EFFECT OF SIGNAL SPACING ON PLATOON DISPERSION

by the

Research Staff
Transportation Research Center
Department of Civil Engineering

Final Report
EES 311

Prepared in cooperation with
the U.S. Department of Transportation, Federal Highway Administration,
and the Ohio Department of Transportation

Engineering Experiment Station
The Ohio State University
Columbus, Ohio 43210

July, 1973
Effect of Signal Spacing on Platoon Dispersion

J. Treiterer, Z. Nemeth and R. Vecellio

The Ohio State University
Engineering Experiment Station
2070 Neil Avenue
Columbus, Ohio 43210

Ohio Department of Transportation
Box 899
Columbus, Ohio 43216

Research conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration

The study was conducted to investigate platoon movements on urban arterials and to relate variation in platoon behavior to variation in signal control and to changes in traffic volumes. This objective was accomplished in the form of determining platoon characteristics for traffic traveling on signalized urban arterials and developing a mathematical model to simulate the behavior of a group of vehicles as it passes through a series of signalized intersections.

The developed model can be used to establish signal settings allowing for the dispersion of traffic and to study the effect of a change in a signal system on expected queue lengths and mean delays per vehicle.
ABSTRACT

A study was conducted to investigate platoon movements on urban arterials and to relate variation in platoon behavior to variation in signal control and to changes in traffic volumes. This objective was accomplished in the form of determining platoon characteristics for traffic traveling on signalized urban arterials and developing a mathematical model to simulate the behavior of a group of vehicles as it passes through a series of signalized intersections.

A helicopter-mounted aerial camera was used to collect continuous spacing and velocity data as traffic platoons traveled through a series of signalized intersections on one-way urban arterials. Time and space patterns of selected descriptive traffic parameters were obtained and the principal variables affecting platoon movement were identified.

Regression equations were developed for relating mean traveltime and the standard deviation of traveltime to signal spacing at a given volume level. A mathematical model was developed to generate queue and delay statistics for any combination of signal spacing and signal offset. Statistical agreement between observed and simulated queue length distributions revealed the model to be an adequate representation of traffic operations through a signalized arterial.
The validated simulation model can be used by the practicing engineer to establish signal settings allowing for the dispersion of traffic and to study the effect of a change in a signal system on expected queue lengths and mean delays per vehicle.
PREFACE

This report describes the work performed on Research Project EES 311 entitled "Effect of Signal Spacing on Platoon Dispersion" since June, 1969. This research project was sponsored by the Ohio Department of Transportation in cooperation with the United States Federal Highway Administration. The work was conducted by the Research Staff of the Transportation Research Center of The Ohio State University under the direction of Dr. Joseph Treiterer, Professor of Civil Engineering. Dr. Zoltan A. Nemeth, Assistant Professor of Civil Engineering, served as Project Coordinator. Principal contributor to the research was Robert L. Vecellio, Research Associate. Other contributors include Doyle R. Clear, Jr., and Jeffrey A. Myers, Research Associates. Recognition is given to Mrs. Barbara Austin for her efforts in preparation of this report.

In addition, thanks are extended to Mr. James Musick, Chief Traffic Engineer, and his staff of the City of Columbus, Division of Traffic Engineering for their assistance in the data collection activities associated with this project.

This study was performed in cooperation with the Ohio Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration. The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Ohio Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification or regulation.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Chapter Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>ii</td>
</tr>
<tr>
<td>PREFACE</td>
<td>iv</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>vii</td>
</tr>
<tr>
<td>LIST OF ILLUSTRATIONS</td>
<td>ix</td>
</tr>
<tr>
<td>CHAPTER ONE INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Description of Research Problem</td>
<td></td>
</tr>
<tr>
<td>1.2 Research Objective</td>
<td></td>
</tr>
<tr>
<td>1.3 Background</td>
<td></td>
</tr>
<tr>
<td>1.4 Scope</td>
<td></td>
</tr>
<tr>
<td>CHAPTER TWO REVIEW OF THE LITERATURE</td>
<td>5</td>
</tr>
<tr>
<td>2.1 Introduction</td>
<td></td>
</tr>
<tr>
<td>2.2 Platoon Behavior Studies</td>
<td></td>
</tr>
<tr>
<td>2.3 Conclusions</td>
<td></td>
</tr>
<tr>
<td>2.4 Closure</td>
<td></td>
</tr>
<tr>
<td>CHAPTER THREE DATA ACQUISITION BY AERIAL PHOTOGRAPHY</td>
<td>19</td>
</tr>
<tr>
<td>3.1 Selection of Study Sites</td>
<td></td>
</tr>
<tr>
<td>3.2 Establishment of Ground Control</td>
<td></td>
</tr>
<tr>
<td>3.3 Development of Signal Indicator Device</td>
<td></td>
</tr>
<tr>
<td>3.4 Data Collection</td>
<td></td>
</tr>
<tr>
<td>3.5 Data Reduction</td>
<td></td>
</tr>
<tr>
<td>CHAPTER FOUR CHARACTERISTICS OF PLATOON BEHAVIOR</td>
<td>41</td>
</tr>
<tr>
<td>4.1 Introduction</td>
<td></td>
</tr>
<tr>
<td>4.2 Characteristics of Photographed Platoons</td>
<td></td>
</tr>
<tr>
<td>4.3 Time and Space Patterns Exhibited by Selected Traffic Parameters</td>
<td></td>
</tr>
</tbody>
</table>
4.4 Analysis of Platoon Characteristics
4.5 Conclusions

CHAPTER FIVE DEVELOPMENT OF SIMULATION MODEL 114

5.1 Introduction
5.2 Development of Single Intersection Model
5.3 Development of Two-Intersection Model
5.4 Development of Multi-Intersection Model

CHAPTER SIX IMPLEMENTATION OF RESEARCH RESULTS 181

6.1 Applicability of Model
6.2 Example of Model Implementation

CHAPTER SEVEN CONCLUSIONS AND RECOMMENDATIONS 189

7.1 Summary of Accomplishments
7.2 Future Research Recommendations

REFERENCES 193

APPENDIX 196

vi
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Characteristics of Summit Street Study Site</td>
<td>22</td>
</tr>
<tr>
<td>3.2</td>
<td>Characteristics of North Fourth Street Study Site</td>
<td>23</td>
</tr>
<tr>
<td>3.3</td>
<td>Characteristics of Data Collection Films</td>
<td>31</td>
</tr>
<tr>
<td>4.1</td>
<td>Size of Initial Platoons</td>
<td>44</td>
</tr>
<tr>
<td>4.2</td>
<td>Lane Distribution of North Fourth Street Traffic</td>
<td>45</td>
</tr>
<tr>
<td>4.3</td>
<td>Lane Distribution of Summit Street Traffic</td>
<td>47</td>
</tr>
<tr>
<td>4.4</td>
<td>Frequency of Lane Change Maneuvers for North Fourth Street Platoons</td>
<td>48</td>
</tr>
<tr>
<td>4.5</td>
<td>Frequency of Lane Change Maneuvers for Summit Street Platoons</td>
<td>49</td>
</tr>
<tr>
<td>4.6</td>
<td>Frequency of Lane Change Maneuvers Per 1000 Feet for Summit Street Platoons</td>
<td>52</td>
</tr>
<tr>
<td>4.7</td>
<td>Frequency of Lane Change Maneuvers Per 1000 Feet for North Fourth Street Platoons</td>
<td>53</td>
</tr>
<tr>
<td>4.8</td>
<td>Velocity Measurements for Platoons 342 and 343</td>
<td>99</td>
</tr>
<tr>
<td>4.9</td>
<td>Lane Effect Test Results - North Fourth Street</td>
<td>101</td>
</tr>
<tr>
<td>4.10</td>
<td>Lane Effect Test Results - Summit Street</td>
<td>102</td>
</tr>
<tr>
<td>5.1</td>
<td>Comparison of Mean Delay Per Vehicle for Cycle Lengths of 60 and 90 Seconds</td>
<td>124</td>
</tr>
</tbody>
</table>
Table

5.2 Delay Characteristics Generated by Normal and Uniform Traveltime Distributions for Specified Offset Conditions with Mean Traveltime = 25 Seconds 139

5.3 Delay Characteristics Generated by Normal and Uniform Traveltime Distributions for Specified Offset Conditions with Mean Traveltime = 35 Seconds 140

5.4 Vehicles Discharged Per Fully Saturated Green Phases 150

5.5 Signal Timings Used in North Fourth Street Model 152

5.6 Signal Timings Used in Summit Street Model 153

5.7 Allowable Queue Lengths Along Summit Street Study Site 155

5.8 Allowable Queue Lengths Along North Fourth Street Study Site 156

5.9 Predicted Traveltime Characteristics Incorporated in Summit Street Model 166

5.10 Predicted Traveltime Characteristics Incorporated in North Fourth Street Model 167

5.11 Simulation Output for Five Independent Sequences of Random Numbers 173

5.12 Kolmogorov-Smirnov Test Results for Summit Street Queue Distributions 175

5.13 Kolmogorov-Smirnov Test Results for North Fourth Street Queue Distributions 176

5.14 Comparison of Queue Length Statistics on Summit Street: Field Data vs. Simulation Output 178

5.15 Comparison of Queue Length Statistics on North Fourth Street: Field Data vs. Simulation Output 179
<table>
<thead>
<tr>
<th>Figure</th>
<th>Illustration Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Summit Street and North Fourth Street Study Sites</td>
<td>21</td>
</tr>
<tr>
<td>3.2</td>
<td>Patterns Displayed by Signal Indicator Device</td>
<td>26</td>
</tr>
<tr>
<td>3.3</td>
<td>The KA-62A Aerial Camera and Bell Helicopter</td>
<td>29</td>
</tr>
<tr>
<td>3.4</td>
<td>Typical Photographs From Aerial Film on Summit Street</td>
<td>33</td>
</tr>
<tr>
<td>3.5</td>
<td>Data Reduction System for Aerial Photographs</td>
<td>35</td>
</tr>
<tr>
<td>3.6</td>
<td>Data Reduction Programming Process</td>
<td>38</td>
</tr>
<tr>
<td>4.1</td>
<td>Set of Vehicle Trajectories for North Fourth Street</td>
<td>55</td>
</tr>
<tr>
<td>4.2</td>
<td>Distance Variation of Traffic Density for Platoon 332</td>
<td>62</td>
</tr>
<tr>
<td>4.3</td>
<td>Distance Variation of Mean Velocity for Platoon 343</td>
<td>64</td>
</tr>
<tr>
<td>4.4</td>
<td>Distance Variation of Mean Velocity for Platoon 232</td>
<td>65</td>
</tr>
<tr>
<td>4.5</td>
<td>Distance Variation of Standard Deviation of Velocity for Platoon 742</td>
<td>67</td>
</tr>
<tr>
<td>4.6</td>
<td>Distance Pattern of Coefficient of Variation of Velocity for Platoon 343</td>
<td>69</td>
</tr>
<tr>
<td>4.7</td>
<td>Distance Pattern of Coefficient of Variation of Velocity for Platoon 353</td>
<td>70</td>
</tr>
<tr>
<td>4.8</td>
<td>Distance Variation of Mean Spacing for Platoon 343</td>
<td>71</td>
</tr>
<tr>
<td>4.9</td>
<td>Distance Variation of Mean Spacing for Platoon 712</td>
<td>73</td>
</tr>
<tr>
<td>4.10</td>
<td>Distance Variation of Mean Headway for Platoon 712</td>
<td>74</td>
</tr>
<tr>
<td>4.11</td>
<td>Distance Variation of Mean Headway for Platoon 343</td>
<td>75</td>
</tr>
</tbody>
</table>
Figure

4.12  Time Variation of Traffic Density for Platoon 743  77
4.13  Time Variation of Traffic Volume for Platoon 743  79
4.14  Time Variation of Mean Velocity for Platoon 232  80
4.15  Time Variation of Mean Velocity for Platoon 742  82
4.16  Time Variation of Standard Deviation of Velocity for Platoon 351  84
4.17  Time Pattern of Coefficient of Variation of Velocity for Platoon 711  85
4.18  Time Pattern of Coefficient of Variation of Velocity for Platoon 712  86
4.19  Time Variation of Mean Spacing for Platoon 742  88
4.20  Time Variation of Mean Spacing for Platoon 343  89
4.21  Time Variation of Mean Headway for Platoon 712  91
4.22  Time Variation of Mean Headway for Platoon 453  92
4.23  Velocity-Time Patterns for Platoons of Various Sizes  94
4.24  Mean Velocity Profiles for Platoons 342 and 343  97
4.25  Observed and Theoretical Speed-Density Relationships  105
4.26  Observed and Theoretical Volume-Density Relationships  107
4.27  Observed Standard Deviation of Velocity-Density Relationship  108
4.28  Observed Coefficient of Variation of Velocity-Density Relationship  110

5.1  Flow Discharge Pattern of Signalized Intersection  116
5.2  Fixed-time Signalized Intersection Model  118
5.3  Frequency Distribution of Delay at a Signalized Intersection  121
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.4</td>
<td>Comparison of Delay-Volume Relationships: Webster's Model and Simulation Model</td>
<td>125</td>
</tr>
<tr>
<td>5.5</td>
<td>Relationship Between Standard Deviation of Delay and Effective Green Time: Simulation Results</td>
<td>127</td>
</tr>
<tr>
<td>5.6</td>
<td>Relationship Between Parameters of Delay Distribution: Simulation Results</td>
<td>128</td>
</tr>
<tr>
<td>5.7</td>
<td>Queue Length - Offset Relationship for 2-Intersection Model</td>
<td>131</td>
</tr>
<tr>
<td>5.8</td>
<td>Mean Delay Per Vehicle - Offset Relationship for 2-Intersection Model</td>
<td>132</td>
</tr>
<tr>
<td>5.9</td>
<td>Functional Relationships Between Traveltime Parameters, Signal Spacings and Signal Offsets</td>
<td>136</td>
</tr>
<tr>
<td>5.10</td>
<td>Empirical Traveltime Distribution on Summit Street, 15th Avenue to 12th Avenue</td>
<td>158</td>
</tr>
<tr>
<td>5.11</td>
<td>Empirical Traveltime Distribution on Summit Street, Maynard Avenue to Lane Avenue</td>
<td>159</td>
</tr>
<tr>
<td>5.12</td>
<td>Relationship Between Mean Traveltime and Signal Spacing on Summit Street</td>
<td>161</td>
</tr>
<tr>
<td>5.13</td>
<td>Relationship Between Standard Deviation of Traveltime and Signal Spacing on Summit Street</td>
<td>162</td>
</tr>
<tr>
<td>5.14</td>
<td>Relationship Between Mean Traveltime and Signal Spacing on North Fourth Street</td>
<td>163</td>
</tr>
<tr>
<td>5.15</td>
<td>Relationship Between Standard Deviation of Traveltime and Signal Spacing on North Fourth Street</td>
<td>164</td>
</tr>
<tr>
<td>5.16</td>
<td>Empirical Headway Distribution on Summit Street at Maynard Avenue</td>
<td>168</td>
</tr>
<tr>
<td>5.17</td>
<td>Empirical Headway Distribution on North Fourth Street at Fifth Avenue</td>
<td>170</td>
</tr>
</tbody>
</table>
CHAPTER ONE

INTRODUCTION

1.1. Description of Research Problem

A queue of vehicles is formed periodically during the red phase of a fixed-time traffic signal. Following release of the queue, traffic arrives a short distance downstream from the signal in a well-identified platoon. As the traffic progresses further downstream from the signal, the platoon begins to lose its consistency. Changes in the spacings and velocities of the individual vehicles are reflected in the behavior of the platoon. As a result, changes in platoon length occur as a function of the distance downstream from the initial signal and the time since the beginning of green. Knowledge of the behavior of vehicular platoons is the most important consideration for programming traffic efficiently through a signal system.

Dispersion of platoons at isolated intersections has been studied quite extensively. However, a considerable amount of experimenting is involved in the coordination of traffic signals on urban arterials. Very little is known of the extent to which different factors influence the behavior of platoons on urban streets. Consequently, there is no design method of timing a linear system of traffic signals in current use which directly takes into account the dispersion
of traffic. Neglecting the effects of dispersion appears to be one of the important contributing factors in the occasional inefficiency of progressive signal systems. It is felt that signal spacing and offset patterns do have a measurable effect on the consistency of platoons.

1.2 Research Objective

The overall objective of this study is to investigate platoon movements on urban arterials and to relate variation in platoon behavior to variation in signal control and to changes in traffic volumes. This objective will be accomplished in the form of determining platoon characteristics for traffic traveling on signalized urban arterials and developing a mathematical model to simulate the behavior of a group of vehicles as it passes through a series of signalized intersections.

1.3 Background

Research on this project was begun in April, 1968. During the first year of this study, platoon data were collected along a four block signalized arterial in downtown Columbus, Ohio, using a ground-based 16mm time-lapse camera. An interim report, EES 311-1 entitled Effect of Signal Spacing on Platoon Dispersion, was published in June, 1969, summarizing the results of the first nine months of this research.

During the second year of research, additional data which had been collected with the 16mm camera was reduced and analyzed. In addition, the collection of comprehensive data over several one-way signalized arterials
using a helicopter-mounted aerial camera was initiated. A statistical design to be used in analyzing the data was formulated. An interim report, EES 311-2 entitled *Effect of Signal Spacing on Platoon Dispersion*, documenting this research was published in June, 1970.

1.4. Scope

The scope of this study can be described in terms of the phases of research which were pursued. The project objectives were accomplished by structuring the research efforts around four principal phases.

The first phase consisted of a review of the literature to describe the few studies of platoon behavior which have been conducted. The literature review provided the basis for identifying the major areas of work which had not been investigated and also illustrated the differences in the research approach used in the present study and the approaches used in all previous studies.

The second phase consisted of the collection and reduction of basic data on platoon movement. Platoon behavior was recorded through use of aerial photography; control point and vehicle coordinates were measured with the aid of a monocomparator. Finally, velocity and spacing measurements were obtained for the photographed platoons with the aid of computer programs.

The third phase of research consisted of identifying platoon characteristics for traffic traveling on urban signalized arterials. Lane of travel, platoon size and traffic volume were used as index variables. The characteristics which were identified were used as input to the fourth phase of research.
The fourth phase involved the development of a mathematical model to simulate the behavior of a group of vehicles as it passes through a series of signalized intersections. The model is able to predict queue lengths and vehicle delays for any combination of signal offset and spacing conditions. The use of the model permits the timing of progressive signal systems allowing for the dispersion of traffic.
CHAPTER TWO
REVIEW OF THE LITERATURE

2.1 Introduction

Only a limited number of studies have been conducted on the behavior of traffic departing from a signalized intersection. The majority of the studies conducted have been experimental in which speed, volume, and headway characteristics of platoon movement have been measured. A few studies have been theoretical in which attempts have been made to describe platoon movement according to several derived mathematical theories of diffusion. In addition, several studies have attempted to combine the theoretical and experimental aspects of platoon behavior. Of the fourteen studies which comprise this literature review, nearly half of them were published within the last five years.

A review of the pertinent studies follows. For each study, care has been taken to document the study approach or methodology and the significant findings.

2.2 Platoon Behavior Studies

One of the earliest experimental studies concerned with platoon movement of traffic from a signalized intersection was conducted by Lewis and
reported at the Highway Research Board Meeting in 1957 (1). Lewis used a 20-pen recorder to collect arrival time data at five points downstream from an isolated signalized intersection located on a semi-expressway type of facility in California. Analysis of the vehicle arrival time frequency distributions revealed that certain measured characteristics of platoon movement were linearly related to the distance downstream from the traffic signal. These platoon characteristics were the maximum ordinates of the equivalent normal distributions of arrival times of the \textit{n}th platoon vehicle (implying that the standard deviation as a measure of platoon dispersion is inversely related to distance traveled downstream from the issuing traffic signal), the mean arrival time of the \textit{n}th platoon vehicle, and the time for a given percentage of vehicles in a platoon to pass a point.

Lewis concluded that a traffic signal placed at any distance up to 0.65 mile from the platoon formation point could be timed in a coordinated manner with the initial signal. The author did not study platoon movement beyond the 0.65 mile point.

Another experimental study of platoon behavior was conducted by Graham and Chenu of the California Division of Highways (2). The purpose of the study was to determine the amount of dispersion of platoons at various distances downstream from a rural isolated signalized intersection. The field data, collected in 1958 by means of road tubes on two lanes of a 4-lane rural expressway, revealed that traffic remained in well-defined platoons for distances of at least one mile beyond the signal. Analysis of the 70 platoons
studied indicated that 91% of the vehicles remained in well-defined platoons up to 1/4 mile from the platoon formation point, 85% of the vehicles remained in platoons up to 1/2 mile, 80% up to 3/4 mile, and 77% up to 1 mile.

Pacey, in an unpublished report of 1956, presented a probabilistic model of platoon dispersion (3). Pacey's model is formulated on two assumptions:

1. Vehicles move independently of each other with a normal distribution of speeds.

2. The velocity of any individual vehicle remains constant as the vehicle moves down the road. That is, no interaction is assumed between vehicles.

Based on these assumptions, Pacey was able to formulate a distribution of travel times applicable to traffic departing from a saturated signalized intersection. The derived traveltime distribution is shown to be a function of the normality of vehicle speeds, the mean and variance of the speed distribution, and the distance over which the traveltimes are distributed. Thus, given an input flow at a point downstream from the traffic signal, one is able to predict the flow at a point further downstream by using Pacey's kinematic model.

To test his model of platoon behavior, Pacey collected flow-time data with manually operated teleprinter tape machines on two expressways in England. In comparing the observed results with those predicted by theory, Pacey concluded that his kinematic model gave a good fit to the data, especially for moderate traffic volumes.
In 1962, Grace and Potts conducted a theoretical investigation of Pacey's platoon dispersion model (4). These researchers showed that the diffusion process predicted by the kinematic model can be described by a one-dimensional diffusion equation satisfied by a function related to traffic density. If one considers a platoon of vehicles moving down a road according to Pacey's assumptions, then the pseudo diffusion equation derived by Grace and Potts is the following:

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

where \( k \) = a traffic density related function

\( \tau = \) a variable related to \((\text{time})^2\)

\( \alpha = \) diffusion constant

\( x = \) preset time, the time added to the beginning of the green phase of a traffic signal to allow for the spreading of a platoon released from a prior signal.

Of particular interest from this equation is the diffusion constant \( \alpha \). If vehicle speeds are normally distributed with mean \( m \), and variance \( \sigma^2 \), then the diffusion constant \( \alpha \), is defined as; \( \alpha = \frac{\sigma}{m} \).

This dimensionless parameter characterizes the rate at which a platoon diffuses and lengthens. If all vehicles travel at the same speed \( m \), then \( \sigma = 0 \) and, hence, there is no platoon spreading \( (\alpha = 0) \). As the dispersion of vehicle speeds increases relative to the mean speed, the diffusion constant indicates greater platoon spreading \( (\alpha \text{ is increasing}) \).

As pointed out by Grace and Potts, an important characteristic of the
diffusion equation is that it relates the microscopic and macroscopic properties of the traffic stream. The microscopic parameters are taken to be the mean and variance describing the speed distribution of the stream vehicles whereas the macroscopic parameter is considered to be the diffusion constant.

Grace and Potts applied their theoretical model to the design of progressive signal systems. For certain assumed initial conditions regarding the traffic density function, the diffusion equation is used to provide analytical solutions to the problem of coordinating two successive signalized intersections allowing for the phenomenon of platoon dispersion.

A study was conducted by Herman, Potts, and Rothery of the General Motors Research Laboratories to test experimentally the kinematic model of traffic platoon behavior and the theoretical results obtained by Grace and Potts (5). Arrival times and speeds were collected at two points downstream from a signalized intersection on a 4-lane divided highway. Pressure tapes were used to collect arrival times which were recorded automatically on magnetic tapes.

The results of the experiments confirmed that the kinematic model well describes the spreading of platoons in medium volume traffic. The diffusion constant was found experimentally to have a value of 0.18. Good agreement between theoretical results and those found from the experiment was established in describing the behavior of the front of the platoons. A total of 88 platoons consisting of about 1800 vehicles were observed.

Several theories have been proposed to describe the diffusion process.
One such theory is credited to the work of Lighthill and Whitham (6). In their classic paper, Lighthill and Whitham proposed a hydrodynamic theory of traffic flow and used it to describe the theoretical behavior of platoons upon release from a traffic signal. The fundamental hypothesis of the theory is that at any point on the road flow is a function of traffic density. The theory can be used to predict the formation of a shock wave when a stationary queue of vehicles begins forward motion.

Using an observed flow-density curve, Pacey applied the Lighthill-Whitham theory to his data collected in London to predict the changes in shape of flow-time histograms as a platoon of vehicles travels away from a saturated signalized intersection. It was concluded that there seems to be some evidence in favor of the Lighthill-Whitham Theory but not of a substantial nature. It must be added that only 20 cycles of observed data were subjected to testing.

In another theoretical paper, Gerlough introduced an analogy between traffic dynamics and wave mechanics to describe the dissipation of a platoon (7). However, Grace and Potts in their paper gave a different interpretation to Gerlough's theoretical results and concluded that the wave mechanics analogy is hardly evident.

In 1957, Hillier and Rothery conducted a study of platoon behavior and showed how the diffusion process can be taken into account in the setting of signals (8). Arrival time data were collected at four sites in London, England. The sites selected consisted of 2-way roadways having 2 lanes in each direction. Observations of platoons discharging from a signalized intersection were made
at four locations downstream from each signal. Arrival times were collected manually and recorded automatically on tapes. Data reduction consisted of computing flows in intervals of 2 seconds for six different flow volumes ranging from approximately 400 to 1400 vehicles per hour.

The researchers established relationships between total delay measured in vehicle-hours per hour of green and offset-time. It is shown that the optimal offset time which minimizes delay is a linear function of the distance from the platoon formation point. This suggests that the progression speed which minimizes total delay is approximately equal to the mean speed of the traffic stream. This latter result was also established by Francis and Lott in a simulation study of linked traffic signals (9).

Practically all of the studies reviewed thus far have dealt with characteristics of an average platoon, that is the average of all types of platoons over a period of time. A slight disadvantage of this method is that it tends to give skewed distributions of vehicle arrivals. To remove this disadvantage, Ferguson analyzed platoons of particular time lengths and volumes (10).

Platoon behavior was observed at two contrasting sites in Glasgow, Scotland. The first site was one direction of a divided 4-lane roadway where overtaking of vehicles was permitted, whereas the second site was one lane of a 2-lane, 2-way roadway where overtaking was highly unlikely. Arrival times at several points downstream from signalized intersections located at each site were collected manually with the aid of stopwatches or an electronic timing unit.
For analysis purposes, the platoons were classified by their time lengths and their volume levels. Platoon time lengths ranged from 4 seconds to 20 seconds; platoon volumes ranged from approximately 0.5 vehicle per second to 1.0 vehicle per second.

