following expression:

\[ q_2 (k) = \sum_i q_1 (i) d (k - i) \]

where

- \( q_1 (i) \) = the flow at the initial point during \( i^{th} \) interval
- \( d(k - i) \) = probability that running time is \( (k - i) \) intervals
- \( q_2 (k) \) = the flow at the downstream point during the \( k^{th} \) interval.

In testing the two theories, Pacey recorded the passage of vehicles and times with manual teleprinter tape machines at two stations on each of the two selected expressways in London, England. The maximum distance between the two sections tested was 1821 feet. Pacey compared the observed results with those predicted by both the kinematic wave theory and the diffusion theory and concluded that under normal flow conditions the results predicted by the diffusion theory fit the observed data quite well. However, under heavy flow conditions the results predicted by the kinematic wave theory gave a better fit to the data than those results predicted by the diffusion theory.
A further test of Pacey's diffusion theory was also conducted by Seddon (4). Seddon used simulated data for testing and reached the same conclusion that the diffusion theory gave an accurate prediction of platoon dispersion under medium traffic flow conditions, provided an accurate mean and standard deviation of speed are used.

In 1962, Grace and Potts, by using a technique of transformation of variables, showed that the traffic density function at a downstream location based on the normal distribution of speeds is of the same form as that of the solution of the following one-dimensional diffusion equation (5):

\[ \frac{\partial k}{\partial \tau} = \alpha^2 \left( \frac{\partial^2 k}{\partial \chi^2} \right) \]

where
- \( k \) = a traffic density function
- \( \tau \) = one-half of time squared, i.e., \( \tau = \frac{1}{2}(t^2) \)
- \( t \) = the time from the beginning of the green phase of the first signal
- \( \chi \) = the downstream distance
- \( m \) = mean speed
- \( \chi' = (\chi/m) - t \), the time added to the beginning of the green phase of the second signal
- \( \sigma \) = standard deviation or dispersion of speeds
- \( \alpha \) = \( \sigma/m \) = diffusion constant

Since the use of the diffusion equation requires the knowledge of the
initial traffic density, Grace and Potts derived equations for predicting the flow past a downstream point based on three types of initial traffic density functions, namely, delta function, rectangular pulse, and trapezoidal pulse.

Two experimental studies (6, 7) were conducted to test the predictability of the diffusion equation by Grace and Potts. One study was conducted at the General Motors Research Laboratories by Herman, Potts, and Rothery (6). Using pressure tape switches, the arrival time data were collected at two downstream points from a signal on a four-lane divided highway. The two downstream points were located at 757 and 2142 feet from the stopline. The diffusion constant was found to have a consistent value of 0.18. The results from this study indicated that the predicted arrival pattern agreed very well with the observed arrival pattern.

The second study conducted to test the applicability of the diffusion equation was conducted by Nemeth and Vecellio at The Ohio State University (7). By means of a 16-mm Bolex camera, the authors collected the data of arrival frequencies at four downstream points on a six-lane urban arterial in the downstream area of the city of Columbus, Ohio. The diffusion constant, based on the average speed between the second and third downstream locations, was found to have a value of 0.15. Although the authors stated that "..... it appears that the kinematic model may be applicable to a wider range of situations than just traffic leaving an isolated intersection.", they did not directly make use of the diffusion constant, but instead, manually simulated traffic and
obtained a theoretical arrival frequency at the fourth location.

Nonetheless, the two studies established somewhat different values of the diffusion constant. For the purpose of comparison the values of the diffusion constant were recalculated and presented to four significant digits as follows:

<table>
<thead>
<tr>
<th>Investigators</th>
<th>Diffusion Constant</th>
<th>Distance between Stations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Herman, Potts and Rothery</td>
<td>0.1762</td>
<td>757</td>
</tr>
<tr>
<td>Herman, Potts and Rothery</td>
<td>0.1783</td>
<td>2142</td>
</tr>
<tr>
<td>Nemeth and Vecellio</td>
<td>0.1488</td>
<td>597</td>
</tr>
</tbody>
</table>

It seems that the value for the diffusion constant depends on the distance travelled. Also, it is noted that the rate of increase seems to decline after the distance of 757 feet. If this is the case, then it indicates that platoon behavior becomes stable after a certain distance. Therefore, more studies are required to establish a proper relationship between the diffusion constant and the distance travelled before the diffusion equation can be applied with confidence.

In addition to the kinematic wave theory and the diffusion theory, Robertson, of the Road Research Laboratory, introduced a recurrence relationship model for predicting platoon behavior (8). Robertson incorporated such a recurrence relationship into his TRANSYT model, which is a method for determining optimal fixed-time traffic signal settings. The recurrence relationship can be expressed as:

\[ q_2 (i + t) = F \cdot q_1 (i) + (1 - F) \cdot q_2 (i + t - 1) \]
where \( q_1(i) \) = initial flow in the \( i \)th time interval

\( q_2(i + t) \) = predicted flow in the \( (i + t) \)th time interval at a downstream point along the road

\( F \) = smoothing factor, \( 0 \leq F \leq 1 \), and

\( t \) = 0.8 of the average journey time over the distance for which the platoon dispersion is being calculated.

To establish a functional relationship between the smoothing factor and the average journey time, Robertson collected the passing times of every vehicle at four observing stations located just beyond the traffic signal and at approximately 300, 600 and 1000 feet downstream, respectively. Observations were made at four different sites in West London, England, each of which possessed different physical characteristics ranging from single lane flow with heavy parking and very restricted overtaking to a multi-lane facility with no parking and relatively free overtaking. Based on the observed flow patterns, Robertson was able to establish the following functional relationship between the values of the smoothing factor that minimized the difference between the observed and predicted arrival patterns, and the average journey times:

\[
F = \frac{1}{1 + 0.4\bar{t}}
\]

where \( \bar{t} \) = the average journey time over the distance for which the platoon dispersion is being calculated.

Despite the empirical derivation of Robertson's recurrence relationship, the method has proven to be successful. For this reason Seddon conducted a
study on the theoretical background of the recurrence relationship (9). By means of a simple algebraic induction, the author showed that the recurrence relationship can be expressed in the same form as the equation given by Pacey, except that the journey time is geometrically distributed rather than normally distributed. Seddon also compared the predictability of the diffusion theory and the recurrence relationship to two different sets of data, and concluded that the predicted flow by both methods fit equally well to the data.

Although the recurrence relationship provides a successful method for predicting platoon dispersion, Robertson reasoned that the smoothing factor should also be a function of site factors such as road width, gradient, parking, opposing flow level, traffic composition and so on. Since all the studies reviewed thus far were conducted under short distances, at most 2142 feet, none of the authors stated any limitations to both the diffusion theory and the recurrence relationship as far as the downstream distance is concerned. It is not believed that platoons continue dispersing without limit, therefore, there must be some limit at which the recurrence relationship can be applied, assuming the present expression of the smoothing factor is maintained. Such a case was evident in a study conducted at The Ohio State University by Rathbone (10). The study was conducted over two unusually long signal spacings on two suburban arterials. Figures 2.1 and 2.2 show the comparisons between the observed arrival frequency and the predicted arrival frequency by using the smoothing factor suggested by Robertson. It is seen that these predictions appear to be much less satis-
Figure 2.1 Observed and Predicted Flow Patterns for $F = (1/(1 + 0.5t))$ (U.S. 35)
Figure 2.2  Observed and Predicted Flow Patterns for $F = 1 / (1 + 0.5t)$ (Olentangy River Road)
factory than the predictions at short downstream locations. Consequently, the author used a different expression of the smoothing factor, as follow:

\[ F = a + b \ln(d) \]

where \( a \) and \( b \) are constants

d is the downstream distance.

Figures 2.3 and 2.4 show the results using this different form of the smoothing factor. Although the figures show substantial improvement, the results are not quite satisfactory. Thus, this is still a subject requiring further research.

The literature reviewed thus far has concentrated on the prediction of platoon dispersion, which is a basic requirement in the synchronization of signals for an urban arterial network. A limited number of studies have been conducted in which the real behavior of platoons has been monitored as traffic departed from a signalized intersection.

In 1957, Lewis investigated how well platoons of vehicles stay together as they travel downstream from a signal (11). Using a 20-pen recorder, Lewis collected arrival time data at downstream distances of 0.03, 0.21, 0.34, 0.50, and 0.65 mile on a four-lane undivided highway in California. Based on the analysis of arrival frequency distributions at the five downstream points, the author concluded that a traffic signal installed at any distance up to 0.65 mile from the platoon formation point could be beneficially synchronized with the initial signal. Such a conclusion was based on the study of platoon dispersion
Figure 2.3 Observed and Predicted Flow Patterns for $F = 1.12 - 0.108 \log_{10} d$
Figure 2.4  Observed and predicted flow patterns for $F = 1.12 - 0.108 \log_{e} d$ (Olentangy River Road)
up to distance of 0.65 mile.

Another study of platoon behavior was conducted by Rudy, on an 11.4 mile section of State Route 15 in Connecticut (12). The purpose of the study was to determine the maximum spacing for a progressive signal system on a high speed rural highway carrying high traffic volumes. Rather than collecting arrival time data, as most studies on platoon behavior did, Rudy measured vehicle speeds and headways, and investigated how well speed control could be maintained under the existing control scheme that provided a 40-second bandwidth at a progression speed of 45 mph. The author also evaluated access as a function of distance from the signal by investigating mean headways at various distances. Based on the two criteria, namely, maintenance of speed control and provision of sufficient access to the highway, Rudy concluded that the maximum practical signal spacing ranges from 4000 to 4600 feet.

In 1958, Graham and Chenu investigated platoon movement on U.S. 40, a four-lane undivided rural expressway in California (13). The purpose of the study was simply to measure platoon dispersion at various downstream distances. The passing time data were collected at five points at downstream distances of 150 feet, 0.25, 0.50, 0.75 and 1.00 mile. The results revealed that 91%, 85%, 80% and 77% of vehicles remained in well-defined platoons up to distances of 0.25, 0.50, 0.75 and 1.00 mile, respectively, from the platoon formation point. The authors did not make any recommendation on the maximum practical signal spacing.
A follow-up study conducted by the Missouri State Highway Department was conducted to measure the degree of platoon dispersion at selected distances from signalized intersections (14). In this study, twenty-pen graphic recorders were used to record the passing times of every vehicle at selected downstream locations along five different study sections. These sites were four-lane highways with varying traffic volumes and different degrees of access control. The results of the study showed that 67-82 percent of vehicles remained in a platoon up to a distance of 0.25 mile, 48-73 percent up to 0.5 mile, 41-63 percent up to 0.75 mile, 37-60 percent up to 1.00 mile, and 45-56 percent up to 1.25 miles. Based on these findings, it was concluded that synchronization of signals should be provided for the maximum signal spacing of one mile.

Although some studies were conducted to determine the maximum signal spacing, it is apparent that each study resulted in a larger maximum signal spacing based on the existing operating conditions at selected sites. The latest study on this issue, the determination of the maximum signal spacing, was conducted by Evans (15). Using the conventional license plate technique, the passing times of each vehicle were recorded at five points located at 200 feet, 0.3, 0.6, 0.9 and 1.2 miles from the stopline on S.R. 3, a two-lane rural roadway near Columbus, Ohio. Using the same criterion as Lewis, percentage of vehicles remaining in bandwidth, Evans found that on the average 87%, 85%, 84%, and 77% of vehicles remained in a bandwidth of 44 seconds; and as a result, it was concluded that the maximum signal spacing can be at a distance of up to
1.2 miles. As the author stated, such a conclusion is based on and limited by the available signal spacings where the research was conducted.

None of the literature reviewed up to now conducted any further steps to derive relationships between signal offsets and some effectiveness measures, such as total delay per period of operation, mean delay per vehicle, and so on, from empirical data. Hillier and Rothery seemed to be the first to derive such relations from platoon profiles which were actually observed in the field (16). By plotting the observed cumulative arrival curve and the cumulative service curve against time, the authors were able to calculate total delay, which is the area between the two curves, at the corresponding offset. After extensively repeating the process for various distances from initial intersections, Hillier and Rothery found that the optimal signal offset is a linear function of the downstream distance. Based on the flow range of their observations, they concluded that traffic volumes appear to have no significant effect on the optimal offset. It was also suggested that a progression speed should be approximately equal to the mean speed so that delay is minimized.

In 1972, Vecellio conducted a study on platoon characteristics on two, one-way urban arterials (17). Using an aerial data collection technique, the author was able to collect relatively continuous data by following some platoons from the initial intersection to the last intersection under observation. Vecellio found that platoon behavior can be well described by graphical patterns of mean velocity, mean spacing, the coefficient of variation, and traffic density. In
analyzing vehicle trajectories, he stated that traffic disturbances can be caused by an improper signal offset, the presence of initial queues at downstream intersections and high frequencies of lane changes at a specific location. In addition, Vecellio concluded that platoon behavior along linearly progressive signal systems can be affected by signal spacing, signal offset, and platoon size. Finally, a computer simulation model was developed to simulate the behavior of vehicles as they travel through a series of nine signalized intersections. Further discussion of Vecellio's model will be presented in the next section.

As a result of some success in using the aerial data collection technique in studying platoon behavior along urban signalized arterials, a follow-up study was conducted to investigate platoon behavior as vehicles travel along suburban or rural signalized arterials that possessed various large signal spacings (18). The results revealed that patterns of mean velocity, mean spacing, and mean headway can be used to describe platoon behavior. It was found that patterns of mean headway are quite stable when vehicles travel between signalized intersections separated by an unusually large distance. This implies that platoons of vehicles apparently disperse until they travel to a certain downstream point where platoon behavior becomes relatively stable.

2.3 Computer Simulation Techniques

Computer simulation techniques are probably the most accurate method in evaluating a traffic network. For this reason they have been widely applied
in many studies for evaluating or comparing different control strategies of traffic. Its applications range from a study of one intersection to a study of an entire road network. Since the simulation technique is an art of studying a complex system, methods of modelling vary from one study to another, depending on the purpose of each individual study. Therefore, it is not the purpose of this review to present the methodology of all previous studies. Rather, some relevant studies will be reviewed and discussed in some detail regarding the methods of modelling as well as the significant findings.

Since most of the simulation models were developed for studying either one-intersection operation or road network operation, the literature to be discussed is essentially reduced to two types of models. The first computer simulation model to be discussed was developed by Watjen (19). The model was intended to simulate traffic behavior which travelling through three signalized intersections, so that effects of various settings of two downstream signals could be studied. At the entry point, vehicles were generated according to an exponential distribution of inter-arrival times and then released from an intersection, after being stopped, with a truncated normal distribution of headway with mean of 2.0 seconds and a standard deviation of 0.5 second. In travelling from one signal to another, all vehicles were assumed to travel at the same speed.

From his simple model of one-lane traffic, Watjen established an interesting finding through the implementation of his simulation model. He found that if the optimal offset cannot be achieved, the second choice of offset should be
less than traveltime, since it causes less delay than when offset is greater than traveltime.

A multi-intersection simulation model was also developed by Vecellio (17). The model is comprised on one-lane of traffic with nine intersections. Unlike Watjen's model, vehicles are generated according to a shifted exponential distribution of inter-arrival times and then released from an intersection by using a constant headway of 2.1 seconds; vehicles require a random amount of time to travel from one intersection to another according to an empirical traveltime distribution for each signal spacing. During the development of the model, the author tested the two-intersection model and found that for a given mean and standard deviation, the shape of traveltime distribution has no significant effect on the resulting delay distribution. However, for a given shape of traveltime distribution, both mean and standard deviation of traveltime have significant effects on the shape of the delay distribution as well as on the dispersion of the delay distribution.

Although the two simulation models reviewed are appreciably simple, they are not capable of simultaneously simulating traffic on two lanes. No turning features were incorporated into either model. The effects of traffic composition and traffic from minor streets can not be realistically investigated. Finally, the implementation of these two models is limited, and consequently is not sufficient to acquire comprehensive knowledge necessary to satisfy the requirements of this type of study.
2.4 Analytical Techniques

There are a number of researchers attempting to use analytical techniques in approaching the problem of optimal control of a traffic network. However, it is not the purpose of this study to exhaustively review all papers using these techniques because the papers are similar and deterministic in nature. Therefore, only a few of these papers will be reviewed and discussed herein.

Bavarez and Newell considered a one-way street that intersects one-way side streets each of which with flows that are independent of signal offset (20). To simplify the problem, the authors made the following assumptions:

1. No turning movements are permitted.
2. Traffic is treated as being fluid.
3. The main street flow at the first intersection is steady at a rate \( q \).
4. The side street flows are steady at rates \( q_j, j = 1, 2, \ldots, n \).
5. On the main street, a platoon of vehicles travels at a constant speed, i.e. no platoon dispersion is considered.
6. Vehicles cross an intersection at saturation flow rates of \( s \) on the main street and \( s_j \) on side street \( j \).

Based on the first assumption, there would be essentially only two phases at each signal. In addition, it was also implicitly assumed that no over saturation would occur. Bavarez and Newell attempted to determine the optimal common cycle length as well as the common split that would minimize: (1) the total number of stops per unit time on all lanes at all intersections, (2) the total delay per unit time at all intersections, and (3) the maximum delay to any driver.
on any street. Such a problem is quite interesting because it presents an aspect different from the normal design practice. However, the results presented in this paper happen to be completely different from the objectives of the present study. Nevertheless, the authors presented two interesting conclusions:

(1) For a given common cycle length and splits at each intersection, there is a choice of offsets (phases) that simultaneously minimizes both the total delay and number of stops, but it is not necessarily the offset that produces a maximum bandwidth.

(2) It is possible to set some signals on one-half or one-third of the cycle length of other signals. This results in a main street delay equal to that for the optimal common cycle length setting, but results in less delay for the side street traffic.

In 1968, Newell presented a paper at the Fourth International Symposium on the Theory of Traffic Flow, held in Karlsruhe, Germany (21). The purpose of this study was to determine the signal settings for a network of two-way roads such that the total delay and the total number of stops to the main street traffic are minimized. Admittedly, Newell stated that the problem of estimating signal offsets is generally quite complicated. But if the flows in both directions are at a saturation level, the optimal offset between a pair of adjacent signals does not depend upon the offsets between other pairs of signals. Consequently, the mathematical problem was reduced to an analysis of delays and stops between only a single pair of intersections.
In this study, most of the assumptions made were precisely the same as those made in the previously reviewed work, except that no consideration was given to delays or stops for the side street traffic. In addition to those assumptions, it was also assumed that all signals operate on a common cycle length; all signals have the same green time and the same capacity; there are no commercial vehicles; and the flows through the system are close to these capacities. Based on the assumptions made, the Newell derived both the total delay and the total number of stops, by means of a superposition method, from various geometries that depended on the offsets.

In spite of considerable effort, the author admitted that the model is severely restricted by the assumptions of a constant traveltime, no turning traffic, and nearly saturated flow. Newell, however, concluded that:

(1) For unequal flows in opposite directions, the optimal offsets are those which produce a progression in the direction of heavier traffic, and the settings which minimize delays and stops are unique and are never simultaneously possible.

(2) For equal flows, there are a number of possible settings that result in the same total delays and total number of stops.

(3) Vehicles leaving the last intersection during different green periods have a difference of traveltime equal to the red period, independent of the number of signals in the system.

(4) Differences in delays between two vehicles travelling in the same direction
will never accumulate.

(5) If the saturation flows in both directions are equal, thus regardless of the values of the offsets, it is possible to set a sequence of signals so as to simultaneously minimize the total delay and the total number of stops, while also reducing the difference in delays between any two vehicles to less than the red period.

Blunden and Pretty conducted another theoretical investigation of the average delay to traffic at downstream signals by using a Fourier Series to represent periodic input and output flows in the system (22). To develop such a model, the authors made the following assumptions: (1) there is a finite number of consecutive cycles over which there is saturated flow through the intersection for all effective green periods; (2) there are no vehicles waiting at the beginning of the first cycle; and, (3) the input flow at time \( t \), \( q(t) \), and the output flow at time \( t \), \( s(t) \) are known and have periods of \( \frac{ac}{b} \) and \( c \), respectively. After some manipulation, Blunden and Pretty developed a general model for average delay, expressed as:

\[
d = \frac{1}{2 \lambda s} \int_{0}^{ac/b} q(t) \left(1 - \frac{2bt}{ac}\right) dt + \frac{1}{2} \left(1 - \lambda\right)c
\]

where

\[d \quad = \quad \text{the average delay per vehicle}
\]

\[\frac{a}{b} \quad = \quad \text{the relative frequency of input to output cycle (a and b are integers having no common factor)}
\]

\[\lambda \quad = \quad \text{the effective green time of the appropriate phase of cycle length, } c, \quad 0 < \lambda < 1
\]
\[ s = \text{the saturation flow} \]
\[ q(t) = \text{the input flow at time } t. \]

This, in fact, is a very interesting model since it can be used to calculate the average delay per vehicle for not only the case of a synchronization problem with a common cycle length \( c \), but also for the case of two isolated signals having different cycle lengths, of \( c' \) and \( c \), which are close to each other. Unfortunately, the practical application of the model seems to be limited to a case of continuously high and saturated flow, otherwise the randomness of traffic would cause an inaccurate estimate of the average delay per vehicle. Furthermore, the model does not include the effect of offset; thus, it is not clear to which condition of signal settings the calculated average delay per vehicle for each traffic condition corresponds.

Two other contemporary works using an analytical approach were presented by Allsop (23), and Buckley and others (24). Both presentations were essentially similar to the work by Newell (21). Allsop concentrated on the determination of the total delay per cycle for all possible cases of platoon duration and the arrival time of the leading vehicle relative to the beginning of the signal cycle. The author investigated for conditions of both uniform and varying arrival rates within each platoon. Allsop concluded that if the arrival time of the leading vehicle of each platoon, within which its arrival rate is uniform, is fixed, then a small variation in the arrival rate between platoons always causes an increase in the mean delay per cycle, while a small variation in the duration
of the platoon causes an increase in the mean delay per cycle in some cases and a decrease in mean delay per cycle in other cases.

