INVESTIGATION OF TRAFFIC DYNAMICS BY AERIAL PHOTOGRAMMETRY TECHNIQUES

by the Research Staff
Transportation Research Center
Department of Civil Engineering

INTERIM REPORT
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ABSTRACT

This report documents the progress made during the period June 1970 to August 1972 on a continuing research project aimed at improving and simplifying an aerial photogrammetry technique for practical applications in highway design and traffic operations. The report describes the results of a theoretical study undertaken for the purpose of developing an improved model for relating the average speed of a freeway traffic platoon to traffic density. The study was conducted in two phases with analysis proceeding from both an empirical and a mathematical approach. The speed-density model proposed is multi-linear in form with a straight line segment representing the speed-density relation in each of a number of mutually exclusive density subregions. Based on these subregions a new level of service framework is developed and a set of freeway control standards using lane occupancy as the control parameter is proposed.

In addition, the results of two pilot studies conducted for the purpose of improving urban freeway operations during the morning and evening peak periods are presented. Both studies were made on Interstate 71 in northern Columbus, Ohio, with one study concentrated on traffic operations during the southbound morning peak and the other on operations during the northbound evening peak. As a result of these studies peak period control strategies featuring fixed-time ramp metering were proposed for both directions of flow.
The southbound control strategy has been implemented in the field and the results of a study made to evaluate its effectiveness in improving flow efficiency is also included in the report. The northbound strategy has yet to be field-tested.
PREFACE

This report summarizes the work performed on Research Project EES 278, "Investigation of Traffic Dynamics by Aerial Photogrammetry Techniques" since June, 1970. This research project is 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 Traffic Research Group under the direction of Dr. Joseph Treitzer, Professor of Civil Engineering. This report was prepared by Jeffrey A. Myers, Research Associate. Contributors to the research include:

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The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Ohio or the Federal Highway Administration.
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Massachusetts. This instrument is connected to an on-line IBM 026 Printing Card Punch through a Mann Type 1945 Data Logger allowing information from the aerial photographs to be punched directly onto cards as it is obtained. The data is then converted to the required information on traffic parameters using a battery of computer programs written in FORTRAN IV G for use on the IBM 360/370 series electronic computer. Through application of this survey technique continuous data can be attained on such attributes of the traffic stream as traffic density, velocity, acceleration and deceleration, weaving movements, headways and spacings.

The research program described in this report was undertaken with the purpose of refining the aerial survey technique and applying it to treat both theoretical and practical problems of highway design and operation. The results presented document the progress made during the period June 1970 through August 1972.

Objectives of the Research

The research program has four specific objectives:

1. To explore practical applications of the aerial photogrammetry technique and to conduct pilot studies in cooperation with the Ohio Department of Transportation

2. To apply the results of the research program to improve traffic flow and increase safety

3. To conduct theoretical studies of traffic dynamics utilizing aerial
photography with emphasis on research in the theory of traffic flow

4. To develop better methods for the reduction and analysis of aerial photography data.

Scope of Research Undertaken Between June 1970 and August 1972

Primary emphasis during the time period covered by this report was placed on Objectives 1 and 2. Pilot studies were conducted on Interstate 71 in northern Columbus, Ohio, to determine the nature and extent of operational problems occurring on this roadway during the southbound morning and northbound evening peak periods (Objective 1). Data was collected using both aerial photography and standard ground-based traffic engineering techniques. This data was then used to develop control strategies for each peak period. These control strategies feature fixed-time ramp metering and are expected to yield significant improvements in traffic movement and motorist safety (Objective 2).

Specific theoretical studies of the dynamics of traffic flow were conducted as a secondary priority (Objective 3). These studies were concentrated on the development of an improved macroscopic model relating average speed and traffic density. No specific efforts were made to modify the aerial data acquisition technique (Objective 4) during the reporting period. The technique continues to function smoothly and adequately satisfies the data needs of the subject research project.
Summary of the Contents of This Report

This report documents the progress made on the subject research project during the period from June 1970 to August 1972. The report is divided into two major sections, each of which is then subdivided into two chapters. A brief description of the contents of each of these chapters is presented below.

Part I of the report presents the results of a theoretical study undertaken with the purpose of developing an improved model for relating the average speed of a platoon of vehicles to traffic density. The basic premise of the study is that different traffic operating conditions can be described in terms of mutually exclusive density subregions. Thus a speed-density relationship is sought which would allow the boundaries of these subregions to be readily identified. The study was conducted in two phases and involved analysis from both an empirical and a mathematical point of view.

The empirical phase of the study is described in Chapter 1. In this phase a speed-density model is developed from an analysis of data on freeway platoon movement collected on Interstate 71 using the aerial photogrammetric technique. The proposed model consists of a series of straight line segments with each segment representing the speed-density relationship existing within a separate subregion of the density domain. The break-points between adjacent subregions are identified from a detailed analysis of the patterns displayed by a selected group of platoon parameters as traffic density varies from zero to jam conditions. The validity of the proposed model is investigated using a
series of three statistical tests. These tests reveal that the multi-linear form model provides a continuous, statistically acceptable description of the field data. A new level of service concept and a set of control standards for freeway traffic are then proposed based on the density subregions identified from the speed-density model.