Analysis of the behavior of the 750 vehicles included in the study yielded the following results:

1. The mean time headway of a platoon departing from a signalized intersection decreased as the time length of a platoon increased.

2. Mean platoon speeds were linearly related to distance from the stop line.

3. Vehicle arrivals at a point downstream from the signal tended to exhibit a normal distribution.

4. Platoon splitting due to lane changing was satisfactorily explained by a 2-platoon theory of overtaking related to platoon speeds.

5. To minimize delay to traffic on a one-way system, it was found necessary to offset the center of each green phase at approximately the central time point of the platoon.

This final conclusion, of course, verifies the results obtained earlier by Hillier & Rothery and by Francis & Lott.

Pacey's kinematic model was shown to be a valid method for predicting the flow downstream from a traffic signal given an initial flow histogram. Another method, due to Robertson, based on a smoothing technique appears to offer much promise.
D. I. Robertson of the Road Research Laboratory developed a method of determining optimum fixed-time traffic signal settings for a network of signalized intersections (11). His TRANSYT model incorporated a technique of allowing for the dispersion of platoons. Robertson was able to predict platoon behavior using the technique of exponential smoothing. A recurrence relationship was established to predict flow downstream from a traffic signal given the input flow, previously predicted flow, and a smoothing factor. The appropriate equation is the following:

\[ q'_{i+t} = F \cdot q_i + (1-F) \cdot q'_{i+t-1} \]

where \( q'_{i+t} \) = predicted flow for time interval \( (i+t) \)

\( F \) = smoothing factor with range \( 0 \leq F \leq 1 \)

\( q_i \) = input flow for \( i^{th} \) time interval

\( q'_{i+t-1} \) = predicted flow for time interval \( (i+t-1) \)

\( t \) = a variable related to the mean travel time over the distance for which platoon dispersion is being calculated.

The formula was applied to data collected at four sites in West London. Passage times of vehicles were recorded manually at four points downstream from a traffic signal. The behavior of over 700 platoons was recorded. Good fits were observed between measured and predicted platoon patterns.

The smoothing factor, \( F \), was found to vary with average travel time. The specific relationship developed by Robertson is the following:

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

where \( \bar{t} \) = average travel time.
The author points out that it is reasonable to expect that the smoothing factor should also be a function of site conditions such as roadway width, gradient, parking, and traffic composition. The role of such factors in platoon dispersion have not yet been investigated.

In Ferguson's study of platoon dispersion, a method is devised whereby the smoothing factor can be calculated from observations of average arrival times and first arrival times ignoring the 2.5 per cent of extremely fast vehicles. Knowing the rate of platoon spread, Ferguson shows how the smoothing factor can be used to derive delay-offset relations such that optimal signal settings on a 2-way street can be established.

Wright, in 1969, developed a bunching model to predict vehicle speeds downstream from a traffic signal as a function of position in the initial queue and the desired speed distribution (12). Unlike Pacey's work, Wright's model does not allow overtaking. The theory implies that velocities downstream are marginally dependent on the amount of green time during saturation flow conditions.

Data for the model were gathered along a one-half mile section of 2-lane road originating at a signalized intersection. Six pneumatic tube detectors were placed at 400 foot intervals downstream from the signal and arrival times were recorded during both the morning and evening peak periods.

Agreement between observed and predicted mean speeds was considered quite good in view of the simplicity of the model. However, the model did predict lower velocities for the first and last vehicles in a platoon than those
measured in the field. The author feels that his model does have potential
application in timing a set of coordinated traffic signals.

Experimental work on platoon dynamics has been conducted by General
Motors Research Laboratories. Herman, Lam and Rothery conducted a series
of experiments to study the starting and stopping characteristics of a platoon
of buses as it operates along an exclusive right of way (13).

The behavior of the lead and last vehicles in a six bus platoon was
monitored through the use of in-vehicle instrumentation. Time and distance
measurements were recorded on magnetic tape as a series of test runs was
conducted for selected values of the control variables: interstation spacing,
operating speed, and intervehicular spacing at the platform.

The authors found that the cyclic dynamics of bus platoons starting at
one station and stopping at another possessed repeatable features which could
be described qualitatively. By examining the velocity-time patterns of both the
lead and last buses, it was established that the motion of the platoon can be
described in terms of three components: a starting transient, a steady state,
and a stopping transient.

The starting transient consists of a starting delay and an initial adjust-
ment followed by a series of final adjustments until a steady-state condition is
reached. On the other hand, the stopping transient consists of a lag time for the
beginning of deceleration, an adjustment during the deceleration period and,
finally, the stopping delay.

Using the same experimental approach, the authors in a recent study
examined the starting characteristics of a 6-vehicle automobile platoon (14). Based on an analysis of time-space trajectories for the first and last platoon positions, it was determined that the starting phase of an automobile platoon could be described in terms of an acceleration phase and a relaxation phase.

The acceleration phase, defined from the time the lead car starts to move to the time the last car reaches the cruising speed of the lead car, was shown to be a function of the initial platoon length, the starting delay, and the acceleration of lead and last cars. The relaxation phase, defined from the end of the acceleration phase to the time the platoon reaches a steady state, was quantified by establishing platoon transit times at points downstream from the initial platoon position.

In view of their earlier work, Herman, Lam, and Rothery concluded that the starting phase of automobile platoons cannot be as easily described as was done for bus platoons due principally to the wide variation of driver and car performance. Instead of only one velocity or acceleration adjustment, automobile drivers were found to make a number of adjustments as they accelerate to the cruising speed.

2.3 Conclusions

As a result of the articles reviewed in this section, the following findings can be stated:

1. The approach taken in all studies of platoon movement reviewed was:
   a) the collection of data either manually or by means of automatic
on-the-road counters to yield vehicle arrival times at a point or points downstream from a signalized intersection, or
b) the collection of platoon data by means of instrumented vehicles operating on a test track.

2. In all experimental studies reviewed, study sites consisted of isolated signalized intersections in a rural or semi-urban environment.

3. Two methods are available for predicting dispersion of platoons downstream from a signalized intersection. These are Pacey's kinematic model of platoon dispersion (a model formulated on probability considerations) and Robertson's dispersion model (a model based on the technique of exponential smoothing). Both models were found to give good agreement with empirical data.

4. Considering the distribution of vehicle speeds, the ratio of the standard deviation and the mean speed characterized the rate at which a platoon diffuses and lengthens. This ratio, termed the diffusion constant, can be derived from theoretical considerations of Pacey's kinematic model.

2.4 Closure

The approach taken in the present research project differs in two important respects from all previous studies of platoon behavior. First, rather than collecting limited data on many platoons, the approach taken is to collect comprehensive data on relatively few platoons. Secondly, rather than studying platoon movement at isolated signalized intersections or on test facilities, the
approach of the present study is to use signalized urban arterials as study sites.
CHAPTER THREE
DATA ACQUISITION BY AERIAL PHOTOGRAPHY

3.1 Selection of Study Sites

To permit extensive coverage of traffic traveling through a linear signal system, it was decided that one-way urban streets would best serve as study sites. Two-way streets were rejected because of the disruptions in flow caused by left turning traffic and because of the usually limited number of lanes of travel in a given direction.

Additional requirements were that the study sites handle appreciable amounts of traffic during peak hour conditions and that a progressive signal system be in operation during peak hour conditions. These conditions would permit the study of the behavior of high volume platoons as they progressed through a series of signalized intersections. A further requirement was that streets selected as study sites should possess variable spacings between signalized intersections. In this manner, the effect of different signal spacings on platoon behavior could be studied.

Based on these requirements, two study sites were selected. Both are one-way signalized urban arterials located in Columbus, Ohio. The sites are situated near and parallel to Interstate 71 and, as such, serve as alternate routes to and from the CBD.
Summit Street is a southbound 5-lane arterial having 9 signalized intersections with distances between signals ranging from approximately 350 feet to 1975 feet. North Fourth Street is a northbound 5-lane arterial having 9 signalized intersections with distances between signals ranging from approximately 350 feet to 2450 feet. Both sites have lane widths of 10 feet with parking permitted on portions of each street.

A schematic of the study sites is presented in Figure 3.1 which illustrates the frequency of intersections on each street and also differentiates between signalized and non-signalized intersections. Cross traffic is considered appreciable only at the signalized intersections.

Characteristics of the Summit Street Study Site are presented in Table 3.1. Platoon behavior was studied from the platoon formation point at Maynard Avenue to a point just south of 5th Avenue.

Characteristics of the North Fourth Street Study Site are presented in Table 3.2. Platoons formed by the signal at 5th Avenue were studied until they approached Hudson Street.

### 3.2 Establishment of Ground Control

Ground control points necessary in the data reduction stage were established along both study sites. Points were selected with the aid of photographs and horizontal distances were measured in the field. The control points selected consisted of the intersections of private walks and sidewalks. All control points were located on one side of the entire length of a study site.
Table 3.1 Characteristics of Summit Street Study Site

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Distance Between Intersections</th>
<th>Number of Lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hudson Street</td>
<td>1458 feet</td>
<td>3 Traffic Lanes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 Parking Lanes*</td>
</tr>
<tr>
<td>Maynard Avenue</td>
<td>1975</td>
<td>4 Traffic Lanes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 Parking Lane - West</td>
</tr>
<tr>
<td>Lane Avenue</td>
<td>1550</td>
<td>4 Traffic Lanes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 Parking Lane - West</td>
</tr>
<tr>
<td>17th Avenue</td>
<td>628</td>
<td>4 Traffic Lanes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 Parking Lane - West</td>
</tr>
<tr>
<td>15th Avenue</td>
<td>1545</td>
<td>4 Traffic Lanes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 Parking Lane - West</td>
</tr>
<tr>
<td>Chittenden Avenue</td>
<td>346</td>
<td>4 Traffic Lanes</td>
</tr>
<tr>
<td>11th Avenue</td>
<td>1630</td>
<td>4 Traffic Lanes</td>
</tr>
<tr>
<td>7th Avenue</td>
<td>1106</td>
<td>4 Traffic Lanes</td>
</tr>
<tr>
<td>5th Avenue</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*East parking lane becomes traffic lane one block south of Hudson Street.
Table 3.2 Characteristics of North Fourth Street Study Site

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Distance Between Intersections</th>
<th>Number of Lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td>5th Avenue</td>
<td>1106 feet</td>
<td>4 Traffic Lanes</td>
</tr>
<tr>
<td>7th Avenue</td>
<td>1630</td>
<td>4 Traffic Lanes</td>
</tr>
<tr>
<td>11th Avenue</td>
<td>346</td>
<td>4 Traffic Lanes</td>
</tr>
<tr>
<td>Chittenden Avenue</td>
<td>1545</td>
<td>4 Traffic Lanes, 1 Parking Lane - West</td>
</tr>
<tr>
<td>15th Avenue</td>
<td>628</td>
<td>4 Traffic Lanes, 1 Parking Lane - West</td>
</tr>
<tr>
<td>17th Avenue</td>
<td>713</td>
<td>4 Traffic Lanes, 1 Parking Lane - West</td>
</tr>
<tr>
<td>19th Avenue</td>
<td>2460</td>
<td>4 Traffic Lanes, 1 Parking Lane - West</td>
</tr>
<tr>
<td>Wyandotte Avenue</td>
<td>1810</td>
<td>3 Traffic Lanes, 1 Parking Lane - West</td>
</tr>
<tr>
<td>Hudson Street</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
A rather large number of points were selected for control: 38 points on North Fourth Street and 36 points on Summit Street. Such a high density of points was selected to permit a minimum coverage of 3 control points per photograph—thus allowing a high degree of accuracy in taking measurements from the photographs. Spacing between control points was approximately 300 feet.

The bearings of the control points were established by photogrammetric methods. Photo coordinates of the control points were used to determine the photographic distances between control points and the law of cosines was then applied to determine the direction angles. Five sets of coordinate readings were averaged to determine a given angle.

This photogrammetric procedure entails very little additional work since the photo coordinates of the ground control points must be established in the data reduction procedure. It has the important advantage of being much less time consuming than the traditional method of measurement of the angles in the field with a transit.

3.3 Development of Signal Indicator Device

In order to relate dispersion characteristics to signal timing, it is necessary to indicate on the film the particular signal indication at one of the signalized intersections. Since a progression system is in operation at the time of the data collection flights, knowledge of the phase change at one of the signalized intersections in addition to knowledge of the offset pattern and the
time interval between photographs then permits determination of the signal phase at each of the remaining signalized intersections. Several devices were designed and tested for this purpose before a final design proved successful.

For the initial flight, a manually operated signal indicator was constructed and was in operation at the time of the flight. The particular signal phase at the intersection where the portable device is located is determined by the direction of a 4 foot x 1 foot rotating arrow mounted on a wooden platform of contrasting color. The developed film revealed the device to be easily identified and, as a result, a modification of the indicator was constructed to permit electronic operation and increased accuracy.

A light bank consisting of 24 signal lamps of varying colors was tested as a signal indicator. However, the failure of the black and white film to clearly distinguish between bulb intensity and even whether a lamp was burning or not dictated its demise.

The final design, which proved to be successful, consists of twelve 2-inch square aluminum rods mounted on a 4 foot x 3 foot 10 inch wooden base. Rotation of the shafts by means of synchronous motors permits a total of four different and distinct designs to be displayed. In operation, the device is connected directly to the traffic control box at the intersection for which the phasing is desired.

A decision was made in the data collection phase to take the majority of photographs at intervals of 3 seconds. Figure 3.2 illustrates the different designs displayed by the signal indicator during such a data collection flight.
Figure 3.2 Patterns Displayed by Signal Indicator Device
Different patterns were displayed at the following times: during the red phase of the signal, at one second after the beginning of green, at two seconds after the beginning of green, and from three seconds after the beginning of green until the end of the amber phase. The signal indicator was located at the intersection where the platoons were formed. Data collection would be initiated before the platoon was released by the signal. In this manner, once the platoon starts moving, it is possible to permit precise determination of the beginning of the green phase by examining the photographs on which the device was captured. The beginning of green could thus be estimated to the nearest one second.

3.4 Data Collection

Basic data on platoon movement was gathered with a helicopter-mounted aerial camera. The camera used is a KA-62A aerial reconnaissance camera manufactured by Chicago Aerial Industries. This metric camera has a focal length of 3 inches and takes photographs having a 4.5 inch by 4.5 inch film format. The film capacity of the camera is 250 feet which yields approximately 600 exposures per roll of film. Film used was Kodak Plus-X Panchromatic having an exposure index of 80. The camera is equipped with automatic exposure control which was of great value during data collection flights since the surveys were flown during the morning and evening traffic peak hours when changing and rather difficult light conditions prevailed.

The helicopter used for the aerial surveys was a Bell Ranger helicopter owned by the Ohio Department of Highways. The helicopter with the mounted
camera is shown in Figure 3.3. The specially designed mount permits rotation of the camera through a horizontal angle of approximately 130 degrees. This allows the camera operator to compensate for drift of the helicopter.

During a data collection flight, the pilot is instructed to be over the intersection at which the platoon is formed a few seconds before the signal turns green. Upon release of the platoon, the pilot then follows the platoon of vehicles attempting to match the helicopter's velocity with that of the platoon. Because of the progression system in operation at the time of the data collection flights, the entire platoon is never stopped at a signal as the platoon moves down the arterial.

A total of eight flights have been made over the study sites. The first flight served as a test flight for purposes of investigating different flight altitudes as to film coverage and ease of vehicle identification, determining the extent of problems associated with overhanging trees adjacent to the curb lanes, and determining the extent of problems associated with shade during peak traffic conditions.

As a result of evaluating the film taken during the initial flight, it was decided that photographs taken from an altitude as low as 1000 feet above ground level would still provide adequate coverage of a study site for determining platoon behavior characteristics. However, flight altitudes in the neighborhood of 2000 feet above ground level were considered preferable because of the wider film coverage available.

It was also discovered that overhanging trees are present only sporadically
Figure 3.3 The KA-62A Aerial Camera and Bell Helicopter
throughout the study sections and consequently posed no major problems. Film analysis also revealed that vehicles could be easily identified from the photographs regardless of the presence of shadows.

Seven rolls of film were used for data collection. Three of these were exposed during the morning peak on Summit Street and four were exposed during the evening peak on North Fourth Street. Table 3.3 summarizes several descriptive characteristics of the data collection films. As indicated in the table, a total of 38 runs was conducted, each run being identified with one platoon or group of vehicles. The number of runs per film is a variable quantity depending on the coverage of each platoon and the time interval between photographs.

One film on each street was exposed with one second time intervals between photographs; all other films were exposed with three second time intervals between photographs. Rate of film movement was controlled by a highly accurate intervalometer which exhibited an average error of only 0.0029 seconds from the designated time interval.

Data collection flights on Summit Street were initiated at Maynard Avenue. During several of the morning flights, the traffic signal at Maynard was operated by a member of the Columbus Traffic Engineering Department. The red time of this signal (normally 22.5 seconds) was held for one complete cycle, 75 seconds, to permit a queue of vehicles to form on Summit Street. The queue was then released at a time so as not to interfere with the signal progression scheme.
Table 3.3 Characteristics of Data Collection Films

<table>
<thead>
<tr>
<th>Film Identification</th>
<th>No. of Runs</th>
<th>Time Interval</th>
<th>Location</th>
<th>Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>Film 2</td>
<td>5</td>
<td>3 secs</td>
<td>Summit St.</td>
<td>7:30 - 8:10 AM</td>
</tr>
<tr>
<td>Film 3</td>
<td>8</td>
<td>3 secs</td>
<td>North 4th St.</td>
<td>5:00 - 5:40 PM</td>
</tr>
<tr>
<td>Film 4</td>
<td>8</td>
<td>3 secs</td>
<td>North 4th St.</td>
<td>4:45 - 5:15 PM</td>
</tr>
<tr>
<td>Film 5</td>
<td>4</td>
<td>1 sec</td>
<td>Summit St.</td>
<td>7:45 - 8:15 AM</td>
</tr>
<tr>
<td>Film 6</td>
<td>5</td>
<td>3 secs</td>
<td>North 4th St.</td>
<td>4:50 - 5:20 PM</td>
</tr>
<tr>
<td>Film 7</td>
<td>4</td>
<td>3 secs</td>
<td>Summit St.</td>
<td>7:40 - 8:15 AM</td>
</tr>
<tr>
<td>Film 8</td>
<td>4</td>
<td>1 sec</td>
<td>North 4th St.</td>
<td>4:45 - 5:15 PM</td>
</tr>
</tbody>
</table>
Data collection flights on North Fourth Street were initiated at Fifth Avenue. The normal red time of 45 seconds was allowed for a queue buildup on North Fourth Street. Because of the high volume of traffic on Fifth Avenue during the evening peak, it was not deemed advisable or considered necessary to extend the red time on North Fourth Street at this signal.

During all flights, radio communication between the helicopter pilot and the ground crew (stationed at the platoon formation point) was established. In this way, the pilot was always aware of how much time remained before the platoon was given the green and he could maneuver the helicopter into proper position. It was important that the helicopter be directly over the intersection when the platoon started its forward motion. This timing procedure between the pilot and ground crew was necessary since the helicopter cannot hover for any substantial length of time.

A typical set of data collection photographs is shown in Figure 3.4. These low altitude photographs illustrate traffic movement during the morning peak on Summit Street. In the photograph on the left, a platoon has formed during the red phase at Maynard Avenue. The photograph on the right shows the position of the platoon 12 seconds later. Approximate altitude is 1000 feet above ground.

The only difficulty experienced during the data collection flights was in attempting to maintain sufficiently high altitudes so as to capture the behavior of very large platoons. The difficulty stemmed from the fact that both study sites were located within the Air Traffic Control Zone of Port Columbus.
International Airport and, in addition, were directly in the flight paths of commercial aircraft operating from the airport. Several times, due to the heavy influx of incoming air traffic, data collection flights had to be terminated and rescheduled. For these reasons, the majority of the data collection flights had to be flown at altitudes of 1500 feet above ground or lower. At these low altitudes, ground coverage was limited and, as a result, it was not possible to capture large platoons on film consistently from frame to frame.

3.5 Data Reduction

Data reduction consisted of measuring the x, y coordinates of all platoon vehicles and the necessary control points on the photographs and then converting the extracted information to headway and velocity measurements. A highly efficient data reduction process developed at the Transportation Research Center was used and will be briefly described herein.

3.5.1 Data Reduction Hardware

The data reduction system for measuring photo coordinates consists of a Type 829D Mann Comparator, a Type 1945 Mann Data Logger, and an IBM 026 Printing Card Punch. The three components of the system are shown in Figure 3.5.

The comparator is a precision instrument capable of measuring distances in the x and y directions on photographic films. Consisting essentially of an assembly of mechanical stages employing precision lead screws and ways, the instrument projects the photographic image using a 20x magnification system.
Figure 3.5 Data Reduction System for Aerial Photographs
onto a 6" x 6" white screen. The operator positions an illuminated dot over the point on the photograph whose coordinates are desired. A calibration check of the instrument revealed an average deviation of ± 0.002mm accounting for both machine errors and reading errors.

The data logger provides a digital display of the coordinate readout from the comparator. Seventeen cold-cathode gas-discharge tubes are used to display the x and y coordinate readings in microns, the direction of the readings with respect to an established reference point (indicated by plus or minus signs), and a three digit frame count. In addition, supplementary information can be entered into the data logger such as film number, frame number and other identifying characteristics.

The IBM card punch is connected on-line with the data logger and permits the total display plus the supplementary information to be transferred directly to punched cards. In operation the rate of output from the data reduction system is controlled by the card punch.

3.5.2 Data Reduction Software

During recent years, the Transportation Research Center has acquired considerable experience in the reduction of data from aerial photographs. A number of programs have been devised for the IBM 370 computer to transfer photo coordinates of vehicles into time and distance measurements. These programs were used in the current project, with some modifications, to yield time headway and velocity data on platoon vehicles. The computer process
necessary to transform the raw data to the desired information is illustrated
diagrammatically in Figure 3.6.

Preliminary to the 4-phase process shown in Figure 3.6 is the establish-
ment of ground control coordinates. This is accomplished through knowledge of
the photo coordinates of each control point in addition to the direction angles and
the ground distances between control points. A plot of the resulting ground
coordinates serves as a visual check on their positions relative to one another.

The purpose of Phase 1 in the data reduction process is to produce an
input deck suitable for the IBM 1620/27 digital incremental plotter. Using the
raw data and the ground coordinates of the control points as input, Phase 1
yields photo coordinates, ground coordinates, and cumulative distance from
some reference point for each vehicle on each photograph being processed.
This output is in the form of binary cards.

Phase 2 is a plotting program. Using the binary output from Phase 1,
this phase produces a time-distance plot of vehicle movement. The program
plots the actual vehicle positions in space and in time on a discrete basis. By
observing the time-distance diagram for a platoon of vehicles, errors in the
photo reduction procedure can be detected. Such errors may include missed
vehicles and incorrect control points on some photographs. When the errors
are corrected, the raw data is suitable for processing in Phase 3. If a time-
distance plot of platoon movement is desired, the corrected data is recycled
through Phases 1 and 2.

The logic of the Phase 3 program is identical to that used in Phase 1.
Figure 3.6 Data Reduction Programming Process
However, the output from Phase 3 (photo coordinates, ground coordinates, and cumulative distances) is in printed form rather than binary form. In addition, vehicle identification numbers can be assigned for this phase as a result of visual examination of the time-distance plots.

Phase 4 represents the final step in the data processing procedure. Using the input from Phase 3, this computer program calculates and prints out time headways and velocities for all vehicles on a given photograph. In addition, measures of traffic density, mean velocity and traffic volume for each photograph are calculated and printed out.

3.5.3 Data Reduction Output

Nearly 1000 photographs of platoon movement have been reduced. All photographs were not selected for reduction purposes because some photographs provided inadequate coverage, caught the presence of an unusual traffic disturbance (an accident or a stalled vehicle) or depicted platoons having a high proportion of bus traffic.

The photographs were reduced on a lane basis. For a section of 4-lane street, only the two interior lanes were reduced. For a section of 5-lane street, the three interior lanes were reduced. The movement of traffic in the curb lanes was not considered for reduction purposes due to the unpredictable influences of buses and turning traffic.

For traffic traveling in a given lane, vehicle trajectories which depict vehicle movement by a plot of distance traveled versus time traveled were
constructed to provide a visual picture of traffic behavior. Trajectories for a total of 28 platoons were obtained: 16 on Summit Street and 12 on North Fourth Street. The velocity and spacing information for these platoons comprises the sample data for this study. A total of approximately 28,000 time-space positions were determined.

No problems were encountered in the data reduction process. Using the comparator, the rate of coordinate measurement was approximately 20 minutes per photograph. This entailed measuring the coordinates of 80 to 100 vehicles per photograph.

Using the metric camera and the monocomparator data reduction system, it is estimated that vehicle spacings were determined with an accuracy of ± 0.50 foot and vehicle velocities within ± 1.00 mph.
CHAPTER FOUR
CHARACTERISTICS OF PLATOON BEHAVIOR

4.1 Introduction

The purpose of this phase of the research is to identify behavioral characteristics of traffic traveling on signalized arterials. As was mentioned in the previous chapter, the sample data for this study consists of time and space information for 28 platoons. For the purposes of this analysis, a platoon is defined as those vehicles discharged during the go period of the initial signal. However, platoon size does not remain constant as the initial group of vehicles moves along the arterial because of additional vehicles picked up at other signals, because of lane changes, and because of low altitude photography. (With regard to the latter, the effect was noticeable only for large-sized platoons traveling at rather low speeds.) For these reasons, a platoon is to be considered a flexible number of vehicles reflecting actual traffic conditions.

In the analysis of platoon characteristics, no rigid requirements were established for distinguishing between platoon members and non-platoon members. In the data reduction stage, the photo coordinates of all vehicles appearing in a given lane of each photograph were measured. The vehicle trajectories for that lane were then constructed using all recorded vehicles.
These trajectories were then examined for purposes of defining a platoon. Vehicles were deleted from consideration if they were found to possess excessively large headways or gaps and their presence did not cause any noticeable changes in the behavior of the platoon members (as manifested in their trajectories). Because the aerial films recorded peak hour traffic conditions at both study sites, few vehicles had to be eliminated for these considerations. Thus, this analysis of platoon movement essentially uses those vehicles which would have actuated a series of detectors had they been located at each intersection in the lane of travel. This approach is considered valuable because any traffic control system which is developed for an urban arterial must rely on information supplied by on-site detectors. Hence, the traffic patterns established by such flexible-sized platoons is thought to be of considerable value.