The presentation of Buckley and others was, however, strictly based on uniform arrival rates. In addition to the expressions for determining total delay per cycle and mean delay per cycle for six delay patterns, the authors also derived expressions for determining the storage queue length and the total number of vehicles delayed for each case. The authors, however, admitted that although all the derived expressions provide no assistance in the devising of an actual optimization procedure, they can be used for comparing alternative sets of cycle lengths, green time, and the signal relative beginning.

2.5 Conclusions

As a result of the literature reviewed in this chapter the following conclusions are offered:

1. Most of the studies using purely empirical data collection techniques concentrated on platoon dispersion as a measurement of traffic behavior.
2. Three theories of platoon diffusion have been validated as useful tools for predicting vehicle arrival times at downstream locations, but only Robertson's recurrence relationship has been used, to date, in practice.
3. It is believed that a different expression of the smoothing factor is required for unusually long signal spacings, so that a more accurate prediction of platoon dispersion can be obtained.
4. The conclusions pertaining to the determination of a maximum signal spacing depend very much upon subjective measures such as, the definition of a platoon and how to measure its behavior.

5. The use of empirical data collection techniques to gain an amount of comprehensive knowledge comparable to that amount acquired from simulation techniques is prohibitively expensive and time consuming. Besides, it is difficult to collect data on many types of traffic and operating conditions. However, the empirical data collection technique is necessary but not sufficient for the present study.

6. By the very definition of 'traveltime', the data on traveltimes depend directly on the traffic and operating conditions during which the data are observed. Yet, many authors have used the definition of 'traveltime' very loosely. This would normally not have a serious effect on the analysis except when traveltime data are used as input to a simulation model. In such a case, the observed traveltime from the simulation model would include delays at each intersection twice unless the input data were observed during the period when all vehicles experienced no delay or when the input traveltimes were calculated from the observed speed data.

7. The models from the two pertinent simulation studies can simulate traffic behavior one lane at a time and do not incorporate commercial traffic.
8. Due to the complication of the stochastic effect on the analytical approach, all analytical works reviewed so far are deterministic in nature, and cannot simultaneously consider the effects of various factors. The derivation has to be case by case. Unfortunately, the sequence of all possible cases happens in a random fashion; therefore, comprehensive evaluation of system performance becomes infeasible.

2.6 Closure

As a consequence of the evaluation of previous research efforts, the computer simulation technique is the most applicable approach to the research set forth in the present study. Therefore, attempts must be undertaken to develop a simulation model such that the objectives of the study can be accomplished while maintaining a reasonable amount of effort and computer time. The development of such a simulation model will be discussed later in Chapter Four of this thesis.
CHAPTER THREE

DATA ACQUISITION

3.1 Description of the Study Sites

Prior to the selection of study sites, some basic requirements were established such that the objective of the study can be achieved. These requirements are:

Firstly, study sites should possess at least 4 to 10 signalized intersections separated by variable spacings between them. In this manner, traffic behavior in the forms of two traffic parameters, namely, mean running time and its corresponding standard deviation can be related functionally to signal spacings measured from stop line to stop line. Preferably, spacings between signals should vary from 0.4 to 1.5 miles.

Secondly, each site should handle appreciable traffic volume and composition during peak periods, with a minimum of pedestrian traffic.

Thirdly, at each site, a signal progressive system should be in operation during peak period conditions, so that there is a minimum interruption at downstream intersections when platoons of vehicles are released from an initial intersection.

Unfortunately, all investigated study sites could not totally satisfy the above requirements. However, with only small deficiencies present, the following sites were selected as sources of information for the study:
1. A section of U.S. 35 near Xenia, Ohio.

2. A section of Olentangy River Road (State Route 315) in Columbus, Ohio.

The section of U.S. 35, a four-lane highway, consists of four signalized intersections, namely, Dayton-Xenia Road, Grange-Hall Road, N. Fairfield Road and Factory Road. Signal spacings between these intersections are approximately 0.63, 1.60 and 1.82 miles respectively. Signals are of a semi-actuated type. Traffic volumes range from 335 to 710 vehicles per hour per lane with 0 to 6 per cent of commercial traffic and no pedestrian influence. The speed limit is 55 miles per hour. This route is under the jurisdiction of the State of Ohio. Figure 3.1 shows a schematic drawing of the study site along U.S. 35. There is no commercial development along the selected section of this site. East Patterson Road and Shakertown Road are the only two secondary roads that intersect the study section as shown in Figure 3.1.

The other section on Olentangy River Road, also a four-lane facility, consists of five signalized intersections. These intersections are at Riverview Drive, the entrance to the Gold Circle Store, W. N. Broadway, Highland Drive, and Henderson Road. These are separated by distances of approximately 0.26, 0.26, 0.66 and 0.87 miles respectively. Each signal is a semi-actuated type. Traffic volumes range from about 275 to 890 vehicles per hour per lane with 0 to 9 per cent of commercial traffic and very little pedestrian influence. A schematic drawing of this site is shown on Figure 3.2. Unlike the study section of U.S. 35, this site is under the jurisdiction of the City of
Figure 3.1    Study Site at U.S. 35-Xenia
Figure 3.2 Study Site at Olentangy River Rd.
Columbus; there are quite a few commercial developments along the section. from Riverview Drive to W.N. Broadway Place, and there exists some light residential development along the remainder of the section.

3.2 Collection and Reduction of the Data

3.2.1 Technique

In the simulation approach, some specific types of information are required as input to the model; the types of information needed depend upon the design of that simulation model.

In this particular study, the developed simulation model, which is discussed in detail in Chapter Four, requires three types of primary input to the model, namely, the inter-arrival time distribution at each entry point, the starting headways, and the running time distribution with corresponding mean and standard deviation for each pair of intersections.

There are many techniques, such as the simple manual method, loop detector, or photographic technique that would be applicable to this study. Each method, however, presents some advantages and disadvantages when applied to collecting the particular type of information needed. It was, therefore, decided to use a simple manual technique with the aid of portable tape recorders and stop watches. Some basic ideas of using a tape recorder as a tool for data collection are:

1. The technique does not require the simultaneous recording of both license plate number and time because the speed of the recording
and play back functions of the tape recorder are basically equal; therefore this can increase the accuracy of the data, especially when observing heavy traffic volumes, because less is required of the observers.

2. It is much less labor intensive for each observation than the conventional manual technique.

3. It offers an economical means for data collection because inexpensive portable tape recorders and replayable cassettes can be utilized.

4. It obviously offers a simple method of data collection as compared to loop detector and photographic techniques.

5. During the data reduction process, it is believed that this technique requires a comparable amount of effort as required by other techniques.

3.2.2 Data Collection Method.

(a) Running Time Data

The technique of using portable tape recorders was first applied to the collection of running time data by means of the conventional license plate technique. However, the conventional license plate technique is subjected to two serious drawbacks when applied to a multiple lane facility during the peak period. Firstly, it is not possible to accurately record the license number as well as the time when vehicles pass the recording station. Secondly, it requires too many workers at a time to overcome the omission of some
vehicles due to lane changes. The use of portable cassette tape recorders can definitely increase the accuracy of the data as well as reduce the number of personnel in half.

Although this technique for collecting running time data was basically the license plate method there are some additional details and precautions deemed useful and necessary to be documented in this report. After each observer, with a synchronized stop watch and a portable cassette tape recorder, was properly located at the data collection station assigned to him, a floating car was used to signal observers at each station as to the beginning of the observation period. The use of the floating car was intended to reduce the waste of efforts during the beginning and ending periods of the observation at each station.

After getting the signal from the floating car each observer recorded a reference time and subsequently the last three digits of each license number at the time a vehicle passed the station, or more precisely a reference line. In the case that an observer missed any license number the word 'miss' was recorded, so that it was possible to later determine these missed vehicles. Each observer was responsible for collecting data from the lane assigned to him. He continuously operated the tape recorder during steady flows of traffic. It was, however, possible to stop operating the recorder when a large gap in vehicles was present. To begin any subsequent observation, a new reference time was recorded. Although it was not necessary to record any
time after a reference time recorded, it was useful to record times at reasonable intervals, at about 2 to 5 minutes. In this manner, it was possible to reduce the time required for the subsequent data reduction process.

In addition, other relevant information was also recorded, such as the identification of trucks, and the beginnings of the green and red phases of signals that were also data collection stations. At the end of each observation, the floating car was used to signal the approximate stopping time. Although there were no further observations after the ending period, the stopwatches were still operated until the same instant stopping time of all the stopwatches could be recorded so that any time differences between them could be detected for the purpose of an adjustment process.

Two methods were tried to locate data collection stations. One was the locating of the stations at downstream locations other than at intersections as indicated by stations A, B, C, and D in Figure 3.1 and 3.2. The other was simply stations at intersections. Even though both methods could provide the required running time information, it was found that the former method was more effective when spacings between signals were large and the systems were not progressive. Besides, the former method also provided the downstream inter-arrival time distribution which could be used to validate the developed simulation model which will be discussed in Chapter Four.

(b) Inter-arrival Time Data

Generally, inter-arrival time or headway data can be affected by the location of the data collection station. The data can be greatly affected by the
queue length if the station is located too close to the intersection. If the station is too far upstream of the intersection, high velocities can also affect the data. Therefore, it was decided to locate each station far enough upstream from the intersection such that the above two adverse effects were minimized.

In applying the technique of using the portable tape recorder to collect the inter-arrival time data, only one observer was responsible for one approach, or two lanes, since this type of information did not require the matching of license numbers, but rather required the presence of a vehicle at the instant it passed the station. Consequently, instead of recording the license number, the observer either recorded 'S' or 'M' depending on whether the vehicle passing by the station was travelling on the shoulder or on the median lane. In the case that two vehicles passed the station about the same instant, 'B' would be recorded instead.

(c) Starting Headway Data

In the simulation approach, there are two choices in releasing vehicles from a signalized intersection, namely, by either saturation flow or starting headway. In this study, it was decided to use the latter method because it was considered to be a more realistic method of simulating actual traffic behavior.

In most studies, this type of data is usually obtained by either manual counts and stopwatches or time-lapse photographs. In this study, portable cassette tape recorders and stopwatches were used. It was, however, not necessary to continuously operate the recorders because the absolute times were not as important as
the relative times when the signal turned green and the front wheels of stopped vehicles passed the recording stating, usually a stop line.

3.2.3 Data Reduction Method

(a) Running Time Data

This process was essentially the transfer of the collected data onto forms and then matching the license numbers. It required at least two playbacks of the tape, the first time to obtain all license numbers and reference times and the second time to obtain synchronized times for every data point. As was previously discussed, any significant differences in time between stop watches were used for the adjustment of reference times. It was sometimes possible that the speed of the playback was slightly different from the recording speed during the actual field observations. If any differences between reference times and times from the playback were significant, say greater than two seconds, a further adjustment was processed by assuming that the reference times from the field data were correct. The final task for the data reduction process was the matching of license numbers at different stations. A sample of the final form is presented in Appendix A.

(b) Inter-arrival Time Data

The process for this type of data was basically the same as the preceding one except that no matching was required. However, it would be much easier for the persons reducing the data if the data were transferred onto the forms one lane at a time. This was due to the fact that for high volumes, the
letters 'S', 'M', and 'B' were less identifiable than license numbers during the playback of the tape.

(c) Starting Headway Data

No further discussions are required for this type of information since the process was basically the same as those previously discussed. The final form of the reduced data is also presented in Appendix A.

3.3 Data Analysis

3.3.1 Frequency of Lane Change Maneuvers

One advantage of the license plate technique is that it is possible to detect lane change occurrences between adjacent observation stations. Only one lane change per vehicle can be detected. Under normal traffic conditions a driver is, however, not likely to make frequent lane changes.

In an attempt to study lane change maneuvering, it was considered easier to define a platoon as a group of vehicles released from an initial signal during the same green period. Each platoon is assigned a three digit number: the first digit refers to the corresponding lane of travel and the last two digits denote the platoon number. Throughout this report lane 1 refers to the median lane and lane 2 refers to the shoulder lane. Any platoon consisting of less than five vehicles was not included in the analysis of lane change maneuvers. To gain some scope as to the frequency of lane change occurrences, the numbers of occurrences per thousand feet of travelled distance were calculated and are
summarized in Table 3.1. The frequency of lane change occurrences per thousand feet, ranging from 0.9 to 2.8 occurrences, did not appear to be very high for the distances travelled.

A further investigation for any possible existence of a relationship between lane change occurrences and traffic volumes was undertaken. It should, however, be made clear that the traffic volumes in this sense are not real traffic volumes, but rather instantaneous volumes which were calculated by dividing platoon sizes by the corresponding initial platoon time length. Initial platoon time length is the time period during which the first and last platoon vehicles pass the initial control station. Figure 3.3 is a plot of the percentage of lane changes against traffic volume. The figure suggests that no strong relationship between the two variables is present.

3.3.2 Analysis of Running Time Data

The purpose of this analysis was threefold:

1) To identify factors that significantly influence the mean running time.

2) To obtain regression models that could be used to predict mean running times and corresponding standard deviations under different signal spacings and other prevailing conditions.

3) To test for goodness of fit of the data to a theoretical distribution.

The analysis of covariance technique was applied to serve the first part of this analysis. For each of the study sites, a separate analysis was conducted.
Table 3.1  Frequency of Lane Change Maneuvers for the Median Lane of U.S. 35.

<table>
<thead>
<tr>
<th>Platoon Identification</th>
<th>Lane Change Maneuvers</th>
<th></th>
<th></th>
<th>Per 1000 ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Entrances (Exits)*</td>
<td>Exits (Entrances)*</td>
<td>Total</td>
<td></td>
</tr>
<tr>
<td>110 (213)*</td>
<td>7</td>
<td>6</td>
<td>13</td>
<td>1.4</td>
</tr>
<tr>
<td>112 (215)</td>
<td>7</td>
<td>10</td>
<td>17</td>
<td>1.9</td>
</tr>
<tr>
<td>113 (217)</td>
<td>11</td>
<td>10</td>
<td>21</td>
<td>2.3</td>
</tr>
<tr>
<td>115 (221)</td>
<td>5</td>
<td>8</td>
<td>13</td>
<td>1.4</td>
</tr>
<tr>
<td>116 (223)</td>
<td>6</td>
<td>10</td>
<td>16</td>
<td>1.7</td>
</tr>
<tr>
<td>118 (226)</td>
<td>2</td>
<td>6</td>
<td>8</td>
<td>0.9</td>
</tr>
<tr>
<td>121 (230)</td>
<td>8</td>
<td>14</td>
<td>22</td>
<td>2.4</td>
</tr>
<tr>
<td>123 (233)</td>
<td>6</td>
<td>6</td>
<td>12</td>
<td>1.3</td>
</tr>
<tr>
<td>125 (236)</td>
<td>5</td>
<td>8</td>
<td>13</td>
<td>1.4</td>
</tr>
<tr>
<td>126 (238)</td>
<td>7</td>
<td>8</td>
<td>15</td>
<td>1.6</td>
</tr>
<tr>
<td>128 (242)</td>
<td>4</td>
<td>9</td>
<td>13</td>
<td>1.4</td>
</tr>
<tr>
<td>129 (244)</td>
<td>8</td>
<td>18</td>
<td>26</td>
<td>2.8</td>
</tr>
<tr>
<td>131 (248)</td>
<td>4</td>
<td>7</td>
<td>11</td>
<td>1.2</td>
</tr>
<tr>
<td>137 (255)</td>
<td>6</td>
<td>3</td>
<td>9</td>
<td>1.0</td>
</tr>
</tbody>
</table>

* Platoon number in parentheses correspond to maneuvers as indicated in parentheses of each column.
Figure 3.3  Plot of Percentage of Lane Changes v.s. Volume
The analyzed factors consisted of both quantitative and qualitative types. In the construction of models for the analysis of covariance, the following notations were used:

- **DIST** = Distance travelled by each vehicle
- **SIZE** = Size of platoon or effect of platoon size
- **LANE** = Effect of lane of travel
  - 1 = median lane
  - 2 = shoulder lane
- **TYPE** = Effect of type of vehicle
  - 1 = passenger car
  - 2 = commercial vehicle

It should be noted that the distance travelled was treated as a quantitative factor while platoon size was first treated as a qualitative factor (Models 1 and 3) and later as a quantitative factor (Models 2 and 4). Table 3.2 summarizes the results of statistical inference at a five per cent level of significance.

There were obviously some inconsistent results obtained. Based on the analysis of the data from U.S. 35 (Models 1 and 2), it appeared that lane of travel and type of vehicle had significant effects upon the mean running time. The analysis of the data from Olentangy River Road, however, did not completely support such conclusions. On the contrary, at the same level of significance it showed that lane of travel did not contribute a significant effect upon the mean running time. Type of vehicle appeared to be a significant factor when platoon size was treated qualitatively (Model 3), but an insignificant factor when platoon size was treated quantitatively (Model 4). Nonetheless, both distance travelled and
Table 3.2  Summary of the Analysis of Covariance

<table>
<thead>
<tr>
<th>Model</th>
<th>Source</th>
<th>D.F.</th>
<th>SS</th>
<th>MS</th>
<th>F</th>
<th>Fat 5%</th>
<th>Conclusion</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$RT_{ijkl} = \mu + (\text{LANE})_i + (\text{TYPE})_j + (\text{SIZE})<em>k + (\text{LANE*TYPE})</em>{ij} + (\text{DIST})<em>l + \varepsilon</em>{ijkl}$</td>
<td>Total</td>
<td>2541</td>
<td>21332034.00</td>
<td>847783.7</td>
<td>15519.56</td>
<td>2.21 significant</td>
<td>1. Data are from U.S.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Total reduction</td>
<td>25</td>
<td>21194593.00</td>
<td>848783.7</td>
<td>15519.56</td>
<td>2.21 significant</td>
<td>1. Data are from U.S.35</td>
</tr>
<tr>
<td></td>
<td>$RT_{ijl} = \mu + (\text{LANE})_i + (\text{TYPE})<em>j + (\text{LANE*TYPE})</em>{ij} + (\text{DIST})_l + (\text{SIZE})<em>l + \varepsilon</em>{ijl}$</td>
<td>Mu</td>
<td>1</td>
<td>2642237.60</td>
<td>2642237.6</td>
<td>48368.90</td>
<td>3.84 significant</td>
<td>2. SIZE is qualitative</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LANE</td>
<td>1</td>
<td>5750.40</td>
<td>5750.4</td>
<td>105.27</td>
<td>3.84 significant</td>
<td>2. SIZE is qualitative</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TYPE</td>
<td>1</td>
<td>749.7</td>
<td>749.7</td>
<td>13.72</td>
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<td>2. SIZE is qualitative</td>
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<tr>
<td></td>
<td></td>
<td>SIZE</td>
<td>20</td>
<td>18106.00</td>
<td>905.3</td>
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<td>2. SIZE is qualitative</td>
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<tr>
<td></td>
<td></td>
<td>LANE*TYPE</td>
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<td>9.545</td>
<td>0.18</td>
<td>3.84 not significant</td>
<td>qualitative</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Distance</td>
<td>1</td>
<td>3902232.30</td>
<td>3902232.3</td>
<td>71434.41</td>
<td>3.84 significant</td>
<td>qualitative</td>
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<td></td>
<td>Remainder</td>
<td>2516</td>
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<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>Total</td>
<td>2541</td>
<td>21332034.00</td>
<td>847783.7</td>
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<td></td>
<td></td>
<td>Total reduction</td>
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<td>58461.25</td>
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<td></td>
<td></td>
<td>Mu</td>
<td>1</td>
<td>2797650.7</td>
<td>2797650.7</td>
<td>46334.86</td>
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<td></td>
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<td>LANE</td>
<td>1</td>
<td>5135.4</td>
<td>5135.4</td>
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<td>3.84 significant</td>
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<td>826.7</td>
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<td>3.84 significant</td>
<td>2. SIZE is quantitative</td>
</tr>
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</tr>
<tr>
<td></td>
<td></td>
<td>Distance</td>
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<td>3902232.3</td>
<td>3902232.3</td>
<td>64629.02</td>
<td>3.84 significant</td>
<td>quantitative</td>
</tr>
<tr>
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<td></td>
<td>SIZE</td>
<td>1</td>
<td>2486.4</td>
<td>2486.4</td>
<td>41.18</td>
<td>3.84 significant</td>
<td>quantitative</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Remainder</td>
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<td>153060.6</td>
<td>60.379</td>
<td>3.84 significant</td>
<td>quantitative</td>
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Table 3.2 (continued)

<table>
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<th>Model</th>
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<th>D.F.</th>
<th>SS</th>
<th>MS</th>
<th>F</th>
<th>F at 5%</th>
<th>Conclusion</th>
<th>Remarks</th>
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<tbody>
<tr>
<td>3</td>
<td>( RT_{ijkl} = \mu + (\text{LANE})_i + (\text{TYPE})_j + (\text{SIZE})<em>k + (\text{LANE} \times \text{TYPE})</em>{ij} + (\text{DIST})<em>l + \varepsilon</em>{ijkl} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>1945</td>
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<td>1.44</td>
<td>significant</td>
<td></td>
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<td>4451550.2</td>
<td>1224540.1</td>
<td>4782.69</td>
<td>3.84</td>
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<td>224540.1</td>
<td>224540.1</td>
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<td>3.84</td>
<td>not significant</td>
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<td>239.9</td>
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<td>14.0</td>
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<td>3.84</td>
<td>not significant</td>
<td></td>
</tr>
<tr>
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<td>Distance</td>
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<td>10152.80</td>
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<tr>
<td></td>
<td>Remainder</td>
<td>1909</td>
<td>89624.8</td>
<td>46.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>( RT_{ijl} = \mu + (\text{LANE})_i + (\text{TYPE})<em>j + (\text{LANE} \times \text{TYPE})</em>{ij} + (\text{DIST})_l + (\text{SIZE})<em>l + \varepsilon</em>{ijl} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>1945</td>
<td>4541175.0</td>
<td>738453.4</td>
<td>12963.37</td>
<td>2.09</td>
<td>significant</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Reduction</td>
<td>6</td>
<td>4430720.6</td>
<td>227305.8</td>
<td>3990.30</td>
<td>3.84</td>
<td>significant</td>
<td></td>
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<td>Mu</td>
<td>1</td>
<td>227305.8</td>
<td>227305.8</td>
<td>0.00</td>
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<td>10.1</td>
<td>10.1</td>
<td>0.18</td>
<td>3.84</td>
<td>not significant</td>
<td></td>
</tr>
<tr>
<td></td>
<td>TYPE</td>
<td>1</td>
<td>184.4</td>
<td>184.4</td>
<td>3.24</td>
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<td></td>
</tr>
<tr>
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<td>LANE \times \text{TYPE}</td>
<td>1</td>
<td>1.7</td>
<td>1.6</td>
<td>0.03</td>
<td>3.84</td>
<td>not significant</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Distance</td>
<td>1</td>
<td>552432.4</td>
<td>552432.4</td>
<td>9697.82</td>
<td>3.84</td>
<td>significant</td>
<td></td>
</tr>
<tr>
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<td>SIZE</td>
<td>1</td>
<td>16471.2</td>
<td>16471.2</td>
<td>289.15</td>
<td>3.84</td>
<td>significant</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Remainder</td>
<td>1939</td>
<td>110454.4</td>
<td>57.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
platoon size appeared to be consistently significant factors; and the interaction between lane of travel and type of vehicle did not show any significant influence.