The mathematical phase of the study is described in Chapter 2. In this phase the movement of a platoon of vehicles along the roadway is analyzed using the concepts of force and work from applied physics. It is hypothesized that the platoon is at any instant of time being acted upon by two force components; one component is responsible for the forward velocity of the platoon (external force) while the other controls the platoon length causing it to expand and contract as it moves (internal force). Since movement is involved in the action of each component, it is logical to assume that each component performs a given amount of work on the platoon. These two types of work can be visualized in the following manner. The external work (that done by the external force component) is responsible for any changes in platoon velocity as it moves along the roadway. The internal work is responsible for changes in platoon length. Since velocity and length change concurrently as a platoon moves and are related such that a slower moving platoon is generally shorter for the same number of vehicles than a faster moving platoon, it was further hypothesized that the two units of work done are proportional to one another.

The two types of work were then expressed in terms of common traffic
parameters (including average speed and traffic density) and the proportionality relationship was manipulated to yield an expression with average speed as the dependent variable. The resulting relation is shown below:

\[ u = A + \beta \left( \frac{C}{B} \right) K \]

where \( u \) represents average speed, \( K \) is traffic density, \( A \) is a constant and \( \beta, C, \) and \( B \) are platoon parameter values. This relation shows that speed is a linear function of density. It is then shown that the parameters \( C \) and \( B \) exhibit different values in different parts of the density domain thus providing mathematical support for the multi-linear speed-density relation proposed in Chapter 1.

Part II of the report describes the progress made in two pilot studies undertaken for the purpose of applying the aerial photogrammetric data collection technique to investigate practical problems of highway design and operation. The two studies were conducted on Interstate 71 in northern Columbus, Ohio, with one study concentrated on traffic operations during the southbound morning peak period and the other on operations during the northbound evening peak. The specific section studied extended from the I-270 interchange on the north to the Fort Hayes Interchange on the south (10.2 miles). The studies were conducted in cooperation with personnel of the Division of Highways of the Ohio Department of Transportation and the City of Columbus Division of Traffic Engineering.

Both studies were conducted using an identical five stage procedure.

1. Conduct several general survey flights over I-71 between the I-270 and Fort Hayes Interchanges in an attempt to locate regions of high
congestion symptomatic of operational problems

2. Conduct a series of ground-based studies concentrated in the regions identified in Stage 1 for the purpose of isolating existing traffic bottlenecks—study techniques to be used include travel time runs, density trap counts and volume counts.

3. Conduct a series of intensive aerial surveys of identified bottleneck areas to determine the following information:
   a. Specific nature of the problem
   b. Parameters which can be used to describe the traffic situation at the bottleneck
   c. Causative factors leading to flow breakdowns and congestion at the problem points

4. Formulate a control strategy for I-71 based on the data collected in stages 1, 2, and 3; inform appropriate State of Ohio and City of Columbus personnel of findings and supply available data for their analysis.

5. Conduct a series of "after" aerial surveys and field studies to determine the effectiveness of the proposed control strategy in improving the operational efficiency of I-71 and provide recommendations concerning needed strategy modifications.

The results of the southbound study are documented in Chapter 1. This study has progressed through all five stages and is considered complete. As a result of this study a control strategy featuring fixed-time metering of three
southbound entrance ramps was proposed. This strategy was approved by both
the cooperating state and city agencies and has been implemented in the field.
An evaluation study showed that the control plan has been successful in increas-
ing the efficiency of morning peak period traffic flow. Chapter 2 presents the
results of the northbound study. This study has reached the stage of control
plan formulation. The proposed control plan calls for fixed-time metering of
four northbound entrance ramps. No decision has yet been made, however,
about the implementation of this strategy and consequently only the first four
stages of the study can be described in this report.
PART I

THEORETICAL STUDIES
OVERVIEW

Provision for the smooth and efficient movement of people and goods is a difficult problem in any country. It is especially complicated in a complex, urbanized society such as ours, however, where daily some 110 million vehicles compete for a limited number of roadway facilities. In order to effectively treat the multitude of operational problems which result from this overcrowding of the roadways, it is necessary to develop an in-depth understanding of the dynamics of traffic flow. Highway researchers have long been aware of this need and consequently much effort has been devoted in recent years to the development of a body of theory pertaining to the description of the characteristics of traffic flow on both a macroscopic and microscopic level.

One major area of traffic flow theory is the study of functional relationships between the three fundamental traffic parameters: density, volume and speed. The basic relationship of traffic flow assumes that traffic volume equals the product of speed (space-mean) and density. Therefore, only one pair of the three basic parameters is needed to study all possible functional relationships. Of the possible pairings speed-density data seems to exhibit the simplest functional pattern and therefore is used most often to investigate the macroscopic characteristics of traffic flow.