Throughout the analysis of behavioral characteristics, lane identity was maintained. Each platoon is assigned a three digit number; the first two digits refer to film and section numbers and the third digit refers to the lane of travel. The lanes of each study site were numbered from left to right looking into the direction of travel. Thus Lane 1 represents the East traffic lane of Summit Street and the West traffic lane of North Fourth Street. Numbers were not assigned to parking lanes.

Each numbered platoon then represents a specified lane of travel. Of the 28 platoons studied, 6 traveled in Lane 1, 11 traveled in Lane 2 and 11 traveled in Lane 3. Data reduction in Lane 1 of the North Fourth Street study
site began just past the signal at Chittenden Avenue and continued to Hudson Street. All other lanes were continuous from the platoon formation point to the terminal signal.

4.2 Characteristics of Photographed Platoons

As a prelude to any analysis, characteristics of the 28 platoons were documented. All of the platoons consisted of passenger vehicles traveling on urban arterials during peak hour conditions. The traffic characteristics examined were platoon size, lane distribution, frequency of lane change maneuvers, and occurrences of traffic breakdowns.

4.2.1 Platoon Size

The photographed platoons were indexed according to platoon size. Table 4.1 presents the sizes of the initial platoons at the platoon formation points. Stopped vehicles were distinguished from those joining the queue of waiting vehicles. The observations were made at the instant the platoon was given the green signal indication. The figures indicate the wide variation in initial platoon sizes obtained. The larger-sized platoons were generally found during the evening peak on North Fourth Street whereas the smaller-sized platoons were generally found during the morning peak on Summit Street.

4.2.2 Lane Distribution

The distribution of traffic by lane of travel was also documented for the data collected. Table 4.2 indicates a reasonably uniform distribution of traffic
Table 4.1 Size of Initial Platoons

<table>
<thead>
<tr>
<th>Platoon Identification</th>
<th>Number of Stopped Vehs</th>
<th>Number of Vehs Joining Queue</th>
<th>Total Vehicles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platoon 212</td>
<td>4</td>
<td>2</td>
<td>6</td>
</tr>
<tr>
<td>213</td>
<td>6</td>
<td>1</td>
<td>7</td>
</tr>
<tr>
<td>232</td>
<td>3</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>233</td>
<td>4</td>
<td>2</td>
<td>6</td>
</tr>
<tr>
<td>312</td>
<td>10</td>
<td>8</td>
<td>18</td>
</tr>
<tr>
<td>313</td>
<td>6</td>
<td>12</td>
<td>18</td>
</tr>
<tr>
<td>332</td>
<td>21</td>
<td>4</td>
<td>25</td>
</tr>
<tr>
<td>333</td>
<td>23</td>
<td>12</td>
<td>35</td>
</tr>
<tr>
<td>342</td>
<td>15</td>
<td>0</td>
<td>15</td>
</tr>
<tr>
<td>343</td>
<td>24</td>
<td>13</td>
<td>37</td>
</tr>
<tr>
<td>351</td>
<td>NA*</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>352</td>
<td>28</td>
<td>13</td>
<td>41</td>
</tr>
<tr>
<td>353</td>
<td>28</td>
<td>14</td>
<td>42</td>
</tr>
<tr>
<td>451</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>452</td>
<td>18</td>
<td>9</td>
<td>27</td>
</tr>
<tr>
<td>453</td>
<td>15</td>
<td>11</td>
<td>26</td>
</tr>
<tr>
<td>711</td>
<td>4</td>
<td>3</td>
<td>7</td>
</tr>
<tr>
<td>712</td>
<td>9</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>713</td>
<td>6</td>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>721</td>
<td>9</td>
<td>2</td>
<td>11</td>
</tr>
<tr>
<td>722</td>
<td>11</td>
<td>2</td>
<td>13</td>
</tr>
<tr>
<td>723</td>
<td>10</td>
<td>2</td>
<td>12</td>
</tr>
<tr>
<td>731</td>
<td>5</td>
<td>2</td>
<td>7</td>
</tr>
<tr>
<td>732</td>
<td>7</td>
<td>5</td>
<td>12</td>
</tr>
<tr>
<td>733</td>
<td>7</td>
<td>7</td>
<td>14</td>
</tr>
<tr>
<td>741</td>
<td>3</td>
<td>3</td>
<td>6</td>
</tr>
<tr>
<td>742</td>
<td>9</td>
<td>1</td>
<td>10</td>
</tr>
<tr>
<td>743</td>
<td>10</td>
<td>0</td>
<td>10</td>
</tr>
</tbody>
</table>

*NA = Not Applicable
Table 4.2 Lane Distribution of North Fourth Street Traffic

<table>
<thead>
<tr>
<th>Identification</th>
<th>Lane 1</th>
<th>Lane 2</th>
<th>Lane 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Run 3-1</td>
<td>-</td>
<td>45%</td>
<td>55%</td>
</tr>
<tr>
<td>Run 3-3</td>
<td>-</td>
<td>48%</td>
<td>52%</td>
</tr>
<tr>
<td>Run 3-4</td>
<td>-</td>
<td>51%</td>
<td>49%</td>
</tr>
<tr>
<td>Run 3-5</td>
<td>38%</td>
<td>32%</td>
<td>30%</td>
</tr>
<tr>
<td>Run 4-5</td>
<td>36%</td>
<td>29%</td>
<td>35%</td>
</tr>
</tbody>
</table>
over the available lanes of travel for the North Fourth Street study site. Regardless of the number of lanes of data reduced, lane usage did not differ by more than 5 per cent from a uniform distribution. The results are not too surprising considering the data does reflect peak hour traffic conditions.

The lane distribution of Summit Street traffic is presented in Table 4.3. On those sections of street where 2 lanes of data were reduced, traffic tended to divide itself equally between the lanes of travel. However, as the table indicates, the 3-lane sections consistently exhibited a lower percentage of traffic traveling in Lane 1 than either Lane 2 or Lane 3. The only reasonable explanation which can be suggested is that the majority of traffic entered Summit Street from the West approach of Hudson Street and conveniently found itself in one of the two closest lanes of travel (that is, Lane 2 or Lane 3). This non-uniform split of traffic occurred on all of the Summit Street films.

4.2.3 Frequency of Lane Change Maneuvers

For the platoons photographed, the frequency of lane changes was noted. Lane change maneuvers were classified as either entrances into a lane or exits from a lane. The number of lane change occurrences observed appears in Table 4.4 for the 12 platoons of North Fourth Street and in Table 4.5 for the 16 platoons of Summit Street. Of the total lane changes which occurred on each street, approximately half of them involved the interaction between the reduced lanes of travel. The remaining fifty per cent of the lane changes consisted of traffic moving to the curb lanes for the purpose of entering a side street.
Table 4.3 Lane Distribution of Summit Street Traffic

<table>
<thead>
<tr>
<th>Identification</th>
<th>Lane 1</th>
<th>Lane 2</th>
<th>Lane 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Run 2-1</td>
<td>-</td>
<td>48%</td>
<td>52%</td>
</tr>
<tr>
<td>Run 2-3</td>
<td>-</td>
<td>49%</td>
<td>51%</td>
</tr>
<tr>
<td>Run 7-1</td>
<td>25%</td>
<td>40%</td>
<td>35%</td>
</tr>
<tr>
<td>Run 7-2</td>
<td>29%</td>
<td>34%</td>
<td>37%</td>
</tr>
<tr>
<td>Run 7-3</td>
<td>21%</td>
<td>37%</td>
<td>42%</td>
</tr>
<tr>
<td>Run 7-4</td>
<td>29%</td>
<td>35%</td>
<td>36%</td>
</tr>
<tr>
<td>Platoon Identification</td>
<td>Lane Change Maneuvers</td>
<td>Entrances</td>
<td>Exits</td>
</tr>
<tr>
<td>-----------------------</td>
<td>-----------------------</td>
<td>-----------</td>
<td>-------</td>
</tr>
<tr>
<td>Platoon 312</td>
<td>4</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>313</td>
<td>18</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>332</td>
<td>5</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>333</td>
<td>9</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>342</td>
<td>10</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>343</td>
<td>3</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>351</td>
<td>25</td>
<td>17</td>
<td>8</td>
</tr>
<tr>
<td>352</td>
<td>45</td>
<td>16</td>
<td>29</td>
</tr>
<tr>
<td>353</td>
<td>52</td>
<td>18</td>
<td>34</td>
</tr>
<tr>
<td>451</td>
<td>24</td>
<td>9</td>
<td>15</td>
</tr>
<tr>
<td>452</td>
<td>18</td>
<td>7</td>
<td>11</td>
</tr>
<tr>
<td>453</td>
<td>23</td>
<td>6</td>
<td>17</td>
</tr>
</tbody>
</table>
Table 4.5 Frequency of Lane Change Maneuvers for Summit Street Platoons

<table>
<thead>
<tr>
<th>Platoon Identification</th>
<th>Lane Change Maneuvers</th>
<th>Entrances</th>
<th>Exits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platoon 212</td>
<td>9</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>213</td>
<td>16</td>
<td>5</td>
<td>11</td>
</tr>
<tr>
<td>232</td>
<td>18</td>
<td>5</td>
<td>13</td>
</tr>
<tr>
<td>233</td>
<td>17</td>
<td>8</td>
<td>9</td>
</tr>
<tr>
<td>711</td>
<td>4</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>712</td>
<td>4</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>713</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>721</td>
<td>9</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>722</td>
<td>11</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>723</td>
<td>12</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>731</td>
<td>8</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>732</td>
<td>9</td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>733</td>
<td>17</td>
<td>8</td>
<td>9</td>
</tr>
<tr>
<td>741</td>
<td>3</td>
<td>3</td>
<td>0</td>
</tr>
<tr>
<td>742</td>
<td>4</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>743</td>
<td>8</td>
<td>4</td>
<td>4</td>
</tr>
</tbody>
</table>
The data suggests the frequency of lane change maneuvers to be a highly variable quantity with lane exits generally exceeding lane entrances. The highest incidence of lane changes occurred on North Fourth Street for Platoons 351, 352 and 353. These 3 platoons represent the largest-sized platoons of all platoons studied. An examination of their vehicle trajectories was made to determine if the recorded lane changes were location oriented.

For Platoon 351, the lane changes did not occur predominantly at any one location. For Platoon 352, the majority of the exits from Lane 2 occurred just past the Chittenden Avenue intersection. Of the 29 vehicles exiting from Lane 2, 23 of them entered Lane 1. The apparent explanation is that because of the heavy left turning movement of vehicles from Lane 1 onto Chittenden Avenue, vehicles in Lane 2 would not enter Lane 1 until past the Chittenden Avenue intersection.

For Platoon 353, more than half of the 34 lane exits occurred prior to or in the vicinity of 15th Avenue. Traffic is moving over to the right curb lane prior to exiting North Fourth Street via 17th Avenue. Access to Interstate 71 is provided by 17th Avenue thus tending to explain the heavy right turning movement at this location.

Platoons 452 and 453 also exhibited lane changes which appeared to be location oriented. Again, the Chittenden Avenue and 15th Avenue intersections were the locations where the frequency of lane changes was the highest. For all other platoons, the lane changes did not occur predominantly at any specific location.
The frequency of lane change maneuvers is a function of the distance over which a platoon moves. This makes the interpretation of the figures given in Tables 4.4 and 4.5 rather difficult since all platoons were not photographed over the same distances. For instance, Platoon 332 was photographed while traveling from 5th Avenue to Chittenden Avenue whereas Platoon 352 was photographed while traveling from 5th Avenue to Hudson Street.

To provide a truer picture of lane maneuvering, the frequency of lane change maneuvers (entrances plus exits) was calculated per 1000 foot section of travel. Table 4.6 contains the results for Summit Street and Table 4.7 contains the results for North Fourth Street.

Comparison of these two tables reveals that the higher incidence of lane changes occurred on North Fourth Street. Rarely did more than two entrances and/or exits occur within a 1000 foot section of Summit Street. The maximum frequency of lane change maneuvers recorded for North Fourth Street was slightly over four per 1000 foot section, a figure which does not seem alarmingly high.

4.2.4 Traffic Disturbances

The time-distance plots depicting traffic movement on the study sites provided a means of evaluating qualitatively traffic performance during peak hour conditions. By a visual examination of the vehicle trajectories, disturbances in the traffic stream were clearly evident and oftentimes their causes could be determined. An example will illustrate this point.
Table 4.6 Frequency of Lane Change Maneuvers
Per 1000 Feet for Summit Street
Platoons

<table>
<thead>
<tr>
<th>Platoon Identification</th>
<th>Lane Change Maneuvers/1000 Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platoon 212</td>
<td>1.15</td>
</tr>
<tr>
<td>213</td>
<td>2.05</td>
</tr>
<tr>
<td>232</td>
<td>1.91</td>
</tr>
<tr>
<td>233</td>
<td>1.77</td>
</tr>
<tr>
<td>711</td>
<td>0.62</td>
</tr>
<tr>
<td>712</td>
<td>0.61</td>
</tr>
<tr>
<td>713</td>
<td>0</td>
</tr>
<tr>
<td>721</td>
<td>1.55</td>
</tr>
<tr>
<td>722</td>
<td>1.83</td>
</tr>
<tr>
<td>723</td>
<td>2.00</td>
</tr>
<tr>
<td>731</td>
<td>1.21</td>
</tr>
<tr>
<td>732</td>
<td>1.41</td>
</tr>
<tr>
<td>733</td>
<td>2.57</td>
</tr>
<tr>
<td>741</td>
<td>0.54</td>
</tr>
<tr>
<td>742</td>
<td>0.74</td>
</tr>
<tr>
<td>743</td>
<td>1.48</td>
</tr>
</tbody>
</table>
Table 4.7 Frequency of Lane Change Maneuvers  
Per 1000 Feet for North Fourth  
Street Platoons

<table>
<thead>
<tr>
<th>Platoon Identification</th>
<th>Lane Change Maneuvers/1000 Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platoon 312</td>
<td>0.91</td>
</tr>
<tr>
<td>313</td>
<td>3.33</td>
</tr>
<tr>
<td>332</td>
<td>1.25</td>
</tr>
<tr>
<td>333</td>
<td>1.96</td>
</tr>
<tr>
<td>342</td>
<td>3.12</td>
</tr>
<tr>
<td>343</td>
<td>0.68</td>
</tr>
<tr>
<td>351</td>
<td>3.57</td>
</tr>
<tr>
<td>352</td>
<td>3.88</td>
</tr>
<tr>
<td>353</td>
<td>4.26</td>
</tr>
<tr>
<td>451</td>
<td>4.00</td>
</tr>
<tr>
<td>452</td>
<td>1.58</td>
</tr>
<tr>
<td>453</td>
<td>2.02</td>
</tr>
</tbody>
</table>
Figure 4.1 illustrates a 110 second portion of a set of vehicle trajectories for traffic traveling in one lane of North Fourth Street from 5th Avenue to 17th Avenue. Traffic signals are located at the six intersections identified on the drawing.

An examination of Figure 4.1 indicates the presence of three kinematic disturbances. The disturbance between 5th and 7th Avenues can be considered a shock wave which has propagated in a direction opposite to traffic movement resulting in the stoppage of 17 vehicles. The probable cause of the wave development is the conflict of the released queue at 5th Avenue (region of low density) with the vehicles waiting at or approaching the red signal at 7th Avenue (region of high density). An examination of the photographs revealed that the vehicles forming the initial queue at 7th Avenue entered North Fourth Street during the green phase on 5th Avenue. Obviously, the presence of an initial queue was not taken into consideration in determining the signal offset.

The disturbance which begins near Chittenden Avenue and is propagated back beyond 11th Avenue is obviously due to the presence of the 6-vehicle queue waiting during the red light at Chittenden. Sixteen vehicles experienced a stoppage or a slowdown.

A slight disturbance is evident from the trajectories approximately 300 feet prior to 15th Avenue. The probable cause appears to be the slowing down of vehicles prior to entering the right curb lane. No disturbance was evident at this location in Lane 2—the adjacent left-hand lane—indicating that the signal offset was not a factor in causing this disturbance.
Figure 4.1 Set of Vehicle Trajectories for North Fourth Street
In like manner, the trajectories of the 12 platoons traveling on North Fourth Street were examined. The incidences of vehicle slowdowns and stoppages were recorded. The following repetitive patterns were identified:

1. Slowdowns at 7th Avenue in the presence of 3-vehicle queues,
2. Slowdowns at the 11th Avenue-Chittenden Avenue intersection pair with slowdowns at Chittenden propagating back to affect traffic flow at 11th,
3. Slowdowns in the presence of an unusually high number of lane changes, and
4. Stoppages at Hudson Street.

The principal factor precipitating the slowdowns at 7th Avenue appeared to be the presence of waiting vehicles at 7th when the traffic released by the signal at 5th Avenue arrived. Only slight slowdowns were observed in the presence of 3- to 4-vehicle queues. However, when the 7th Avenue queues exceeded 6 vehicles per lane, momentary stoppages would occur.

The factors responsible for the congestion evident at the 11th Avenue-Chittenden Avenue couplet were identified as the combination of an improper signal offset and the presence of waiting vehicles at Chittenden Avenue. Between 11th and Chittenden Avenues, an 8.0 second offset was observed over a distance of only 346 feet. It became apparent from the trajectories that vehicles released from the signal at 11th Avenue would reach the queue of vehicles waiting at Chittenden Avenue before the latter signal turned green. It was noted that a 4.0 second offset was observed between these same two inter-
sections during the morning peak on Summit Street with no interference between intersections apparent.

Although the trajectories indicated that a high frequency of lane changes would cause a slowdown, this phenomenon occurred only twice for the platoons photographed. In both instances, only slowdowns and not stoppages were recorded.

On the other hand, the stoppages at the Hudson Street signal were frequent occurrences. Traffic movement on North Fourth Street at this location is impeded because of the backup of Eastbound traffic between the Hudson Street intersection and the adjacent freeway interchange. The dual right turning movements from North Fourth Street onto Hudson Street are severely restricted causing oftentimes 30-vehicle queues to form at the Hudson Street signal.

Examination of the trajectories for the 16 platoons traveling on Summit Street revealed smoother flow patterns than were found on North Fourth Street. However, congestion was evident at the following locations:

1. Slowdowns at Chittenden Avenue in the presence of queues ranging from 3 to 8 vehicles, and

2. Slowdowns at 5th Avenue in the presence of 3-vehicle queues.

As before, the probable cause was identified as the combination of an improper signal offset and the presence of queued vehicles. The 29.0 second offset between 7th and 5th Avenues was considered too long to efficiently accommodate the progression of traffic on Summit Street, particularly when queues were
present at 5th Avenue. It is to be noted that a 22.0 second offset was observed between the same two intersections on North Fourth Street. For queues of less than 4 vehicles, somewhat improved flow was noted for the shorter offset.

In summary, it was observed that traffic disturbances resulting in vehicle slowdowns or stoppages were caused by one or more of the following events:

1. An improper signal offset,
2. The presence of initial queues at interior signalized intersections,
3. High frequency of lane changes, and
4. A bottleneck precipitated by side street traffic at the terminal signal.

4.3 Time and Space Patterns Exhibited by Selected Traffic Parameters

The variation of platoon behavior was depicted by determining the patterns exhibited by various traffic parameters. It was felt that the identification of those parameters which exhibit a repetitive pattern as a platoon of traffic moves through a series of signalized intersections would aid toward achieving a better understanding of platoon behavior.

Nine parameters were selected for inclusion in the variation analysis. The parameters which were used to describe the behavior of a group of vehicles were as follows:

1. Traffic Density
2. Traffic Volume
3. Mean Velocity

4. Standard Deviation of Velocity

5. Coefficient of Variation of Velocity

6. Mean Spacing

7. Standard Deviation of Spacing

8. Coefficient of Variation of Spacing

9. Mean Time Headway

The first three parameters, traffic density, volume and velocity, were selected because in traffic flow literature, they are considered the fundamental parameters of a traffic stream. The standard deviation and coefficient of variation of velocity were selected as indicators of dispersion. As was noted in the literature review of Chapter Two, the coefficient of variation of velocity has both a theoretical and an empirical basis. It has been used at rural locations to characterize the rate at which a platoon diffuses and lengthens.

Mean spacing and its associated dispersion measures, the standard deviation of spacing and the coefficient of variation of spacing, were selected primarily to provide additional information with regard to traffic density. These measures of dispersion help to distinguish between similar levels of density.

Mean time headway is simply the reciprocal of traffic volume. Sometimes, a more regular pattern can be observed by viewing the variation of the reciprocal of a parameter rather than the variation of the parameter itself.
It has been observed that mean time headway due to its lower variance does exhibit more of a steady pattern. Although the information was readily available, the individual time headways of vehicles were not considered for analysis purposes. Using aerial photography data, a vehicle's time headway is obtained by the ratio of its spacing to the preceding vehicle and its velocity. For very low velocities, time headways become exorbitantly large causing any function of time headways to become biased. The interpretation of any associated dispersion measures then becomes extremely difficult.

The Omnitab programming system was used to supply time and space plots of each parameter because of its facility to produce plots on an on-line printer. Following a group of vehicles on a time basis permitted data analysis in relation to the beginnings of the green phases at each of the signalized intersections. Following a group of vehicles on a distance basis permitted data analysis in relation to the spacings between the signalized intersections.

Plotting the time and distance variation of the nine parameters for the 28 platoons resulted in a total of 504 individual graphs. Since space does not permit the inclusion of all of the graphs, selected plots of the relationships studied are included in the following sections, particularly those relationships which were found to be stable.

4.3.1 Space Patterns Exhibited by Selected Traffic Parameters

The variation of the nine parameters over distance was obtained for
each of the 28 platoons studied. Each street was divided into 200 foot regions and the values of the parameters within each region were calculated. For a given platoon, each region then contributed one ordinate value. These values were plotted versus distance traveled from platoon formation point. Because of the wide range of initial queue lengths, an abscissa value of 2100 feet was selected to represent the stopline at 5th Avenue on North Fourth Street and an abscissa value of 1100 feet was selected to represent the stopline at Maynard Avenue on Summit Street. All other stopline locations for the signalized intersections on each street are identified on the drawings.

A description of the distance relationships which were established in this phase of the research follows. Specific plots are used to illustrate the findings.

A. Traffic Density - Distance Relationship

The traffic density profile with distance is reflective of the conditions encountered as traffic moves from signal to signal. Figure 4.2 illustrates the increase in density at each signal for a platoon which encountered waiting vehicles at each of the signalized intersections. Different levels of density were evident in the initial platoon prior to discharge from the 5th Avenue signal. Because of vehicles arriving at the signal, the tail of the platoon was found to have a density of approximately 125 vehicles per mile which was much lower than the density found at the front of the platoon (approximately 200 vehicles per mile).

Due to lane changes, the presence of waiting vehicles, and changes
Figure 4.2 Distance Variation of Traffic Density for Platoon 332
in platoon size, a typical traffic density profile with distance did not exist for
the platoons studied.

B. Traffic Volume - Distance Relationship

The traffic volume measurement used in this analysis is really a
"derived volume," obtained by the product of traffic density and space mean
speed. Observations of the traffic volume-distance plots revealed that no
stable patterns could be identified for the platoons studied. The flow rate of
the platoons was found to fluctuate widely from section to section.

C. Mean Velocity - Distance Relationship

For all of the platoons evaluated, a cyclic relationship between
mean velocity and distance was observed with peaks occurring approximately
midway between signals. Local minima occurred at the signalized intersec-
tions.

The cyclic patterns were evident even in the absence of waiting
vehicles at the interior signalized intersections. However, in the presence of
queued vehicles, the cyclic behavior was quite pronounced. Figure 4.3 shows
the mean velocity pattern observed on North 4th Street for Platoon 343 and
Figure 4.4 shows the pattern observed on Summit Street for Platoon 232.
Although progression systems were in operation on these two streets, it is
apparent that traffic did not flow as one would predict from a theoretical
time-space diagram.

A recurrent observation made from examining the velocity-distance
plots was that platoons would undergo reductions in velocity in the immediate
Figure 4.3 Distance Variation of Mean Velocity for Platoon 343
Figure 4.4 Distance Variation of Mean Velocity for Platoon 232.
vicinity of signalized intersections. These reductions would be of the order of 3 to 5 miles per hour in the absence of queued vehicles and as high as 15 miles per hour in the presence of large queues. Because velocities were examined within 200 feet of each signal, the reductions were easily detected. It is interesting to note that no velocity reductions were apparent at any of the non-signalized intersections.

D. Standard Deviation of Velocity – Distance Relationship

The general tendency as reflected in the standard deviation of velocity-distance plots was for the standard deviation of velocity to decrease and operate within a small range (1 to 5 mph) as a platoon progressed along the arterial unless disturbed by other vehicles. Figure 4.5 shows the standard deviation pattern exhibited by Platoon 742, a platoon which traveled through the progression system at approximately 30 miles per hour.

Interpretation of traffic behavior from sole knowledge of the value of the standard deviation of velocity is difficult because a one to one relationship does not exist between the standard deviation and traffic performance. For example, an extremely low value of the standard deviation can indicate either uniform free flow conditions or congested traffic conditions. Obviously, the speed of the traffic is also an important variable.

E. Coefficient of Variation of Velocity – Distance Relationship

The coefficient of variation of velocity, defined as the ratio of the standard deviation of velocity and mean velocity, is a parameter which does seem to reflect traffic conditions. For congested conditions, it was established that the coefficient exhibits much higher values than for conditions of undisturbed
Figure 4.5 Distance Variation of Standard Deviation of Velocity for Platoon 742
platoon movement. For this reason, an examination of a coefficient of variation of velocity-distance plot can provide information regarding "how well" a platoon of traffic is moving through a series of signalized intersections.

To illustrate this point, consider first the pattern exhibited by Platoon 343 as shown in Figure 4.6. This platoon encountered no slowdowns or stopped vehicles upon release from the 5th Avenue signal. It is noted that beyond the platoon formation point, values of the coefficient remained within a narrow band of 5 to 15 per cent.

Now consider the coefficient of variation of velocity pattern exhibited by Platoon 353 as shown in Figure 4.7. In traveling from 5th Avenue to Hudson Street, Platoon 353 encountered queued vehicles at 7th, 11th, and Chittenden Avenues and encountered a slowdown because of lane changes at 15th Avenue. At each of these locations, values of the coefficient in excess of 40 per cent were found. Beyond 15th Avenue, the platoon encountered no further disturbances and moved without difficulty. Consequently, values of the coefficient over this distance did not exceed 15 per cent.