The reason of the above inconsistencies could be attributable to the effect of locations of the study sites. As was previously described, the section of Olentangy River Road is located in a highly suburbanized area while the section of U.S. 35 is a highway located in a rural area.

Since three of the four models investigated agreed upon the significance of the type of vehicle, it is desirable to determine the extent to which the differences in operating conditions due to vehicle types are significant. To quantify the effect of trucks, which are taken as any vehicle larger than a standard American pickup truck, mean values of the running times, adjusted for distance travelled and platoon size, were calculated. The results are given in Table 3.3.

Since it was desirable to be able to predict means and standard deviations of running times so that the traffic behavior under different signal spacings can be simulated, the technique of regression analysis was employed. Based on the analysis of the lane-by-lane data from both study sites, it was found that speed limit was an implicitly significant variable, not by itself but when it was used to create a new variable named 'expected running time' which was defined as the running time at the observed speed limit. Comparisons between the models using distance travelled as signal spacing and 'expected running time' as independent variables are summarized in Table 3.4. It is obvious that the 'expected running time' is considered to be a better predictor
Table 3.3  Equivalency of Truck in Terms of Passenger Car Unit.

<table>
<thead>
<tr>
<th>Data from</th>
<th>Speed limit</th>
<th>Type i*</th>
<th>Number observed</th>
<th>$RT_i$</th>
<th>Equivalency $= \frac{RT_2}{RT_1}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>U.S. 35</td>
<td>55</td>
<td>1</td>
<td>1913</td>
<td>82.9</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>32</td>
<td>85.4</td>
<td></td>
</tr>
<tr>
<td>Olentangy River</td>
<td>45</td>
<td>1</td>
<td>2424</td>
<td>44.6</td>
<td>1.08</td>
</tr>
<tr>
<td>Road</td>
<td></td>
<td>2</td>
<td>117</td>
<td>47.2</td>
<td></td>
</tr>
</tbody>
</table>

* type 1 = passenger car      type 2 = truck

Table 3.4  Summary of Regression Analysis of Mean Running Time and Standard Deviation.

<table>
<thead>
<tr>
<th>Regression Model</th>
<th>$R^2$</th>
<th>$S^2$</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 MRT = 0.8808 + 1.1015 (EXP)</td>
<td>0.982</td>
<td>27.3</td>
<td>Median lane, U.S. 35 + Olentangy River Rd.</td>
</tr>
<tr>
<td>MRT = 6.6701 + 0.0120 (DIST)</td>
<td>0.941</td>
<td>92.1</td>
<td></td>
</tr>
<tr>
<td>2 STD = 1.0388 + 0.0790 (EXP)</td>
<td>0.877</td>
<td>1.09</td>
<td></td>
</tr>
<tr>
<td>STD = 1.4429 + 0.0009 (DIST)</td>
<td>0.843</td>
<td>1.40</td>
<td></td>
</tr>
<tr>
<td>3 MRT = 4.4765 + 1.1282 (EXP)</td>
<td>0.988</td>
<td>18.6</td>
<td>Shoulder lane, U.S. 35 + Olentangy River Rd.</td>
</tr>
<tr>
<td>MRT = 13.2309 + 0.0130 (DIST)</td>
<td>0.967</td>
<td>49.3</td>
<td></td>
</tr>
<tr>
<td>4 STD = 3.6694 + 0.0602 (EXP)</td>
<td>0.454</td>
<td>5.03</td>
<td></td>
</tr>
<tr>
<td>STD = 4.2938 + 0.0007 (DIST)</td>
<td>0.410</td>
<td>5.44</td>
<td></td>
</tr>
</tbody>
</table>
than the distance travelled or signal spacing. This conclusion is based on two criteria, namely, minimum variance and high coefficient of correlation.

The last part of the analysis of the running time data is testing the goodness of fits. Although an empirical distribution could be used as input to the developed simulation model, it was more desirable to use a theoretical distribution if the data could be well approximated by such a distribution associated with its parameters. By means of the Chi-squared test for goodness of fit it was found that the normal distribution could well approximate the empirical distributions. Figures 3.4 to 3.6 show some histograms of running time distributions for different travelled distances or signal spacings.

3.3.3 Analysis of Inter-arrival Time Data

Although a number of theoretical distributions of headway have been proposed by various authors, it is desirable to use the distribution that provides a good fit over an extensive range of traffic volumes. So far, only a composite exponential distribution proposed by Schuhl (25) has been tested, with the parameters estimated in two studies, one by Kell (26) and the other by Grecco and Sword (27). Kell showed that his results of parameter estimations could be used over traffic volumes ranging from about 100 vph to almost 1200 vph. Normally, this range is considered to cover traffic volumes of rural and suburban arterials.

Accordingly, it was decided to test the observed data for goodness of fit to the composite exponential distribution of inter-arrival times, of which its
Figure 3.4  Histogram of the Observed Running Times on the Shoulder Lane of U.S. 35 between Stations 1 and 3.
Figure 3.5  Histogram of the Observed Running Times on the Median Lane of U.S. 35 between Stations 1 and 4.

Test of Normal Dist.
\[ n = 418 \]
\[ \bar{x} = 127.9 \]
\[ s = 10.1 \]

calculated \[ \chi^2 = 18.7 \]

critical \[ \chi^2_{0.05} = 30.1 \]

Normal distribution can be assumed.
Figure 3.6  Histogram of the Observed Running Times on the Shoulder lane of U.S. 35 between N. Fairfield Road and Grange-Hall Road.

Test of Normal Dist.

\[ n = 279 \]
\[ \bar{x} = 123 \]
\[ s = 9.4 \]

Calculated \( \chi^2 = 23.5 \)

Critical \( \chi^2_{0.05} = 26.3 \)

Normal distribution can be assumed.
parameters were estimated by Kell. To proceed with such tests, a computer program (see Appendix B) was written to compute cumulative distributions for various traffic volumes; the Kolmogorov-Smirnov one sample test procedure was employed. The test procedure is well discussed in a nonparametric statistics book by Siegel (28). The test results indicated that the observed data could be well represented by the composite exponential distribution with corresponding traffic volumes. Figures 3.7 to 3.9 show some of the plots comparing the observed and the theoretical distributions. The test results are also shown in these figures.

3.3.4 Analysis of Starting Headway Data

In any simulation of traffic at a signalized intersection, the choice of methods of releasing vehicles during the green period is a matter of individual opinion. In this study the starting headway approach was utilized because it was considered to be more realistic than the saturation flow approach. There are a number of documents that have been addressed to this type of information. Probably the most widely used information of this nature is the paper written by Greenshields in 1946 (29). Following his initial investigation were a number of studies, with the results generally indicating a decreasing trend of starting headways.

Accordingly, it was decided to conduct a study on starting headways. The total number of cycles observed was 124. Table 3.5 summarizes the starting headways from various study results as well as conclusions from
Test of Composite Exponential Dist.

\[ n = 176 \]

Volume = 875 vph.

Maximum D = 0.057

Critical D = 0.103 (at \( \alpha = 5\% \))

Do not reject the composite exponential Distribution.

Figure 3.7  Comparison between the Observed Inter-arrival Time Distribution and the Composite Exponential Distribution for the Corresponding Traffic Volume (median land at Riverview Drive.)
Test of Composite Exponential Dist.

$n = 212$

Volume = 1070 vph.

Maximum $D = 0.083$

Critical $D_{0.05} = 0.093$

Do not reject the composite exponential distribution.

Figure 3.8 Comparison between the Observed Inter-arrival Time Distribution and the Composite Exponential Dist. for the Corresponding Traffic Volume (shoulder lane at Riverview Dr.)
Figure 3.9  Comparison between the Observed Inter-arrival Time Distribution and the Composite Exponential Distribution for the Corresponding Traffic Volume (shoulder lane at W.N. Broadway Pl.)
Table 3.5 Summary of the Starting Headways from Various Study Results.

<table>
<thead>
<tr>
<th>Vehicle Position</th>
<th>Present Study</th>
<th>Crit. Z</th>
<th>Greenshields's study</th>
<th>Drew and Pinnell's study</th>
<th>Carstens's study</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
<td>n</td>
<td>s</td>
<td>X&lt;sub&gt;g&lt;/sub&gt;</td>
<td>Z&lt;sub&gt;g&lt;/sub&gt;</td>
</tr>
<tr>
<td>1</td>
<td>2.32</td>
<td>124</td>
<td>0.72</td>
<td>1.960</td>
<td>2.575</td>
</tr>
<tr>
<td>2</td>
<td>2.25</td>
<td>124</td>
<td>0.58</td>
<td>1.960</td>
<td>2.575</td>
</tr>
<tr>
<td>3</td>
<td>2.23</td>
<td>124</td>
<td>0.48</td>
<td>1.960</td>
<td>2.575</td>
</tr>
<tr>
<td>4</td>
<td>2.05</td>
<td>124</td>
<td>0.42</td>
<td>1.960</td>
<td>2.575</td>
</tr>
<tr>
<td>5</td>
<td>1.95</td>
<td>124</td>
<td>0.40</td>
<td>1.960</td>
<td>2.575</td>
</tr>
<tr>
<td>6</td>
<td>1.99</td>
<td>124</td>
<td>0.44</td>
<td>1.960</td>
<td>2.575</td>
</tr>
</tbody>
</table>

Remarks:
1) Year of each study: Greenshields (1946), Drew and Pinnell (1961), Carstens (1971)
2) Z scores are calculated as follow:

\[ Z_i = \frac{X - X_i}{s/\sqrt{n}} , \]  

where \( i = g, d, c \)
eighteen statistical tests. Most of the results of the present study were significantly different from those of previous studies. This seems to further indicate a decreasing trend of starting headways as previously observed, thus indicating the aggressive behavior of drivers under present traffic conditions.

3.4 Closure

An emphasis of the discussions in this chapter was placed upon the technique of data collection and reduction employed such that experiences from this study can serve as a guideline if this technique is used in the future. Evaluation of the accuracy of the results from this study indicates that by using a conventional stop watch the absolute time varies within plus or minus one half second. However, more accurate results can be obtained, by using a digital stop watch or perhaps the timing ability of Hewlett Packard's HP-55 calculator. This will probably decrease the time for the data reduction process in addition to improving the quality of data.
CHAPTER FOUR

DEVELOPMENT OF A SIMULATION MODEL

4.1 Introduction

Since the pioneer traffic simulation studies of Gerlough (32) and of Goode, Pollmar and Wright (33), both from 1956, computer simulation technique has proved to be a very valuable research tool in traffic engineering. The technique has been widely applied in numerous studies of traffic problems, especially those encountered at intersections. Major reasons for employing a simulation technique are:

1. It offers a means of conducting traffic studies without disturbing real world traffic conditions.
2. Different traffic conditions can be studied in an economical manner.
3. Unlike the analytical approach, the technique does not require a high level of mathematical knowledge.
4. The computer simulation technique itself is an art of finding solutions to research problems.

Therefore, a simulation model for any traffic study can be developed to realistically simulate traffic behavior as much as possible, although the increasing difficulty in programming may act as a constraint at some level of
detail. Besides, too many factors incorporated into a developed model can cause intractable output. Of the numerous reports documenting results of studies employing a simulation technique, the studies conducted by Watjen (19) and Vecellio (17) seem to be most similar in nature to this study. Both studies did not, however, incorporate a factor of vehicle type and were not capable of handling traffic from a minor road. Furthermore, the models developed by Watjen and Vecellio considered traffic only one lane at a time. Consequently, a simulation model incorporating more factors is required in the study.

The purpose of this phase of the study is to develop a simulation model such that it can be used to simulate the behavior of vehicles as they travel through a series of signalized intersections so that the effectiveness of a signal progressive system under various traffic and operating conditions can be studied. Based on the ultimate objective of this study, a computer simulation model was written in General Purpose Simulation System (GPSS) language because of its apparent applicability to traffic problems as well as its simplicity. A description of the model will be presented in the next section.

4.2 Design of the Simulation Model

4.2.1 Factors Incorporated into the Model

To achieve the purpose of the simulation model the following factors are incorporated into the model: major traffic volume, minor traffic volume joining the main stream of traffic, lane of travel, type of vehicle, cycle lengths and splits, and signal offset. The obvious importance of these factors obviates the need for any further clarification of reasons for their incorporation in the model.
As a result of previous analysis, lane of travel and type of vehicle contribute some significant influence on the mean running time, an important parameter in simulating the behavior of vehicles travelling from one intersection to another intersection. Lane of travel was incorporated into the model by simultaneously simulating traffic on both the median and the shoulder lanes. In incorporating the type of vehicle, three equivalency factors were included in the model in calculating straight-thru and right-turn accelerations as well as travel between two intersections. The results from an extensive study by Carsten (31) in 1971 were used. The following is a summary of the equivalency factors used in the simulation model:

- 1.63 for straight-thru acceleration time of trucks
- 1.81 for right-turn acceleration time of trucks
- 1.20 for right-turn acceleration time of passenger cars
- 1.03 for travelling from one intersection to another intersection.

The last equivalent factor of 1.03 was derived from the previous analysis presented in Chapter Three of this report.

4.2.2 The Model

Figure 4.1 presents a schematic drawing of the basic roadway configuration incorporated into the model. For the purpose of this study only one direction of traffic was included. However, the design of the model was aimed not only towards direct applicability to this study but also to flexibility for future modifications without affecting the existing structure of the model. For instance, the model can be modified to a two-way simulation model without any change in
Figure 4.1 Basic Roadway Configuration with 12-foot Lane Width
the one-way model by simply adding a traffic portion that will simulate traffic in the other direction.

Basically, the model is divided into three main segments as well as four different portions of input or initial and definition cards. Each of these portions can be easily identified from the listings of the program. The listings of both two-intersection and three-intersection models are included in Appendix C. The three main model segments include the signal segment, the traffic segment and the control segment. The remainder of the model includes input or initial cards, function definition cards, variable definition cards and table definition cards. Only the main model segments require further discussion in detail.

a) Signal Segment

The signal segment was designed to function periodically as a real signal operation. That is, the signal indicates green phases according to the corresponding cycle splits of which the basic arrangement has been presented in Figure 4.2. Between each green phase there is an effective amber period of four seconds. To eliminate any cumbersome microscopic decisions of drivers approaching a signalized intersection, the effective green and amber periods were incorporated in the signal segment rather than the real green and amber periods. In this manner, a more efficient model, in terms of the ease of programming and computer time, can be obtained.

At the beginning of each green phase, the signal segment initializes all vehicle-position indicators of the corresponding traffic lanes in that phase. It also updates the departure times of leading vehicles in the corresponding lanes.
Phase 1

Phase 2

Phase 3

Figure 4.2 Simulated Phasing of the Signal at Each Intersection
The last function of the signal segment is to collect queue length statistics from each corresponding traffic lane. Figure 4.3 is a simplified flow diagram of a signal simulator; the diagram can be applied to any other signal simulator.

b) Traffic Segment

This segment simulates traffic movements. It generates traffic according to the pre-specified inter-arrival time distribution for each traffic lane. The composite exponential distributions with the parameters estimated by Kell (26) have been previously verified to provide acceptable representatives of the observed inter-arrival time data, and thus were incorporated into the model. The type of each generated vehicle will be randomly assigned according to the pre-specified distribution of traffic composition for each lane.

The median lane is assumed to handle only straight-thru and left-turn traffic while the shoulder lane will handle only straight-thru and right-turn traffic. At an intersection, a vehicle on the median lane either proceeds straight or turns left according to the pre-assigned percentage of left turns. A left-turning vehicle proceeds to queue in the left-turn pocket lane provided the existing left turn queue does not exceed the capacity of the pocket lane. If the capacity is exceeded, the turning vehicle has to wait in the median lane until the green for left turns appears and the stopped vehicles are released. Sometimes a left-turning vehicle cannot proceed to the left-turn pocket lane because the queue length for straight-thru traffic on the median lane extends beyond the taping of the pocket lane. This possibility was also included in this segment.
Generate Signal Indicator at a Specified Offset

Signal Turns Green for Phase I = 1

Record Time when Signal Turns Green

Signal is in Phase I as Long as Effective Green

Signal Turns Amber for Phase I

Set All Corresponding Vehicle Position Indices for Phase I to 1

Signal is Amber as Long as Effective Amber

Collect any Needed Data on Queue Length

If Phase I = 3

YES

I = 1

NO

I = I + 1

Figure 4.3 Simplified Flow Diagram of a Signal Simulator
A vehicle on the shoulder lane arriving at an intersection either proceeds straight or turns right according to the pre-specified percentage of right-turning traffic. The characteristics of right turn movements were established in the model by including the equivalency factors that were discussed in the preceding section. In addition, the model allows for continuous right turn movements for the lead vehicles.

If the percentages of left turn and right turn traffic are not specified at the beginning of each run, the default value of zero will be assumed as the number of turning vehicles in both lanes.

Regardless of the lane of travel, each vehicle first proceeds thru an intersection without any delay if there are not any stopped vehicles ahead and if the corresponding signal phase is green, and then travels to the next intersection according to the pre-specified normal distribution of running time truncated at 95 percent. At a downstream intersection, each vehicle behaves the same way as discussed before.

If a vehicle cannot, however, proceed thru the intersection immediately, it will join the preceding queue. In such a case, the vehicle is subjected to both stopping and accelerating delays. This feature is quite different from the simulation model developed by Vecellio (17) in 1973. A study conducted from aerial photographic data at the Ohio State University (18) in 1976 showed that a distance of 195 feet is a reasonable distance for acceleration of any vehicle from a stopped position; the average acceleration period of nine
seconds was also determined from such data. Such results agree very well with the results from a study by Dockerty (34).

The rationale of including such a feature is to emphasize two important kinds of delays that will be components of traveltime data. That is, the travel-time of a delayed vehicle includes stopped delay, acceleration time up to a distance of 195 feet from the upstream stop line, and a random running time. For a vehicle that is not delayed at both upstream and downstream intersections, its traveltime is equal to the randomly assigned running time.

Since the influence of a truck upon following vehicles is considered more pronounced when delayed at an intersection than when travelling between intersections, any vehicle following a truck during the acceleration period will have essentially the same acceleration time as that of the leading truck. In other words, the factor of vehicle position was also incorporated into the model.

To study the effect of minor road traffic, two routines for generating minor road traffic are included in the model. The two routines are for traffic turning left from the north approach and heading eastbound and for traffic turning right from the south approach and heading eastbound. Since the two routines are conflicting, all vehicles from the north approach are assumed to accept a gap size of 4.5 seconds, which is approximately the median value from a study of gap acceptances by left turning vehicles on four-lane, two-way streets conducted by Gerlough and Wagner (35).

Figure 4.4 depicts a simplified flow diagram of the general portion of the traffic segment, showing the flow of vehicles on any lane, as they
Figure 4.4  Simplified Flow Diagram of a Traffic Simulator
arrive at an initial intersection and travel to an immediate downstream intersection. Upon arriving at the immediate downstream intersection, the vehicles behave precisely the same as when arriving at the initial intersection.

c) Control Segment

This segment was intended to control the running time for each simulation run as well as to cope with a tactical problem encountered in computer simulation. Following the procedure proposed by Conway (36), it was decided that the first ten cycles of data will be excluded from the data collection for each run. Some further discussions of tactical problems are found in Appendix D.

4.2.3 The Model Input

Although details of the model and its input have been discussed quite extensively, it is deemed helpful to summarize all the required input in one review. The following are required as basic input to the model:

1. Inter-arrival time or headway distribution at each entry point of the system.

2. Signal splits at each intersection.

3. Mean and standard deviation of running time for each lane between each pair of intersections.

4. Distance between intersections, measured from stop line to stop line.

5. Vehicle storing capacity for each section between a pair of intersections and storing capacities for left-turn pocket lanes.
6. Percentage of left-turning traffic at each intersection (optional, the default value of zero will be assumed if not pre-specified).

7. Percentage of right-turning traffic at each intersection (optional, the default value of zero will be assumed if not pre-specified).

8. Percentage of truck traffic at each entry point of the system (optional, the default value of zero will be assumed if not pre-specified).

9. Signal offset at each intersection (optional, simultaneous system will be assumed if not pre-specified).

Each of these inputs can be easily prepared and inserted in its proper place of the input deck, and quickly identified by comment cards.

In determining lane capacity, a vehicle is assumed to occupy a space of 23.8 feet. This figure was obtained from a study of aerial data conducted at The Ohio State University. It is based upon an average of spacings for vehicles stopped at the intersection of Henderson Road and Olentangy River Road in Columbus, Ohio.

4.2.4 The Model Output

Each simulation run can generate the following items of the model output:

1. Maximum queue length.

2. Mean and standard deviation of queue lengths for all vehicles.

3. Mean queue length for delayed vehicles.

4. Percentage of delayed vehicles.
5. Queue length distribution.

6. Mode and median of queue lengths.

7. Mean and standard deviation of delays per vehicle.

8. Mean delay per delayed vehicle.

9. Distribution of delays per vehicle.

10. Mode and median of delays per vehicle.

11. Total delay per unit time of operation (i.e., per cycle or per hour).

The above items of the output will be automatically printed out for every main traffic lane at each intersection. In addition, each simulation run also results in distributions of traveltimes between the initial intersection and subsequent intersections on each lane.

Of all items of the model output, only three will be discussed and analyzed such that the objectives of the study can be efficiently accomplished. Discussions and results of analyses will be presented in Chapter Five.

4.3 Verification and Validation of the Simulation Model

Prior to the implementation of a simulation model, it is necessary to verify the internal logic of the model and, if feasible, to validate the model by comparing selected performance measures obtained from the field data with the output from the simulation run under actual conditions.

During the development stage, the model was carefully and accurately developed to simulate as closely as possible the real progressive signal system. Therefore, it was decided to divide the validation process into two
parts. The first part is to validate the signal model segment of one signal operation; the second part is to validate traffic behavior as vehicles progress through a signalized system.

There exist a number of theoretical methods for determining mean delay per vehicle at a fixed-time signalized intersection. These methods have been established by Webster (37), Newell (38), Blunden (39), Miller (40), and Beckmann, McGuire and Winston (41). Since Webster's delay formula has proven to be a successful empirical formula, it was decided to compare the mean delays per vehicle from the simulation output with the mean delays calculated from Webster's formula.

By a combination queueing theory and computer simulation, Webster (37) shows that the mean delay per vehicle on a particular lane can be estimated by the following formula:

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

where
- \( d \) = mean delay, seconds per vehicle,
- \( c \) = cycle length, seconds,
- \( \lambda \) = proportion of the cycle length that is effectively green (that is, effective green/cycle length),
- \( q \) = flow rate or traffic volume, vehicles per second,
- \( s \) = saturation flow, vehicles per second
- \( x \) = degree of saturation (that is, \( q/\lambda s \)).

The first term in this expression is the delay due to uniform vehicle arrivals,
the second term is the delay due to random arrivals, and the third term is a
correction factor empirically derived from the results of traffic simulation.

Figures 4.5 and 4.6 show the comparisons between mean delay-volume
relationships as calculated from the above Webster delay equation and the
simulation results which were based on 0% left-turns, 0% right-turns, and 0% truck
traffic. It is obvious that the simulation results for both 75 and 90 second cycle
lengths are in good agreement with the results from Webster's formula.

In the second part of the validation process, an attempt was undertaken
to validate the entire model with two intersections in the system. Unfortunately,
the existing signal systems of the study sites are not of a fixed type, and
consequently not in progression. Therefore, the only alternative remaining is
to compare inter-arrival time distributions at downstream locations where data
were collected. Table 4.1 summarizes the tests and the results of the compari-
sions. Based on the two-sample Kolmogorov-Smirnov Test procedure presented
by Siegel (28), it was found that at a five percent level of significance all six
tests resulted in 'not reject' the null hypothesis, namely, that the simulated
inter-arrival time distributions at various downstream locations can represent
the actual distributions from field data at the same downstream locations.
Figure 4.5 Comparison of Delay-Volume Relationships: Webster's Model vs. Simulation Model, for Cycle Length = 75 Seconds

Sat. Flow = 1800 vph.
Cycle length = 75 secs.

Effective Green Time = 20 secs.
Figure 4.6 Comparison of Delay-Volume Relationships: Webster's Model vs. Simulation Model, for Cycle Length = 90 Seconds

Effective Green Time = 20 secs

Sat. Flow = 1800 vph.
Cycle length = 90 secs.

- Webster's
- Simulation

Traffic Volume, Vehicles per Hour

Mean Delay, Seconds per Vehicle
Table 4.1 Comparison of Cumulative Distribution of Inter-arrival Time; Observed Results vs. Simulation Results

<table>
<thead>
<tr>
<th>Interarrival time seconds</th>
<th>Station 2 (2562 ft.)</th>
<th></th>
<th>Station 3 (6079 ft.)</th>
<th></th>
<th>Station 4 (9441 ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Median Lane</td>
<td>Shoulder Lane</td>
<td>Median Lane</td>
<td>Shoulder Lane</td>
<td>Median Lane</td>
</tr>
<tr>
<td>0 - 1.5</td>
<td>.187</td>
<td>.121</td>
<td>.066</td>
<td>.160</td>
<td>.153</td>
</tr>
<tr>
<td>1.5 - 2.5</td>
<td>.635</td>
<td>.597</td>
<td>.038</td>
<td>.430</td>
<td>.437</td>
</tr>
<tr>
<td>2.5 - 3.5</td>
<td>.803</td>
<td>.765</td>
<td>.038</td>
<td>.645</td>
<td>.552</td>
</tr>
<tr>
<td>3.5 - 4.5</td>
<td>.851</td>
<td>.813</td>
<td>.038</td>
<td>.700</td>
<td>.652</td>
</tr>
<tr>
<td>4.5 - 5.5</td>
<td>.900</td>
<td>.852</td>
<td>.048</td>
<td>.740</td>
<td>.684</td>
</tr>
<tr>
<td>5.5 - 6.5</td>
<td>.916</td>
<td>.874</td>
<td>.042</td>
<td>.788</td>
<td>.710</td>
</tr>
<tr>
<td>6.5 - 7.5</td>
<td>.938</td>
<td>.882</td>
<td>.056</td>
<td>.823</td>
<td>.712</td>
</tr>
<tr>
<td>7.5 - 8.5</td>
<td>.945</td>
<td>.894</td>
<td>.051</td>
<td>.843</td>
<td>.828</td>
</tr>
<tr>
<td>8.5 - 9.5</td>
<td>.958</td>
<td>.889</td>
<td>.060</td>
<td>.870</td>
<td>.854</td>
</tr>
<tr>
<td>9.5 - 10.5</td>
<td>.961</td>
<td>.906</td>
<td>.055</td>
<td>.889</td>
<td>.861</td>
</tr>
<tr>
<td>10.5 - 11.5</td>
<td>.972</td>
<td>.908</td>
<td>.064</td>
<td>.909</td>
<td>.869</td>
</tr>
<tr>
<td>11.5 - 12.5</td>
<td>.972</td>
<td>.908</td>
<td>.064</td>
<td>.925</td>
<td>.874</td>
</tr>
<tr>
<td>12.5 - 13.5</td>
<td>.975</td>
<td>.908</td>
<td>.067</td>
<td>.938</td>
<td>.882</td>
</tr>
<tr>
<td>13.5 - 14.5</td>
<td>.975</td>
<td>.910</td>
<td>.065</td>
<td>.941</td>
<td>.885</td>
</tr>
<tr>
<td>14.5 - 15.5</td>
<td>.977</td>
<td>.912</td>
<td>.056</td>
<td>.950</td>
<td>.890</td>
</tr>
<tr>
<td>15.5</td>
<td>1.00</td>
<td>1.00</td>
<td>0.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

| No. observed | 457 | 494 | 406 | 384 | 437 | 487 | 426 | 408 | 464 | 492 | 399 | 387 |

| Max. Diff. | .067 | .060 | .007 | .055 | .057 | .047 |

| Critical D at 3% | .088 | .097 | .090 | .094 | .088 | .097 |

| Remarks | Not Reject | Not Reject | Not Reject | Not Reject | Not Reject | Not Reject |

+ Distance from stop line.
4.4 Closure

From the above discussions of the developed simulation model, it is noted that the model combines both semi-microscopic and macroscopic behavior of vehicles. Such a feature is considered essential for the purpose of this study. The design of the model is such that the model can be modified for the inclusion of more intersections in the simulation without difficulty. The application of the model therefore can become a method to search for the optimum setting of signal offsets for any particular traffic condition and configuration.
5.1 Performance Measures

In order to evaluate a setting of a signal progression system, some measure of performance is necessary. In fact, there are quite a number of measures to choose from, namely, mean delay per vehicle, total delay, mean traveltime, mean queue length, total number of delayed vehicles, maximum throughput, and even maximum queue length. For the same operating and traffic conditions, these measures are all interrelated. For instance, a high mean queue length usually corresponds to a high mean delay per vehicle, and vice versa.

For the purpose of this study three measures were selected: mean traveltime, mean queue length and mean delay per vehicle at a downstream intersection. The data on queue length and delay at the first intersection were not included in the evaluation because the first signal can be assumed to operate independently of the other signals regardless of the coordination of the signal system. Mean queue length and mean delay are relatively straightforward criteria, but mean traveltime requires some explanation.
Mathematically, traveltime of a vehicle can be shown as:

\[ T_k = \sum_{i=1}^{N} D_i + \sum_{i<j} R_{ij} + \sum_{i<j} R'_{ij} + \sum_{i=1}^{N-1} A_i \]

where

- \( T_k \) = total traveltime by vehicle \( k \) through \( N \) intersections
- \( D_i \) = stopped delay at intersection \( i \)
- \( R_{ij} \) = random running time between intersection \( i \) and intersection \( j \), stop line to stop line with the condition that \( R_{ij} = 0 \), if \( D_i \neq 0 \).
- \( R'_{ij} \) = random running time between intersection \( i \) and intersection \( j \), from a distance 195 ft. downstream of intersection \( i \) to stop line at intersection \( j \) with the condition that \( R'_{ij} = 0 \), if \( D_i = 0 \).
- \( A_i \) = acceleration time from a stop line at intersection \( i \) up to a distance of 195 feet. (\( A_i = 0 \), if \( D_i = 0 \)).

For a particular vehicle, the minimum traveltime is equal to the summation of all random running times between intersections. Also, the maximum traveltime is equal to the summation of random stopped delays at every intersection, acceleration times at immediate downstream distance of 195 feet past each intersection, and random running times between sections from 195 feet downstream of each intersection to the stop lines of each subsequent downstream intersection. Obviously, the minimum traveltime case can normally be attained if downstream signals are properly set.

5.2 Analyses of Factors Affecting the Selected Performance Measures

In order to investigate factors that affect the selected performance measures and thus the effectiveness of a setting of a signal system, the
simulation model was utilized to perform appropriate computer experiments by holding constant all variables except the factor to be investigated. In this manner, the effects of each factor on the performance measures can be analyzed.

5.2.1 Signal Offset

Signal offset was the first factor investigated by employing the validated simulation model. To start with, a signal spacing of 2000 feet was arbitrarily selected and tried. At both intersections, a ninety second cycle length and a forty second effective green period were set. For the purpose of investigating the signal offset factor, the traffic from minor roads was excluded. The selected main traffic volumes were 550 and 350 vph on the median and shoulder lanes respectively. Figure 5.1 (a) to 5.1 (c) present the results obtained when varying only the signal offset. It is clear that any of the three criteria can indicate the effectiveness of the signal settings. These figures also suggest similar relationships between the signal offset and the selected performance measures. These relationships are parabolic in form, and thus there should always exist an optimum signal offset at a downstream signal.

It should be noted that two curves are present on each figure; one curve corresponds to the median lane while the other parabolic curve represents the shoulder lane. The optimum signal offset for each lane is determined by
Figure 5.1 (a) The Effect of Signal Offset on Mean Delay; Signal Spacing = 2000 feet and G/C = 40/90
Figure 5.1 (b) The Effect of Signal Offset on Mean Traveltime; Signal Spacing = 2000 feet and G/C = 40/90
Figure 5.1 (c) The Effect of Signal Offset on Mean Queue Length
Signal Spacing of 2000 feet and G/C = 40/90
using different means and standard deviations of running times presented in
Chapter Three. Figure 5.1 (a) through 5.1 (c) also suggest that the optimum
signal offset for one signal spacing is somewhere between the optimum offset
of each lane. However, this will not be the case if it happens that the
means and standard deviations of the running times are the same for both lanes.

5.2.2. Main Traffic Volume

To study the effect of main traffic volumes, the simulation model was
implemented by varying main traffic volumes for a signal spacing of 5000 feet, a
ninety second cycle length, a forty-second effective green period and offsets of
fifty, sixty, seventy, and eighty seconds. The results are illustrated in Figures
5.2(a) to 5.2(c). These figures indicate some interactions between traffic
volumes and signal offsets at the downstream signal. Consequently, further
investigations were conducted at signal spacings of 6500 and 9500 feet for the
same cycle length and effective green period as the preceding simulation runs.
Figures 5.3(a) to 5.3(c) and Figures 5.4(a) to 5.4(c) show the results at signal
spacings of 6500 and 9500 feet respectively. It should be noted that Figures 5.3
and 5.4 display relationships between the performance measures and the ratios
of signal offsets to means of running times, instead of the absolute signal
offset as previously presented.

The usual definition of the signal offset, which varies from zero to the
cycle length, is one important drawback that requires attention. Such a defini-
tion is not proper at a signal spacing over which the mean running time is very
Figure 5.2 (a): The Effect of Main Traffic Volumes on Mean Delay; Signal Spacing = 5000 feet and G/C = 40/90
Figure 5.2 (b) : The Effect of Main Traffic Volumes on Mean Traveltime; Signal Spacing = 5000 feet and G/C = 40/90.
Figure 5.2 (c)  The Effect of Main Traffic Volume on Mean Queue Length; Signal Spacing = 5000 feet and G/C = 40/90
Figure 5.3 (a): The Effect of Main Traffic Volumes on Mean Delay; Signal Spacing = 6500 feet and G/C = 40/90.
Figure 5.3 (b): The Effect of Main Traffic Volume on Mean Traveltime; Signal Spacing = 6500 feet and G/C = 40/90.
Figure 5.3 (c) : The Effect of Main Traffic Volumes on Mean Queue Length; Signal Spacing = 6500 feet and G/C = 40/90.
Figure 5.4 (a) The Effect of Main Traffic Volumes on Mean Delay at each Corresponding Offset; Signal Spacing = 9500 feet and G/C = 40/90
Figure 5.4 (b) The Effect of Main Traffic Volumes on Mean Traveltime at each Corresponding Offset; Signal Spacing = 9500 feet and G/C=40/90
Figure 5.4 (c) The Effect of Main Traffic Volumes on Mean Queue Length at each Corresponding Offset; Signal Spacing = 9500 feet and G/C = 40/90
close to or longer than one cycle length. This drawback becomes even more pronounced when dealing with mathematical models which will relate the performance measures to the explanatory variables. Accordingly, a new definition of offset is required in this study and is defined to be the addition of the multiple of cycle length close to mean running time and the real offset time. As has been mentioned, this definition is applied whenever the mean running time is close to or longer than one cycle length. In addition to this new definition of offset, it is also desirable to transform the new variable into a ratio of the offset to the mean running time (to be referred to later as simply 'ratio') such that it will be possible to develop mathematical models for predicting the three performance measures at the first downstream signalized intersection. The above explanation will be further discussed in the subsequent enumeration of practical applications.

Despite the substantial difference in signal spacings, the results of Figure 5.3 and 5.4 show similar relationships between the performance measures and the traffic volumes, as well as between the performance measures and the ratios of the offsets to the mean running times. Besides direct relationships, the figures also illustrate that there are certainly some interactions between the traffic volumes and the ratios. Thus, for higher volumes, the corresponding optimum signal offsets should be changed accordingly. Such a finding has never been realized before since it has previously been understood that the optimum signal offset depends solely upon the mean traveltime, as

5.2.3 Minor Traffic Volume

To investigate the effect of minor traffic volume on the effectiveness of signal progression, computer experiments were performed at a signal spacing of 2000 feet, with a ninety second cycle length and a forty second effective green period at both intersections. Figures 5.5 (a) to 5.5 (c) show the results of incorporating minor traffic volumes ranging from 0 to 200 vph in conjunction with a fixed main traffic volume of 550 vph. There is very little or no effect in terms of the optimum signal offset when mean delay per vehicle at the downstream intersection and mean traveltime are used as criteria. However, there is a slight effect present when mean queue length is used as a measure of performance, but such effect seems to be very little when considering the real sensitivity of the timing mechanism in the signal control device.

5.2.4 Trucks

Trucks have been thought of as retarding factors in traffic flow, especially when vehicles, delayed by a previous signal, follow a truck at an intersection. To further investigate such an effect, the amount of truck traffic was varied from zero to twenty percent in several simulation runs at ten second and twenty second offsets. These experiments were performed at a signal spacing of 2000 feet, with a ninety second cycle length, and a forty second effective green period at both intersections. Figures 5.6 (a)
Figure 5.5 (a) The Effect of Minor Traffic Volumes on Mean Delay:
Signal Spacing 2000 feet and G/C = 40/90
Figure 5.5 (b) The Effect of Minor Traffic Volumes on Mean Traveltime; Signal Spacing 2000 feet and G/C = 40/90
Figure 5.5 (c) The Effect of Minor Traffic Volumes on Mean Queue Length; Signal Spacing 2000 feet and G/C = 40/90
Figure 5.6 (a) The Effect of Truck Traffic on Mean Delay;
Signal Spacing = 2000 feet, G/C = 40/90 and
Main Traffic Volume = 700 vph.
Figure 5.6 (b) The Effect of Truck Traffic on Mean Traveltime; Signal Spacing = 2000 feet, G/C = 40/90 and Main Traffic Volume = 700 vph.
Figure 5.6 (c) The Effect of Truck Traffic on Mean Queue Length;
Signal Spacing = 2000 feet, G/C = 40/90, and
Main Traffic Volume = 700 vph.
to 5.6 (c) present results which clearly indicate the linear effect of truck traffic on all three measures of performance.

5.2.5 Signal Spacing

In an attempt to determine the maximum spacing at which the provision of signal progression is still beneficial, a traffic volume of 550 vehicles per hour on the median lane was held fixed while the offset of a downstream signal, positioned at a spacing of 2000, 4000, 6000, 8000 and 10000 feet, was varied. The results of such experiments are presented in Figures 5.7 (a) and (b). In this set of experiments, mean travel times are not presented because of large differences in the results of different spacings.

From Figures 5.7 (a) and (b), it is apparent that up to a distance of 10,000 feet there still exists an optimal setting of a downstream signal. Furthermore, such an optimal setting reduces substantially both mean delay per vehicle and mean queue length. This implies that the provision of signal progression for a spacing of up to 10,000 feet is still beneficial. However, this implication is based on experiments that utilized a common cycle length of ninety seconds. Therefore, additional experiments at a signal spacing of 9500 feet were performed with different cycle lengths of 75, 90 and 120 seconds. The experimental roadway was assumed to serve a traffic volume of 800 vehicles per hour. Minor traffic and trucks were not included in this set of experiments.

Figures 5.8 (a) to (c) display the results of such experiments. It is
Figure 5.7 (a) The Effect of Signal Spacing on Mean Delay; Traffic Volume = 550 vph, and G/C = 40/90
Figure 5.7 (b) The Effect of Signal Spacing on Mean Queue Length; Traffic Volume = 550 and G/C = 40/90
obvious that the relationships between the three performance measures and the ratios are similar for all cycle lengths to those relationships shown on Figures 5.7 (a) and (b). The results presented above lead to the conclusion that there are always some benefits to be gained in the provision of signal progression. Operationally, this means that there is no limit to the maximum signal spacing for the provision of signal progression. Such a finding is quite different from previous studies (11, 13, 14, 15). The conclusions from previous studies were based on the results of microscopic studies of platoon dispersion. On the contrary, the conclusion from this study is based not only on the possibility of finding an optimal setting of a downstream signal but also on the relative amount of improvement, as measured by reductions in the criteria of mean traveltime, mean delay, and mean queue length.

In addition, Figures 5.8 (a) to (b) also show that employing different criteria would sometimes result in different optimal cycle lengths. For instance, among the three cycle lengths tested under similar conditions, the 75-second cycle length would be the optimal cycle length if the mean traveltime is used, while the 90-second cycle length would be the optimal one if the mean delay per vehicle is used. However, both 75- and 90-second cycle lengths yield practically the same minimum mean queue length. Such a finding shows that the two-intersection simulation model can be implemented to help a traffic engineer in his task to find the cycle length that would yield the 'global' optimal operating conditions.
Figure 5.8 (a) The Effect of Cycle Length on Mean Delay;
Signal Spacing = 9500 and Traffic Volume = 800 vph.
Figure 5.8 (b) The Effect of Cycle Length on Mean Traveltime;
Signal Spacing = 9500 feet and Traffic Volume = 800 vph.
Figure 5.8 (c) The Effect of Cycle Length on Mean Queue Length; Signal Spacing = 9500 feet and Traffic Volume = 800 vph.
5.3 Practical Applications

Based upon the results presented in the preceding section, three regression models were developed to be used in the formulation of guidelines pertaining to the introduction of signal progression on major arterials, as well as in the prediction of traveltime, mean delay per vehicle, and mean queue length, under different traffic and operating conditions.