F. Mean Spacing-Distance Relationship

Observations of the mean spacing-distance plots revealed the cyclic behavior exhibited by mean spacing as platoons progressed through several signalized intersections. In fact, these curves oscillated in a manner quite similar to the mean velocity-distance curves. Comparison of Figures 4.8 and 4.3 which pertain to the same group of vehicles traveling on North Fourth
Figure 4.6 Distance Pattern of Coefficient of Variation of Velocity for Platoon 343
Figure 4.7 Distance Pattern of Coefficient of Variation of Velocity for Platoon 353
Figure 4.8 Distance Variation of Mean Spacing for Platoon 343
Street will verify this observation. The indication is that any change in platoon spacing is reflected in a change of platoon velocity.

Figure 4.9 shows a typical mean spacing pattern observed on Summit Street. Again a cyclic relationship is apparent as traffic adjusts and readjusts itself as it proceeds through the six signalized intersections.

G. Standard Deviation of Spacing - Distance Relationship

For the platoons studied, the standard deviation of spacing exhibited no discernible pattern over distance. It appears to be a very sensitive parameter able to change abruptly over very short distances. Wide fluctuations of the parameter were observed for the various platoons with the result that no stable patterns could be identified.

H. Coefficient of Variation of Spacing - Distance Relationship

Like the standard deviation of spacing, the coefficient of variation of spacing exhibited no discernible pattern over distance. There was no common tendency for the parameter to either increase or decrease with distance traveled. Very abrupt changes in the parameter values were recorded, particularly whenever the size of the platoon changed.

I. Mean Headway - Distance Relationship

Stable patterns of mean headway were recorded as platoons traveled on the signalized arterials. Figure 4.10 indicates that once Platoon 712 was discharged from the Maynard Avenue signal, the time headway varied in a rather small range about the 2.00 second value. Figure 4.11 shows the same result for platoon 343 traveling on North Fourth Street once it was
Figure 4.9 Distance Variation of Mean Spacing for Platoon 712
Figure 4.10 Distance Variation of Mean Headway for Platoon 712
Figure 4.11 Distance Variation of Mean Headway for Platoon 343
discharged from the signal at 5th Avenue. For both platoons, the minimum recorded mean headway was 1.7 seconds.

4.3.2 Time Patterns Exhibited by Selected Traffic Parameters

For each of the 28 platoons, the variation of the nine parameters over time was obtained. Parameter values were obtained for each photograph and plotted as ordinates at either one or three second time intervals, the particular value being dependent on the actual time observed between photographs. With the aid of the signal indicator device described in Chapter Three, the beginnings of the green phases at each of the signalized intersections were determined and their values indicated on the time plots.

A description of the time relationships which were observed follows. Again, specific plots are used to illustrate the findings.

A. Traffic Density - Time Relationship

The time behavior of traffic density is indicative of the amount of platoon spreading and contracting which occurs as a group of vehicles moves through a series of signalized intersections. Because the platoons studied did not encounter the same traffic conditions, the density profiles reflect a variety of shapes.

Figure 4.12 illustrates the traffic density-time relationship for Platoon 743. From an initial density of 186 vehicles/mile, traffic decreased to and operated within a small density range of 40 to 60 vehicles/mile. A slight jump in the density profile occurred at Lane Avenue because of the
Figure 4.12 Time Variation of Traffic Density for Platoon 743
addition to the platoon of four vehicles. Otherwise, the platoon remained intact while traveling through the six signalized intersections.

B. Traffic Volume – Time Relationship

Because of the wide fluctuations in the density patterns, the derived volume parameter was found to exhibit widespread variation over a time domain. This is clearly evident from the time pattern of traffic volume for Platoon 743 shown in Figure 4.13. The major volume reduction which occurred coincided with the beginning of the green phase at Lane Avenue because of a 4-vehicle queue at Lane. Beyond Lane Avenue, the platoon proceeded uninterrupted through the remaining signals. It is noted that extremely high volume rates can occur though lasting only short periods of time. Due to the second by second fluctuations of the volume parameter, not much information is provided with respect to the smoothness of traffic flow.

C. Mean Velocity – Time Relationship

The monitoring of mean velocity as a platoon of vehicles progresses through a series of signalized intersections can indicate how well the platoon adapts to the particular signal timing for the given traffic conditions encountered. Figure 4.14 illustrates the velocity–time profile for Platoon 232 traveling on Summit Street. From a stopped position, the platoon rapidly increased its velocity to an average maximum of 37 mph (Speed limit = 35 mph = design progression speed) but was eventually forced to reduce its velocity because of the continued red indication by the Lane Avenue signal. When the
Figure 4.13 Time Variation of Traffic Volume for Platoon 743
Figure 4.14 Time Variation of Mean Velocity for Platoon 232
platoon was finally given the green at Lane Avenue, its velocity had decreased to 24 mph.

After passing through the Lane Avenue intersection, the platoon recovered to its previous maximum velocity but was able to sustain it for only a short period of time. Because of the continued red indication at 17th Avenue and the presence of two waiting vehicles, the platoon was again forced to slow down to 24 mph. As the platoon moved further down the arterial, another similar slowdown was precipitated by a 3-vehicle queue at Chittenden Avenue. A major slowdown occurred at 5th Avenue. Approaching this intersection, the platoon was forced to reduce its velocity to 15 mph in the face of a 6-vehicle queue.

It is quite evident then that Platoon 232 did not adapt itself to the given signal timing for the traffic conditions encountered. Platoon spreading and the presence of initial queues appear to be two factors which were not taken into consideration in designing the signal progression system.

A platoon which fared somewhat better in traveling through the Summit Street signal system was Platoon 743. Its mean velocity profile is presented in Figure 4.15. The only major velocity reduction reflected in this time plot occurred at Lane Avenue in the presence of a 2-vehicle queue. Otherwise, the platoon progressed through the system at a mean velocity of about 30 mph.

D. Standard Deviation of Velocity – Time Relationship

Although an examination of the standard deviation of velocity-time
Figure 4.15 Time Variation of Mean Velocity for Platoon 742
plots for the various platoons revealed no definite patterns, a few characteristics regarding the parameter values could be made. For undisturbed movement, the platoons generally exhibited standard deviations from 1 to 6 mph. This range would be exceeded only upon starting from a stopped position or when the platoon encountered disturbances in its lane of travel.

The extremely large-sized platoons exhibited very little change in the value of the standard deviation parameter over time. For example, Figure 4.16 illustrates the time pattern of the standard deviation of velocity for Platoon 351. This 15-vehicle platoon, formed in Lane 1 just past the Chittenden Avenue intersection, underwent moderate platoon spreading once given the green signal at 15th Avenue. The platoon then settled down to exhibit a standard deviation of velocity value of around 4 mph. Very little change in the parameter value occurred until the platoon encountered a backup at the Hudson Street signal. Much higher standard deviation values then occurred.

E. Coefficient of Variation of Velocity - Time Relationship

The coefficient of variation of velocity is a parameter which provides a valid comparison of platoon spreading for platoons traveling at different velocities. Figures 4.17 and 4.18 illustrate the coefficient of variation of velocity patterns displayed by Platoons 711 and 712 respectively. A comparison of these plots reveals that the 12-vehicle Platoon 712 exhibited greater variability in velocities than the 7-vehicle Platoon 711 upon discharge from the Maynard Avenue signal. The patterns for these platoons traveling in adjacent traffic lanes were then quite similar until the platoons reached the vicinity of
Figure 4.16 Time Variation of Standard Deviation of Velocity for Platoon 351
Figure 4.17 Time Pattern of Coefficient of Variation of Velocity for Platoon 711
Figure 4.18 Time Pattern of Coefficient of Variation of Velocity for Platoon 712
15th Avenue. The front of Platoon 711 was forced to slow down after the beginning of the green phase at 15th Avenue resulting in the higher values of the coefficient than were exhibited by Platoon 712. This latter platoon encountered no disturbance in its lane of travel at this location. As the diagrams indicate, both platoons were forced to slow down coincident with the beginning of the green phase at 7th Avenue.

F. Mean Spacing - Time Relationship

Mean spacing is a much more sensitive parameter to changing traffic conditions than mean velocity. Changes in platoon size are directly reflected in the mean spacing parameter. Owing to the aerial data collection technique, abrupt changes in the mean spacing parameter occur when an additional vehicle first appears on the leading edge of a photograph. For this reason, jumps occur in the corresponding mean spacing-time plots. However, for those platoons which did not undergo much variation in platoon size, the mean spacing profiles indicated reasonably similar patterns.

The variation of mean spacing with time for a Summit Street platoon is shown in Figure 4.19. As the platoon moved through the six-signal system, mean platoon spacing fluctuated between 80 and 135 feet.

The variation of mean spacing for a tighter platoon traveling on North Fourth Street is shown in Figure 4.20. For Platoon 343, the range of mean spacing was from 38 to 94 feet.

G. Standard Deviation of Spacing - Time Relationship
Figure 4.19 Time Variation of Mean Spacing for Platoon 742
Figure 4.20 Time Variation of Mean Spacing for Platoon 343
Because of the variable platoon size, the standard deviation of spacing did not portray any type of non-random pattern over time.

II. Coefficient of Variation of Spacing – Time Relationship

The coefficient of variation of spacing exhibited too many abrupt changes in its time plots for any kind of pattern to be identified.

I. Mean Headway – Time Relationship

Stable patterns of mean headway with time were observed.

Figure 4.21 illustrates the time pattern of mean headway for a platoon traveling on Summit Street. As the diagram indicates, the platoon consistently exhibited a 1.9 second headway except at 7th Avenue where a number of lane changes occurred.

The mean headway-time pattern for a platoon traveling on North Fourth Street is shown in Figure 4.22. Again, a headway value around 2.00 seconds appears to be the modal value. Once the traffic received the green at 19th Avenue, a considerable amount of lane changing occurred resulting in a 9-vehicle reduction in platoon size. The remaining 6-vehicle platoon underwent spreading until it eventually ran into a queue of stopped vehicles at the Hudson Street signal.

4.4 Analysis of Platoon Characteristics

From the review of the literature, it was established that two important variables affecting the behavior of traffic movement on urban arterials are signal spacing and signal offset. The fact that these two variables influence
Figure 4.21 Time Variation of Mean Headway for Platoon 712
Figure 4.22 Time Variation of Mean Headway for Platoon 453
platoon behavior also became evident from the parameter variation analysis. Distance patterns were interpreted in relation to the spacing between signalized intersections and time patterns were interpreted with respect to the values of the signal offsets.

Two other variables were investigated to determine their influence, if any, on platoon behavior. These were platoon size and lane of travel. In addition, an investigation of traffic behavior was conducted treating traffic density as the independent variable.

4.4.1 Effect of Platoon Size

To demonstrate whether platoons of different sizes behave differently under essentially the same conditions, mean velocity was monitored for platoons of various sizes emanating from a signalized intersection. To eliminate any possible bias due to lane of travel, platoons selected for this analysis were restricted to a common lane. The common lane selected was the middle traffic lane thereby eliminating any effect caused by the presence of parked vehicles along the curb lanes. Also, care was taken to select platoons which did not undergo any lane changes for the length of period studied.

These conditions led to the selection of four platoons traveling in Lane 2 of Summit Street. For each of the constant-sized platoons selected, mean velocities were determined at intervals of three seconds once the platoon was discharged from the Maynard Avenue intersection. A plot of the resulting velocity patterns is given in Figure 4.23. The behavior of the platoons was
Figure 4.23 Velocity-Time Patterns for Platoons of Various Sizes
recorded for a 25 second time period. Beyond this point several of the platoons underwent lane changes. Signal offset was not considered a factor affecting the behavior of the platoons since the observed offset of 39 seconds far exceeds the 25 second time period studied.

The patterns appearing in Figure 4.23 are not to be considered typical of all Summit Street traffic since they apply to only four specific platoons at a particular location. Rather, they are presented to demonstrate the effect of platoon size on platoon behavior as manifested in this instance by platoon velocity.

The figure illustrates several important features regarding platoon movement. First, platoon size does have a measurable effect on platoon behavior. Specifically, platoon velocity tends to decrease with increasing platoon size. A maximum velocity differential of approximately 20 miles per hour was observed 13 seconds past the beginning of green and this differential decreased to approximately 6 miles per hour at the 25 second point. The implication is that the velocity differential will continue to decrease until all platoons are traveling at or about the 35 mph posted speed limit.

Secondly, the graph indicates that the initial acceleration characteristics of the smaller-sized platoons (4 to 6 vehicles) are substantially different from the characteristics of the larger-sized platoons (10 to 13 vehicles). Extreme values observed were 0.5 g and 0.2 g over the first three seconds.
4.4.2 Effect of Lane of Travel

Data presented at the beginning of this chapter indicated that drivers traveling on the study sites tended to distribute themselves equally over the available lanes of travel barring any unusual circumstances. Also it was observed from the aerial films that queues found in the different approach lanes at any given signalized intersection were found to be of approximately the same size. Lane of travel then seems to have no apparent effect on platoon size.

An investigation was conducted to determine whether platoons of approximately the same size traveling in adjacent traffic lanes on the same street at the same time would behave in a similar manner or would exhibit significantly different behavioral characteristics. In other words, can any observed differences in platoon behavior be attributed to lane of travel?

An analysis of variance procedure was used to test for a lane effect. Profiles of mean velocity over distance for platoons traveling side by side were compared statistically. The variables of signal offset and signal spacing were assumed to affect the different lane platoons equally. A specific example will illustrate the statistical procedure employed.

Consider the velocity profiles presented in Figure 4.24. These velocity profiles were exhibited by Platoons 342 and 343 traveling in adjacent lanes through three signalized intersections of North Fourth Street. Because of the presence of queued vehicles at 7th and 11th Avenues, both platoons which were discharged from the signal at 5th Avenue encountered slowdowns. The patterns
Figure 4.24 Mean Velocity Profiles for Platoons 342 and 343
appear to be quite similar to the observer's eye. To determine whether the patterns can be considered similar in the statistical sense, the hypothesis to be tested is whether the mean difference in velocities is zero. Based on the assumption that velocity differences obey the normal distribution, the test statistic for determining acceptance or rejection of the hypothesis is given by the following equation:

\[
F = \frac{\frac{1}{n} \sum_{i=1}^{n} y_i^2}{\frac{1}{n-1} \sum_{i=1}^{n} (y_i - \bar{y})^2}
\]

where \( F \) = numerical value of test statistic
\( n \) = sample size
\( y_i \) = velocity difference at location \( i \)
\( \bar{y} \) = mean velocity difference

A brief derivation of the test statistic is given in the Appendix where it is shown that the statistic is distributed according to the F-distribution having \( n \) and \( n-1 \) degrees of freedom.

For the two platoons under investigation, the numerical data presented in Table 4.8 reveals the observed mean velocity difference to be 2.31 miles per hour. Calculations will reveal the computed F value to be 1.35, which is less than the critical F value of 2.32 at the 5 per cent level of significance. Therefore, the hypothesis that the population mean velocity difference is zero can be accepted. The interpretation is that lane of travel has no significant effect on the behavior of Platoons 342 and 343.
Table 4.8 Velocity Measurements for Platoons 342 and 343

<table>
<thead>
<tr>
<th>n</th>
<th>Platoon Velocity, mph</th>
<th>Velocity Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>342</td>
<td>343</td>
</tr>
<tr>
<td>1</td>
<td>1.86</td>
<td>3.47</td>
</tr>
<tr>
<td>2</td>
<td>7.51</td>
<td>11.79</td>
</tr>
<tr>
<td>3</td>
<td>14.95</td>
<td>21.77</td>
</tr>
<tr>
<td>4</td>
<td>19.90</td>
<td>27.53</td>
</tr>
<tr>
<td>5</td>
<td>25.75</td>
<td>30.37</td>
</tr>
<tr>
<td>6</td>
<td>28.75</td>
<td>31.49</td>
</tr>
<tr>
<td>7</td>
<td>19.50</td>
<td>27.51</td>
</tr>
<tr>
<td>8</td>
<td>14.98</td>
<td>22.04</td>
</tr>
<tr>
<td>9</td>
<td>23.64</td>
<td>26.15</td>
</tr>
<tr>
<td>10</td>
<td>28.47</td>
<td>29.29</td>
</tr>
<tr>
<td>11</td>
<td>30.27</td>
<td>29.83</td>
</tr>
<tr>
<td>12</td>
<td>32.76</td>
<td>30.80</td>
</tr>
<tr>
<td>13</td>
<td>33.32</td>
<td>30.08</td>
</tr>
<tr>
<td>14</td>
<td>31.50</td>
<td>29.74</td>
</tr>
<tr>
<td>15</td>
<td>27.89</td>
<td>26.79</td>
</tr>
<tr>
<td>16</td>
<td>18.76</td>
<td>21.18</td>
</tr>
<tr>
<td>17</td>
<td>22.50</td>
<td>21.77</td>
</tr>
<tr>
<td>Mean</td>
<td>22.49</td>
<td>24.80</td>
</tr>
</tbody>
</table>
This statistical procedure was applied to the 12 platoons traveling on North Fourth Street and the 14 platoons traveling on Summit Street. Differences in velocities at 200 foot intervals were obtained for the platoons traveling in each lane over a common length of street. For those sections of street in which two lanes of data were available pairwise velocity differences were computed. If three lanes of data were to be compared, the three possible lane pairs were checked individually.

A summary of the total test results for North Fourth Street is presented in Table 4.9. Although the mean velocities in Lane 3 were generally higher than those in Lane 2 which, in turn, were generally higher than those in Lane 1, the pairwise differences were not found to be statistically significant.

A summary of the test results for Summit Street is presented in Table 4.10. Although Lane 1 exhibited slightly higher mean velocities than Lane 2, which, in turn, exhibited higher mean velocities than Lane 3, again the differences are not considered statistically significant.

Based on the data analyzed, it can be concluded that lane of travel exhibits no significant effect on platoon behavior. The importance of this conclusion will be demonstrated in the succeeding chapter.

4.4.3 Traffic Density Relationships

In Section 4.4.1, platoon size was shown to be an important variable affecting platoon behavior. Knowledge of platoon size however, does not in itself indicate anything about traffic performance. For a constant-sized
<table>
<thead>
<tr>
<th>Sample Size</th>
<th>Mean Velocity, mph</th>
<th>FCR</th>
<th>F</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>312 - 313</td>
<td>23</td>
<td></td>
<td>0.96</td>
<td>accept</td>
</tr>
<tr>
<td>332 - 333</td>
<td>19</td>
<td>25.77</td>
<td>2.04</td>
<td>accept</td>
</tr>
<tr>
<td>342 - 343</td>
<td>17</td>
<td>22.04</td>
<td>0.98</td>
<td>accept</td>
</tr>
<tr>
<td>351 - 352</td>
<td>36</td>
<td>2.31</td>
<td>2.21</td>
<td>accept</td>
</tr>
<tr>
<td>351 - 352</td>
<td>36</td>
<td>4.75</td>
<td>1.74</td>
<td>accept</td>
</tr>
<tr>
<td>451 - 452</td>
<td>31</td>
<td>1.32</td>
<td>1.74</td>
<td>accept</td>
</tr>
<tr>
<td>451 - 452</td>
<td>31</td>
<td>1.74</td>
<td>1.74</td>
<td>accept</td>
</tr>
<tr>
<td>452 - 453</td>
<td>31</td>
<td>1.85</td>
<td>1.85</td>
<td>accept</td>
</tr>
<tr>
<td>452 - 453</td>
<td>31</td>
<td>1.85</td>
<td>1.85</td>
<td>accept</td>
</tr>
</tbody>
</table>

Table 4.9 Lane Effect Test Results - North Fourth Street
<table>
<thead>
<tr>
<th>Platoons</th>
<th>Sample Size</th>
<th>Mean Velocity, mph</th>
<th>Mean Diff.</th>
<th>F</th>
<th>F&lt;sub&gt;CR&lt;/sub&gt;</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>212 - 213</td>
<td>40</td>
<td>29.58 28.25</td>
<td>1.46</td>
<td>1.04</td>
<td>1.70</td>
<td>accept</td>
</tr>
<tr>
<td>232 - 233</td>
<td>46</td>
<td>30.62 27.55</td>
<td>3.07</td>
<td>1.29</td>
<td>1.64</td>
<td>accept</td>
</tr>
<tr>
<td>711 - 712</td>
<td>33</td>
<td>29.54 26.74</td>
<td>2.80</td>
<td>1.47</td>
<td>1.80</td>
<td>accept</td>
</tr>
<tr>
<td>711 - 713</td>
<td>33</td>
<td>29.54 25.53</td>
<td>4.01</td>
<td>1.60</td>
<td>1.80</td>
<td>accept</td>
</tr>
<tr>
<td>712 - 713</td>
<td>33</td>
<td>26.74 25.53</td>
<td>1.21</td>
<td>1.12</td>
<td>1.80</td>
<td>accept</td>
</tr>
<tr>
<td>721 - 722</td>
<td>30</td>
<td>27.82 26.48</td>
<td>1.34</td>
<td>1.03</td>
<td>1.85</td>
<td>accept</td>
</tr>
<tr>
<td>721 - 723</td>
<td>30</td>
<td>27.82 25.04</td>
<td>2.79</td>
<td>1.22</td>
<td>1.85</td>
<td>accept</td>
</tr>
<tr>
<td>722 - 723</td>
<td>30</td>
<td>26.48 25.04</td>
<td>1.44</td>
<td>1.03</td>
<td>1.85</td>
<td>accept</td>
</tr>
<tr>
<td>731 - 732</td>
<td>33</td>
<td>27.85 26.52</td>
<td>1.33</td>
<td>1.02</td>
<td>1.80</td>
<td>accept</td>
</tr>
<tr>
<td>731 - 733</td>
<td>33</td>
<td>27.85 27.46</td>
<td>0.39</td>
<td>0.98</td>
<td>1.80</td>
<td>accept</td>
</tr>
<tr>
<td>732 - 733</td>
<td>33</td>
<td>26.52 27.46</td>
<td>0.94</td>
<td>0.99</td>
<td>1.80</td>
<td>accept</td>
</tr>
<tr>
<td>741 - 742</td>
<td>28</td>
<td>31.52 29.21</td>
<td>2.31</td>
<td>1.12</td>
<td>1.90</td>
<td>accept</td>
</tr>
<tr>
<td>741 - 743</td>
<td>28</td>
<td>31.52 29.00</td>
<td>2.52</td>
<td>1.15</td>
<td>1.90</td>
<td>accept</td>
</tr>
<tr>
<td>742 - 743</td>
<td>28</td>
<td>29.21 29.00</td>
<td>0.21</td>
<td>0.97</td>
<td>1.90</td>
<td>accept</td>
</tr>
</tbody>
</table>
platoon, platoon characteristics will vary depending on the nature of traffic conditions encountered.

A parameter which does take into account platoon size as well as the amount of platoon spreading is traffic density. Usually expressed as vehicles per mile of roadway, traffic density is considered a sensitive indicator of traffic change. That this is the case is borne out by the fact that vehicle occupancy, a linear function of traffic density, is currently being used to control traffic on urban freeways (15, 16).

In Section 4.3, valuable information was provided by the time and distance variations of the different parameters investigated. At that time, several observations were made regarding how well certain of the parameters reflected actual traffic conditions. For example, it was suggested that the coefficient of variation of velocity may be a valid indicator of traffic conditions.

To quantify such an observation, it was decided to treat traffic density as an independent variable and to observe the changes in the dependent variables of mean velocity, traffic volume, standard deviation of velocity, and coefficient of variation of velocity.

The relationships established in this section are necessarily restricted to the density range observed on Summit and North Fourth Streets. Vehicle trajectories for all platoons were examined in three specific regions: vehicles starting from a stopped position, vehicles moving at a constant velocity, and vehicles slowing to a stopped position.

Vehicles operating in the region characterized by parallel constant
speed trajectories referred to hereafter as the "steady state region" exhibited density values from 40 to 70 vehicles per mile. Vehicles starting from or slowing to a stopped position exhibited density values from 60 to 170 vehicles per mile. Thus for the density domain studied (40 to 170 vehicles per mile), values of the dependent variables were evaluated and relationships with traffic density were established.

A. Mean Velocity - Density Relationship

In Figure 4.25, the decrease in mean speed with increasing traffic density is apparent. For the density range studied, it was found that a second degree equation gives a much better fit to the observed data than a linear relationship. The theoretical speed-density relationship formulated assumes the following form:

\[ u = 0.0015k^2 - 0.57k + 56 \]

where \( u \) = mean velocity in miles per hour

\( k \) = traffic density in vehicles per mile.

Because of the limited density range studied, the equation cannot be used to predict any boundary values such as free flow speed or jam density. Independent estimates of these important parameters must be made.

Free flow speed can simply be estimated as 35 miles per hour, the speed limit of the two streets from which the data was gathered. An estimate of jam density can be made from actual photographic measurements of vehicles in stopped positions. This procedure was used and the results, which are
Figure 4.25 Observed and Theoretical Speed-Density Relationships
presented in Chapter Five, indicated maximum density to be 231 vehicles per mile.

B. Traffic Volume - Density Relationship

For the same set of data, the volume-density relationship is presented in Figure 4.26. Defining volume as the product of mean speed and density, the plotted curve is the theoretical curve derived from the aforementioned speed-density relationship. The derived curve provides a maximum volume of 1635 vehicles per hour at an optimum density of 65 vehicles per mile. The corresponding velocity is 25 miles per hour.

In the parameter variation analyses, undisturbed traffic was seen to operate in the approximate density range of 50 to 70 vehicles per mile. In this density range, Figure 4.26 indicates volume observations ranging from 1400 to 1800 vehicles per hour. Because of this wide variation in volume, the 1635 maximum volume figure is not to be considered a reliable estimate of maximum throughput.

C. Standard Deviation of Velocity - Density Relationship

The standard deviation of velocity-density relationship presented in Figure 4.27 confirms the observation made in Section 4.3.1 that a one to one relationship does not exist between the standard deviation of velocity and traffic performance. Although no theoretical curve is presented, it is apparent from the trend of the plotted points that the same value of the standard deviation can occur at two distinct density levels. For the data gathered, maximum
Figure 4.26 Observed and Theoretical Volume-Density Relationships
dispersion in velocities occurs in the neighborhood of 100 vehicles per mile. Low dispersions are associated with traffic operating within the steady state region - thus tending to confirm the name applied to the region of parallel, constant speed trajectories.

D. Coefficient of Variation of Velocity - Density Relationship

From the literature review and the parameter variation analysis, the coefficient of variation of velocity was suggested to be a parameter which can distinguish different traffic conditions. That this is the case is borne out by the plot appearing in Figure 4.28.

It is evident from the plotted points that the higher values of the coefficient are associated with high density, and hence, congested traffic conditions. The higher the value of the coefficient, the greater the amount of congestion. On the other hand, the low values are associated with steady state traffic conditions.