Before beginning any discussion on practical applications from the results of this study, it is useful to briefly discuss the development of such regression models. In developing the three regression models, the following notations are used to represent independent variables:

- **RATIO** = the ratio of offset to mean running time
- **SQRAT** = \((RATIO)^2\)
- **MRT** = mean running time, seconds
- **MSTD** = standard deviation of running time, seconds
- **VOL** = main traffic volume, vehicles per hour
- **MINOR** = minor traffic volume that merges into main traffic, vehicles per hour
- **TRUCK** = percentage of trucks
- **CYC1** = common cycle length, seconds
- **XMRT** = interaction between ratio and mean running time
- **XTRK** = interaction between ratio and percentage of trucks
- **XVOL** = interaction between ratio and major traffic volume
- **XMIN** = interaction between ratio and minor traffic volume
G1 = effective green period at Intersection 1
G2 = effective green period at Intersection 2
GOVC1 = G1 / CYC1
GOVC2 = G2 / CYC1.

The following variables are performance measures or dependent variables:
TRAV = mean traveltime, seconds
DLAY = mean delay, seconds per vehicle
QUE = mean queue length, vehicles

Most of the variables listed above are self-explanatory and need no further discussion, except XMRT. Since the relationships between system performance measures and signal offsets are parabolic in form, the use of an absolute value of offset would not allow the development of a simple regression model. Such a drawback can be clearly seen in Figure 5.9, where an ideal case is presented. This necessitates the transformation of real values of offsets into ratios of 'new offsets' divided by mean running times. As was stated in the previous section, the 'new offset' is the addition of the real offset and a multiple cycle length that is close to the corresponding mean running time, such that the range of ratios would include an optimum ratio, expectedly between 0.5 and 1.8.

In spite of the desirable advantage of this new definition, it is subjected to one minor problem. One unit of a ratio at a shorter spacing would correspond to less absolute offset than one unit at a longer spacing. Such an effect on a performance
Figure 5.9  
Ideal Relationship between a Criterion and Offset at Various Signal Spacings, $D_i$ 
($D_1 < D_2 < D_3 < D_4 < D_5$)
measure can be clearly shown in Figure 5.10. Mathematically, this implies that the coefficients $A$, $B$, and $C$ of the parabolic expression $y = Ax^2 + Bx + C$ are not constant, but rather functions of mean running time.

Based on the above discussion several regression models for all three performance measures were investigated. Using three criteria, namely, 'minimum' mean squared error, 'maximum' coefficient of determination ($R^2$), and the significance level of explanatory variables (as measured by t-statistic), three 'best' models were selected and are presented in computer output form as Tables 5.1 to 5.3. It should be noted that the 'best' models for different performance measures contain some different explanatory variables, and also that the effect of trucks, when added to the variables composing the 'best' models, was found to be 'insignificant', in spite of its intuitively appealing contribution. Perhaps, developing these types of regression models for each cycle length and performing more experiments would result in the future inclusion of 'insignificant' independent variables such as TRUCK, GOVC 1, GOVC 2 and so on.

The three regression models were found to be satisfactory in the sense that 92.0, 99.7 and 93.6 percent of mean delays per vehicle, mean travel times and mean queue lengths, respectively, can be explained by the models. Two desirable applications can be found from these models, namely, (1) predictions of the three performance measures under different traffic and operation conditions are possible, and (2) a guideline to obtain an optimal signal setting
Figure 5.10  Ideal Relationship between a Criterion and Offset Ratio at Various Signal Spacings, $D_1$

($D_1 < D_2 < D_3 < D_4 < D_5$)
Table 5.1  Computer Output of Regression Analysis of Mean Delay Model

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<td>0.4097</td>
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Table 5.2  Computer Output of Regression Analysis of Mean Traveltime Model

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In the analysis of variance table, the regression model is evaluated with a sum of squares of 15054137, where the fitted regression is 15941.4. The residual sum of squares is 216736, and the corrected sum of squares is 172167, indicating a good fit. The model's R² value is 0.81529, suggesting a strong relationship between the variables. The t-values for each predictor variable are also provided, with significant values indicating a statistically significant relationship with the dependent variable. The standard errors for the regression coefficients and the t-values are calculated to assess the significance of the predictors.
### Table 5.3 Computer Output of Regression Analysis of Mean Queue Length Model

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- **C.V. (Coefficient of Variation):** 42.29319
- **Mean Queue Length:** 2.99466623
- **Std Dev:** 7.09072

This table presents the computer output of a regression analysis for the mean queue length model. The table includes the sequential sum of squares, mean square, F value, and p-value for both the model and residual. The analysis reveals that the model is statistically significant with a p-value of 0.0001, indicating a good fit for the model.
for the introduction of signal progression and to evaluate the difference between
an existing setting and an optimal setting can be determined.

To illustrate the two applications it is helpful to present the three
developed regression models as follows:

\[
DLAY = 17.3142 \times (RATIO)^2 - 40.1686 \times (RATIO) + 0.0368 \times (VOLUME) \\
+ 0.2206 \times (MRT) - 0.1955 \times (MRT \times RATIO) + 0.2675 \times (CYCLE) \\
- 0.0168 \times (VOLUME \times RATIO) + 0.0111 \times (MINOR) \\
+ 0.9511 \times (MSTD) \quad \ldots \ldots \quad \text{(Eq. 1)}
\]

\[
TRAV = 18.3266 \times (RATIO)^2 - 42.8455 \times (RATIO) + 0.0657 \times (VOLUME) \\
+ 1.2021 \times (MRT) - 0.1974 \times (MRT \times RATIO) + 0.6064 \times (CYCLE) \\
- 0.0162 \times (VOLUME \times RATIO) + 38.0344 \times (GOVC 1) \\
+ 1.9384 \times (MSTD) \quad \ldots \ldots \quad \text{(2)}
\]

\[
QUE = 6.1847 \times (RATIO)^2 - 10.9797 \times (RATIO) + 0.0242 \times (VOLUME) \\
+ 0.0798 \times (MRT) - 0.0460 \times (MRT \times RATIO) + 0.0520 \times (CYCLE) \\
- 0.0111 \times (VOLUME \times RATIO) + 0.0110 \times (MINOR) \\
- 8.0282 \times (GOVC 2) \quad \ldots \ldots \quad \text{(3)}
\]

**Application I**

Suppose that the following represent traffic and operating conditions
between two intersections:

Traffic volume \(= 605 \text{ vph.}\)

Distance between signals \(= 3500 \text{ feet}\)

Minor traffic volume (that merges into the main traffic) \(= 175 \text{ vph.}\)

Speed limit \(= 55 \text{ mph.}\)

Cycle length \(= 85 \text{ seconds}\)

Green period (signal 1) \(= 40 \text{ seconds}\)

Green period (signal 2) \(= 38 \text{ seconds}\)
Amber period = 5 seconds

Signal offset (signal 2) = 33 seconds

It is desired to estimate mean delay per vehicle at the downstream signal.

To apply the regression model for mean delay per vehicle it is first necessary to estimate the mean and standard deviation of the running times. This can be accomplished by applying the regression model summarized in Table 3.4 on page 50. By applying the models for mean and standard deviation of running times on the median lane, the mean running time and the associated standard deviation are estimated to be 48.7 and 4.9 seconds, respectively. In this case, it is not necessary to transform the real offset into a new offset. Therefore, the ratio can be calculated directly and is equal to 0.6776. Substituting all the variables appearing in Equation 1 on page 120 yields a mean delay of 29.7 seconds per vehicle.

Now, if the signal spacing is 5500 feet, the mean and standard deviation of running time can be estimated to be 76.0 and 6.8 seconds, respectively. As a general rule for applying the developed regression models, if the real offset is less than one-half the mean running time, then the transformed offset should be used in calculating its corresponding ratio. The real offset should be transformed so that the resultant ratio is close to 1. This rule was applied during the preparation of the simulated data.

In following the above rule, the signal offset of 33 seconds should be transformed to 118 seconds, which results in a value for the ratio of 1.5526. Substituting all the values of the required variables yields an estimated mean
delay of 10.7 seconds per vehicle. The second approach in estimating any of
the three performance measures is to run the computer simulation by preparing
all the information as input to the two-intersection model.

Application II

The second application is considered more useful than the first one,
although it does require an application of the classical optimization technique
by differentiating mean delay per vehicle, mean traveltime, and mean queue
length, with respect to the ratio and then equating such derivatives to zero.

Further simplification results in three models, to be referred to as guideline
models, which can be used to determine an optimum offset of the downstream
signal based on the minimization of mean delay per vehicle, mean traveltime, or
mean queue length, respectively, as follows:

(i) Based on minimum mean delay per vehicle;

\[
\text{OPTIMUM RATIO} = \frac{1}{34.6284} \left[ 0.1955(MRT) + 0.0168(VOLUME) + 40.1686 \right] \quad \ldots \ldots (4)
\]

(ii) Based on minimum mean traveltime;

\[
\text{OPTIMUM RATIO} = \frac{1}{36.6532} \left[ 0.1974(MRT) + 0.0162(VOLUME) + 42.8455 \right] \quad \ldots \ldots (5)
\]

(iii) Based on minimum mean queue length

\[
\text{OPTIMUM RATIO} = \frac{1}{12.3694} \left[ 0.0460(MRT) + 0.0111(VOLUME) + 10.9797 \right] \quad \ldots \ldots (6)
\]

As an illustration, it is proposed to provide the progression of traffic between
two signalized intersections separated by a distance of 3500 feet. Previous data
show that the normal mean running time between the two intersections is 50.6 seconds, and the normal traffic volume is 645 vehicles per hour. It is desired to find an optimal setting of the downstream signal such that mean traveltime can be minimized.

Based on the above information the optimal ratio can be calculated from Equation (5) as:

\[
\text{OPTIMUM RATIO} = \frac{1}{36.6532} \left[ 0.1974(50.6) + 0.0162(645) + 42.8455 \right]
\]

\[= 1.7265\]

The optimal offset can now be obtained by simply multiplying the ratio by the mean running time. This yields an optimal offset of 87.4 seconds. The real offset depends on the cycle length. For example, for cycle lengths of 60, 75 and 90 seconds, the real offsets would be 27.4, 12.4 and 87.4 seconds, respectively. However, if the criterion is changed from minimum mean travel-time to minimum mean delay per vehicle, this would result in an optimal offset of 89.0 seconds, using Equation (4). Similarly, if mean queue length is the criterion, then the optimal offset, from Equation (6), would be 83.7 seconds. Although the optimal offsets based on different criteria are not exactly the same value, the differences are very small when compared to the accuracy of the control mechanism of the signal. If the two signals have already been set, the result can be used to examine if the setting is sub-optimal.

One particularly interesting feature of the guideline models is that they do not directly include distance or signal spacing as an explanatory variable.
Including the mean running time as an explanatory variable allows the applications of the guideline models to roadways with speed limits other than 55 miles per hour. The guideline models can always be applied as long as the information on mean running time is available.

5.4 Closure

The material presented in this chapter is the result of the implementation of the validated simulation model. Five factors, namely, signal offset, main traffic volume, minor traffic volume, trucks and signal spacing have been extensively investigated. The effects of both mean running time and cycle lengths have also been presented. Among these factors, it has been found that signal offset is the most influential factor while mean running time, main traffic volume, cycle length and minor traffic volume have some significant effects on the performance measures. The percentage of trucks, however, appears to be a relatively insignificant factor.

Furthermore, it has been found that there is no operational limit on the maximum spacing for signal progression. Such a conclusion is based on two criteria; (1) the existence of an optimal setting corresponding to each combination of traffic and operating conditions, and (2) the relative amount of the reduction of mean delay per vehicle, mean traveltime and mean queue lengths. However, the question of ascertaining a practical limit is still a subject for future research.

Finally, the most important result from the implementation of the two-
intersection simulation model is the development of three regression models which can be ultimately used to determine a guideline for the introduction of signal progression. The developed guideline models can be easily applied to the setting of signals on any arterial provided that the information on mean running time and traffic volume is available. From the guideline models it is worth noting that optimal offset increases with traffic volume. The increase depends upon the criterion used. For example, if the criterion is mean delay per vehicle, then, optimal offset increases by about 1.7 seconds for each 100 vehicles per hour increment of traffic volume.
CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

Before presenting a summary of the results attained during the course of this study, it is appropriate to review the objectives to which this study was addressed and the techniques that were employed to achieve those objectives. In this manner, a framework can be established from which the value of the investigation can be judged.

The primary objective of this study was to develop criteria for the introduction of signal progression on major state-controlled signalized arterials in suburban-to-rural areas. This has been accomplished by developing, validating, and implementing the two-intersection simulation model. Three parameters, namely, mean delay per vehicle, mean traveltime, and mean queue length, were selected as criteria to measure the effectiveness of different signal settings at a downstream intersection. Among the factors investigated, signal offset was found to be the most influential factor upon the three measures of effectiveness. Other significant factors included mean running time, main traffic volume, cycle length, and minor traffic volume that merges into the main traffic stream. The percentage of trucks was a relatively insignificant factor.
By means of regression analysis techniques, the simulated data were utilized in the development of three predictive models, corresponding to the three measures of effectiveness, and subsequently three guideline models. Some examples of the application of both predictive and guideline models were illustrated in Chapter Five. The guideline models are useful because they are directly applicable in determining an advantageous procedure for the introduction of signal progression between a pair of signalized intersections.

The secondary objective of this study was to determine the maximum signal spacing at which the provision of progression is still beneficial. In accomplishing this objective, the developed simulation model was implemented to investigate whether there is any operational limit to maximum signal spacing. Several computer experiments with varying traffic and operating conditions were performed, with the result that regardless of the specific signal spacing and cycle length: (1) there always exists an optimal setting of a downstream signal, and (2) the relationships between the three performance measures (mean delay per vehicle, mean traveltime and mean queue length) and the signal offset are always parabolic in form, indicating that a substantial improvement is attainable at the optimal setting.

Based on the findings of this study it can be concluded that there is no defined operational limit to the maximum signal spacing. If the main traffic volume is large enough to warrant signal operation, there is always some benefit to be realized by providing signal progression.
6.2 Recommendations

Despite considerable effort spent in developing and implementing the operational simulation model, there still exists, as with any predictive modelling process, further experimental research that should be conducted. Thus, the following recommendations for future work are presented:

(1) Separate predictive models for different cycle lengths need to be developed because firstly, the relationships between the three performance measures and the cycle length are not conceptually linear, and secondly, the intuitively appealing factor of percentage of trucks, which was shown to be insignificant, should, however, contribute to improving the regression models.

(2) Both economical and operational comparative analyses between implemented "optimal" signal progression systems and semi-actuated systems (which may or may not be progressive) should be undertaken.

(3) Investigation of the effect of using different theoretical distributions of interarrival times to represent the real traffic arrival distribution is recommended. The motivation is that a simpler theoretical distribution, such as the negative exponential distribution, can be adequately utilized without jeopardizing the validity of the results.
APPENDIX A

DATA FORMS AND SOME SAMPLES OF DATA

FORM A: Running Time and Arrival Time

FORM B: Starting Time Data
## FORM A: Running Time and Arrival Time

**PLATOON No. 110**

**SITE:** U.S. 35  
**LANE:** Median  
**DIRECTION:** EB  

**OBSERVATION DATE:** 8/27/74  
**TIME:** 4:30-6:00 p.m.

<table>
<thead>
<tr>
<th>License Number</th>
<th>Vehicle Type</th>
<th>$T_i$ = Time passing station $i$, $i = A, B, C, D$</th>
</tr>
</thead>
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<td></td>
<td>$T_A$ 25:04 $T_{AE}$ 36 $T_B$ 25:40 $T_{BC}$ 40 $T_C$ 26:20 $T_{CD}$ 45 $T_D$ 27:05</td>
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<tr>
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</tr>
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</tr>
</tbody>
</table>

**Remarks:**  
$T$ = truck  
(blank) = passenger car  
$^*$ change to the other lane  
$^{**}$ change to the original lane
### Form B: Starting Time Data

**Intersection:** W. N. Broadway  
**Site:** Olentangy R. Rd.  
**Lane:** Median  
**Direction:** NB  
**Observation Date:** 4/9/76  
**Time:** 4:30 - 5:45

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<td>2.00</td>
<td>1.91</td>
<td>1.94</td>
<td>2.00</td>
<td>1.98</td>
<td>2.00</td>
<td>2.38</td>
</tr>
<tr>
<td>Hdwy.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
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</tbody>
</table>
APPENDIX B

LISTING OF PROGRAM STATEMENTS FOR CALCULATING COMPOSITE EXPONENTIAL DISTRIBUTIONS FOR VARIOUS TRAFFIC VOLUMES
In 1955, Schuhl (25) proposed that the traffic stream be considered as composed of a combination of two types of vehicles, namely, free-flowing vehicles and restrained vehicles. The free-flowing vehicles are not influenced by vehicles in front of them, and thus, can pass these vehicles at will. On the contrary, restrained vehicles cannot pass front vehicles at will and thus possess minimum headway. Mathematically, the composite exponential distribution proposed by Schuhl can be expressed as

\[ P(h > t) = (1-a) \exp \left( -\frac{t}{T_1} \right) + a \exp \left( -\frac{t - \tau}{T_2 - \tau} \right) \quad (1) \]

where

- \( 1 - a \) = fraction of free-flowing vehicles
- \( T_1 \) = average headway of free-flowing vehicles
- \( a \) = fraction of restrained vehicles
- \( T_2 \) = average headway of restrained vehicles
- \( \tau \) = minimum headway of restrained vehicles.

In 1962, Kell (26) utilized a slightly modified version of the composite exponential distribution. He considered free-flowing vehicles to also possess a minimum headway which results in the following expression:

\[ P(h > t) = (1-a) \exp \left( -\frac{t - \lambda}{T_1 - \lambda} \right) + a \exp \left( -\frac{t - \tau}{T_2 - \tau} \right) \quad (2) \]

where

- \( \lambda \) = minimum headway of free-flowing vehicles.

By transforming Equation (2), Kell obtained

\[ P(h > t) = \exp \left( A - \frac{t}{K_1} \right) + \exp \left( C - \frac{t}{K_2} \right) \quad (3) \]

where

- \( A = \frac{\lambda}{T_1 - \lambda} + \ln (1-a) \)
In estimating the above transformed parameters, Kell collected extensive field data from two-lane urban streets having traffic volumes ranging from about 100 to 1200 vph. This resulted in the following relationships:

\[
\begin{align*}
K_1 &= T_1 - \lambda \\
C &= \frac{\tau}{T_2 - \tau} + \ln(a) \\
K_2 &= \frac{\tau}{T_2 - \tau}
\end{align*}
\]

\[
\begin{align*}
K_1 &= \frac{4827.9}{V^{1.024}} \\
A &= -0.046 - 0.000448 V \\
K_2 &= 2.659 - 0.00120 V \\
C &= \left[ \exp (-10.503 + 2.829 \ln V - 0.173 (\ln V)^2) \right] - 2
\end{align*}
\]

where \( V \) = lane volume, vehicles per hour

Also, he found that best fit between the theoretical curve and the observed data occurred where \( 0.9 < \lambda < 1.0 \) and \( 1.2 < \tau < 1.36 \).

In order to test goodness-of-fit between the observed data in this study and the theoretical distribution using the above estimated parameter, a computer program was developed to calculate the cumulative distribution for every five vehicle per hour increment ranging from 100 to 1200 vph. To obtain an accurate distribution, each cumulative distribution was calculated at an increment of 0.25 second. Figure B-1 represents the listing of program statements. Some of the results are plotted and shown in Figure B-2.

In 1968, Greco and Sword (27) used the following expression of the
THE PROGRAM FOR COMPUTING CUMULATIVE COMPOSITE EXPONENTIAL DISTRIBUTION OF WHICH ITS PARAMETERS ARE FROM KELL'S REPORT ON "ANALYZING VEHICULAR DELAY AT INTERSECTIONS THROUGH SIMULATION", HIGHWAY RESEARCH BOARD BULLETIN 356, 1962, PAGE 28-35. THE OUTPUT IS FOR VOLUMES FROM 100 TO 1200 VPH AT AN INCREMENT OF 5 VPH AND EACH CUMULATIVE DISTRIBUTION IS CALCULATED AT EVERY 0.25 SECOND.

ELIMINATE LOOP #1 IF ONLY A SPECIFIC VOLUME, SAY 766 VPH, IS NEEDED; THEN REPLACE THE CARD V = 100.0 BY V = 766.0.

V = APPROACH VOLUME IN VEHICLES PER HOUR PER LANE

\( \Lambda = \) MINIMUM HEADWAY OF FREE FLOW TRAFFIC PORTION = 0.90 SEC.

\( \tau = \) MINIMUM HEADWAY OF 'RESTRAINED' TRAFFIC PORTION = 1.22 SEC.

\( \alpha_1 = \) PERCENTAGE OF FREE FLOW TRAFFIC

\( \alpha_2 = \) PERCENTAGE OF 'RESTRAINED' TRAFFIC

\( \text{PROB}(i) = \) PROBABILITY THAT HEADWAY IS GREATER THAN OR EQUAL TO I SECOND.