4.4.4 Effect of Signal Visibility

Another factor thought to affect traffic performance through a progressive signal system is signal visibility. It seems probable that drivers will react more appropriately to a signal system if the signals can be observed far in advance of an intersection than otherwise. In a recent study conducted by Bleyl, deviations in speed profiles for traffic approaching a signalized intersection were apparent when drivers first observed a red signal indication (17). Although the signal was first observed at a point 1200 feet from the intersection,
the changes in velocities which occurred were not considered significant until
the drivers were within 500 feet of the intersection.

Bleyle's results were based on the movement of isolated vehicles.
Traffic volume is also an important variable which may mitigate the effect of
signal visibility. In a study by Desrosiers and Leighty, it was established
that drivers did not adapt to an unannounced change in progression speed of an
urban signal system from 27 to 33 miles per hour after a period of two months
nor to a change in progression speed from 33 to 40 miles per hour after a
period of one month. (18) The authors observed that large traffic volumes
were a limiting factor in the ability of drivers to adapt to a change in speed
of progression.

For the two study sites investigated in this project, signal spacings
ranged from 0.07 mile to 0.47 mile with no appreciable changes in vertical or
horizontal alignment present. It was determined that for all signalized inter-
sections studied, the signals at any given intersection were clearly visible
from the immediately preceding signalized intersection. Consequently, the
effect of signal visibility on platoon behavior could not be investigated. All of
the results found in this project, then, must be interpreted under these
conditions.

4.5 Conclusions

Based on the analyses of characteristics of passenger car platoons
traveling on the one-way signalized arterials of Summit and North Fourth Streets
during peak traffic hours, the following findings can be stated:

1. Traffic disturbances resulting in vehicle slowdowns or stoppages as traffic moves through a progressive signal system can be caused by an improper signal offset, the presence of initial queues at interior signalized intersections, an unusually high frequency of lane changes at a specific location, or a bottleneck precipitated by side street traffic.

2. The principal variables affecting platoon behavior as traffic moves through a progressive signal system are signal spacing, signal offset, and platoon size. (Signal visibility, roadway gradient, and amount of commercial traffic as factors affecting platoon movement were not investigated in the current project because of their inapplicability to the study sites investigated.)

3. Lane of travel exhibits no significant effect on the behavior of platoons traveling through a linear signal system.

4. For traffic traveling through a coordinated signal system, stable patterns of platoon velocity, mean spacing, mean headway, and the coefficient of variation of velocity with distance traveled were observed. The patterns of mean velocity and mean spacing were found to be cyclic in nature with peaks occurring between signalized intersections.

5. The coefficient of variation of velocity is considered to be a valid indicator of traffic conditions.
6. Platoon movement along an urban arterial can best be characterized by patterns of mean velocity or mean spacing, the coefficient of variation of velocity, and traffic density.
CHAPTER FIVE

DEVELOPMENT OF SIMULATION MODEL

5.1 Introduction

The principal variables affecting platoon movement on one-way signalized arterials were identified in the previous chapter. The task at hand is to incorporate these variables into a mathematical model so that signal settings can be determined allowing for the dispersion of traffic. The purpose of this phase of the research, then, is to simulate the behavior of a group of vehicles as it passes through a series of signalized intersections. To accomplish this objective, the approach taken was to develop the model in a series of stages. The first stage was concerned with the development of a single intersection model, the second stage was concerned with the development of a two-intersection model, and the third stage was concerned with the development of a multi-intersection model.

The models are written in a special-purpose simulation language termed GPSS (General Purpose Simulation System). This language is ideally suited for studying macroscopic systems, such as the behavior of queues. The advantages of using GPSS are the speed with which programs can be written and the ease of learning the language. All programs have been run on the IBM 370/165.
In the initial stages of model development, data from the aerial films was not available for incorporation into the models. For this reason, the models constructed were analyzed to determine the correctness of the logic used and to determine their adequacy in giving realistic results of signalized intersection operation.

5.2 Development of Single Intersection Model

The British concept of signalized intersection operation was employed in model development. Consider a queue of vehicles waiting at a signalized intersection. During the green and amber periods, the pattern of queue discharge is as shown in Figure 5.1. As depicted in the diagram, the rate of flow is lower during the first few seconds of the green phase (due to starting delays) and during the amber phase (due to the deceleration of the queue). Otherwise, the queue discharges during the green period at a more or less constant rate, termed saturation flow. Saturation flow can be defined as the flow which would be obtained if there were an infinite queue of vehicles and they were given a continuous green.

During the portion of the green and amber periods, the time lost due to starting delays and to deceleration of the queue is termed "lost time." Defining effective green time as the time during which saturation flow is assumed to take place, the discharge pattern shown in Figure 5.1 can be replaced by a rectangle, the base of which is equal to effective green time and the vertical sides of which are equal to saturation flow. In this manner, the summation of effective green time and lost time would equal actual green time.
Figure 5.1 Flow Discharge Pattern of Signalized Intersection
(the amount of green time displayed by the signal) plus amber. Thus, fixed-time signalized intersection operation can be depicted on a flow-time diagram as shown in Figure 5.2. The quantity represented by the area of a rectangle equals the number of vehicles discharged during one cycle.

The model developed to simulate traffic operations at a fixed-time signalized intersection consists of five major components. A brief description of each component follows.

1. Signal Timing Subroutine - This subroutine is used to represent the cyclic operation of a pretimed traffic signal. Serving as a continually running clock, this subroutine stops all movement at the intersection for a period corresponding to the effective red time of the signal (the amount of red time displayed by the signal plus lost time) and permits vehicle movement through the intersection for a period equal to the effective green time of the signal. Cycle length and cycle split thus completely identify the particular subroutine used.

2. Function Definition Cards - This phase of the computer program consists of the definition of all mathematical functions used in the model. For intersection operation, both the arrival and service mechanisms must be specified. Probability distributions are easily incorporated in the model by specifying their cumulative distribution functions. Random number generators are used to sample from the defined distributions.

3. Main Program - This phase is used to relate the different components of the model so that vehicles can be processed for the purposes of
Figure 5.2 Fixed-time Signalized Intersection Model
simulation. For the intersection model, traffic is assumed to arrive on a random basis. Vehicles queue up if they arrive during the red phase of the signal or during the green phase in the presence of a queue. Traffic is assumed to depart from the queue according to a saturation flow discharge pattern. Vehicles which arrive during the green phase in the absence of waiting vehicles proceed through the intersection without delay.

4. Table Definition Cards - The statistics of the simulation to be gathered in table form are specified in this phase. Distributions of delay and time spent in the system are obtained quite readily in table format.

5. Control Cards - Control cards are used to provide proper dimensioning of all variables incorporated in the model and to provide limits on the length of simulation desired. The duration of the simulation can be governed either by the number of vehicles to be processed or by the real time to be simulated.

Numerical values were assigned to the model variables to permit manipulation of the simulation program. This program simulates the behavior of vehicles as they proceed through one lane of a fixed-time signalized intersection. Turning and opposing traffic are not considered.

To simulate one lane of an isolated signalized intersection, the following characteristics were used:

Random arrival rates:  
1 veh per 7 secs (515 vph)  
1 veh per 6 secs (600 vph)  
1 veh per 5 secs (720 vph)
Constant rate of saturation: 1 veh per 2 secs.

The simulation of 500 vehicles through the intersection for each of the volume levels and for a cycle length of 60 seconds (30 secs effective green) and for a cycle length of 90 seconds (42 secs effective green) gave the following results for the time spent in the queue:

<table>
<thead>
<tr>
<th>Volume Level</th>
<th>60 sec Cycle</th>
<th>90 sec Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean Delay per Veh</td>
<td>(\sigma)</td>
</tr>
<tr>
<td>515 vph</td>
<td>11.78 secs</td>
<td>10.91</td>
</tr>
<tr>
<td>600 vph</td>
<td>13.68</td>
<td>10.84</td>
</tr>
<tr>
<td>720 vph</td>
<td>18.90</td>
<td>13.09</td>
</tr>
</tbody>
</table>

The stopped-time delay distribution corresponding to the 90 sec cycle at the 600 vph volume level is shown in Figure 5.3. This distribution can be considered a typical delay distribution for traffic at a fixed-time signalized intersection. Certain features are noteworthy: 1) the rather high percentage of vehicles which proceeded through the intersection with little or no delay, 2) the rather low percentage of vehicles which experienced delays approaching the duration of the cycle length, and 3) the rather high variance associated with a delay distribution.

Theoretical methods for predicting mean delay per vehicle at fixed-time signalized intersections have been well established by Webster, Newell, Blunden, Miller, and Beckmann, McGuire and Wisten (19, 20, 21, 22, 23). Of these methods, Webster's delay formula is the most well known and accepted.
Volume = 600 v.p.h.
90 second cycle
500 Observations
\( \mu_d = 24.81 \)
\( \sigma_d = 19.19 \)

Figure 5.3 Frequency Distribution of Delay at a Signalized Intersection
Webster derived an empirical formula to relate average delay per
vehicle to length of green phase, cycle length, traffic volume and saturation
flow. The specific relationship is given as follows:
\[
\mu_d = \frac{c(1 - \lambda)^2}{2(1 - \lambda x)} + \frac{x^2}{2q(1 - x)} - 0.65 \frac{c^3}{q^2} \frac{x^2 + 5\lambda}{3}
\]
where \( \mu_d \) = mean delay per vehicle, secs
\( c = \) cycle length, secs
\( \lambda = g/c, \) proportion of cycle which is effectively green
\( q = \) traffic volume, vehicles per sec
\( s_e = \) saturation flow, vehicles per sec
\( x = \frac{q}{\lambda s_e}, \) degree of saturation.

The first term of this equation represents the delay for uniform arrivals, the
second term accounts for the randomness of the arrivals, and the third term
is an empirical correction term.

To determine whether the simulation model produced realistic delays,
a comparison was made with the delays obtained by Webster's equation. The
basic assumption of Webster's model is random arrivals. To duplicate this
in the simulation model, arrivals were considered to follow a Poisson distribu-
tion, or equivalently, interarrival times were considered to follow a negative
exponential distribution. To avoid unrealistic low headways, the shifted
exponential distribution was used with a shift factor of 1.00 second. A mini-
mum headway of 1.00 second agrees with studies conducted by Bieyl, Gerlough,
and Kell (24, 25, 26). In addition, the aerial data reduced for this project
revealed an average minimum headway of 1.00 second for individual vehicles for the two study sites investigated.

The simulation results are compared to the results obtained by Webster's model for the traffic characteristics assumed previously. The results for the 60 second and 90 second cycle lengths are presented in Table 5.1. The results indicate good agreement between the two methods but with small differences for the higher cycle length.

To provide a comparison on a more extensive basis, the 60 second cycle length was selected. Mean delays per vehicle were obtained for effective green times ranging from 15 to 45 seconds and volumes ranging from approximately 100 to 900 vehicles per hour. The results are presented in Figure 5.4. The solid lines give delays computed by Webster's formula whereas the plotted points give delays obtained by the single intersection simulation model. Each point represents the results of processing 500 vehicles through the model. The results indicate the simulation model to be an adequate representation of traffic operations at a fixed-time signalized intersection.

The standard deviation of the delay distribution is considered an important parameter of intersection operation because it indicates the extent of delays associated with a given mean. The simulation model was manipulated to determine the relationship of the standard deviation to other parameters.

It was established that the standard deviation of delays can be directly related to the amount of effective green time (or equivalently, the amount of
Table 5.1 Comparison of Mean Delay per Vehicle for Cycle Lengths of 60 and 90 Seconds

<table>
<thead>
<tr>
<th>Traffic Volume</th>
<th>Mean Delay Per Vehicle, Seconds</th>
</tr>
</thead>
<tbody>
<tr>
<td>515 vph</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Simulation</td>
</tr>
<tr>
<td>600 vph</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Simulation</td>
</tr>
<tr>
<td>720 vph</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Simulation</td>
</tr>
</tbody>
</table>
Figure 5.4 Comparison of Delay-Volume Relationships: Webster's Model and Simulation Model.
effective red time) but bears no relationship to traffic volume. The relationship between the standard deviation and effective green time is shown in Figure 5.5. This relationship is volume independent. That is, for the volume range studied (100 vph to 900 vph), the curve shown in Figure 5.5 remains unchanged.

For delay distributions, high variances are generally associated with high means and low variances with low means. To quantify this relationship, the parameters of the delay distributions from the results of 29 simulation runs were plotted against each other. The results are shown in Figure 5.6. A linear regression line was fitted to the data resulting in the following equation:

$$\mu_d = 0.085 \sigma_d^2 + 0.99$$

where $\mu_d =$ mean delay per vehicle, seconds

$\sigma_d =$ standard deviation of delay distribution, seconds.

The correlation coefficient of 0.97 indicates an excellent fit. A test to determine whether the slope differed significantly from zero indicated a "t" value of 3.18. Since this value exceeds the critical value of 2.05 (df = 27, $\chi^2 = 0.05$), the hypothesis that the slope is zero can be rejected. Thus the regression line can be considered a significant fit.

As a result of these analyses, the single intersection simulation model appears to be a realistic approximation to actual traffic operations. This conclusion is based on relationships documented in the literature and duplicated in the model.
Figure 5.5 Relationship Between Standard Deviation of Delay and Effective Green Time: Simulation Results
Figure 5.6 Relationship Between Parameters of Delay Distribution: Simulation Results

\[ \mu_t = 0.085 \sigma_t^2 + 0.99 \]
5.3 Development of Two-Intersection Model

A second model was developed to simulate a single lane of traffic passing through two signalized intersections. This model was constructed to simulate the following conditions:

Poisson Arrivals
1 veh/5.25 secs

42 secs Effective Green
48 secs Effective Red

55 secs Effective Green
35 secs Effective Red

These characteristics pertain to the first two intersections of the North Fourth Street Study Site. In addition, a 25 second constant travel time is assumed for vehicles traveling from 5th Avenue to 7th Avenue. This corresponds to a velocity of 30 mph.

For processing traffic through each intersection, a distribution of saturation flow values was assumed: 1 sec with probability of 0.10, 2 secs with probability of 0.80 and 3 secs with probability of 0.10. This distribution corresponds closely to that found by Watjen (27).
To determine sensitivity of the model, the offset between the two signals was varied in the simulation and the length of queue and mean delay per vehicle were monitored. For each offset, a total of 500 vehicles was processed through the model. The resulting queue length-offset relationship is presented in Figure 5.7. The mean delay per vehicle-offset relationship is shown in Figure 5.8.

A careful analysis of the graphs illustrates the following relationships:

1. Both queue length and mean delay per vehicle attain minimums when the signal offset is set equal to the mean travel time between signals.

2. As indicators of level of service, queue length and mean delay correlate well.

3. It is better to time two adjacent signals with an offset less than travel time than with an offset greater than travel time by the same amount, if it is impossible to time the signals with the optimum offset.

The particular form of the relationships shown in Figures 5.7 and 5.8 has been verified numerous times in the literature, thus indicating, at least, the validity of the logic used in the two-intersection model (28, 29, 30).

5.4 Development of Multi-Intersection Model

The extension of the two-intersection model to a model having n-signalized intersections was accomplished by defining the characteristics of each signalized intersection to be added to the model and by incorporating
Figure 5.7 Queue Length-Offset Relationship for 2-Intersection Model
Figure 5.8 Mean Delay per Vehicle-Offset Relationship for 2-Intersection Model
certain features in the model to increase its sophistication. Model variables were defined and numerical values assigned to permit the simulation of traffic through nine signalized intersections on North Fourth Street and through nine signalized intersections on Summit Street. These two models were then validated using data collected from the study sites.

5.4.1 Model Structure

The basis of the multi-intersection model is the following. An arrival distribution of traffic is considered an input to a series of signalized intersections. The initial arrival distribution is considered a function of volume. During the red phase, a queue of vehicles builds up at the first intersection. The queue is then released during the green phase. Defining a "block" as the distance between signals, dispersion of the queue takes place as a function of block length, traffic volume, and signal offset. Because of the dispersion of the traffic, a distribution of vehicle arrivals occurs at the succeeding signal which is different, yet related, to the former arrival distribution. This process of dispersion and definition of new arrival distributions then occurs in a repetitive fashion, though with different characteristics per block length, as the platoon progresses along the arterial.

In Chapter Four, it was established that lane of travel exhibits no significant effect on platoon behavior. For this reason, it is necessary then to incorporate only one lane of travel into the simulation model. This approach is considered realistic since the purpose of the model is to generate mean delay
and queue statistics at signalized intersections rather than attempting to predict the precise behavior of traffic between intersections.

As such, the multi-intersection model must be capable of handling two important functions: the processing of traffic through signalized intersections and the processing of traffic between signalized intersections. The former function is structured around the British concept of signalized intersection operation. Traffic is processed through each signalized intersection in the same manner as was described for the single intersection model. The model variables include the signal timing at each intersection, saturation flow, and lost time. In addition, the arrival distribution of traffic (and hence, its volume) at the initial signal must also be specified.

The processing of traffic between signalized intersections is achieved in the model by relating traffic dispersion to site-related traveltime parameters. In the preceding chapter, it was concluded that the variables which have a pronounced effect on platoon behavior are signal spacing, signal offset, and platoon size. To simulate the movement of traffic between intersections, it was considered necessary to incorporate these variables into some kind of traveltime function. The requirements of the function selected were that it be readily defined (so as to permit usage by practicing engineers) and that it allow for the dispersion of traffic. Also, ease of incorporating the function into the simulation program was a desirable, though not a necessary, objective.

In the two-intersection model, a constant traveltime was assumed between the two simulated intersections. This was considered adequate for the
purpose of validating the logical structure of the model. However, to more nearly duplicate actual field conditions, it was necessary that the constant traveltime restriction be removed. Field data suggested a distribution of traveltimes to be more appropriate.

The use of a distribution of traveltimes in the model fulfills the three conditions previously specified. Empirical traveltime distributions are easily obtainable through the collection of field data and incorporating the functions in the simulation model poses no difficulties. The most important reason, however, for utilizing traveltime distributions is that the parameters defining the distributions can be related to the significant variables affecting platoon movement. It will be shown that for a given volume level or equivalently, for platoons of approximately the same size, the mean and standard deviation of traveltimes for traffic proceeding from signal to signal can be directly related to signal spacings and signal offsets. It is envisioned that a family of curves can be developed so that from knowledge of signal spacing, signal offset, and volume level, the parameters of the traveltime distributions can be accurately predicted.

To illustrate the functional relationships expected, assume the hypothetical curves presented in Figure 5.9 were developed for traffic conditions during the peak hour on an urban arterial. Further assume rather uniform flow during the peak period so that, on the average, the platoon size or volume level can be considered fixed.

The curves are formulated on the observations that greater signal spacings result in greater dispersion of traveltimes and that signal offsets
Figure 5.9 Functional Relationships Between Traveltime Parameters, Signal Spacings and Signal Offsets
much greater than optimum result in greater vehicle delays than offsets less than optimum. If one considers three separate sets of offset values \( \{ O_1 \} \), \( \{ O_2 \} \) and \( \{ O_3 \} \) such that for a given signal spacing \( O_1 < O_2 < O_3 \), where \( O_2 \) represents the optimum offset, then the curves would be labeled as shown in Figure 5.9.

If it is desired to simulate traffic conditions along the arterial represented by the hypothetical plots, one would predict from the curves the mean and standard deviation of the traveltime distributions for each combination of signal spacing and signal offset found along the arterial. These values would then serve as model input. Thus, the pertinent model variables for handling this function include signal spacing, signal offset, and traffic volume.

In using this procedure, the implication is that the type of traveltime distribution is known a priori. Since specifying the parameters of a distribution does not completely identify that distribution, it was initially felt that the type of traveltime distribution had to be specified also so that field conditions could be duplicated in the model. Observations of Summit Street and North Fourth Street traveltime data (collected for purposes of model validation) for the various combinations of signal spacings and signal offsets found during the peak hour suggest that traveltimes between signals tend to follow a normal or a uniform probability distribution.

To determine the effect of the type of traveltime distribution on the resulting delay distribution, the constant traveltime restriction in the two-intersection model was replaced with a distribution of traveltimes. For
purposes of simulation, vehicles in traveling from one intersection to another were assigned traveltimes selected randomly from the defined distribution.

Using the same arrival rate, cycle lengths, signal splits and saturation flow values previously specified for the two-intersection model, traffic movement was simulated for the following conditions:

- **Type of Travelt ime Distribution:** Normal, Uniform
- **Mean Travelt ime:** 25, 35 seconds
- **Standard Deviation of Travelt ime:** 0.50, 1.00, 2.00, 5.00 seconds
- **Signal Offset:** 0, 60 seconds

The 25 second travelt ime corresponds to a mean velocity of approximately 30 miles per hour from 5th Avenue to 7th Avenue on North Fourth Street whereas the 35 second travelt ime corresponds to a mean velocity of about 21 miles per hour. The selected range of values of the standard deviation of traveltimes is representative of conditions found along the study sites. The two values of signal offsets were chosen to represent one offset much less than optimum and one offset much greater than optimum.

The parameter values of the delay distribution at the 7th Avenue intersection were obtained for each of the 32 conditions simulated. These results are presented in Table 5.2 for the 25 second mean travelt ime and in Table 5.3 for the 35 second mean travelt ime. Each table entry is the result of processing 500 vehicles through the model resulting in approximately 45 minutes of real time simulated per run.

The results indicate no significant differences in prediction of the para-
Table 5.2 Delay Characteristics Generated by Normal and Uniform Traveltime Distributions for Specified Offset Conditions with Mean Traveltime = 25 Seconds

<table>
<thead>
<tr>
<th>Traveltime Distribution</th>
<th>Standard Deviation of Traveltimes</th>
<th>Offset = 0 Secs</th>
<th>Offset = 60 Secs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean Delay</td>
<td>Std. Dev. Delay</td>
</tr>
<tr>
<td>Normal</td>
<td>0.50 secs</td>
<td>8.34 secs</td>
<td>150.69 secs</td>
</tr>
<tr>
<td>Uniform</td>
<td>0.50</td>
<td>8.43</td>
<td>151.37</td>
</tr>
<tr>
<td>Normal</td>
<td>1.00</td>
<td>8.42</td>
<td>151.25</td>
</tr>
<tr>
<td>Uniform</td>
<td>1.00</td>
<td>8.42</td>
<td>151.25</td>
</tr>
<tr>
<td>Normal</td>
<td>2.00</td>
<td>8.00</td>
<td>147.12</td>
</tr>
<tr>
<td>Uniform</td>
<td>2.00</td>
<td>8.25</td>
<td>149.13</td>
</tr>
<tr>
<td>Normal</td>
<td>5.00</td>
<td>8.20</td>
<td>146.56</td>
</tr>
<tr>
<td>Uniform</td>
<td>5.00</td>
<td>8.20</td>
<td>146.44</td>
</tr>
</tbody>
</table>
Table 5.3  Delay Characteristics Generated by Normal and Uniform Traveltime Distributions for Specified Offset Conditions with Mean Traveltime = 35 Seconds

<table>
<thead>
<tr>
<th>Traveltime Distribution</th>
<th>Standard Deviation of Traveltimes</th>
<th>Offset = 0 Secs</th>
<th>Offset = 60 Secs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean Delay</td>
<td>Std. Dev. Delay</td>
<td>Mean Delay</td>
</tr>
<tr>
<td>Normal</td>
<td>0.50 secs</td>
<td>16.73 secs</td>
<td>24.32 secs</td>
</tr>
<tr>
<td>Uniform</td>
<td>0.50</td>
<td>16.62</td>
<td>24.32</td>
</tr>
<tr>
<td>Normal</td>
<td>1.00</td>
<td>16.82</td>
<td>24.33</td>
</tr>
<tr>
<td>Uniform</td>
<td>1.00</td>
<td>16.71</td>
<td>24.33</td>
</tr>
<tr>
<td>Normal</td>
<td>2.00</td>
<td>16.46</td>
<td>24.35</td>
</tr>
<tr>
<td>Uniform</td>
<td>2.00</td>
<td>16.30</td>
<td>24.33</td>
</tr>
<tr>
<td>Normal</td>
<td>5.00</td>
<td>15.97</td>
<td>24.40</td>
</tr>
<tr>
<td>Uniform</td>
<td>5.00</td>
<td>15.46</td>
<td>24.37</td>
</tr>
</tbody>
</table>
meter values of the delay distributions for the two traveltime distributions investigated. The maximum observed difference in mean delay was only 0.51 seconds and the maximum observed difference in standard deviation of delay was merely 3.26 seconds. Thus, for a given mean and standard deviation of traveltimes, the shape of the traveltime distribution has no effect on the resulting delay distribution.

Comparison of the two tables reveals that the traveltime mean has a pronounced effect on the shape of the delay distribution. It is apparent that an increase in traveltime mean can cause a significant increase or a significant decrease in average delay per vehicle. For the simultaneous signal system, an increase in traveltime mean resulted in a doubling of mean delay per vehicle. For the 60 second offset condition, however, the traveltime increase actually resulted in a reduction of mean delay per vehicle. The model appears to be quite sensitive to changes in both signal offset and traveltime mean.

The effect of platoon spreading on delay can also be observed from the data. The effect is quite noticeable for the 25 second traveltime mean - 60 second offset condition. To provide a comparison with the constant traveltime condition, the parameter values of the delay distribution assuming a uniform distribution of traveltimes are presented below:

<table>
<thead>
<tr>
<th>Std. Dev. Traveltime</th>
<th>Mean Delay</th>
<th>Std. Dev. Delay</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 secs</td>
<td>34.12 secs</td>
<td>45.06 secs</td>
</tr>
<tr>
<td>2</td>
<td>33.01</td>
<td>63.88</td>
</tr>
<tr>
<td>5</td>
<td>30.73</td>
<td>87.81</td>
</tr>
</tbody>
</table>
Although the variation of the standard deviation of travel times caused little change in mean delay per vehicle, the effect on the spreading of the delay distribution is quite evident.

As a result of this analysis, it can be concluded that traveltime distributions defined only by their means and variances can be used to simulate traffic conditions between signalized intersections. Since the type of the distribution is not an important factor, the uniform distribution of travel times, being the more easily defined and programmable distribution, is incorporated in the multi-intersection model.

As was done in this abbreviated analysis, random sampling from defined traveltime distributions is incorporated in the multi-intersection model. Since the model is essentially macroscopic in nature rather than microscopic, this approach is considered adequate for the purpose of determining average characteristics of platoon behavior over a number of signal cycles.

5.4.2 Model Input

Having outlined the model structure, a list of the input variables required can now be presented. The multi-intersection simulation model requires the following input:

1. Signal timing at each intersection
2. Signal offset between signalized intersections
3. Distances between signalized intersections
4. Saturation flow at each signalized intersection
5. Lost time at each signalized intersection

6. Storage capacity between signalized intersections

7. Arrival distribution at the initial signal

8. Traveltime parameters between signalized intersections

Of these eight variables, six are considered independent in that variables 6 and 8 are dependent upon signal spacing and signal offset. Of the independent variables, signal timing, signaloffset, and signal spacing can be classified as deterministic since these values are generally known precisely and need not be estimated. However, values of the remaining variables must be estimated from operational data.