\( \text{DIMENSION PPPP}(200), \text{PROB}(200) \)

REAL \( K_1, K_2, \Lambda, \alpha_1 \)

\( V = 100.0 \)

\( \Lambda = 0.50 \)

\( \tau = 1.22 \)

DO 1 I = 1, 220

\( K_1 = \frac{4827.4}{V^{1.024}} \)

\( A = -0.046 - 0.046 * V/100.0 \)

\( K_2 = 2.556 - 0.120 * V/100.0 \)

\( C = \exp(10.503 + 2.829 * \log(V) - 0.173 * (\log(V)^2)) - 2.0 \)

\( \alpha_1 = \exp(\Lambda - \Lambda \div K_1) \times 100.0 \)

\( \alpha_2 = 100.0 - \alpha_1 \)

WRITE(6, 200) V, \( \lambda, \tau, \alpha_1, A, \alpha_2, C, \alpha_2 \)

200 FORMAT('6(2X, 'COMPOSITE EXPONENTIAL DISTRIBUTION\n
142X, 'APPROACH VOLUME = ', F6.0, ' VPH.\n
242X, 'HEADWAY = ', F6.3, ' SECONDS.\n
342X, 'MINIMUM HEADWAY OF FREE FLOW TRAFFIC PORTION = ', F6.3, ' SECONDS.\n
442X, 'MINIMUM HEADWAY OF RESTRAINED TRAFFIC PORTION = ', F6.3, ' SECONDS.\n
542X, 'PERCENT OF FREE FLOW TRAFFIC = ', F6.3, '.\n
642X, 'PERCENT OF RESTRAINED TRAFFIC = ', F6.3, '.\n
742X, 'PERCENT OF FREE FLOW TRAFFIC = ', F6.3, '.\n
842X, 'PERCENT OF RESTRAINED TRAFFIC = ', F6.3, '.\n
942X, 'PERCENT OF TOTAL VOLUME = ', F6.3, '.')
Figure B-1 (Continued)

```
T = 0.0
DO 2 J=1,2000
  T = T+0.25
  P1 = A-(T/K1)
  PPPP(J) = EXP(A-T/K1)+EXP(C-T/K2)
  PROB(J) = 1.0-PPPP(J)
  H = J
  IF(PROB(J).LT.0.999996) GO TO 2
  AUX = M/4.
  MM = AUX
  BUX = AUX-MM
  IF(BUX.TQ.0.0) GO TO 888
  2 CONTINUE
888 WRITE(6,201)
201 FORMAT(3X,' T PPPP(T) PROB(T)',6X)/
  GO 3 J=1,J,4
  J2 = J+1
  J3 = J+2
  J4 = J+3
  T1 = J*0.25
  T2 = J2*0.25
  T3 = J3*0.25
  T4 = J4*0.25
  3 WRITE(6,202) T1,PPPP(J),PROB(J),T2,PPPP(J2),PROB(J2),
     * T3,PPPP(J3),PROB(J3),T4,PPPP(J4),PROB(J4)
202 FORMAT(3X,4(F6.2,3X,F7.5,3X,F7.5,4X))
  V = V+5.0
  1 CONTINUE
STOP
END
```
Figure B-2  Cumulative Composite Exponential Distribution for Different Traffic Volumes.
Schuhl equation:

\[ P(h \geq t) = (1-a) \exp\left(-\frac{t}{T_1}\right) + a \exp\left(-\frac{t - \tau}{T_2}\right) \]  

(4)

In their study of headway distribution using the form of Equation (4), Greco and Sword obtained the following relationships:

\[ a = 0.00115 V \]
\[ T_1 = 24 - 0.0122 V \]
\[ T_2 = 2.5 \]
\[ \tau = 1.0 \]

where \( V = \) lane volume, vehicles per hour

Although both the study by Kell and the study by Greco and Sword resulted in estimating relationships between the parameters of the composite exponential distribution and traffic volume, Kell's results seem to be more realistic in the sense that the average headways of restrained vehicles are not constant for all traffic volumes. Besides, all tests of goodness-of-fit in this study indicate that the theoretical curves from Kell's study can satisfactorily represent the observed headway distributions in this study.
APPENDIX C

LISTINGS OF SIMULATION MODELS OF TWO- AND THREE-INTERSECTION SYSTEMS
During the development of the simulation model, some easily identifiable names for the block, facility, and queue entities of the model were developed. Because of this feature, it is easy to supplement the program with more traffic lanes or even opposite direction traffic lanes. The following is a list of some of the names used in the model:

- **GRN11** = effective green period for Phase 1 at Intersection 1.
- **GRN21** = effective green period for Phase 2 at Intersection 2.
- **GRN31** = effective green period for Phase 3 at Intersection 3.
- **OFF12** = offset at Intersection 2 with respect to Intersection 1.
- **CAP12** = storing capacity for a lane between Intersections 1 and 2.
- **CWAL1** = storing capacity of the left-turn pocket lane for west approach at Intersection 1.
- **CWAL2** = storing capacity of the left-turn pocket lane for west approach at Intersection 2.
- **LWN1** = percentage of left-turning traffic from west approach to northbound at Intersection 1.
- **LES1** = percentage of left-turning traffic from east approach to southbound at Intersection 1.
- **RWS1** = percentage of right-turning traffic from west approach to southbound at Intersection 1.
- **RWS2** = percentage of right-turning traffic from west approach to southbound at Intersection 2.
- **AWAll** = arrival function for the traffic on the median lane, west approach at Intersection 1.
- **AWA21** = arrival function for the traffic on the shoulder lane (2), west approach (WA) at Intersection 1.
- **ANA11** = arrival function for the traffic on the median lane, north approach (NA) at Intersection 1.
- **ASA21** = arrival function for the traffic on the shoulder lane, south approach (SA) at Intersection 1.
- **MAXM1** = mean running time for section 1 (from Intersection 1 to Intersection 2) on the median lane (M).
- **MAXS1** = mean running time for Section 1 on the shoulder lane (S).
- **SAXM1** = standard deviation of running time for Section 1 on the median lane.
- **SAXS1** = standard deviation of running time for Section 1 on the shoulder lane.
DIS12 = signal spacing or distance between Intersection 1 and 2 in feet (from stop line to stop line).

DIS23 = signal spacing or distance between Intersection 2 and 3 in feet (from stop line to stop line).

The other entity names are not included in the above list because the comment on each card is sufficient in explaining each name. However, it should be noted that the unit of time used in this model is 0.1 second. For example, if the mean running time between Intersections 1 and 2 (Section 1) on the median lane is 35.9 seconds then MAXM1 will take a value of 359. This applies to all input dealing with signal splits, and means and standard deviations of running times, but not to the arrival functions. The unit of time for the arrival functions is second not 0.1 second. If the arrival function has been prepared on the unit 0.1 second, then a slight change is required on the GENERATE block of all traffic generators. For example,

"GENERATE 10, FN$AWA11,,10,F" must be changed to "GENERATE 1, FN$AWA11,,10,F" and so on.

Besides the names listed, all other dummy names of block entities can usually be identified in a similar fashion. That is, the second and third letters indicate the approach from which that traffic has been generated. There exist some exceptions. For example, the name of the QUEUE block for the west approach is QUE11 rather than QWA11, which has been used as the name of the table entity for queue length of the corresponding queue entity.
* SIMULATION MODEL OF MULTI-SIGNAL SYSTEM (TWO INTERSECTIONS)
* BASIC TIME UNIT IS 0.1 SECOND

* INITIAL OR INPUT CARDS

* CYCLE SPLIT: EFFECTIVE GREEN
  INITIAL XH$GRN11,400/XH$GRN21,220/XH$GRN31,160 INT1
  INITIAL XH$GRN12,400/XH$GRN22,220/XH$GRN32,160 INT2

* SIGNAL OFFSETS AT DOWNSTREAM SIGNALS
  INITIAL XH$OFF1,000/XH$OFF2,300

* PERCENTAGE OF TRUCKS (MAIN STREAM TRAFFIC)
  INITIAL XH$TUKM1,05/XH$TUKS1,07 MEDIAN: SHOULDER = 5% : 7%

* PERCENTAGE OF RIGHT-TURN TRAFFIC
  INITIAL XH$RWS1,10/XH$RWS2,15 RIGHT TURN 10% AND 15%

* PERCENTAGE OF LEFT-TURN TRAFFIC
  INITIAL XH$LWN1,06/XH$LWN2,09 LEFT TURN 6% AND 9%

* MEAN AND STANDARD DEVIATION OF RUNNING TIME (STOP LINE TO STOP LINE)
  INITIAL XH$MAX1,692/XH$MAXS1,744/XH$SAX1,063/XH$SAXS1,074 5000 FT.

* CAPACITY OF LANE FOR STORING VEHICLES
  INITIAL XH$CAP12,205/XH$CAP22,163 CAPACITIES 5000 AND 4000 FT.
  INITIAL XH$CWAL1,6/XH$CWAL2,16

* DISTANCE BETWEEN SIGNALIZED INTERSECTIONS
  INITIAL XH$DIS12,5000 DISTANCE BETWEEN INT1 AND INT2 IS 5000 FT.
FACT1 FUNCTION P7,D2  EQUIVALENCY FOR STRAIGHT-THRU VEHICLE
1,100/2,163
*
FACT2 FUNCTION P7,D2  EQUIVALENCY FOR RIGHT-TURN VEHICLE
1,120/2,180
*
FACT3 FUNCTION P7,D2  EQUIVALENCY FOR RUNNING TIME
1,160/2,103
*
START FUNCTION P10,D5  STARTING HEADWAYS
1,23/2,23/3,22/4,21/5,20
*

******************************************************************************************
****************************************************************************************** VARIABLE DEFINITION CARDS  ******************************************************************************************
******************************************************************************************

FLOWS EVARIABLE FN$START*FN$FACT1/100
FLOWR EVARIABLE FN$START*FN$FACT2/100
START EVARIABLE  OG*FN$FACT1/100  ACCELERATION TIME, STRAIGHT
RIGHT EVARIABLE  OG*FN$FACT2/100  ACCELERATION TIME, RIGHT
RUNM1 EVARIABLE (XH$MAXM1+XH$MAXM1*FN$TNORM)*FN$FACT3/100
RUNM2 EVARIABLE (XH$MAXM1+XH$MAXM1*FN$TNORM)*FN$FACT3/100
VEL12 EVARIABLE XH$DIS12/P8  AVG. RUNNING SPEED (INT1 TO INT2)
RUS12 EVARIABLE P8-105/V$VEL12  RUNNING TIME (195 FT. TO INT2)
BSAR1 EVARIABLE LR$PHS21*P9'E'O*P5'LE'1
BWAP1 EVARIABLE LR$PHS11*P9'E'O*P5'LE'1
BWAR2 EVARIABLE LR$PHS12*P9'E'O*P5'LE'1
GAPS1 EVARIABLE FNU$ISAS1*W$SSS21'E'O
HEADWAY EVARIABLE P2-P2
LEFT1 EVARIABLE P5-XH$XWAM1  POSITION ID EX FOR LEFT-TURN AT INT1
LEFT2 EVARIABLE P5-XH$XWAM2  POSITION ID EX FOR LEFT-TURN AT INT2

******************************************************************************************
****************************************************************************************** SIGNAL MODEL SEGMENT  ******************************************************************************************
******************************************************************************************
* * * COMMENT FOR SIGNAL 1 CAN ALSO BE APPLIED TO OTHER SIGNALS.
* *

**GENERATE** "\$XH\$OFF1,1,3,F" SIGNAL AT INT1

**SIGN1**

**LOGIC S**

PHS11 SIGNAL TURNS GREEN (PHASE 1)

**MARK** 1 RECORD TIME WHEN PHASE 1 IS GREEN IN P1

**SAVE VALUE**

HWAM1,P1

HWAS1,P1

HEAM1,P1

HEAS1,P1

**ADVANCE** "\$XH\$CRN11" EFFECTIVE GREEN OF PHASE 1 AT INT1

**LOGIC R**

PHS11 SIGNAL TURNS AMBER AND RED (PHASE 1)

**SAVE VALUE**

XWAM1,K1,H SET VEH. POSITION INDEX FOR WAM1 TO 1

XWAS1,K1,H SET VEH. POSITION INDEX FOR WAS1 TO 1

XEAM1,K1,H SET VEH. POSITION INDEX FOR EAM1 TO 1

XEAS1,K1,H SET VEH. POSITION INDEX FOR EAS1 TO 1

**ADVANCE** 40 EFFECTIVE AMBER PERIOD 4.0 SECONDS

**TABULATE** QNA11 AND QSA21 HERE, IF DESIRED

**LOGIC S**

PHS21 SIGNAL IS IN PHASE 2

**MARK** 2 RECORD TIME WHEN PHASE 2 IS GREEN IN P2

**SAVE VALUE**

HNAM1,P2

HNAS1,P2

HSAM1,P2

HSAS1,P2

**ADVANCE** "\$XH\$GRN21" EFFECTIVE GREEN PHASE 2 AT INT1

**LOGIC R**

PHS21 SIGNAL TURNS AMBER AND RED (PHASE 2)

**SAVE VALUE**

XNAM1,K1,H SET VEH. POSITION INDEX FOR NAM1 TO 1

XNAS1,K1,H SET VEH. POSITION INDEX FOR NAS1 TO 1

XSAM1,K1,H SET VEH. POSITION INDEX FOR SAM1 TO 1

XSAS1,K1,H SET VEH. POSITION INDEX FOR SAS1 TO 1

**ADVANCE** 40 EFFECTIVE AMBER PERIOD 4.0 SECONDS

**LOGIC S**

PHS31 SIGNAL AT INT1 IS IN PHASE 3

**ADVANCE** "\$XH\$GRN31" EFFECTIVE GREEN FOR PHASE 3

**LOGIC R**

PHS31 SIGNAL TURNS AMBER AND RED (PHASE 3)

**SAVE VALUE**

XWAL1,K1,H SET VEH. POSITION INDEX FOR WAL1 TO 1
SAVEVALUE XEAL1,K1,H  SET VEH. POSITION INDEX FOR EAL1 TO 1
ADVANCE  40     EFFECTIVE AMBER PERIOD 4.0 SECONDS
TABULATE QWA11   COLLECT QUEUE LENGTH STATISTICS, QWAM1
TABULATE QWA21   COLLECT QUEUE LENGTH STATISTICS, QWAS1
TRANSFER ,SIGN1  REPEAT THE SIGNAL OPERATION AT INT1

*****************************************************************************
*****************************************************************************

GENERATE ,,XH$OFF2,1,,3,F  SIGNAL AT INT2
SIGN2 LOGIC S  PHS12
MARK  1
SAVEVALUE HWAM2,P1
SAVEVALUE HWAS2,P1
SAVEVALUE HEAM2,P1
SAVEVALUE HEAS2,P1
ADVANCE XH$GRN12
LOGIC R  PHS12
SAVEVALUE XHAM2,K1,H
SAVEVALUE XHAS2,K1,H
SAVEVALUE XEAM2,K1,H
SAVEVALUE XEAS2,K1,H
ADVANCE  40
MARK  2
LOGIC S  PHS22
SAVEVALUE HNAM2,P2
SAVEVALUE HNAS2,P2
SAVEVALUE HSAM2,P2
SAVEVALUE HSAS2,P2
ADVANCE XH$GRN22
LOGIC R  PHS22
SAVEVALUE XNAM2,K1,H
SAVEVALUE XNAS2,K1,H
SAVEVALUE XSAM2,K1,H
SAVEVALUE XSAS2,K1,H
ADVANCE  40
MARK  3
**LOGIC S**

**PHS32**

**ADVANCE**

**XHSGRN32**

**LOGIC R**

**PHS32**

**SAVEVALUE**

**XWAL2,K1,H**

**SAVEVALUE**

**XEAL2,K1,H**

**ADVANCE**

**4D**

**TABULATE**

**QWA12**

**TABULATE**

**QWA22**

**TRANSFER**

**,SIGN2**

*--------------------------------------------------------------------------------------------------------------------------*

**-------------------------------------------------------------------------------------------------------------------------- TRAFFIC MODEL SEGMENT -----------------------------------------------------------------------------**

*--------------------------------------------------------------------------------------------------------------------------*

**GENERATE**

10,FNSWAWA11,,,1C,F ARRIVAL ON MEDIAN LANE

**ASSIGN**

4,7 ASSIGN RANDOM NUMBER STREAM

**ASSIGN**

5,Q1QWAM1 RECORD QUEUE LENGTH AT INT1 ON QWAM1

**TRANSFER**

.XH1TKM1,,TRUK1 CHECK IF IT IS A TRUCK

**ASSIGN**

7,K1 PASSENGER CARS ARE CODED AS 1

**TRANSFER**

,QUE1

**TRUK1 ASSIGN**

7,K2 TRUCKS ARE CODED AS 2

**QUEUE**

QWAM1 JOIN THE QUEUE AHEAD, QWAM1

**TRANSFER**

.XH1LWN1,,LWAW1 CHECK IF IT IS A LEFT-TURNER

**ASS11 ASSIGN**

8,W$DWA11 RECORD NO. OF VEH. IN BLOCK DWA11 IN P8

**ASSIGN**

8,W$ZWA11 ADD NO. OF VEH. IN BLOCK ZWA11 TO P8

**ASSIGN**

8,W$GWA11 ADD NO. OF VEH. IN BLOCK GWA11 TO P8

**ASSIGN**

8,W$WHA11 ADD NO. OF VEH. IN BLOCK WWA11 TO P8

**ASSIGN**

8,W$MWA11 ADD NO. OF VEH. IN BLOCK MWA11 TO P8

**ASSIGN**

8,Q$QWAM2 ADD NO. OF VEH. IN QWAM2 TO P8

**TEST L**

PR,XHSCAP12,ASS11 CHECK IF QUEUE WILL BLOCK INT1

**GATE LS**

PHS11 CHECK IF SIGNAL IS GREEN

**SEIZE**

IWAM1 IF GREEN, OCCUPY THE INTERSECTION INT1

**ASSIGN**

10,XH$UXWAM1 ASSIGN NO. CORRESPONDING TO ITS POS.

**TRANSFER**

SIM,FWA11,DWA11 CHECK IF THERE IS ANY VEH. AHEAD

**DWA11 ADVANCE**

V$FLWS IF YES, SUBJECTED TO STARTING DELAY

**DEPART**

QWAM1 LEAVE QUEUE QWAM1
SAVEVALUE XWAM1+,K1,H UPDATE THE VEH-POSITION INDEX
RELEASE IWAM1 LEAVE INTERSECTION INT1
MARK 2 RECORD VEH. ARRIVING TIME IN P2
TRANSFER ,YWA11 GO TO ACC. ROUTINE, START AT YWA11
FWA11 DEPART QWAM1 VEH. WITHOUT START DELAY LEAVE QWAM1
RELEASE IWAM1 AND PASSES INTERSECTION INT1
ZWA11 ADVANCE VERUMM1 TRAVEL TO INT2 WITH RANDOM RUNNING TIME
TRANSFER ,CHN11 GO TO SAME ROUTINE AS IT ARRIVES INT1
YWA11 TEST E P10,K1,UWA11 CHECK IF IT IS A LEADING VEH.
VWA11 SAVEVALUE ACCM1,V$STAIT,H IF YES, RECORD THE ACC. TIME
GWA11 ADVANCE XH$ACCM1 ACC. UPTO A DISTANCE OF 195 FT.
SAVEVALUE HWAM1+,V$FLOWS RECORD LEAVING TIME OF EACH VEH.
ASSIGN $,V$RUMM1
TRANSFER ,MWA11 GO TO ROUTINE OF TRAVEL, 195 FT. TO STOPLINE
UWA11 ASSIGN 3,XH$WAM1 RECORD LEAVING TIME OF PRECEDING VEH.
TEST E P7,K1,VWA11 CHECK IF IT IS A PASSENGER CAR
TEST LE V3H$WAY,50,VWA11 IF YES, TEST IF HDWY IS LE 5.0 SEC.
WFA11 ADVANCE XH$ACCM1 IF YES, ITS ACC. IS SAME AS PRECEDING VEH.
SAVEVALUE HWAM1+,V$FLOWS UPDATE RECORD OF LEAVING TIME
MFA11 ADVANCE V$RUS12 TRAVEL FROM 195-FOOT DIST. TO INT2
* TRANSFER .12,CHN21 12 % CHANGE TO THE SHOULDER LANE
CHN11 ASSIGN 5,Q1,QWAM2 JOIN QUEUE AT INT2, QWAM2
QUEUE QWAM2
TRANSFER .XH$LNN?,LWA12
ASS12 ASSIGN 8,0,$DWA12
ASSIGN 8+,0,0,WAM3
TEST L P6,XH$CAP23,ASS12
GATE LS PHS12
SEIZE IWAM2
ASSIGN 10,XH$XWAM2
TRANSFER SIM,FWA12,DWA12
DWA12 ADVANCE V$FLOWS
DEPART QWAM2
SAVEVALUE XWAM2+,K1,H
RELEASE IWAM2
TRANSFER ,TRN11
FWA12 DEPART QWAM2
RELEASE IWAM2
TRN11 TABULATE TWM12
TERMINATE

*
<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<
*
GENERATE 10, FN$AWA21,,10,F ARRIVAL ON SHOULDER LANE
ASSIGN 4,8
ASSIGN 5,0$QWA1
TRANSFER *XH$TKS1,,TRUK2 CHECK IF THIS IS A TRUCK
ASSIGN 7,K1
TRANSFER *QUE2
TRUK2 ASSIGN 7,K2
QUE2 QUEUE QWA1
TRANSFER *XH$RS1,SWA21,RWA21
RWA21 ASSIGN 9,K0 RIGHT-TURN VEHICLE
TRANSFER *CHK21
SWA21 ASSIGN 9,K1 STRAIGHT-THRU, VEHICLE
ASS21 ASSIGN 8,WS$AWA21
ASSIGN 8+,WSGWA21
ASSIGN 8+,W$WFA21
ASSIGN 8+,WMBWAS1
ASSIGN 8+,W$ZWA21
ASSIGN 8+,WS$WAS2
CHK21 TEST E BV$Wavar1,K1,KWA21 CHECK FOR CONTINUOUS RIGHT TURN
SEIZE IWAS1 READY TO TURN RIGHT IF POSSIBLE
TWA21 GATF NU INAS1
GATF NU IEAL1
TRANSFER SIM,PWA21,TWA21
PWA21 ASSIGN 10, XH$XWAS1
ADVANCE V$FLOWR
DEPART QWA1
GATE LS PHS11,NWA21
SAVEVALUE XWAS1+,K1,H
NWA21 RELEASE IWAS1
TERMINATE