5.4.3 Model Output

The model has been programmed to generate delay and queue statistics at each of the signalized intersections simulated. Specific output from the multi-intersection model comprises the following items. Items 1 through 11 apply to each signalized intersection simulated.

1. Queue length distribution

2. Mean queue length for all vehicles

3. Mean queue length for queued vehicles

4. Maximum queue length

5. Modal queue length

6. Number of undelayed vehicles

7. Number of stopped vehicles
8. Distribution of waiting times
9. Mean delay per vehicle
10. Mean delay per stopped vehicle
11. Standard deviation of delay
12. Distribution of travel times through the system

Each of these twelve statistics can be gathered for a fixed number of vehicles, over a specified length of real time, or for a predetermined number of signal cycles.

5.4.4 Model Preparation for Validation Purposes

Model validation is viewed as consisting of two essential parts: a check of the logical structure of the model and a check of the predictive capability of the model. Constructing the multi-intersection model in stages permitted testing individual parts of the model. In Section 5.2 the single intersection model was used to verify the appropriateness of using the British concept of signalized intersection operation for the processing of traffic through a fixed-time signalized intersection. Comparison of the simulated mean delay per vehicle with the mean delay predicted by a validated theoretical equation over an arbitrary range of traffic variables indicated the model to be an adequate representation of traffic movement at an isolated intersection.

In Section 5.3 the two-intersection model was used to check the sensitivity of several model components. Treating signal offset as an independent variable, the changes in mean delay per vehicle and queue length were
monitored. The relationships obtained were found to be of the same general type as those documented in the literature, thereby indicating the validity of the logic used in constructing the model.

Testing the components of the total simulation model does not constitute, in itself, an adequate test of model validity. A check of the predictive capability of the model is considered necessary before proper model implementation can be achieved.

The strategy employed to validate the model was to simulate peak hour traffic conditions on North Fourth Street and on Summit Street and to compare the simulated results with actual field measurements. Prior to running the model, numerical values were assigned to the model variables. The primary source of these values was the analysis of the collected platoon data. Several traffic characteristics which could not be accurately estimated from the aerial data were determined from past studies of intersection performance and verified in the field.

Two separate models were developed: one to simulate the movement of traffic on North Fourth Street and one to simulate the movement of traffic on Summit Street. The specific model inputs and resulting characteristics which were used are described herein.

A. Saturation Flow

Saturation flow is a site-related parameter. Estimates of saturation flow were made by examining vehicle trajectories approximately midway between
signalized intersections. At such points, a sufficient number of vehicles would have been discharged from the previous signal and would not have been measurably influenced by the indication of the following signal. Care was taken to choose only constant speed trajectories in estimating saturation flow.

Based on the behavior of 20 platoons, saturation flow along the study sites was estimated to be approximately 1700 vehicles per hour of green. This figure applies to through traffic on 10 foot lanes, level roadway, and no commercial traffic. No significant differences were found between the estimates made on Summit Street and those made on North Fourth Street.

In 1971, a comprehensive study concerned with the timing of fixed-time signalized intersections was conducted in Columbus, Ohio (31). In this study, Wentzel accurately measured over 21 hours of saturation flow occurring in 38 different approach lanes. One of the objectives of the study was to relate saturation flow to lane width, amount of turning traffic, and amount of commercial traffic. For a 10 foot lane width, no turning traffic and no commercial traffic, Wentzel found saturation flow to vary from 1640 to 1785 vehicles per hour of green, the particular value depending on other friction factors present at the site (such as gradient, pavement condition, and location in the city). It is to be noted that the range does incorporate the 1700 vehicles per hour of green estimated from the aerial data.

In addition, it is interesting to note that the mean headway under the measured saturation flow conditions is $\frac{3600}{1700} = 2.1$ seconds per vehicle.
This value corresponds precisely to that found by Greenshields under saturation conditions (32).

Thus the 1700 vph figure appears to be a typical value for the conditions stated and is, indeed, representative of traffic behavior on Summit and North Fourth Streets. This figure was used at each intersection simulated in the models except at Hudson Street in the North Fourth Street model. At this terminal intersection, approximately 20% of the entering traffic proceeds through to the residential section situated just north of Hudson Street. Saturated flow was measured in the field for the particular approach lane incorporated in the model and was found to be 1510 vehicles per hour of green for the combination through and right turning lane. A discharge headway of 2.4 seconds is used in the model at this location.

B. Lost Time

Lost time has two components: initial lost time (due to starting delays) and final lost time (due to reduced flow during amber periods). Estimates of lost time could not be accurately made from examination of the vehicle trajectories. Since the minimum time interval between photographs was one second, lost time could only be measured to the nearest integer of a second. Measurements from several trajectories indicated initial lost time to be 2.00 to 3.00 seconds per phase. Final lost times could not be estimated since trajectories for vehicles discharging during an entire green and amber period were not available.
In the aforementioned study, Wentzel measured 2503 periods of initial lost times and 350 periods of final lost times using a speed and delay pen recorder. The results indicated the following 90% confidence intervals on the measured times:

\[
2.21 \text{ secs} \leq \text{initial lost time} \leq 2.35 \text{ secs}
\]

\[
1.66 \text{ secs} \leq \text{final lost time} \leq 2.28 \text{ secs}
\]

\[
3.83 \text{ secs} \leq \text{total lost time} \leq 4.57 \text{ secs}
\]

For the simulation models, a value of 4.00 seconds per phase is assumed for lost time.

C. Vehicles Discharged per Green Phase

An important requirement of any intersection simulation model is that it permit a realistic number of vehicles to be discharged per saturated "go" phase. Such figures can be verified from past studies but, more importantly, can easily be checked in the field.

In his study of traffic performance at signalized intersections, Greenshields established starting delay values according to queue position. This enables an estimate to be made of the maximum number of vehicles capable of being discharged per green interval.

Another estimate can be made from the work of Drew, Capelle and Pinnell (33, 34). Assuming a fully saturated green and amber period, the maximum departures per lane per cycle can be determined as a function of length of green and amber, average minimum headway, starting delay, and the time for the last platoon vehicle to cross the intersection.
Table 5.4 indicates the expected maximum number of vehicles to be discharged for varying lengths of green periods by Greenshields, by Drew, Capelle and Pinnell, and by the method adopted in the simulation model. The simulation values assume saturation flow of 1700 vehicles per hour of green and 4.00 seconds of lost time per phase.

The green periods chosen in this table represent the range of green periods displayed at the various signals of the study sites during the peak hour. Each signal displays a 3.75 second amber period.

From the results presented in Table 5.4, the conclusion can be made that the simulation model adequately allows for the maximum number of vehicles to be discharged per cycle. Several spot checks were made at the signalized intersections in the field and further substantiated the tabular results.

D. Signal Timings, Signal Offsets and Signal Spacings

Several changes were made in the timing of the traffic signals along the study sites between the time of the aerial data collection flights and the time of data collection for model validation purposes. For this reason, the signal offsets used in the parameter variation analysis of Chapter Four are somewhat different than the signal offsets used in the simulation models. Several of the offsets were adjusted to prevent backup problems which were evident from the aerial photographs.

The most significant change which occurred was the installation of a new traffic signal at the intersection of 12th Avenue and Summit Street. A
Table 5.4 Vehicles Discharged per Fully Saturated Green Phases

<table>
<thead>
<tr>
<th>Green Period</th>
<th>Greenshields</th>
<th>Drew, Capelle and Pinnell</th>
<th>Simulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>33 secs</td>
<td>13 vehs</td>
<td>15 vehs</td>
<td>15 vehs</td>
</tr>
<tr>
<td>41</td>
<td>17</td>
<td>19</td>
<td>19</td>
</tr>
<tr>
<td>42</td>
<td>18</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>45</td>
<td>19</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>50</td>
<td>22</td>
<td>24</td>
<td>23</td>
</tr>
<tr>
<td>55</td>
<td>24</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>59</td>
<td>26</td>
<td>28</td>
<td>28</td>
</tr>
<tr>
<td>60</td>
<td>26</td>
<td>29</td>
<td>28</td>
</tr>
<tr>
<td>66</td>
<td>29</td>
<td>32</td>
<td>31</td>
</tr>
</tbody>
</table>
check with the personnel of the City Traffic Engineering Division revealed the primary reason for the justification of the signal was the high pedestrian volumes at this location. The intersection is situated in the vicinity of Ohio State University and serves a significant amount of pedestrian traffic.

With the addition of the 12th Avenue intersection to the Summit Street model, each of the models was designed to simulate traffic through nine signalized intersections. The effective green times, effective red times, and signal offsets used in the models are presented in Table 5.5 for the North Fourth Street model and in Table 5.6 for the Summit Street model. The North Fourth Street signal timings constitute a 90 second cycle, 35 mile per hour progression scheme from 4:00 to 6:00 PM whereas the Summit Street signal timings constitute a 75 second cycle, 35 mile per hour progression scheme from 6:00 to 9:00 AM.

The signal spacings which were previously presented in Tables 3.1 and 3.2 are those used in the models with one exception. The 1545 foot spacing between Chittenden Avenue and 15th Avenue on Summit Street is broken up into a 400 foot spacing from Chittenden Avenue to 12th Avenue and an 1145 foot spacing from 12th Avenue to 15th Avenue.

E. Storage Capacity Between Signalized Intersections

The maximum storage capacity between signalized intersections is an important model variable because when attained it affects the rate of vehicle movement from the preceding signal. The model is designed so that when the
Table 5.5 Signal Timings Used in North Fourth Street Model

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Effective Green</th>
<th>Effective Red</th>
<th>Offset</th>
</tr>
</thead>
<tbody>
<tr>
<td>5th Avenue</td>
<td>42.0 secs</td>
<td>48.0 secs</td>
<td>22.0 secs</td>
</tr>
<tr>
<td>7th Avenue</td>
<td>55.0</td>
<td>35.0</td>
<td>32.0</td>
</tr>
<tr>
<td>11th Avenue</td>
<td>60.0</td>
<td>30.0</td>
<td>8.0</td>
</tr>
<tr>
<td>Chittenden Avenue</td>
<td>59.0</td>
<td>31.0</td>
<td>29.0</td>
</tr>
<tr>
<td>15th Avenue</td>
<td>60.0</td>
<td>30.0</td>
<td>29.0</td>
</tr>
<tr>
<td>17th Avenue</td>
<td>60.0</td>
<td>30.0</td>
<td>10.5</td>
</tr>
<tr>
<td>19th Avenue</td>
<td>59.0</td>
<td>31.0</td>
<td>15.5</td>
</tr>
<tr>
<td>Wyandotte Avenue</td>
<td>68.0</td>
<td>24.0</td>
<td>48.0</td>
</tr>
<tr>
<td>Hudson Street</td>
<td>41.0</td>
<td>49.0</td>
<td>36.5</td>
</tr>
<tr>
<td>Intersection</td>
<td>Effective Green</td>
<td>Effective Red</td>
<td>Offset</td>
</tr>
<tr>
<td>-------------------</td>
<td>-----------------</td>
<td>---------------</td>
<td>---------</td>
</tr>
<tr>
<td>Maynard Avenue</td>
<td>49.5 secs</td>
<td>25.5 secs</td>
<td>42.0 secs</td>
</tr>
<tr>
<td>Lane Avenue</td>
<td>45.0</td>
<td>30.0</td>
<td>29.5</td>
</tr>
<tr>
<td>17th Avenue</td>
<td>38.0</td>
<td>37.0</td>
<td>11.0</td>
</tr>
<tr>
<td>15th Avenue</td>
<td>47.0</td>
<td>28.0</td>
<td>27.5</td>
</tr>
<tr>
<td>12th Avenue</td>
<td>49.5</td>
<td>25.5</td>
<td>12.5</td>
</tr>
<tr>
<td>Chittenden Avenue</td>
<td>41.0</td>
<td>34.0</td>
<td>0.0</td>
</tr>
<tr>
<td>11th Avenue</td>
<td>41.0</td>
<td>34.0</td>
<td>36.0</td>
</tr>
<tr>
<td>7th Avenue</td>
<td>41.0</td>
<td>34.0</td>
<td>29.0</td>
</tr>
<tr>
<td>5th Avenue</td>
<td>33.5</td>
<td>41.5</td>
<td></td>
</tr>
</tbody>
</table>
maximum permissible queue length is reached vehicles are no longer dis-
charged from the preceding signal. The storage capacity is determined from
an estimate of the spacing occupied per stopped vehicle.

Vehicle trajectories for 28 platoons were examined and distances
occupied by queued vehicles were recorded. A check of the velocities of the
selected vehicles revealed that vehicles were assumed to be members of a
queue if their computed velocities were less than 1.00 mile per hour.

As a result of 285 observations, it was found that the mean spacing
occupied per queued vehicle was 22.82 feet. This measurement was based on
a distribution having a standard deviation of 3.13 feet. Thus, the 95% confi-
dence interval for the mean spacing ranges from 22.45 feet to 23.18 feet.

Tables 5.7 and 5.8 give the maximum 1-lane queue lengths per-
mitted in the model at the various intersections along Summit Street and North
Fourth Street, respectively. The figures take into account the condition that
queues along the study sites will not prevent the movement of East-West traffic
through any non-signalized intersections.

The figures indicate the Chittenden Avenue-11th Avenue intersection
pair to be the most critical of all intersection pairs from the standpoint of
signal coordination. Only a small variation in selection of signal offset is
permitted such that a queue of vehicles at one of the intersections will not inter-
fere with traffic movement at the adjacent intersection.

Besides enabling the prediction of lane storage capacities, knowledge
of the mean spacing per queued vehicle also provides an estimate of jam density.
Table 5.7 Allowable Queue Lengths Along Summit Street Study Site

<table>
<thead>
<tr>
<th>Signalized Intersection</th>
<th>Allowable Queue Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maynard Avenue</td>
<td>58 vehicles</td>
</tr>
<tr>
<td>Lane Avenue</td>
<td>81</td>
</tr>
<tr>
<td>17th Avenue</td>
<td>62</td>
</tr>
<tr>
<td>15th Avenue</td>
<td>24</td>
</tr>
<tr>
<td>12th Avenue</td>
<td>42</td>
</tr>
<tr>
<td>Chittenden Avenue</td>
<td>15</td>
</tr>
<tr>
<td>11th Avenue</td>
<td>13</td>
</tr>
<tr>
<td>7th Avenue</td>
<td>66</td>
</tr>
<tr>
<td>5th Avenue</td>
<td>45</td>
</tr>
</tbody>
</table>
Table 5.8 Allowable Queue Lengths Along North Fourth Street Study Site

<table>
<thead>
<tr>
<th>Signalized Intersection</th>
<th>Allowable Queue Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>5th Avenue</td>
<td>65 vehicles</td>
</tr>
<tr>
<td>7th Avenue</td>
<td>45</td>
</tr>
<tr>
<td>11th Avenue</td>
<td>66</td>
</tr>
<tr>
<td>Chittenden Avenue</td>
<td>13</td>
</tr>
<tr>
<td>15th Avenue</td>
<td>57</td>
</tr>
<tr>
<td>17th Avenue</td>
<td>24</td>
</tr>
<tr>
<td>19th Avenue</td>
<td>26</td>
</tr>
<tr>
<td>Wyandotte Avenue</td>
<td>93</td>
</tr>
<tr>
<td>Hudson Street</td>
<td>69</td>
</tr>
</tbody>
</table>
The 22.82 feet of spacing per vehicle corresponds to a traffic density of 231 vehicles per mile. This is the value reported in the speed-density analysis of Chapter Four.

F. Traveltime Parameters

The theory of using signal spacings and signal offsets to predict the parameters of traveltime distributions was presented in Section 5.4.1. To translate the theory into practice, traveltime data was collected for the sixteen different combinations of signal spacings and signal offsets found at the study sites. Since it was desired to simulate traffic operations during the peak period, field data was collected when the signal timings presented in Tables 5.5 and 5.6 were in operation.

For each signalized intersection pair, empirical distributions of traveltimes for traffic traveling in one lane from stopline to stopline were obtained. Representative sets of data are presented in histogram form in Figures 5.10 and 5.11 to verify the observation that the traveltimes tend to be normally or uniformly distributed.

The means and standard deviations of the sixteen traveltime distributions were calculated and plotted against signal spacing. Curve fitting procedures were then employed to generate equations which could be used to predict the traveltime parameters.

Based on the analysis of the Summit Street data, the relationships established between the traveltime parameters and signal spacings are as
Figure 5.10 Empirical Traveltime Distribution on Summit Street, 15th Avenue to 12th Avenue

Fit of Normal Dist.
\( \mu_t = 27.43 \)
\( \sigma_t = 3.46 \)
calc. \( \chi^2 = 6.05 \)
crit. \( \chi^2 = 87.52 \)
\( \therefore \) Good Fit
Figure 5.11 Empirical Traveltime Distribution on
Summit Street, Maynard Avenue to
Lane Avenue

Fit of Uniform Distribution

\[ \mu_t = 43.00 \]

\[ \sigma_t = 4.69 \]

calc. \( \chi^2 = 7.28 \)

crit. \( \chi^2 = 63.62 \)

\[ \therefore \text{Good Fit} \]
follows:

\[ \mu_t = 21.95s + 0.58 \]

\[ \sigma_t = -0.636s^2 + 3.318s + 0.590 \]

where \( \mu_t \) = mean traveltime, seconds

\( \sigma_t \) = standard deviation of traveltime, seconds

\( s \) = signal spacing, thousands of feet

These relationships are plotted in Figures 5.12 and 5.13.

A correlation coefficient of 0.996 was established for the linear equation. A test of the hypothesis that the y-intercept equals zero resulted in an "F" value of 752.19. Since this value exceeds the critical value of 5.99 at the 5% significance level, the hypothesis is rejected. Thus, the constant term in the equation is significantly different from zero and must be retained in predicting mean traveltime.

It was established that a second degree equation resulted in a better fit to the observed standard deviation of traveltime-spacing data than an equation of the first degree. The given polynomial explains 99.3% of the total variation in the data.

Excellent fits were also obtained on North Fourth Street. The theoretical relationships established are as follows:

\[ \mu_t = 21.41s + 1.64 \]

\[ \sigma_t = -0.723s^2 + 3.375s + 1.231 \]

where the symbols are as defined before. These equations are plotted in Figures 5.14 and 5.15.
Figure 5.12 Relationship Between Mean Traveltime and Signal Spacing on Summit Street

\[ \mu_c = 21.95s + 0.58 \]

Signal Spacing, 1000 Feet

Mean Traveltime, Secs
Figure 5.13 Relationship Between Standard Deviation of Traveltime and Signal Spacing on Summit Street

\[ \sigma^2 = -0.636s^2 + 3.318s + 0.590 \]
Figure 5.14 Relationship Between Mean Traveltime and Signal Spacing on North Fourth Street
Figure 5.15 Relationship Between Standard Deviation of Traveltime and Signal Spacing on North Fourth Street

\[ \sigma_t = 0.023S^2 + 3.975S + 1.231 \]
The linear equation for North Fourth Street resulted in a correlation coefficient of 0.989. The hypothesis that the y-intercept equals zero was rejected ("F" value = 265.66) at the 5% significance level. Thus, the constant term again makes a significant contribution in predicting mean traveltime.

A second degree relationship between the standard deviation of traveltime and spacing was established along North Fourth Street. The given polynomial explains 98.8% of the total variation in the observed data.

The four equations were used to predict the traveltime parameters for use in the simulation models. The parameter values generated by the equations are presented in Table 5.9 for the Summit Street model and in Table 5.10 for the North Fourth Street model. Also shown in the tables are the minimum and maximum traveltimes permitted by the uniform probability distribution.

G. Arrival Distribution

The final item of input is the arrival distribution at the initial signal of each model. At the time data was being collected to validate the models, arrival data at Maynard Avenue on Summit Street and at 5th Avenue on North Fourth Street were obtained. In the field, arrival times were recorded when vehicles joined the queue or crossed the stopline in the absence of a queue. The headway distribution found at Maynard Avenue is presented in Figure 5.16. This distribution has a mean of 6.69 seconds per vehicle corresponding to a volume level of 538 vph.
Table 5.9 Predicted Traveltime Characteristics Incorporated in Summit Street Model

<table>
<thead>
<tr>
<th>Signal Spacing</th>
<th>Mean Traveltime</th>
<th>Std. Dev. of Traveltime</th>
<th>Minimum Traveltime</th>
<th>Maximum Traveltime</th>
<th>Street Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>346 ft</td>
<td>8.17 secs</td>
<td>1.66 secs</td>
<td>5.30 secs</td>
<td>11.04 secs</td>
<td>Chittenden-11th</td>
</tr>
<tr>
<td>400</td>
<td>9.36</td>
<td>1.81</td>
<td>6.23</td>
<td>12.49</td>
<td>12th-Chittenden</td>
</tr>
<tr>
<td>628</td>
<td>14.36</td>
<td>2.42</td>
<td>10.17</td>
<td>18.55</td>
<td>17th-15th</td>
</tr>
<tr>
<td>1106</td>
<td>24.86</td>
<td>3.48</td>
<td>18.83</td>
<td>30.89</td>
<td>7th-5th</td>
</tr>
<tr>
<td>1145</td>
<td>25.71</td>
<td>3.55</td>
<td>19.56</td>
<td>31.86</td>
<td>15th-12th</td>
</tr>
<tr>
<td>1550</td>
<td>34.60</td>
<td>4.20</td>
<td>27.33</td>
<td>41.87</td>
<td>Lane-17th</td>
</tr>
<tr>
<td>1630</td>
<td>36.36</td>
<td>4.31</td>
<td>28.90</td>
<td>43.82</td>
<td>11th-7th</td>
</tr>
<tr>
<td>1975</td>
<td>43.93</td>
<td>4.66</td>
<td>35.86</td>
<td>52.00</td>
<td>Maynard-Lane</td>
</tr>
</tbody>
</table>
Table 5.10 Predicted Traveltime Characteristics Incorporated in North Fourth Street Model

<table>
<thead>
<tr>
<th>Signal Spacing</th>
<th>Mean Traveltime</th>
<th>Std. Dev. of Traveltime</th>
<th>Minimum Traveltime</th>
<th>Maximum Traveltime</th>
<th>Street Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>346 ft</td>
<td>9.08 secs</td>
<td>2.31 secs</td>
<td>5.08 secs</td>
<td>13.08 secs</td>
<td>11th–Chittenden</td>
</tr>
<tr>
<td>628</td>
<td>15.08</td>
<td>3.06</td>
<td>9.78</td>
<td>20.38</td>
<td>15th–17th</td>
</tr>
<tr>
<td>713</td>
<td>16.91</td>
<td>3.27</td>
<td>11.25</td>
<td>22.57</td>
<td>17th–19th</td>
</tr>
<tr>
<td>1106</td>
<td>25.32</td>
<td>4.08</td>
<td>18.25</td>
<td>32.39</td>
<td>5th–7th</td>
</tr>
<tr>
<td>1545</td>
<td>34.72</td>
<td>4.72</td>
<td>26.54</td>
<td>42.90</td>
<td>Chittenden–15th</td>
</tr>
<tr>
<td>1630</td>
<td>36.54</td>
<td>4.81</td>
<td>28.21</td>
<td>44.87</td>
<td>7th–11th</td>
</tr>
<tr>
<td>1810</td>
<td>40.39</td>
<td>4.97</td>
<td>31.78</td>
<td>49.00</td>
<td>Wyandotte–Hudson</td>
</tr>
<tr>
<td>2460</td>
<td>54.31</td>
<td>5.16</td>
<td>45.37</td>
<td>63.25</td>
<td>19th–Wyandotte</td>
</tr>
</tbody>
</table>
Figure 5.16 Empirical Headway Distribution on Summit Street at Maynard Avenue

Observations Plotted = 214
Observations Not Plotted = 34
A greater amount of traffic was observed on North Fourth Street. The headway distribution found at 5th Avenue is presented in Figure 5.17. A mean headway of 4.93 seconds per vehicle was established corresponding to a volume level of 730 vph.

Having assigned values to the model variables, the simulation models were prepared for running on the computer. The length of simulation run was determined by the amount of field data collected for validation purposes.

5.4.5 Model Validation

The principal items of model output are queue length and delay measurements at each of the signalized intersections. The literature has revealed that the validation of intersection simulation models has traditionally been based on mean delay per vehicle and total delay measurements. Because of the difficulty of measuring delay accurately in the field and because of the large number of personnel required to measure delay at nine different intersections simultaneously, it was decided to base the validation of the multi-intersection models on queue length distributions. This is considered an appropriate decision since it was established in Section 5.3 of this chapter that mean delay and mean queue length correlate well and are both considered adequate measures of intersection performance.

The data collected, then, for model validation purposes consisted of queue length measurements at each of the signalized intersections along the study sites on a cycle by cycle basis during the peak hour. A common lane
Figure 5.17 Empirical Headway Distribution on North Fourth Street at 5th Avenue
along each street was selected for observation. Since the data had to be collected simultaneously to insure proper model validation, eleven observers were required on each street: one stationed at each of the nine signalized intersections to record queue lengths and two stationed at the initial intersection to record vehicle arrivals.

One hour of data was collected during the morning peak on Summit Street on a Friday morning. Queue lengths were recorded at the end of each red period at each signalized intersection in the traffic lane adjacent to the left curb lane. Vehicle arrivals were recorded at the Maynard Avenue intersection.

An hour of data was also collected during the evening peak on North Fourth Street on a Thursday. Queue lengths were recorded in the traffic lane adjacent to the left curb lane. Vehicle arrivals were recorded at the 5th Avenue intersection.

The initial idea was to use one hour of field data to validate each model. However, during data collection on North Fourth Street, an accident occurred during the latter part of the hour and flow was disrupted for approximately 25 minutes. Data collection activities on Summit Street were curtailed after 35 minutes because of rain. Because of these disruptions, the first half-hour of field data was summarized for each site and was used to validate each model. The validation period was from 7:25 to 7:55 AM on Summit Street and from 4:30 to 5:00 PM on North Fourth Street.

Each model was run on the computer to simulate approximately 30
minutes of real time. Specifically, the Summit Street model was run for 24 signal cycles and the North Fourth Street model for 20 signal cycles. The models were manipulated to generate the distribution of vehicles waiting at the end of the red phase for each signalized intersection.

The multi-intersection model makes extensive use of random sampling from probability distributions. For each computer run, the user has the option of employing either a common sequence of random numbers or a unique sequence of numbers. Since the models were developed to predict characteristics over a period of time rather than on an individual cycle basis, the question is whether a half hour of real time simulated is sufficiently long for consistent results to be obtained.

In order to answer this question, the strategy was to develop the population distribution of total travel times through each street system. This was accomplished by compiling the results of five independent runs of each model. For each computer run, new sequences of random numbers were used. Table 5.11 presents the number of vehicles simulated and the mean travel time for each of the ten computer runs.

Treating the simulated travel time distributions as multinomial distributions, the Chi-Square test for independent samples was used to determine whether the five distributions on each street came from the same population or from significantly different populations. For the Summit Street distributions, the $\chi^2$ statistic was calculated to be 38.80 which is less than the critical value of 46.19 at the 5% level of significance, indicating acceptance of the hypothesis that the distri-
Table 5.11 Simulation Output for Five Independent Sequences of Random Numbers

<table>
<thead>
<tr>
<th>Random Number Sequence</th>
<th>Summit Street</th>
<th>North Fourth Street</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean Traveltime</td>
<td>Vehs</td>
</tr>
<tr>
<td>1</td>
<td>3.69 min</td>
<td>243</td>
</tr>
<tr>
<td>2</td>
<td>3.67</td>
<td>225</td>
</tr>
<tr>
<td>3</td>
<td>3.65</td>
<td>244</td>
</tr>
<tr>
<td>4</td>
<td>3.62</td>
<td>231</td>
</tr>
<tr>
<td>5</td>
<td>3.63</td>
<td>230</td>
</tr>
</tbody>
</table>
distributions are members of the same population.

The same conclusion was reached for the North Fourth Street distributions since the critical value of 101.90 exceeded the calculated value of 87.26 at the 5% level of significance. The implication of this analysis is then that only one run of each model is necessary to generate consistent output for simulating 30 minutes of real time.

Using the results from one computer run of each model, the simulated and the actual queue length distributions were statistically compared. The Kolmogorov-Smirnov Two-Sample Test was used to test agreement between the two cumulative distributions (35). The test statistic employed is the maximum deviation between the two cumulative distribution functions being tested. The two-tailed test used is sensitive to any kind of difference in the distributions, whether it be central tendency or skewness.

The test was applied to the simulated and observed queue length distributions at each of the signalized intersections simulated except the Chittenden Avenue intersection in the North Fourth Street model. At this location, the model predicted no stopped vehicles and, hence, a queue length distribution was not obtained.

A summary of the test results appears in Table 5.12 for the Summit Street queue distributions and in Table 5.13 for the North Fourth Street queue distributions. The Summit Street results were based on a critical value of 0.417 for the Kolmogorov-Smirnov statistic at the 5% level of significance and a sample size of 24. The North Fourth Street results were based on a critical
### Table 5.12 Kolmogorov-Smirnov Test Results for Summit Street Queue Distributions

<table>
<thead>
<tr>
<th>Location</th>
<th>Max. Deviation</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maynard Ave.</td>
<td>0.208</td>
<td>accept</td>
</tr>
<tr>
<td>Lane Ave.</td>
<td>0.250</td>
<td>accept</td>
</tr>
<tr>
<td>17th Ave.</td>
<td>0.375</td>
<td>accept</td>
</tr>
<tr>
<td>15th Ave.</td>
<td>0.375</td>
<td>accept</td>
</tr>
<tr>
<td>12th Ave.</td>
<td>0.416</td>
<td>accept</td>
</tr>
<tr>
<td>Chittenden Ave.</td>
<td>0.167</td>
<td>accept</td>
</tr>
<tr>
<td>11th Ave.</td>
<td>0.375</td>
<td>accept</td>
</tr>
<tr>
<td>7th Ave.</td>
<td>0.412</td>
<td>accept</td>
</tr>
<tr>
<td>5th Ave.</td>
<td>0.750</td>
<td>reject</td>
</tr>
</tbody>
</table>
Table 5.13 Kolmogorov-Smirnov Test Results for North Fourth Street Queue Distributions

<table>
<thead>
<tr>
<th>Location</th>
<th>Max. Deviation</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>5th Ave.</td>
<td>0.150</td>
<td>accept</td>
</tr>
<tr>
<td>7th Ave.</td>
<td>0.650</td>
<td>reject</td>
</tr>
<tr>
<td>11th Ave.</td>
<td>0.400</td>
<td>accept</td>
</tr>
<tr>
<td>15th Ave.</td>
<td>0.050</td>
<td>accept</td>
</tr>
<tr>
<td>17th Ave.</td>
<td>0.200</td>
<td>accept</td>
</tr>
<tr>
<td>19th Ave.</td>
<td>0.250</td>
<td>accept</td>
</tr>
<tr>
<td>Wyandotte Ave.</td>
<td>0.400</td>
<td>accept</td>
</tr>
<tr>
<td>Hudson Street</td>
<td>0.250</td>
<td>accept</td>
</tr>
</tbody>
</table>
value of 0.450 at the 5% level of significance and a sample size of 20.

For the Summit Street study site, the results indicate that in all but one case the hypothesis that the simulated and observed queue length distributions are members of the same population is accepted. A distinctly different distribution was generated by the simulation model at 5th Avenue than was observed in the field. The simulated mean queue length of 1.33 vehicles is much less than the observed queue length of 5.25 vehicles. The reason for the discrepancy is thought to be the presence of an appreciable amount of side street traffic entering Summit Street from 7th Avenue during the data collection period.

For the North Fourth Street study site, the results also indicate agreement between observed and theoretical queue length distributions in all cases except one. The reason for rejection at the 7th Avenue location is thought to be the large amount of traffic entering North Fourth Street from 5th Avenue. Side street traffic was not taken into account in developing the simulation model.

Further comparison between the observed and simulated distributions can be made from the data presented in Tables 5.14 and 5.15. These tables present mean, maximum and modal queue lengths at each of the signalized intersections. Table 5.14 contains the queue characteristics for Summit Street whereas Table 5.15 contains the queue characteristics for North Fourth Street.

As a result of these analyses, it can be concluded that the multi-intersection model offers much promise in simulating the behavior of traffic on a signalized arterial. Specifically, the validation tests revealed that the Summit Street model and the North Fourth Street model accurately predicted observed queueing statistics in 16 of 18 locations.
Table 5.14 Comparison of Queue Length Statistics on Summit Street:
Field Data vs. Simulation Output

<table>
<thead>
<tr>
<th>Location</th>
<th>Mean</th>
<th></th>
<th>Maximum</th>
<th></th>
<th>Mode</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Data</td>
<td>Model</td>
<td>Data</td>
<td>Model</td>
<td>Data</td>
<td>Model</td>
</tr>
<tr>
<td>Maynard Ave.</td>
<td>3.83 vehs</td>
<td>3.46 vehs</td>
<td>7 vehs</td>
<td>7 vehs</td>
<td>3 vehs</td>
<td>2 vehs</td>
</tr>
<tr>
<td>Lane Ave.</td>
<td>2.33</td>
<td>1.58</td>
<td>6</td>
<td>5</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>17th Ave.</td>
<td>3.17</td>
<td>1.46</td>
<td>10</td>
<td>4</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>15th Ave.</td>
<td>1.00</td>
<td>0.04</td>
<td>2</td>
<td>1</td>
<td>0,2</td>
<td>0</td>
</tr>
<tr>
<td>12th Ave.</td>
<td>1.42</td>
<td>0.54</td>
<td>3</td>
<td>2</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Chittenden Ave.</td>
<td>1.21</td>
<td>0.88</td>
<td>4</td>
<td>2</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>11th Ave.</td>
<td>1.71</td>
<td>0.83</td>
<td>4</td>
<td>4</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>7th Ave.</td>
<td>1.29</td>
<td>0.42</td>
<td>3</td>
<td>2</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>5th Ave.</td>
<td>5.25</td>
<td>1.33</td>
<td>9</td>
<td>4</td>
<td>6,7</td>
<td>0,1</td>
</tr>
</tbody>
</table>
Table 5.15 Comparison of Queue Length Statistics on North Fourth Street: Field Data vs. Simulation Output

<table>
<thead>
<tr>
<th>Location</th>
<th>Mean</th>
<th></th>
<th>Maximum</th>
<th></th>
<th>Mode</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Data</td>
<td>Model</td>
<td>Data</td>
<td>Model</td>
<td>Data</td>
<td>Model</td>
</tr>
<tr>
<td>5th Ave.</td>
<td>9.00 vehs</td>
<td>8.75 vehs</td>
<td>15 vehs</td>
<td>17 vehs</td>
<td>8 vehs</td>
<td>8 vehs</td>
</tr>
<tr>
<td>7th Ave.</td>
<td>0.90</td>
<td>0.05</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>11th Ave.</td>
<td>0.60</td>
<td>0.05</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Chittenden Ave.</td>
<td>1.75</td>
<td>0.00</td>
<td>4</td>
<td>0</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>15th Ave.</td>
<td>0.45</td>
<td>0.45</td>
<td>2</td>
<td>2</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>17th Ave.</td>
<td>0.40</td>
<td>0.65</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>19th Ave.</td>
<td>1.00</td>
<td>0.60</td>
<td>3</td>
<td>2</td>
<td>0.1</td>
<td>0</td>
</tr>
<tr>
<td>Wyandotte Ave.</td>
<td>0.75</td>
<td>0.15</td>
<td>3</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Hudson Street</td>
<td>5.65</td>
<td>8.20</td>
<td>11</td>
<td>17</td>
<td>6</td>
<td>8</td>
</tr>
</tbody>
</table>
Statement listings of the simulation programs appear in the Appendix.

Two separate listings are presented: one for the Summit Street model and one for the North Fourth Street model. For each model, a favorable simulation time to real time ratio was observed. One hour of real time was simulated in one minute on the computer.
CHAPTER SIX

IMPLEMENTATION OF RESEARCH RESULTS

This chapter discusses the applicability of the research findings. The field data required to implement the model are itemized and the ability to adapt the model to the study of special conditions is discussed. Finally, an example is presented to demonstrate an application of the research findings.

6.1 Applicability of Model

The results of the research efforts documented in this report illustrate that the movement of traffic on an urban signalized one-way arterial can be simulated on the computer to generate reliable queue and delay statistics. In the model which was developed, an arrival distribution of traffic is considered an input to a series of fixed-time signalized intersections. The green-amber-red periods of each traffic signal are transformed into effective green and effective red periods, taking into account lost times due to starting delays and to deceleration of the queue.

At each signalized intersection, vehicles queue up if they arrive during the red phase of the signal or during the green phase in the presence of a queue. Traffic is assumed to depart from the queue according to a constant saturation flow discharge rate. Vehicles are permitted to leave the queue only if the lane storage capacity to the next signalized intersection is not attained. Vehicles
which arrive during the green phase in the absence of waiting vehicles proceed through the intersection without delay.

The processing of traffic between signalized intersections is achieved in the model by relating traffic dispersion to site-related traveltime parameters. The mean and standard deviation of traveltime distributions are related through regression techniques to signal spacings and signal offsets for a given volume level.

Assumptions of the model include passenger car movement, no turning traffic, no entering traffic from adjacent lanes, and no consideration of signal visibility affecting platoon behavior. These assumptions were made since the study sites for which two models were specifically developed exhibited negligible commercial traffic, low frequency of lane changing, generally insignificant side street traffic entering the lane being simulated, and adequate downstream signal visibility at all intersections. In addition, the model incorporates only one lane of traffic since it was established that no significant differences in platoon movement could be attributed to lane of travel.

To apply the model to a specific one-way signalized arterial, some field data is necessary to estimate certain variables of the model. The following input is required before the model can be implemented:

1. Signal timing at each intersection
2. Signal offset between signalized intersections
3. Distances between signalized intersections
4. Saturation flow at each signalized intersection
5. Lost time at each signalized intersection
6. Storage capacity between signalized intersections
7. Arrival distribution at the initial signal
8. Traveltime parameters between signalized intersections

The physical characteristics of the arterial to be simulated determine the input variable of signal spacing. The conditions to be simulated are reflected in the values assigned to the variables of signal timing and signal offset. The remaining input variables must be estimated from operational data.

Having assigned values to all model variables, the model can be used to generate queue, delay and traveltime characteristics for traffic traveling through the simulated street system. Specific model output comprises the following items:

1. Queue length distribution
2. Mean queue length for all vehicles
3. Mean queue length for queued vehicles
4. Maximum queue length
5. Modal queue length
6. Number of undelayed vehicles
7. Number of stopped vehicles
8. Distribution of waiting times
9. Mean delay per vehicle
10. Mean delay per stopped vehicle
11. Standard deviation of delay

12. Distribution of travel times through the system

The user has the option of gathering these statistics for a fixed number of vehicles, for a specified length of real time, or for a predetermined number of signal cycles.

In order to validate the logical structure of the model and to ascertain its predictive capability, the model was applied to the simulation of traffic movement along two signalized arterials located in Columbus, Ohio. The model was formulated to simulate peak hour traffic traveling through a progressive signal system on each street. The model variables which required the collection of field data to estimate and the methods by which the estimates were obtained comprised the following:

<table>
<thead>
<tr>
<th>Model Variable</th>
<th>Method of Value Estimation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturation flow</td>
<td>field measurement</td>
</tr>
<tr>
<td>Lost time</td>
<td>field measurement</td>
</tr>
<tr>
<td>Storage capacity</td>
<td>aerial photography</td>
</tr>
<tr>
<td>Arrival distribution</td>
<td>empirical distribution</td>
</tr>
<tr>
<td>Mean traveltime</td>
<td>regression equation</td>
</tr>
<tr>
<td>Standard deviation of travel times</td>
<td>regression equation</td>
</tr>
</tbody>
</table>

Queue length distributions generated at each of the signalized intersections simulated were used as the basis for model validation. The statistical agreement between simulated and observed queue length distributions at traffic
volume levels of 538 vehicles per hour and 730 vehicles per hour was established and revealed the models to be valid representations of traffic behavior through the intersections simulated.

The approach used in this research to simulate traffic is considered a valid one. The developed model must be used in the macroscopic sense. That is, characteristics of platoon behavior over a number of signal cycles can be obtained but the model is not to be used to predict the precise behavior of traffic movement from intersection to intersection.

One of the principal features of the model is its flexibility. It can readily be adapted to handle a variety of traffic situations. More importantly, model assumptions made to apply the model to a specific location can be relaxed, if required, in order to apply the model to another location.

For example, if it is desired to simulate an arterial having significant amounts of commercial traffic and/or turning traffic occurring consistently throughout the time period to be simulated, one could take both factors into account by a reduction in the value assigned to saturation flow. Relationships between saturation flow, amount of turning traffic, and proportion of commercial vehicles in the traffic stream are documented in the literature and thus could be incorporated in the model.

The assumption that no side street traffic enters the lane of travel being simulated can be relaxed provided estimates of either the amount of such entering traffic or the size of the additional queue developed as a result at the nearest downstream signal are available. Side street traffic could be generated in the model at desired locations if the average volume levels are known.
Initial queues could be generated at specific intersections and their effect on platoon movement studied.

Changes required in the model to relax these assumptions can be accomplished without much difficulty in programming. The incorporation of such changes, however, does require the collection of additional field data.

6.2 Example of Model Implementation

The validated simulation model can be used by the practicing engineer to determine signal settings based on the dispersion of traffic and it can be used to study the effect of a change of one of the model variables (e.g., signal timing, signal offset, or saturation flow) on expected queue lengths and mean delays per vehicle. In this manner, the model can be used to assist the engineer in decision-making toward implementing changes in traffic control. An example will illustrate this point.

Consider the 5th to 7th Avenue intersection pair of North Fourth Street. Assume that during the evening peak it is observed that a significant amount of traffic enters North Fourth Street from 5th Avenue. Field estimates show that on the average, as a result of the entering side street traffic, a queue of five vehicles is found waiting at the 7th Avenue intersection when the traffic released from the 5th Avenue intersection arrives. The existing progression system is thus disturbed and inefficiency results. The problem is to determine the adjustment in signal settings required because of the initial queue so as to minimize delay.
The problem can be solved by the use of the North Fourth Street simulation model. The characteristics of these two intersections incorporated in the model are the following:

- Effective green time at 5th Avenue = 42 secs
- Effective red time at 5th Avenue = 48 secs
- Effective green time at 7th Avenue = 55 secs
- Effective red time at 7th Avenue = 35 secs
- Cycle length = 90 secs
- Lost time = 4.00 secs per phase
- Saturation flow = 1700 vehicles per hour
- Maximum storage capacity = 65 vehicles
- Traffic volume = 730 vehicles per hour
- Mean of empirical arrival distribution = 1 vehicle per 4.93 secs
- Mean traveltime = 25.32 secs
- Standard deviation of traveltime = 4.08 secs

Assume the mean and variance of the traveltime distribution are not adversely affected by the presence of the 5-vehicle queue. This appears to be a reasonable assumption since the queue represents less than eight per cent of the maximum possible storage capacity between the two intersections.

For the condition where no initial queue is present and the signals are set for a 35 mile per hour progression speed (signal offset = 22.0 seconds), the model predicts a mean delay per vehicle of 0.01 seconds based on one-half hour of real time simulated. If the signal offset is not changed, the 5-vehicle
initial queue occurring every cycle causes a mean delay per vehicle of 14.61 seconds. This figure is also based on the simulation of 20 cycles.

To determine the change in signal setting required so as to minimize delay, signal offset was varied and mean delay per vehicle was monitored. The simulation results are the following:

<table>
<thead>
<tr>
<th>Signal Offset</th>
<th>Mean Delay per Vehicle</th>
</tr>
</thead>
<tbody>
<tr>
<td>14 secs</td>
<td>11.17 secs</td>
</tr>
<tr>
<td>15</td>
<td>11.20</td>
</tr>
<tr>
<td>16</td>
<td>10.85</td>
</tr>
<tr>
<td>17</td>
<td>10.90</td>
</tr>
<tr>
<td>18</td>
<td>11.01</td>
</tr>
</tbody>
</table>

Thus, to minimize mean delay per vehicle, the signal offset should be reduced from 22.0 seconds to 16.0 seconds. This improvement in signal offset will result in an expected hourly savings of 45 vehicle-minutes of delay at a volume level of 730 vehicles per hour.
CHAPTER SEVEN

CONCLUSIONS AND RECOMMENDATIONS

The overall aim of this research was to investigate platoon movements on urban arterials and to relate variation in platoon behavior to variation in signal control and to changes in traffic volumes. A brief summary of how this objective was achieved is presented herein.

7.1 Summary of Accomplishments

The objective of this study was accomplished in the form of determining platoon characteristics for traffic traveling on signalized urban arterials and developing a mathematical model to simulate the behavior of a group of vehicles as it passes through a series of signalized intersections. The research efforts were structured around four principal phases.

The first phase consisted of a review of the literature to describe the few studies of platoon behavior which have been conducted. The significant findings for the fourteen studies reviewed were reported. It was established that the research approach adopted in the present study differed in two important respects from all previous studies of platoon behavior. These were the collection of comprehensive data on platoon movement by using a helicopter-mounted aerial camera and the use of urban signalized arterials as study sites.

The second phase of research consisted of the collection and reduction
of continuous velocity and spacing data as platoons of traffic traveled through progressive signal systems during peak hours. Two study sites, each consisting of nine signalized intersections, were selected in the Columbus, Ohio area. For the twenty-eight platoons photographed, vehicle trajectories were constructed to provide a visual picture of traffic movement.

The third phase of research consisted of identifying platoon characteristics for traffic traveling on the signalized arterials. It was established that improper signal offsets, the presence of initial queues at interior signalized intersections, and high frequency of lane changes at a specific location can cause inefficiency in the operation of a progressive signal system.

The principal variables affecting platoon movement through linear signal systems were identified as signal spacing, signal offset, and platoon size. (Signal visibility, roadway gradient, and amount of commercial traffic as factors affecting platoon behavior were not investigated because of their inapplicability to the study sites investigated.) It was established that lane of travel exhibits no significant effect on the behavior of platoons in traveling from signal to signal.

Finally, it was determined as a result of viewing time and space patterns of selected traffic variables that platoon movement can best be described by patterns of mean velocity or mean spacing, traffic density, and the coefficient of variation of velocity. The latter parameter was shown to be a valid indicator of traffic conditions.

The fourth phase of research involved the development of a mathematical
model to simulate the behavior of a group of vehicles progressing through a series of signalized intersections. The British concept of intersection operation was used for processing traffic through each signalized intersection. The processing of traffic between intersections was achieved in the model by relating traffic dispersion to site-related traveltime parameters. The mean and standard deviation of traveltime distributions were related through regression equations to signal spacings and signal offsets for a given level of traffic volume.

Two specific models were developed to simulate traffic conditions at the study sites. The statistical agreement between simulated and observed queue length distributions was established, thus revealing the models to be adequate representations of traffic behavior through the intersections simulated.

7.2 Future Research Recommendations

As a result of this completed investigation of platoon dispersion characteristics of traffic travelling through signalized intersections, the following recommendations for future research are made:

1. It is recommended that the model be applied to the study of other one-way signalized arterials having different signal spacings and offsets than the sites analyzed in this study.

2. It is recommended that additional regression equations be generated relating traveltime parameters to signal spacings and signal offsets for a variety of traffic volume levels. Comparisons should be made between off-peak and peak hour traffic conditions.
3. It is recommended that a comprehensive sensitivity analysis of all variables incorporated in the model be conducted in order to precisely identify the effect of a change of one model variable on model output.
REFERENCES


APPENDIX
DERIVATION OF TEST STATISTIC USED IN TESTING FOR LANE EFFECT

Let \( y_i \) = observed velocity difference between vehicles traveling in adjacent traffic lanes at location \( i \)

\[ \overline{y} = \text{sample mean} = \frac{1}{n} \sum_{i=1}^{n} y_i \]

\[ s^2 = \text{sample variance} = \frac{1}{n-1} \sum_{i=1}^{n} (y_i - \overline{y})^2 \]

\( \mu = \text{population mean} \)

\( \sigma^2 = \text{population variance} \)

\( m = \text{number of locations} \)

Assumption: \( \{y_i\} \) is a set of independent, identically distributed normal random variables.

\( y_i \sim N(\mu, \sigma^2) \) for all \( i \)

Therefore:\n
\[ \left( \frac{y_i - \mu}{\sigma} \right) \sim N(0, 1) \]

\[ \left( \frac{y_i - \mu}{\sigma} \right)^2 \sim \chi^2_1 \]

\[ \sum_{i=1}^{m} \left( \frac{y_i - \mu}{\sigma} \right)^2 \sim \chi^2_m \]

Also: \( \frac{(m-1) \sigma^2}{\sigma^2_0} \sim \chi^2_{m-1} \)

Hence:\n
\[ \frac{\sum_{i=1}^{m} \left( \frac{y_i - \mu}{\sigma} \right)^2}{m-1} \sim F_{1, m-1} \]

Under the null hypothesis, \( H_0: \mu = 0 \), the test statistic becomes:

\[ \frac{m \sum_{i=1}^{m} y_i^2}{s^2} \sim F_{1, m-1} \]
*SIMULATION OF SUMMIT STREET

* SIMULATION OF INTERSECTIONS 1, 2, 3, 4, 5, 6, 7, 8, 9

* INTERSECTION 1 IS MAYNARD AVENUE
* INTERSECTION 2 IS 8TH AVENUE
* INTERSECTION 3 IS 17TH AVENUE
* INTERSECTION 4 IS 15TH AVENUE
* INTERSECTION 5 IS 12TH AVENUE
* INTERSECTION 6 IS CHITTY AVENUE
* INTERSECTION 7 IS 11TH AVENUE
* INTERSECTION 8 IS 21AVENUE
* INTERSECTION 9 IS 5TH AVENUE

* BASIC TIME UNIT IS 0.10 SEC

* RANDOM NUMBER SEQUENCE 5
  RMULT. 9, 11, 13, 15, 17, 19, 21, 23, 25

* SIMULATE

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 1
  GENERATE 9, 5, 1
  GREN1 ADVANCE 495 49.5 SECS OF EFF GREEN
  PREEMPT INT11
  ADVANCE 255 25.5 SECS OF EFF RED
  RETURN INT11 END OF PREEMPT STATE
  TABULATE 2 ENTER LENGTH OF QUE11 IN TABLE 2
  TRANSFER GREN1 BEGIN NEW GREEN

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 2
  GENERATE 7, 1
  ADVANCE 420 42.0 SEC OFFSET, INT1 TO INT2
  GREN2 ADVANCE 450 45.0 SECS OF EFF GREEN
  PREEMPT INT21
  ADVANCE 300 30.0 SECS OF EFF RED
  RETURN INT21 END OF PREEMPT STATE
  TABULATE 4 ENTER LENGTH OF QUE21 IN TABLE 4
  TRANSFER GREN2 BEGIN NEW GREEN

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 3
  GENERATE 11, 1
  ADVANCE 715 71.5 SEC OFFSET, INT1 TO INT3
  GREN3 ADVANCE 380 38.0 SECS OF EFF GREEN
  PREEMPT INT31
  ADVANCE 370 37.0 SECS OF EFF RED
  RETURN INT31 END OF PREEMPT STATE
  TABULATE 6 ENTER LENGTH OF QUE31 IN TABLE 6
  SPLIT 1, TERM
  TRANSFER GREN3 BEGIN NEW GREEN

TERM TERMINATE 1

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 4
  GENERATE 9, 1
ADVANCE  75    07.5 SEC OFFSET, INT1 TO INT4
GREN4 ADVANCE  470    47.0 SECS OF EFF. GREEN
PREEMPT  INT41
ADVANCE  280    28.0 SECS OF EFF RED
RETURN  INT41    END OF PREEMPT STATE
TABULATE  8    ENTER LENGTH OF QUE41 IN TABLE 8
TRANSFER  ,GREN4    BEGIN NEW GREEN

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 5

GENERATE  ,1
ADVANCE  350    35.0 SEC OFFSET, INT1 TO INT5
GREN5 ADVANCE  495    49.5 SECS OF EFF GREEN
PREEMPT  INT51
ADVANCE  255    25.5 SECS OF EFF RED
RETURN  INT51    END OF PREEMPT STATE
TABULATE  10    ENTER LENGTH OF QUE51 IN TABLE 10
TRANSFER  ,GREN5    BEGIN NEW GREEN

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 6

GENERATE  ,1
ADVANCE  475    47.5 SEC OFFSET, INT1 TO INT6
GREN6 ADVANCE  410    41.0 SECS OF EFF GREEN
PREEMPT  INT61
ADVANCE  340    34.0 SECS OF EFF RED
RETURN  INT61    END OF PREEMPT STATE
TABULATE  12    ENTER LENGTH OF QUE61 IN TABLE 12
TRANSFER  ,GREN6    BEGIN NEW GREEN