KWA21 TEST L P8, XH$CAP12, ASS11 RIGHT-TURN TRAFFIC AT INT1
GATE LS PHS11
SEIZE IWAS1
ASSIGN 10, XH$XWAS1
TEST F P9, K0, JWA21
TRANSFER S17, FWAR1, DWA1

DWAR1 ADVANCE VW$FLOW1
SAVE VALUE XWAS1+, K1, H
FWAR1 DEPART QWAS1
RELEASE IWAS1
TERMINATE
JWA21 TRANSFER S17, FWA21, DWA21 STRAIGHT-THRU TRAFFIC AT INT1
DWA21 ADVANCE VW$FLOW1
SAVE VALUE XWAS1+, K1, H
DEPART QWAS1
RELEASE IWAS1
MARK 2
TRANSFER , YWA21
FWA21 DEPART QWAS1
RELEASE IWAS1
ZWA21 ADVANCE VW$RUMS1
TRANSFER , CHN21
YWA21 TEST E P10, K1, UWA21
VWA21 SAVE VALUE ACS11, VW$STAI1, H
GWA21 ADVANCE XH$ACC1
SAVE VALUE HWAS1+, VW$FLOW1
ASSIGN & , VW$RUMS1
TRANSFER , MWA21
UWA21 ASSIGN 3, XH$WAS1
TFST F P7, K1, VWA21
TEST LE VW$HIGHWAY, 50, VWA21 IF HIGHWAY LF 5.0 SEC., ACC. IND.
WWA21 ADVANCE XH$ACC1
SAVE VALUE HWAS1+, VW$FLOW1
MWA21 ADVANCE VW$RUMS12
* TRANSFER 10, CHN11 10% CHANGE TO MEDIAN LANE
CHN21 ASSIGN 5,Q$QWAS2
QUEUE QWAS2
TRANSFER $XH$+RWS2,SWA22,RWA22

RWA22 ASSIGN 9,K0
TRANSFER ,CHK22

SWA22 ASSIGN 9,K1

ASSIGN 8+WIDWA22
ASSIGN 6+,Q10WAS3

CHK22 TEST E BV16WAR2,K1,KWA22 CHECK FOR CONTINUOUS RIGHT TURN
SEIZE IWAS2 READY TO TURN RIGHT IF POSSIBLE

TWA22 GATE NU INAS2
GATE NU IREAL2
TRANSFER SIM,PWA22,TWA22

PWA22 ASSIGN 10,XH$XWAS2
ADVANCE VSPFLOWR
DEPART QWAS2
GATE LS PHS12,NWA22
SAVEVALUE XWAS2+,K1,H

NWA22 RELEASE IWAS2
TRANSFER ,TPN21

KWA22 TEST L PB,XH$CAP23,ASS22 RIGHT-TURN TRAFFIC AT INT2
GATE LS PHS12
SEIZE IWAS2
ASSIGN 10,XH$XWAS2
TEST E P9,K0,JWA22
TRANSFER SIM,FWAR2,DWAR2

DWAR2 ADVANCE VSPFLOWR
SAVEVALUE XWAS2+,K1,H

FWAR2 DEPART QWAS2
RELEASE IWAS2
TRANSFER ,TPN21

JWA22 TRANSFER SIM,FWAR2,DWAR2 STRAIGHT-THRU TRAFFIC AT INT2

DWAR2 ADVANCE VSPFLOWS
SAVEVALUE XWAS2+,K1,H
DEPART QWAS2
RELEASE IWAS2
TRANSFER TRN21
FWA22 DEPART QWAS2
RELEASE IWAS2
TRN21 TABULATE TWS12
TERMINATE

* ***********************************************
*** TRAFFIC FROM MINOR ROAD  *******************************************
* GENERATE 10,FNSANA11,,,,10,F NORTH APPROACH, MEDIAN LANE
ASSIGN 4,7
ASSIGN 7,K1
QUEUE QNAM1
ASSNA ASSIGN 8,W1DNA11
ASSIGN 8+,W1DNA11
ASSIGN 8+,W1GNA11
ASSIGN 8+,W1SNA11
ASSIGN 8+,W1SWA11
ASSIGN 8+,W1MWA11
ASSIGN 8+,W1ZWA11
ASSIGN 8+,Q1QWAM2
TEST L P8,XHLCAP12,ASSNA
GATE LS PHS21
SFIZE INAM1
ASSIGN 10,XH+XNAM1
TRANSFER SIM,FNA11,DNA11
DNA11 ADVANCE V1FLOWs
SAVEVALUE XNAM1+,K1,H
TFSTE BVSGAPS1,K1
DFPART QNAM1
RELEASE INAM1
MARK 2
TRANSFER YNA11
FNA11 DEPART QNAM1
RELEASE INAM1
TRANSFER, ZWA11
YNA11 TEST E P10, K1, UNA11
VNA11 SAVEVALUE ACSM1, VS$RIGHT, H
GNA11 ADVANCE XH$ACSM1
SAVEVALUE HNAM1+, VS$FLOWS
TRANSFER, MWA11
UNA11 ASSIGN 3, XH$HNAM1
TEST F P7, K1, VNA11
TEST L VS$HDWAY, 40, VNA11
WNA11 ADVANCE XH$ACSM1
SAVEVALUE HNAM1+, VS$FLOWS
TRANSFER, MWA11

*******************************************************************************

GENERATE 10, FN$ASA21, 1, 10, F SOUTH APPROACH, SHOULDER LANE
ASSIGN 4, 8
ASSIGN 5, O$QSAS1
ASSIGN 7, K1
TRANSFER .55, SSA21, RSA21 55% OF TRAFFIC TURN RIGHT
RSA21 ASSIGN 9, K0
TRANSFER, SSS21
SSA21 ASSIGN 9, K1
SSS21 ADVANCE 45
QUEUE QSAS1
ASSSA ASSIGN 8, W$GSA21
ASSIGN 8+, W$GWA21
ASSIGN 8+, W$GSA21
ASSIGN 8+, W$GWA21
ASSIGN 8+, W$WSA21
ASSIGN 8+, W$WMA21
ASSIGN 8+, W$ZWA21
ASSIGN 8+, O$QWAS2
TEST E BV$SAR1, K1, KSA21 START CONTINUOUS RIGHT-TURN ROUTINE
SEIZE ISAS1
TSA21 GATE NU IWM1
GATE NU I WAS1
TRANSFER SIM, PSA21, TSA21
PSA21 TEST L P8, XH$CAP17, ASSSA
ASSIGN 10, XH$XSAS1
ADVANCE V$FLOWR
DEPART QSAS1
RELEASE ISAS1
MARK 2
TRANSFER , YSA21
KSA21 GATE LS PHS21
SIZE ISAS1
ASSIGN 10, XH$XSAS1
TEST E P9, K1, JSA21
TRANSFER SIM, FSA21, ESA21
DSA21 ADVANCE V$FLOW
SAVEVALUE XSAS1+, K1, H
FSA21 DEPART QSAS1
RELEASE ISAS1
TERMINATE
JSA21 TRANSFER SIM, FSA21, ESA21
DSAR1 ADVANCE V$FLOWR
SAVEVALUE XSAS1+, K1, H
DEPART QSAS1
RELEASE ISAS1
MARK 2
TRANSFER , YSA21
FSAR1 DEPART QSAS1
RELEASE ISAS1
TRANSFER , ZWA21
YSA21 TEST E P10, K1, USA21
VSA21 SAVEVALUE ACSS1, V$RIGHT, H
GSA21 ADVANCE XH$ACSS1
SAVEVALUE HSAS1+, V$FLOWR
TRANSFER , MWA21
USA21 ASSIGN 3, X1HSAS1
TEST E P7, K1, VSA21
WSA21 ADVANCE XH$ACSS1
SAVEVALUE HSAS1+,V$FLOWR
TRANSFER ,MWA21

*  ******************************************************************************
***************  LEFT TURN FROM THE MAIN STREAM TRAFFIC  ***************
*   ******************************************************************************
*  LWA11 TEST L  Q$QWAL1,XH$CWAL1 CHECK IF QWAL1 IS LE STORING CAPACITY
TEST L  V$LEFT1,XH$CWAL1 CHECK IF VEH. AHEAD BLOCK THE TAPER
DEPART QWAL1
QUEUE QWAL1
GATE LS PHS31
SEIZE IWAL1
TRANSFER SIM,FWA01,DWA01
DWA01 ADVANCE V$FLOWS
SAVEVALUE XWAL1+,K1,H
FWA01 DEPART QWAL1
RELEASE IWAL1
TERMINATE

*  ******************************************************************************
*   ******************************************************************************
*  LWA12 TEST L  Q$QWAL2,XH$CWAL2 CHECK IF QWAL2 IS LE STORING CAPACITY
TEST L  V$LEFT2,XH$CWAL2 CHECK IF VEH. AHEAD BLOCK THE TAPER
DEPART QWAL2
QUEUE QWAL2
GATE LS PHS32
SEIZE IWAL2
TRANSFER SIM,FWA02,DWA02
DWA02 ADVANCE V$FLOWS
SAVEVALUE XWAL2+,K1,H
FWA02 DEPART QWAL2
RELEASE IWAL2
TERMINATE
TABLE DEFINITION CARDS

QWA11 TABLE Q$QWAM1,0,1,25 QUEUE LENGTH STATISTICS, QWAM1
DWA11 QTABLE QWAM1,0,50,20 DELAY STATISTICS, QWAM1
QWA21 TABLE Q$QWAS1,0,1,25 QUEUE LENGTH STATISTICS, QWAS1
DWA21 QTABLE QWAS1,0,50,20 DELAY STATISTICS, QWAS1
QWA12 TABLE Q$QWAM2,0,1,25 QUEUE LENGTH STATISTICS, QWAM2
DWA12 QTABLE QWAM2,0,50,20 DELAY STATISTICS, QWAM2
QWA22 TABLE Q$QWAS2,0,1,25 QUEUE LENGTH STATISTICS, QWAS2
DWA22 QTABLE QWAS2,0,50,20 DELAY STATISTICS, QWAS2
TWM12 TABLE M1,100,100,30 TRAVEL TIME DISTRIBUTION, INT1 TO INT2
TWS12 TABLE M1,100,100,30 TRAVEL TIME DISTRIBUTION, INT1 TO INT2

MODEL CONTROL SEGMENT

GENERATE ,,,1
TEN ADVANCE 9000
SPLIT 1,TERM
TRANSFER ,TEN
TERM TERMINATE 1
START 1,NP
RESET
START 8
END

9000 TIME UNITS ARE EQUIVALENT TO 15 MINUTES
EXCLUDE THE FIRST FIFTEEN MINUTES OF DATA
SIMULATE

* SIMULATION MODEL OF MULTI-SIGNAL SYSTEM (THREE INTERSECTIONS)

* BASIC TIME UNIT IS 0.1 SECOND

*****************************************************************************
* INITIAL OR INPUT CARDS *****************************************************************************
*****************************************************************************

* CYCLE SPLIT: EFFECTIVE GREEN

INITIAL  XH$GRN1,400/XH$GRN2,220/XH$GRN3,160  INT1
INITIAL  XH$GRN1,380/XH$GRN2,230/XH$GRN3,170  INT2
INITIAL  XH$GRN1,450/XH$GRN2,180/XH$GRN3,150  INT3

* SIGNAL OFFSETS AT DOWNSTREAM SIGNALS

INITIAL  XH$OFF1,000  OFFSET AT INT1 TO INT1 IS 00.0 SEC.
INITIAL  XH$OFF2,300  OFFSET AT INT2 TO INT1 IS 30.0 SEC.
INITIAL  XH$OFF3,505  OFFSET AT INT3 TO INT1 IS 50.5 SEC.

* PERCENTAGE OF TRUCKS (MAIN STREAM TRAFFIC)

INITIAL  XH$TKM1,05/XH$TKS1,07. MEDIAN:SHOULDER = 5% : 7%

* PERCENTAGE OF RIGHT-TURN TRAFFIC

INITIAL  XH$RWS1,10/XH$RWS2,13/XH$RWS3,08

* PERCENTAGE OF LEFT-TURN TRAFFIC

INITIAL  XH$LWN1,06/XH$LWN2,09/XH$LWN3,12

* MEAN AND STANDARD DEVIATION OF RUNNING TIME (STOP LINE TO STOP LINE)

INITIAL  XH$MAXM1,692/XH$MAXS1,744/XH$SAXM1,063/XH$SAXS1,074  5000 FT.
INITIAL  XH$MAXM2,896/XH$MAXS2,954/XH$SAXM2,078/XH$SAXS2,085  6500 FT.

* CAPACITY OF LANE FOR STORING VEHICLES

INITIAL  XH$CAP12,205/XH$CAP23,268/XH$CAP34,310
INITIAL XH$CWAL1,6/XH$CWAL2,16/XH$CWAL3,08

* DISTANCE BETWEEN SIGNALIZED INTERSECTIONS
  INITIAL XH$DIS12,5000 DISTANCE BETWEEN INT1 AND INT2 IS 5000 FT.
  INITIAL XH$DIS23,6500 DISTANCE BETWEEN INT2 AND INT3 IS 6500 FT.

* *

******************************************************************************************
******************************************************************************************
FUNCTION DEFINITION CARDS  *****************************************************************
******************************************************************************************
******************************************************************************************

T: NORM FUNCTION RN4,C33 STANDARD TRUNCATED NORMAL DISTRIBUTION
0.025,-1.96/03,-1.681/04,-1.751/05,-1.645/06,-1.555/07,-1.476/
08,-1.405/09,-1.341/10,-1.282/15,-1.026/20,-0.842/25,-0.674/
30,-0.524/35,-0.385/40,-0.253/45,-0.126/50,0.000/55,0.126/
60,0.253/65,0.385/70,0.524/75,0.674/80,0.842/85,1.036/90,1.282/
91,1.341/92,1.405/93,1.476/94,1.555/95,1.645/96,1.751/97,1.881/
975,1.96

AWA11 FUNCTION RN1,C18 WA SHOULDER VOLUME 650 VPH.
0000,0.9/2472.0/3961.3/5263.4/6202.5/6901.6/7438.7/7861.8/8202.9/8481.10/8906.12/9323.15/
9693.20/9860.25/.9971.35/.9990.42/.9999.57/.1000.76

AWA21 FUNCTION RN2,C24 VOLUME OF 600 VPH.
0660,0.9/1967.2/3739.3/4995.4/5916.5/6613.6/7158.7/7594.8/7949.9/8245.10/8493.11/8763.12/
9882.13/9935.14/9980.16/9962.18/.9957.20/9805.25/
9906.30/9954.35/.9978.40/.9989.45/.9995.25/.1000.83.0

FACT1 FUNCTION P7,D2 EQUIVALENCY FOR STRAIGHT-THRU VEHICLE
1,100/2,163

FACT2 FUNCTION P7,D2 EQUIVALENCY FOR RIGHT-TURN VEHICLE
1,120/2,180
FACT3 FUNCTION  P7,D2  EQUIVALENCY FOR RUNNING TIME
1,100/2,103
*
START FUNCTION  P10,D5  STARTING HEADWAYS
1,23/2,23/3,22/4,21/5,20
*

************************************************************************* VARIABLE DEFINITION CARDS *************************************************************************

* FLOWS FVARIABLE FN$START*FN$FACT1/100
FLOWR FVARIABLE FN$START*FN$FACT2/100
START FVARIABLE 90*FN$FACT1/100 ACCELERATION TIME, STRAIGHT
RIGHT FVARIABLE 90*FN$FACT2/100 ACCELERATION TIME, RIGHT
RUMM1 FVARIABLE (XHSMAXM1+XHSAXM1*FN$TNRM)*FN$FACT3/100
RUMM2 FVARIABLE (XHSMAXM2+XHSAXM2*FN$TNRM)*FN$FACT3/100
RUMS1 FVARIABLE (XHSMAXS1+XHSAXS1*FN$TNRM)*FN$FACT3/100
RUMS2 FVARIABLE (XHSMAXS2+XHSAXS2*FN$TNRM)*FN$FACT3/100
VEL12 FVARIABLE XHS DIS12/P8 AVG. RUNNING SPEED (INT1 TO INT2)
VEL23 FVARIABLE XHS DIS23/P8 AVG. RUNNING SPEED (INT2 TO INT3)
RUS12 FVARIABLE P8-195/VVEL12 RUNNING TIME (195 FT. TO INT2)
RUS23 FVARIABLE P8-195/VVEL23 RUNNING TIME (195 FT. TO INT3)
BSAR1 BVARIABLE LR$PHS21*P$E1*P5*LE1
BSAR2 BVARIABLE LR$PHS11*P$E1*P5*LE1
BSAR3 BVARIABLE LR$PHS12*P$E1*P5*LE1
NOWAY VARIABLE P2-P3
LEFT1 VARIABLE P5-XH$XWAM1 POSITION IDEX FOR LEFT-TURN AT INT1
LEFT2 VARIABLE P5-XH$XWAM2 POSITION IDEX FOR LEFT-TURN AT INT2
LEFT3 VARIABLE P5-XH$XWAM3 POSITION IDEX FOR LEFT-TURN AT INT3
*
************************************************************************* SIGNAL MODEL SEGMENT*************************************************************************

*************************************************************************
* COMMENT FOR SIGNAL 1 CAN ALSO BE APPLIED TO OTHER SIGNALS.

* GENERATE "XH1OFF1,1,3,F SIGNAL AT INT1

SIGN1 LOGIC S PHSI1 SIGNAL TURNS GREEN (PHASE 1)
MARK 1 RECORD TIME WHEN PHASE 1 IS GREEN IN P1
SAVEVALUE HNAM1,P1
SAVEVALUE HWAS1,P1
SAVEVALUE HEAM1,P1
SAVEVALUE HEAS1,P1
ADVANCE XH$GRN11 EFFECTIVE GREEN OF PHASE 1 AT INT1
LOGIC R PHS11 SIGNAL TURNS AMBER AND RED (PHASE 1)
SAVEVALUE XHAM1,K1,H SET VEH. POSITION INDEX FOR WAM1 TO 1
SAVEVALUE XWAS1,K1,H SET VEH. POSITION INDEX FOR WAS1 TO 1
SAVEVALUE XEAM1,K1,H SET VEH. POSITION INDEX FOR EAM1 TO 1
SAVEVALUE XEAS1,K1,H SET VEH. POSITION INDEX FOR EAS1 TO 1
ADVANCE 40 EFFECTIVE AMBER PERIOD 4.0 SECONDS

* TABULATE QNA11 AND QSA21 HERE, IF DESIRED
LOGIC S PHS21 SIGNAL IS IN PHASE 2
MARK 2 RECORD TIME WHEN PHASE 2 IS GREEN IN P2
SAVEVALUE HNAM1,P2
SAVEVALUE HWAS1,P2
SAVEVALUE HSAM1,P2
SAVEVALUE HSAS1,P2
ADVANCE XH$GRN21 EFFECTIVE GREEN PHASE 2 AT INT1
LOGIC R PHS21 SIGNAL TURNS AMBER AND RED (PHASE 2)
SAVEVALUE XNAM1,K1,H SET VEH. POSITION INDEX FOR NAM1 TO 1
SAVEVALUE XNAS1,K1,H SET VEH. POSITION INDEX FOR NAS1 TO 1
SAVEVALUE XSAM1,K1,H SET VEH. POSITION INDEX FOR SAM1 TO 1
SAVEVALUE XSAS1,K1,H SET VEH. POSITION INDEX FOR SAS1 TO 1
ADVANCE 40 EFFECTIVE AMBER PERIOD 4.0 SECONDS
LOGIC S PHS31 SIGNAL AT INT1 IS IN PHASE 3
ADVANCE XH$GRN31 EFFECTIVE GREEN FOR PHASE 3
LOGIC R PHS31 SIGNAL TURNS AMBER AND RED (PHASE 3)
SAVEVALUE XHAL1,K1,H SET VEH. POSITION INDEX FOR HALL1 TO 1
SAVEVALUE XEAL1,K1,H SET VEH. POSITION INDEX FOR EAL1 TO 1
ADVANCE 40  EFFECTIVE AMBER PERIOD 4.0 SECONDS
TABULATE QWA11  COLLECT QUEUE LENGTH STATISTICS, QWAM1
TABULATE QWA21  COLLECT QUEUE LENGTH STATISTICS, QWAS1
TRANSFER ,SIGN1  REPEAT THE SIGNAL OPERATION AT INT1

**************************************************************************************
*  GENERATE ,,XH$OFF2,1,,3,F  SIGNAL AT INT2
SIGN2 LOGIC S  PHS12
MARK 1
SAVEVALUE HWAM2,P1
SAVEVALUE HWAS2,P1
SAVEVALUE HEAM2,P1
SAVEVALUE HEAS2,P1
ADVANCE XH$GRN12
LOGIC R  PHS12
SAVEVALUE XWAM2,K1,H
SAVEVALUE XWAS2,K1,H
SAVEVALUE XEAM2,K1,H
SAVEVALUE XEAS2,K1,H
ADVANCE 40
MARK 2
LOGIC S  PHS22
SAVEVALUE HNAM2,P2
SAVEVALUE HNAS2,P2
SAVEVALUE HSAM2,P2
SAVEVALUE HSAS2,P2
ADVANCE XH$GRN22
LOGIC R  PHS22
SAVEVALUE XNAM2,K1,H
SAVEVALUE XNAS2,K1,H
SAVEVALUE XSAM2,K1,H
SAVEVALUE XSAS2,K1,H
ADVANCE 40
MARK 3
LOGIC S  PHS32
**ADVANCE**  XHS$GRN32
**LOGIC R**  PHS32
**SAVEVALUE**  XWAL2,K1,H
**SAVEVALUE**  XEAL2,K1,H
**ADVANCE**  40
**TABULATE**  QWA12
**TABULATE**  QWA22
**TRANSFER**  SIGN2