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 7

GENERATE  ,1
ADVANCE  475    47.5 SEC OFFSET, INT1 TO INT7
GREN7 ADVANCE  410    41.0 SECS OF EFF GREEN
PREEMPT  INT71
ADVANCE  340    34.0 SECS OF EFF RED
RETURN  INT71    END OF PREEMPT STATE
TABULATE  14    ENTER LENGTH OF QUE71 IN TABLE 14
TRANSFER  ,GREN7    BEGIN NEW GREEN

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 8

GENERATE  ,1
ADVANCE  85.    08.5 SEC OFFSET, INT1 TO INT8
GREN8 ADVANCE  410    41.0 SECS OF EFF GREEN
PREEMPT  INT81
ADVANCE  340    34.0 SECS OF EFF RED
RETURN  INT81    END OF PREEMPT STATE
TABULATE  16    ENTER LENGTH OF QUE81 IN TABLE 16
TRANSFER  ,GREN8    BEGIN NEW GREEN

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 9

GENERATE  ,1
ADVANCE  375    37.5 SEC OFFSET, INT1 TO INT9
GREN9 ADVANCE  335    33.5 SECS OF EFF GREEN
PREEMPT  INT91
<table>
<thead>
<tr>
<th>Function</th>
<th>Minimum</th>
<th>Maximum</th>
<th>Mean</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.70</td>
<td>3.71</td>
<td>3.70</td>
<td>0.50</td>
</tr>
<tr>
<td>2</td>
<td>3.70</td>
<td>3.71</td>
<td>3.70</td>
<td>0.50</td>
</tr>
<tr>
<td>3</td>
<td>3.70</td>
<td>3.71</td>
<td>3.70</td>
<td>0.50</td>
</tr>
<tr>
<td>4</td>
<td>3.70</td>
<td>3.71</td>
<td>3.70</td>
<td>0.50</td>
</tr>
<tr>
<td>5</td>
<td>3.70</td>
<td>3.71</td>
<td>3.70</td>
<td>0.50</td>
</tr>
<tr>
<td>6</td>
<td>3.70</td>
<td>3.71</td>
<td>3.70</td>
<td>0.50</td>
</tr>
<tr>
<td>7</td>
<td>3.70</td>
<td>3.71</td>
<td>3.70</td>
<td>0.50</td>
</tr>
<tr>
<td>8</td>
<td>3.70</td>
<td>3.71</td>
<td>3.70</td>
<td>0.50</td>
</tr>
</tbody>
</table>
FUNCTION & HAS MEAN = 24.86 SECS, STD DEV = 3.48 SECS

* GENERATE CUE11
SEIZE INT11
ASSIGN 1,INT011
ASSIGN 1,INT121
TEST L P1,81
TRANSFER SIM,FAS11,SLO11

SLO11 ADVANCE 21
FAS11 RELEASE INT11
DEPART QUE11
LEAVE QUEUE AT INT1,LANE 1

TTT12 ADVANCE 10,FN2
QUEUE QUE21
TRAVEL TIME,INT1 TO INT2
JOIN QUEUE AT INT2, LANE 1
OBTAIN FACILITY 2 WHEN FREE, LANE 1

ASSIGN 2,INT21
ASSIGN 2,INT22
ASSIGN 2,INT23
TEST L P2,62
TRANSFER SIM,FAS21,SLO21

SLO21 ADVANCE 21
FAS21 RELEASE INT21
DEPART QUE21
LEAVE QUEUE AT INT2,LANE 1

TTT23 ADVANCE 10,FN3
QUEUE QUE31
TRAVEL TIME,INT2 TO INT3
JOIN QUEUE AT INT3, LANE 1
OBTAIN FACILITY 3 WHEN FREE, LANE 1

ASSIGN 3,INT31
ASSIGN 3,INT32
ASSIGN 3,INT33
ASSIGN 3,INT34
TEST L P3,24
TRANSFER SIM,FAS31,SLO31

SLO31 ADVANCE 21
FAS31 RELEASE INT31
DEPART QUE31
LEAVE QUEUE AT INT3,LANE 1

TTT34 ADVANCE 10,FN4
QUEUE QUE41
TRAVEL TIME,INT3 TO INT4
JOIN QUEUE AT INT4, LANE 1
OBTAIN FACILITY 4 WHEN FREE, LANE 1

ASSIGN 4,INT41
ASSIGN 4,INT42
ASSIGN 4,INT43
ASSIGN 4,INT44
TEST L P4,42
TRANSFER SIM,FAS41,SLO41

SLO41 ADVANCE 21
FAS41 RELEASE INT41
DEPART QUE41
LEAVE QUEUE AT INT4,LANE 1

TTT45 ADVANCE 10,FN5
TRAVEL TIME,INT4 TO INT5
| QUEUE | DUE51 | JOIN QUEUE AT INT5, LANE 1 |
| SEIZE | INT51 | OBTAIN FACILITY 5 WHEN FREE, LANE 1 |
| ASSIGN | 5, +5 SLO51 |
| ASSIGN | 5, +5 TTT56 |
| ASSIGN | 5, +5 061 |
| TEST L | P5, +5 | ALLOWABLE QUEUE = 15 VEHs |
| TRANSFER | SIM; FAS51, SLO51 |
| SL051 ADVANCE | 21 | SATURATION FLOW = 2.1 SECS PER VEH |
| EAS51 RELEASE | INT51 | FREE FACILITY 5 FOR NEXT ARRIVAL, LANE 1 |
| DEPART | OUE51 | LEAVE QUEUE AT INT5, LANE 1 |
| TTT56 ADVANCE | 10, +10 | TRAVELTIME, INT5 TO INT6 |
| QUEUE | OUE51 | JOIN QUEUE AT INT6, LANE 1 |
| SEIZE | INT61 | OBTAIN FACILITY 6 WHEN FREE, LANE 1 |
| ASSIGN | 6, +6 SLO61 |
| ASSIGN | 6, +6 TTT67 |
| ASSIGN | 6, +6 071 |
| TEST L | P6, +6 | ALLOWABLE QUEUE = 13 VEHs |
| TRANSFER | SIM; FAS61, SLO61 |
| SL061 ADVANCE | 21 | SATURATION FLOW = 2.1 SECS PER VEH |
| FAS61 RELEASE | INT61 | FREE FACILITY 6 FOR NEXT ARRIVAL, LANE 1 |
| DEPART | OUE61 | LEAVE QUEUE AT INT6, LANE 1 |
| TTT67 ADVANCE | 10, +10 | TRAVELTIME, INT6 TO INT7 |
| QUEUE | OUE71 | JOIN QUEUE AT INT7, LANE 1 |
| SEIZE | INT71 | OBTAIN FACILITY 7 WHEN FREE, LANE 1 |
| ASSIGN | 7, +7 SLO71 |
| ASSIGN | 7, +7 TTT78 |
| ASSIGN | 7, +7 081 |
| TRANSFER | SIM; FAS71, SLO71 |
| SL071 ADVANCE | 21 | SATURATION FLOW = 2.1 SECS PER VEH |
| FAS71 RELEASE | INT71 | FREE FACILITY 7 FOR NEXT ARRIVAL, LANE 1 |
| DEPART | OUE71 | LEAVE QUEUE AT INT7, LANE 1 |
| TTT78 ADVANCE | 10, +10 | TRAVELTIME, INT7 TO INT8 |
| QUEUE | OUE81 | JOIN QUEUE AT INT8, LANE 1 |
| SEIZE | INT81 | OBTAIN FACILITY 8 WHEN FREE, LANE 1 |
| ASSIGN | 8, +8 SLO81 |
| ASSIGN | 8, +8 TTT89 |
| ASSIGN | 8, +8 091 |
| TEST L | P8, +8 | ALLOWABLE QUEUE = 45 VEHs |
| TRANSFER | SIM; FAS81, SLO81 |
| SL081 ADVANCE | 21 | SATURATION FLOW = 2.1 SECS PER VEH |
| FAS81 RELEASE | INT81 | FREE FACILITY 8 FOR NEXT ARRIVAL, LANE 1 |
| DEPART | OUE81 | LEAVE QUEUE AT INT8, LANE 1 |
| TTT89 ADVANCE | 10, +10 | TRAVELTIME, INT8 TO INT9 |
| QUEUE | OUE91 | JOIN QUEUE AT INT9, LANE 1 |
| SEIZE | INT91 | OBTAIN FACILITY 9 WHEN FREE, LANE 1 |
| TRANSFER | SIM; FAS91, SLO91 |
| SL091 ADVANCE | 21 | SATURATION FLOW = 2.1 SECS PER VEH |
| FAS91 RELEASE | INT91 | FREE FACILITY 9 FOR NEXT ARRIVAL, LANE 1 |
| DEPART | OUE91 | LEAVE QUEUE AT INT9, LANE 1 |
| TABULATE | 19 | ENTER SYSTEM TIME IN TABLE 19 |
| TERMINATE | | REMOVE VEH FROM SYSTEM |
**TABLE DEFINITION CARDS**

<table>
<thead>
<tr>
<th>No.</th>
<th>TABLE</th>
<th>Queue</th>
<th>Distribution of Time Spent In Queue</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>QTABLE</td>
<td>0$ QUE11</td>
<td>0, 50, 20 DIST OF TIME SPENT IN QUE11</td>
</tr>
<tr>
<td>2</td>
<td>TABLE</td>
<td>0$ QUE11</td>
<td>0, 1, 20 DIST OF QUEUE LENGTHS, QUE11</td>
</tr>
<tr>
<td>3</td>
<td>QTABLE</td>
<td>0$ QUE21</td>
<td>0, 50, 20 DIST OF TIME SPENT IN QUE21</td>
</tr>
<tr>
<td>4</td>
<td>TABLE</td>
<td>0$ QUE21</td>
<td>0, 1, 20 DIST OF QUEUE LENGTHS, QUE21</td>
</tr>
<tr>
<td>5</td>
<td>QTABLE</td>
<td>0$ QUE31</td>
<td>0, 50, 20 DIST OF TIME SPENT IN QUE31</td>
</tr>
<tr>
<td>6</td>
<td>TABLE</td>
<td>0$ QUE31</td>
<td>0, 1, 20 DIST OF QUEUE LENGTHS, QUE31</td>
</tr>
<tr>
<td>7</td>
<td>QTABLE</td>
<td>0$ QUE41</td>
<td>0, 50, 20 DIST OF TIME SPENT IN QUE41</td>
</tr>
<tr>
<td>8</td>
<td>TABLE</td>
<td>0$ QUE41</td>
<td>0, 1, 20 DIST OF QUEUE LENGTHS, QUE41</td>
</tr>
<tr>
<td>9</td>
<td>QTABLE</td>
<td>0$ QUE51</td>
<td>0, 50, 20 DIST OF TIME SPENT IN QUE51</td>
</tr>
<tr>
<td>10</td>
<td>TABLE</td>
<td>0$ QUE51</td>
<td>0, 1, 20 DIST OF QUEUE LENGTHS, QUE51</td>
</tr>
<tr>
<td>11</td>
<td>QTABLE</td>
<td>0$ QUE61</td>
<td>0, 50, 20 DIST OF TIME SPENT IN QUE61</td>
</tr>
<tr>
<td>12</td>
<td>TABLE</td>
<td>0$ QUE61</td>
<td>0, 1, 20 DIST OF QUEUE LENGTHS, QUE61</td>
</tr>
<tr>
<td>13</td>
<td>QTABLE</td>
<td>0$ QUE71</td>
<td>0, 50, 20 DIST OF TIME SPENT IN QUE71</td>
</tr>
<tr>
<td>14</td>
<td>TABLE</td>
<td>0$ QUE71</td>
<td>0, 1, 20 DIST OF QUEUE LENGTHS, QUE71</td>
</tr>
<tr>
<td>15</td>
<td>QTABLE</td>
<td>QUE81</td>
<td>0, 50, 20 DIST OF TIME SPENT IN QUE81</td>
</tr>
<tr>
<td>16</td>
<td>TABLE</td>
<td>QUE81</td>
<td>0, 1, 20 DIST OF QUEUE LENGTHS, QUE81</td>
</tr>
<tr>
<td>17</td>
<td>QTABLE</td>
<td>QUE91</td>
<td>0, 50, 20 DIST OF TIME SPENT IN QUE91</td>
</tr>
<tr>
<td>18</td>
<td>TABLE</td>
<td>QUE91</td>
<td>0, 1, 20 DIST OF QUEUE LENGTHS, QUE91</td>
</tr>
<tr>
<td>19</td>
<td>TABLE</td>
<td>M1</td>
<td>2000, 100, 30 DIST OF TIME SPENT IN SYSTEM</td>
</tr>
</tbody>
</table>

**CONTROL CARD**

START... 24... SIMULATE 24 CYCLES...

END
* SIMULATION OF NORTH FOURTH STREET
  * SIMULATION OF INTERSECTIONS 1, 2, 3, 4, 5, 6, 7, 8, 9
  * INTERSECTION 1 IS 5TH AVENUE
  * INTERSECTION 2 IS 7TH AVENUE
  * INTERSECTION 3 IS 11TH AVENUE
  * INTERSECTION 4 IS CHITTENDEN AVENUE
  * INTERSECTION 5 IS 15TH AVENUE
  * INTERSECTION 6 IS 17TH AVENUE
  * INTERSECTION 7 IS 19TH AVENUE
  * INTERSECTION 8 IS WYANDOTTE AVENUE
  * INTERSECTION 9 IS HUDSON STREET
  * BASIC TIME UNIT IS 0.10 SEC
  * RANDOM NUMBER SEQUENCE 2
    RMULT 3, 5, 7, 9, 11, 13, 15, 17, 19
  * SIMULATE
    *
    * BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 1
      GENERATE 1
      GREN1 ADVANCE 420 42.0 SECS OF EFF GREEN
      PREEMPT INT11
      ADVANCE 480 48.0 SECS OF EFF RED
      RETURN INT11 END OF PREEMPT STATE
      TABULATE 2 ENTER LENGTH OF QUEUE1 IN TABLE 2
      TRANSFER ,GREN1 BEGIN NEW GREEN
    *
    * BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 2
      GENERATE 1
      GB01 ADVANCE 220 22.0 SEC OFFSET, INT1 TO INT2
      PREEMPT INT21
      ADVANCE 350 35.0 SECS OF EFF RED
      RETURN INT21 END OF PREEMPT STATE
      TABULATE 4 ENTER LENGTH OF QUEUE2 IN TABLE 4
      TRANSFER ,GREN2 BEGIN NEW GREEN
    *
    * BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 3
      GENERATE 1
      GREN3 ADVANCE 600 60.0 SECS OF EFF GREEN
      PREEMPT INT31
      ADVANCE 300 30.0 SECS OF EFF RED
      RETURN INT31 END OF PREEMPT STATE
      TABULATE 6 ENTER LENGTH OF QUEUE3 IN TABLE 6
      TRANSFER ,GREN3 BEGIN NEW GREEN
    *
    * BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 4
      GENERATE 1
      GREN4 ADVANCE 620 62.0 SEC OFFSET, INT1 TO INT4
    *
PREEMPT INT41
ADVANCE 310
RETURN INT41
TABULATE 8
TRANSFER ,GREN4

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 5
GENERATE ,,,1
ADVANCE 010
GREN5 ADVANCE 600
PREEMPT INT51
ADVANCE 300
RETURN INT51
TABULATE 10
TRANSFER ,GREN5

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 6
GENERATE ,,,1
ADVANCE 115
GREN6 ADVANCE 600
PREEMPT INT61
ADVANCE 300
RETURN INT61
TABULATE 12
TRANSFER ,GREN6

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 7
GENERATE ,,,1
ADVANCE 270
GREN7 ADVANCE 590
PREEMPT INT71
ADVANCE 310
RETURN INT71
TABULATE 14
TRANSFER ,GREN7

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 8
GENERATE ,,,1
ADVANCE 750
GREN8 ADVANCE 660
PREEMPT INT81
ADVANCE 240
RETURN INT81
TABULATE 16
SPLIT 1,TERM
TRANSFER ,GREN8
TERM TERMINATE 1

* BLOCK DEFINITION CARDS SIGNAL TIMING INTERSECTION 9
GENERATE ,,,1
ADVANCE 215
GREN9 ADVANCE 410
PREEMPT INT91
<table>
<thead>
<tr>
<th>FUNCTION</th>
<th>Rn4,c26</th>
<th>MEAN</th>
<th>49.0 SECS OF EFF RED</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>220</td>
<td>490</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>583</td>
<td>END OF PREEMPT STATE</td>
</tr>
<tr>
<td>TRANSFER</td>
<td></td>
<td>3</td>
<td>ENTER LENGTH OF DUE91 IN TABLE 18</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>780</td>
<td>BEGIN NEW GREEN</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>811</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>894</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>903</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>915</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>940</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>991</td>
<td></td>
</tr>
</tbody>
</table>

### Empirical Headway Distribution

<table>
<thead>
<tr>
<th>Function</th>
<th>Mean</th>
<th>Std Dev</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>25.32 SECS</td>
<td>4.08 SECS</td>
</tr>
<tr>
<td>2</td>
<td>26.54 SECS</td>
<td>4.81 SECS</td>
</tr>
<tr>
<td>3</td>
<td>34.72 SECS</td>
<td>4.72 SECS</td>
</tr>
<tr>
<td>4</td>
<td>15.08 SECS</td>
<td>3.06 SECS</td>
</tr>
<tr>
<td>5</td>
<td>16.91 SECS</td>
<td>3.27 SECS</td>
</tr>
<tr>
<td>6</td>
<td>54.31 SECS</td>
<td>5.16 SECS</td>
</tr>
<tr>
<td>7</td>
<td>45.37 SECS</td>
<td>5.16 SECS</td>
</tr>
<tr>
<td>8</td>
<td>40.01 SECS</td>
<td>5.16 SECS</td>
</tr>
</tbody>
</table>
* FUNCTION 9 HAS MEAN = 40.36 SECS, STD DEV = 4.97 SECS
FUNCTION RM7.C19
900 31.78 .013 32 .071 33 .129 34 .187 35 .245 36
303 37 .361 38 .419 39 .477 40 .535 41 .593 42
651 42 .710 44 .768 45 .826 46 .884 47 .942 48
1.000 49

GENERATE 10,FN1
QUEUE QUE11
SEIZE INT11
ASSIGN 1,MSL011
ASSIGN 1,MSLTT12
ASSIGN 1,021
TEST L P1.45
TRANSFER SIM,FAS11,SL011

SL011 ADVANCE 21
FAS11 RELEASE INT11
DEPART QUE11
CUE INT11
SEIZE INT21
ASSIGN 2,MSL021
ASSIGN 2,MSLTT23
ASSIGN 2,031
TEST L P1.45
ALLOWABLE QUEUE = 45 VEHS

FAS11 RELEASE INT11
DEPART QUE11
CUE INT11
SEIZE INT21
ASSIGN 2,MSL021
ASSIGN 2,MSLTT23
ASSIGN 2,031
TEST L P1.45
ALLOWABLE QUEUE = 45 VEHS

SATURATION FLOW = 2.1 SECS PER VEH
FREE FACILITY 1 FOR NEXT ARRIVAL, LANE 1
LEAVE QUEUE AT INT1, LANE 1
TRAVELTIME, INT1 TO INT2
JOIN QUEUE AT INT2, LANE 1
OBTAIN FACILITY 1 WHEN FREE, LANE 1

SATURATION FLOW = 2.1 SECS PER VEH
FREE FACILITY 1 FOR NEXT ARRIVAL, LANE 1
LEAVE QUEUE AT INT1, LANE 1
TRAVELTIME, INT1 TO INT2
JOIN QUEUE AT INT2, LANE 1
OBTAIN FACILITY 1 WHEN FREE, LANE 1

SATURATION FLOW = 2.1 SECS PER VEH
FREE FACILITY 2 FOR NEXT ARRIVAL, LANE 1
LEAVE QUEUE AT INT2, LANE 1
TRAVELTIME, INT2 TO INT3
JOIN QUEUE AT INT3, LANE 1
OBTAIN FACILITY 2 WHEN FREE, LANE 1

SATURATION FLOW = 2.1 SECS PER VEH
FREE FACILITY 2 FOR NEXT ARRIVAL, LANE 1
LEAVE QUEUE AT INT2, LANE 1
TRAVELTIME, INT2 TO INT3
JOIN QUEUE AT INT3, LANE 1
OBTAIN FACILITY 2 WHEN FREE, LANE 1

SATURATION FLOW = 2.1 SECS PER VEH
FREE FACILITY 3 FOR NEXT ARRIVAL, LANE 1
LEAVE QUEUE AT INT3, LANE 1
TRAVELTIME, INT3 TO INT4
JOIN QUEUE AT INT4, LANE 1
OBTAIN FACILITY 3 WHEN FREE, LANE 1

SATURATION FLOW = 2.1 SECS PER VEH
FREE FACILITY 3 FOR NEXT ARRIVAL, LANE 1
LEAVE QUEUE AT INT3, LANE 1
TRAVELTIME, INT3 TO INT4
JOIN QUEUE AT INT4, LANE 1
OBTAIN FACILITY 3 WHEN FREE, LANE 1

SATURATION FLOW = 2.1 SECS PER VEH
FREE FACILITY 4 FOR NEXT ARRIVAL, LANE 1

SATURATION FLOW = 2.1 SECS PER VEH
FREE FACILITY 4 FOR NEXT ARRIVAL, LANE 1
DEPART QUE41 LEAVE QUEUE AT INT4, LANE 1
TTT45 ADVANCE 10, FM5 TRAVELTIME, INT4 TO INT5
QUEUE QUE51 JOIN QUEUE AT INT5, LANE 1
SEIZE INT51 OBTAIN FACILITY 5 WHEN FREE, LANE 1
ASSIGN 5, #5SL051
ASSIGN 5, #5TTT56
ASSIGN 5, #061
TEST L P5, 24
TRANSFER SIM, FAS51, SL051

ALLOWABLE QUEUE = 24 VEHS

SL051 ADVANCE 21 SATURATION FLOW = 2.1 SECS PER VEH
FAS51 RELEASE INT51 FREE FACILITY 5 FOR NEXT ARRIVAL, LANE1
DEPART QUE51 LEAVE QUEUE AT INT5, LANE 1
TTT56 ADVANCE 10, FM6 TRAVELTIME, INT5 TO INT6
QUEUE QUE61 JOIN QUEUE AT INT6, LANE 1
SEIZE INT61 OBTAIN FACILITY 6 WHEN FREE, LANE 1
ASSIGN 6, #5SL061
ASSIGN 6, #5TTT67
ASSIGN 6, #071
TEST L P6, 26
TRANSFER SIM, FAS61, SL061

ALLOWABLE QUEUE = 26 VEHS

SL061 ADVANCE 21 SATURATION FLOW = 2.1 SECS PER VEH
FAS61 RELEASE INT61 FREE FACILITY 6 FOR NEXT ARRIVAL, LANE1
DEPART QUE61 LEAVE QUEUE AT INT6, LANE 1
TTT67 ADVANCE 10, FM7 TRAVELTIME, INT6 TO INT7
QUEUE QUE71 JOIN QUEUE AT INT7, LANE 1
SEIZE INT71 OBTAIN FACILITY 7 WHEN FREE, LANE 1
ASSIGN 7, #5SL071
ASSIGN 7, #5TTT78
ASSIGN 7, #081
TEST L P7, 93
TRANSFER SIM, FAS71, SL071

ALLOWABLE QUEUE = 93 VEHS

SL071 ADVANCE 21 SATURATION FLOW = 2.1 SECS PER VEH
FAS71 RELEASE INT71 FREE FACILITY 7 FOR NEXT ARRIVAL, LANE1
DEPART QUE71 LEAVE QUEUE AT INT7, LANE 1
TTT78 ADVANCE 10, FM8 TRAVELTIME, INT7 TO INT8
QUEUE QUE81 JOIN QUEUE AT INT8, LANE 1
SEIZE INT81 OBTAIN FACILITY 8 WHEN FREE, LANE 1
ASSIGN 8, #5SL081
ASSIGN 8, #5TTT89
ASSIGN 8, #091
TEST L P8, 69
TRANSFER SIM, FAS81, SL081

ALLOWABLE QUEUE = 69 VEHS

SL081 ADVANCE 21 SATURATION FLOW = 2.1 SECS PER VEH
FAS81 RELEASE INT81 FREE FACILITY 8 FOR NEXT ARRIVAL, LANE1
DEPART QUE81 LEAVE QUEUE AT INT8, LANE 1
TTT89 ADVANCE 10, FM9 TRAVELTIME, INT8 TO INT9
QUEUE QUE91 JOIN QUEUE AT INT9, LANE 1
SEIZE INT91 OBTAIN FACILITY 9 WHEN FREE, LANE 1
TRANSFER SIM, FAS91, SL091

SL091 ADVANCE 24 SATURATION FLOW = 2.4 SECS PER VEH
FAS91 RELEASE INT91 FREE FACILITY 9 FOR NEXT ARRIVAL, LANE1
DEPART QUE91 LEAVE QUEUE AT INT9, LANE 1
TABULATE 1° ENTER SYSTEM TIME IN TABLE 19
**TABLE DEFINITION CARDS**

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>QUE1,0,50,20</td>
</tr>
<tr>
<td>2</td>
<td>Q$QUE11,0,1,25</td>
</tr>
<tr>
<td>3</td>
<td>QUE21,0,50,20</td>
</tr>
<tr>
<td>4</td>
<td>Q$QUE21,0,1,25</td>
</tr>
<tr>
<td>5</td>
<td>QUE31,0,50,20</td>
</tr>
<tr>
<td>6</td>
<td>Q$QUE31,0,1,25</td>
</tr>
<tr>
<td>7</td>
<td>QUE41,0,50,20</td>
</tr>
<tr>
<td>8</td>
<td>Q$QUE41,0,1,25</td>
</tr>
<tr>
<td>9</td>
<td>QUE51,0,50,20</td>
</tr>
<tr>
<td>10</td>
<td>Q$QUE51,0,1,25</td>
</tr>
<tr>
<td>11</td>
<td>QUE61,0,50,20</td>
</tr>
<tr>
<td>12</td>
<td>Q$QUE61,0,1,25</td>
</tr>
<tr>
<td>13</td>
<td>QUE71,0,50,20</td>
</tr>
<tr>
<td>14</td>
<td>Q$QUE71,0,1,25</td>
</tr>
<tr>
<td>15</td>
<td>QUE81,0,50,20</td>
</tr>
<tr>
<td>16</td>
<td>Q$QUE81,0,1,25</td>
</tr>
<tr>
<td>17</td>
<td>QUE91,0,50,20</td>
</tr>
<tr>
<td>18</td>
<td>Q$QUE91,0,1,25</td>
</tr>
<tr>
<td>19</td>
<td>M1,200Q,100,50</td>
</tr>
</tbody>
</table>

**CONTROL CARD**

START

END

SIMULATE 20 CYCLES.