******************

**GENERATE**  ,,XH$OFF3,1,,3,F  SIGNAL AT INT3

**SIGN3**  LOGIC S  PHS13
**MARK**  1
**SAVEVALUE**  XWAM3,P1
**SAVEVALUE**  HWS3,P1
**SAVEVALUE**  HEAM3,P1
**SAVEVALUE**  HEAS3,P1
**ADVANCE**  XH$GPN13
**LOGIC R**  PHS13
**SAVEVALUE**  XWAM3,K1,H
**SAVEVALUE**  XWAS3,K1,H
**SAVEVALUE**  XEAM3,K1,H
**SAVEVALUE**  XEAS3,K1,H
**ADVANCE**  40
**MARK**  2
**LOGIC S**  PHS23
**SAVEVALUE**  HNAM3,P2
**SAVEVALUE**  HNAS3,P2
**SAVEVALUE**  HSAM3,P2
**SAVEVALUE**  HSAS3,P2
**ADVANCE**  XH$GRN23
**LOGIC R**  PHS23
**SAVEVALUE**  XNAM3,K1,H
**SAVEVALUE**  XNAS3,K1,H
**SAVEVALUE**  XSAM3,K1,H
**TRAFFIC MODEL SEGMENT**

*GENERATE 10,FNSAWA11,,10,F ARRIVAL ON MEDIAN LANE*
*ASSIGN 4,7 ASSIGN RANDOM NUMBER STREAM*
*ASSIGN 5,Q$QWAM1 RECORD QUEUE LENGTH AT INT1 ON QWAM1*
*TRANSFER ,XH$TUKM1,,TRUK1 CHECK IF IT IS A TRUCK*
*ASSIGN 7,K1 PASSENGER CARS ARE CODED AS 1*
*TRANSFER ,QUE1 TRUK1 ASSIGN 7,K2 TRUCKS ARE CODED AS 2*
*QUE1 QUEUE QWAM1 JOIN THE QUEUE AHEAD, QWAM1*
*TRANSFER ,XH$IWLN1,,LWA11 CHECK IF IT IS A LEFT-TURNER*
*ASSIGN 8,W1DWA11 RECORD NO. OF VEH. IN BLOCK DWA11 IN P8*
*ASSIGN 8+,W1ZWA11 ADD NO. OF VEH. IN BLOCK ZWA11 TO P8*
*ASSIGN 8+,W1GWA11 ADD NO. OF VEH. IN BLOCK GWA11 TO P8*
*ASSIGN 8+,W1HWA11 ADD NO. OF VEH. IN BLOCK WWA11 TO P8*
*ASSIGN 8+,W1MWA11 ADD NO. OF VEH. IN BLOCK MWA11 TO P8*
*ASSIGN 8+,Q$QWAM2 ADD NO. OF VEH. IN QWAM2 TO P8*
*TEST L P8,XH$CAP12,ASS11 CHECK IF QUEUE WILL BLOCK INT1*
*GATE LS PHS11 CHECK IF SIGNAL AT INT1 IS GREEN*
*SEIZE IWAM1 IF GREEN, OCCUPY INTERSECTION INT1*
*ASSIGN 10,XH$XWAM1 ASSIGN NO. CORRESPONDING TO ITS POS.*
TRANSFER SIM,FWA11,DWA11 CHECK IF THERE IS ANY VEH. AHEAD
DWA11 ADVANCE VSFLows IF YES, SUBJECTED TO STARTING DELAY
DEPART QWAM1 LEAVE QUEUE QWAM1
SAVEVALUE XWAM1+,K1,H UPDATE THE VEH-POSITION INDEX
RELEASE IWAM1 LEAVE INTERSECTION IN1
MARK 2 RECORD VEH. ARRIVING TIME IN P2
TRANSFER ,YWA11 GO TO ACC. ROUTINE, START AT YWA11
FWA11 DEPART QWAM1 VEH. WITHOUT START DELAY LEAVE QWAM1
RELEASE IWAM1 AND PASSES INTERSECTION IN1
ZWA11 ADVANCE V$RUMM1 TRAVEL TO INT2 WITH RANDOM RUNNING TIME
TRANSFER ,CHN11 GO TO SAME ROUTINE AS IT ARRIVES INT1
.YWA11 TEST E P10,K1,UWA11 CHECK IF IT IS A LEADING VEH.
VWA11 SAVEVALUE ACCM1,V$STAI1,H IF YES, RECORD THE ACC. TIME
GWA11 ADVANCE X$ACCM1 ACC. UP TO A DISTANCE OF 195 FT.
SAVEVALUE HWAM1+,V$FLows RECORD LEAVING TIME OF EACH VEH.
ASSIGN E,X$RUMM1 RECORD ITS RANDOM RUNNING TIME IN PB
TRANSFER ,MWA11 GO TO ROUTINE OF TRAVEL, 195 FT. TO STOPLINE
UWA11 ASSIGN 3,X$HWAM1 RECORD LEAVING TIME OF PRECEDING VEH.
TEST E P7,K1,VWA11 CHECK IF IT IS A PASSENGER CAR
TEST LF V$HWAY,50,VWA11 IF YES, TEST IF HWAY IS LE 5.0 SEC.
WWA11 ADVANCE X$ACCM1 IF YES, ITS ACC. IS SAME AS PRECEDING VEH.
SAVEVALUE HWAM1+,V$FLows UPDATE RECORD OF LEAVING TIME
MWA11 ADVANCE V$US12 TRAVEL FROM 195-FOOT DIST. TO INT2
TRANSFER *.12,,CHN21 12 % CHANGE TO THE SHOULDER LANE
CHN11 ASSIGN 5,Q1QWAM2 RECORD NO. OF VEH. AHEAD IN P5
QUEUE QWAM2 JOIN QUEUE AT INT2*,QWAM2
TRANSFER *.X$LWN2,,LWA12 CHECK IF IT IS LEFT TURN VEH.
ASS12 ASSIGN 8+W$1HWA12 RECORD NO. OF VEH. IN BLOCK DWA12 IN P8
ASSIGN 8+W$ZWA12 ADD NO. OF VEH. IN BLOCK ZWA12 TO PB
ASSIGN 8+W$GWAM12 ADD NO. OF VEH. IN BLOCK GWA12 TO PB
ASSIGN 8+W$HWA12 ADD NO. OF VEH. IN BLOCK WWA12 TO PB
ASSIGN 8+W$WAM12 ADD NO. OF VEH. IN BLOCK MWA12 TO PB
ASSIGN 8+W$QWAM3 ADD NO. OF VEH. IN QWAM3 TO PB
TEST L PB,X$H4CAP23,ASS12 CHECK IF QUEUE WILL BLOCK INT2
GATE LS PHS12 CHECK IF SIGNAL AT INT2 IS GREEN
SE1ZF IWAM2 IF GREEN, OCCUPY INTERSECTION INT2
ASSIGN 10, XH$XWAM2. ASSIGN NO. CORRESPONDING TO ITS POS.
TRANSFER SIM, FWA12, DWA12 CHECK IF THERE IS ANY VEH. AHEAD
DWA12 ADVANCE V$FLOWS IF YES, SUBJECTED TO STARTING DELAY
DEPART QWAM2 LEAVE QUEUE QWAM2
SAVEVALUE XMAM2+, K1, H UPDATE THE VEH-POSITION INDEX
RELEASE IWAM2 LEAVE INTERSECTION INT1
TABULATE TWM12 COLLECT TRAVELTIME DATA FOR MEDIAN LANE
MARK 2 RECORD VEH. ARRIVING TIME IN P2
TRANSFER YWA12 GO TO ACC. ROUTINE, START AT YWA12
FWA12 DEPART QWAM2 VEH. WITHOUT START DELAY LEAVE QWAM2
RELEASE IWAM2 AND PASSES INTERSECTION INT2
TABULATE TWM12
ZWA12 ADVANCE V$RUMMM2
TRANSFER CHN12
YWA12 TEST E P10, K1, UWA12
VWA12 SAVEVALUE ACCM2, V$STATIC, H
GWA12 ADVANCE XH$ACCM2
SAVEVALUE HWAM2+, V$FLOWS
ASSIGN 8, V$RUMMM2
TRANSFER , MWA12
UWA12 ASSIGN 3, XH$WAM2
TEST E P7, K1, VWA12
TEST LE V$HDWAY, 50, VWA12
WVA12 ADVANCE XH$ACCM2
SAVEVALUE HWAM2+, V$FLOWS
MWA12 ADVANCE V$RUS23
TRANSFER , UWA022
CHN12 ASSIGN 5, O$: QWAM3
QUEUE QWAM3
TRANSFER XH$JWN3, LWA13
ASS13 ASSIGN 8, W$: UWA13
ASSIGN 8+, 0$: QWAM4
TEST L P8, XH$CAP24, ASS13
GATE LS PHS13
SEIZE IWAM3
ASSIGN 10, XH$XWAM3
TRANSFER SIM,FWA13,DWA13
DWA13 ADVANCE V$FLows
DEPART QWAM3
SAVEVALUE XWAM3,K1,H
PELFASF IWAM3
TRANSFER TRN13
FWA13 DEPART QWAM3
RELEASE IWAM3
TRN13 TABULATE TWM13
TERMINATE

************************************************************************
************************************************************************
GENERATE 10,FN$AWA21,,10,F ARRIVAL ON SHOULDER LANE
ASSIGN 4,8
ASSIGN 5,QGWAS1
TRANSFER XH+TUKS1,TRUK2 CHECK IF THIS IS A TRUCK
ASSIGN 7,K1
TRANSFER QUE2
TRUK2 ASSIGN 7,K2
QUE2 CUFUE CWAS1
TRANSFER XH+RWS1,SWA21,RWA21
RWA21 ASSIGN 9,K0 RIGHT-TURN VEHICLE
TRANSFER CHK21
SWA21 ASSIGN 9,K1 STRAIGHT-THRU VEHICLE
ASS21 ASSIGN 8,W$DWA21
ASSIGN 8+,W$GWA21
ASSIGN 8+,W$WWA21
ASSIGN 8+,W$MWA21
ASSIGN 8+,W$ZWA21
ASSIGN 8+,Q$WAS2
CHK21 TEST E BV$WARI1,K1,KWA21 CHECK FOR CONTINUOUS RIGHT TURN
SEIZE IWAS1 READY TO TURN RIGHT IF POSSIBLE
TWA21 GATF NU INAS1
GATF NU IEAL1
TRANSFER SIM,PWA21,TWA21
PWA21 ASSIGN 10, XH$XWAS1
ADVANCE V$FLOWR
DEPART QWAS1
GATE LS PHS11, NWA21
SAVEVALUE XWAS1+, K1, H
NWA21 RELEASE I WAS1
TERMINATE
KWA21 TEST L P8, XH$CAP12, ASS21 RIGHT-TURN TRAFFIC AT INT1
GATE LS PHS11
SEIZE IWAS1
ASSIGN 10, XH$XWAS1
TEST E P9, K0, JWA21
TRANSFER SIM, FWAR1, DWAR1
DWAR1 ADVANCE V$FLOWR
SAVEVALUE XWAS1+, K1, H
FWAR1 DEPART QWAS1
RELEASE IWAS1
TERMINATE
JWA21 TRANSFER SIM, FWA21, DWA21 STRAIGHT-THRU TRAFFIC AT INT1
DWA21 ADVANCE V$FLOWR
SAVEVALUE XWAS1+, K1, H
DEPART QWAS1
RELEASE IWAS1
MARK 2
TRANSFER , YWA21
FWA21 DEPART QWAS1
RELEASE IWAS1
ZWA21 ADVANCE V$RUMS1
TRANSFER , CHN21
YWA21 TEST E P10, K1, UWA21
VWA21 SAVEVALUE ACCS1, V$STA1T, H
GWA21 ADVANCE XH$ACC51
SAVEVALUE HVAS1+, V$FLOWR
ASSIGN 8, V$RUMS1
TRANSFER , MWA21
UWA21 ASSIGN 3, XH$HVAS1 RECORD LEAVING TIME OF PRECEDING VEH

167
W A21
M W A 2 1
K C H N 21
R W A 2 ?
S W A 2 2
ASS22
*
TRANSFER .10+,CHN11
10 % CHANGE TO MEDIAN LANE
CHN21 ASSIGN 5,Q$QWAS2
QUEUE QWAS2
TRANSFER .XHSRSW2,SWA22,RWA22
RWA22 ASSIGN 9,K0 RIGHT-TURN VEHICLE
TRANSFER ,CHK22
SWA22 ASSIGN 9,K1 STRAIGHT-THRU VEHICLE
ASS22 ASSIGN 8,W$DWA22
* ASSIGN 8+,WHGWAS2
ASSIGN 8+,Q$QWAS3
CHK22 TEST E BV$WAR2,K1,KWA22 CHECK FOR CONTINUOUS RIGHT TURN
SEIZE IWAS2 READY TO TURN RIGHT IF POSSIBLE
TWA22 GATE NU INAS2
GATE NU IEAL2
TRANSFER SIM,PWA22,TWA22
PWA22 ASSIGN 10,XHSXHAS2
ADVANCE V$FLOWR
DEPART QWAS2
GATE LS PHS12,NWA22
SAVEVALUE XWAS2+,K1,H
NWA22 RELEASE IWAS2
TRANSFER ,TRN21
KWA22 TEST L P8,XHS$CAP23,ASS22 RIGHT-TURN TRAFFIC AT INT2
GATE LS PHS12
SEIZE IWAS2
ASSIGN 10,XHSXHAS2
TEST E P9,K0,JWA22
TRANSFER SIM,FWAR2,DWAR2
DWAR2 ADVANCE V$FLOWR
SAVEVALUE XWAS2+,K1,H
FWAR2 DEPART QWAS2
RELEA SE I\$WS2
TRN21 TABULATE TWS12
TERMINATE
JWA22 TRANSFER S1M,FWA22,DWA22 STRAIGHT-THRU TRAFFIC AT INT2
DWA22 ADVANCE V\$FLOWS
SAVEVALUE X\$WS2+,K1,H
DEPART Q\$WS2
RELEASE I\$WS2
TABULATE TWS12
MARK 2
TRANSFER ,YWA22
FWA22 DEPART Q\$WS2
RELEASE I\$WS2
TABULATE TWS12
ZWA22 ADVANCE V\$RUMS2
TRANSFER ,CHN22
YWA22 TEST F P10,K1,UWA22
VWA22 SAVEVALUE ACCS2,V$STAYT,H
GWA22 ADVANCE XH\$ACCS2
SAVEVALUE MH\$WS2+,V$FLOWS
ASSIGN 8,V\$RUMS2
TRANSFER ,UWA22
UWA22 ASSIGN 3,X\$WS2
TEST E P7,K1,VWA22
TEST LE V\$HDWAY,50,VWA22 IF HDWAY LE 5.0 SEC., ACC. INDEPENDENTLY
WWA22 ADVANCE XH\$ACCS2
SAVEVALUE MH\$WS2+,V$FLOWS
MWA22 ADVANCE V\$RUS23
* TRANSFER ,08,,CHN12
CHN22 ASSIGN 5,Q\$GWAS3
QUEUE Q\$WS3
TRANSFER XH\$WS3,SWA23,RWA23
RWA23 ASSIGN 9,K0
TRANSFER ,CHK23
SWA23 ASSIGN 9,K1 STRAIGHT-THRU VEHICLE
ASS23 ASSIGN 8,W$DW\#23
ASSIGN 8+Q$QWAS4

CHK23 TEST E BV$&WAR3,K1,KWA23 CHECK FOR CONTINUOUS RIGHT TURN

SEIZE IWAS3

READY TO TURN RIGHT IF POSSIBLE

TWA23 GATE NU INAS3

GATE NU IEAL3

TRANSFER SIM,PWA23,TWA23

PWA23 ASSIGN 10XHSXWAS3

ADVANCE VSFLOWR

DEPART QWAS3

GATE LS PHS13,NWA23

SAVEVALUE XWAS3+,K1,H

NWA23 RELEASE IWAS3

TRANSFER ,TRN23

KWA23 TEST L PR,XHS$CAP34,ASS23 RIGHT-TURN TRAFFIC AT INT3

GATE LS PHS13

SEIZE IWAS3

ASSIGN 1G,XHSXWAS3

TFS1E P9,K0,JWA23

TRANSFER SIM,FWAR3,DWAR3

DWAR3 ADVANCE VSFLOWR

SAVEVALUE XWAS3+,K1,H

FWAR3 DEPART QWAS3

RELEASE IWAS3

TRANSFER ,TRN22

JWA23 TRANSFER SIM,FWA23,DWA23 STRAIGHT-THRU TRAFFIC AT INT3

DWA23 ADVANCE VSFLCWS

SAVEVALUE XWAS3+,K1,H

DEPART QWAS3

RELEASE IWAS3

TRANSFER ,TRN23

FWA23 DEPART QWAS3

RELEASE IWAS3

TPN23 TABULATE TWS13

TERMINATE

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**TABLE DEFINITION CARDS**

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**MODEL CONTROL SEGMENT**

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Note: The table above represents a portion of a simulation model, probably for a traffic control system, with various tables defining different aspects of the model.
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9000 TIME UNITS ARE EQUIVALENT TO 15 MINUTES

EXCLUDE THE FIRST FIFTEEN MINUTES OF DATA
APPENDIX D

DISCUSSION OF SOME TACTICAL PROBLEMS IN THE COMPUTER SIMULATION OF TRAFFIC
The second method is more realistic and effective, even though the first and the second approaches are both practical. Therefore, only the second approach was further investigated to determine an approximate length of a transient period at the beginning of each simulation run. In fact, there are many suggestions derived from the experiences of many authors (36, 42, 43, 44, 45). Conway's approach (36) was considered simple, practical, and quite effective.

Since the GPSS language provides such a simple way to obtain periodical summary reports, it was decided to start with one signal operation. A summary report was requested for every two cycles of the signal operation up to a total of sixty cycles. Figure D-1 shows the system behavior through the variation of mean queue length from the beginning of a cycle length up to the time a summary report is requested. In Figure D-1 there are four curves, all of which correspond to different random number streams. Based on the mean queue length as a performance measure, it is observed that random number streams have some effect on the variation of mean queue length. For a longer period of a simulation run, the effect of random number streams becomes small.

Figure D-2 serves the same purpose as Figure D-1 except that mean delay per vehicle is used rather than mean queue length. Obviously, the effect of random number streams is more pronounced here than on the previous plots of Figure D-1. This indicates that mean delay per vehicle is more sensitive to any variation of the system components than to mean queue length,
Figure D-1  Plots of Mean Queue Length against Number of Cycles: Traffic Volume = 400 vph.
and G/C = 40/90
Figure D-2  Plots of Mean Delay per Vehicle against Number of Cycles: Traffic Volume = 400 vph.
and  G/C = 40/90
even though both performance measures are closely related. Nonetheless, both figures indicate a good agreement on the approximate transient period of ten cycles for each simulation run. It should, however, be noted that this is a result of only one simple signal operation. Should a system of many signals be simulated, a longer transient period would certainly be expected. Unfortunately, the limitations of computer time and study period did not allow any further investigation in this area. Therefore, for a multi-signal system, it was considered sufficient to eliminate a transient period of ten cycles and thus to increase the data collection periods to two hours.

One additional investigation of the system behavior was conducted by examining the differences in the coefficients of variation of selected performance measures. The rationale of this investigation is that the mean value of the performance measure, per se, may not be able to sufficiently indicate the 'real' behavior of the system because each mean value is normally subjected to a different dispersion. Therefore, the use of the coefficient of variation should provide a better indicator of the system behavior. Figure D-3 depicts plots of the coefficient of variation of delay against the number of cycles, showing the 'real' behavior of the system. It can be seen that the coefficient of variation is much more sensitive to the variation of random number streams during the transient period, but it does not vary much once the system is in the steady-state period. Theoretically, this is the way a system should behave.

Based on this investigation, it is believed that the use of the coefficient of
Figure D-3  Plots of Coefficient of Variation against Number of Cycles: Traffic Volume = 400 vph.

and  G/C = 40/90
Generally, a simulation run must start from an empty condition which is not the existing situation of the problem being studied. Furthermore, during the beginning period of each simulation run the system is subjected to transient behavior which is normally undesirable in most studies. Because of these two problems, starting condition and transient behavior, it is undesirable to start collecting data from the very beginning of each simulation run. The data collected during the initial period would be likely to create an undesirable bias of the final results.

These difficulties were first referred to as tactical problems in computer simulation by Conway (36).

Shannon (45) summarized that there are at least three possible ways to reduce the biasing effect on the data to be collected:

1. Collect data over a sufficiently long period such that the effect of the data collected during the transient period would be relatively small.

2. Exclude the information during the beginning period from the data to be analyzed.

3. Start a simulation run with the most appropriate starting conditions that are observed to be a steady-state condition.

Although the three approaches are possible, the third method is ruled out as the complexity of the system under observation increases. This is especially true in a study of a network of traffic operation. Any attempt to apply the third approach without a real knowledge of a steady-state condition would undoubtedly cause a biasing effect on the data.
variation of a specific measure is a better method to study the system behavior than the use of the mean value of the measure.

Based on the preceding discussions, the following conclusions can be drawn:

1. The sensitivity of the system behavior to different random number streams depends on the type of performance measure used as the indicator.

2. The coefficient of variation of a performance measure is considered to be a better indicator of the system behavior than the mean value of the same performance measure.

3. The length of the transient period is dependent on:
   (i) The complexity of the system to be simulated.
   (ii) The type of the performance measure.
   (iii) The location in the system where the performance measure is observed.
   (iv) The travel time of each vehicle in the simulation model.
REFERENCES