Previous work in dealing with speed-density relationships has been summarized in the paper "A Statistical Analysis of Speed Density Hypotheses,"
by Drake, Schofer and May (1). The authors used a set of data collected on the Eisenhower Expressway in Chicago to evaluate the adequacy of seven different speed-density hypotheses proposed by various researchers. The selected hypotheses are summarized graphically in Figure I-1. Several statistical tests were applied in conjunction with a group of intuitive discriminating criteria in an attempt to determine which hypothesis corresponded best to the pattern displayed by the field data. As a conclusion to their investigation, the authors stated that their results tended to support the three-regime linear hypothesis and Edie's hypothesis above all others tested. However, from a less rigid standpoint of application, all hypotheses (except the two-regime linear) performed well enough to warrant continued use.

A study quite similar to that of Drake, Schofer and May was conducted as part of the subject research project and was reported upon in the last Interim Report (2). This study utilized data collected on Interstate 71 in Columbus, Ohio using the aerial photogrammetric technique. Instead of collecting data at a point on the roadway several platoons of vehicles were followed as they passed through various operating conditions. In this manner, the influences due to different drivers and different vehicles as well as the inherent differences between queue forming and queue releasing conditions could be sorted out and analyzed. The same seven hypotheses considered by Drake et al. were evaluated using the I-71 data. As a result of this study it was determined that a different functional relationship exists between speed and density for queue releasing as compared to queue forming conditions. Of the seven hypotheses
Figure I-1 Proposed Speed-Density Hypotheses
considered queue forming conditions were best described by a three-regime linear relationship while two linear regimes provided the best fit for queue releasing conditions.

Based on the results of this study, an expanded analysis was conducted during the second phase of the research project. Rather than testing existing hypotheses against field data, this study undertook to use the data to develop a new speed-density hypothesis. The basic premise of the investigation was that different traffic operating conditions can be defined in terms of mutually exclusive density subregions. Hence, the speed-density relationship chosen should be in a form which would allow the boundaries of these subregions to be identified. Once the subregions are established, it should be possible to develop a set of control criteria for providing a given level of operating conditions on the freeway using density as the control parameter.

The results of the second phase study are summarized in the following two chapters. In Chapter 1 the data is presented and a speed-density relationship comprised of a number of linear segments is proposed. Each linear segment represents the relationship existing in a separate subregion of the density domain. The break points between segments which define the subregion boundaries are established through a detailed analysis of platoon characteristics. Parameters of platoon movement considered in the analysis include velocity, volume, kinetic energy, headway and spacing. Once the break-points are defined, a linear segment is fitted to the data of each subregion using a least-squares procedure. The resulting multi-linear speed-density function is
then tested statistically to establish the following:

1. The linearity of the data within each subregion

2. The continuity of the fitted segments at the break-points

3. The difference of segment slopes between subregions.

These tests revealed that the multi-linear function provides a continuous, statistically acceptable description of the field data and that a statistically different speed-density relationship does exist in each subregion. Using these subregions as a basis, a level of service concept is then developed employing traffic density as the fundamental parameter. A set of control standards is also proposed for regulating freeway operating conditions by control of traffic density.

Chapter 2 is devoted to an attempt to establish a mathematical basis for the empirical speed-density relationship developed in the preceding chapter. The approach employed uses the concepts of force and work from applied physics to analyze the movement of a platoon of vehicles through a kinematic disturbance. It is hypothesized that a platoon moving along a section of roadway is at any instant of time being acted upon by two imaginary force components. First, there is an external force component which causes the platoon to move in space at some given velocity. Second, there is an internal force component which controls the length of the platoon causing it to expand and contract as it moves along the roadway. Since both force components involve movement of the platoon as a whole or of its vehicular components separately, each can be said to be performing a certain amount of work on the platoon. During an infinitesimal
interval of time these two kinds of work can be written as:

\[ dW_E = F_E dx \]
\[ dW_I = F_I dy \]

where:
\[ dW_E = \text{infinitesimal external work done} \]
\[ dW_I = \text{infinitesimal internal work done} \]
\[ F_E = \text{imaginary external force component} \]
\[ F_I = \text{imaginary internal force component} \]
\[ dx = \text{infinitesimal distance the platoon is moved} \]
\[ dy = \text{infinitesimal length change for the platoon} \]

It is further hypothesized that these two kinds of work are related in the following manner.

\[-dW_E = \alpha dW_I\]

A mathematical expression is developed for each type of work using straight substitution and dimensional analysis. These expressions are substituted into the above relationship and this equation is manipulated into a form expressing platoon velocity as a function of traffic density. The relation obtained is:

\[ u = A + \beta (C/B)K \]

where:
\[ u = \text{platoon velocity} \]
\[ A = \text{constant of integration} \]
\[ \beta, C \text{ and } B = \text{platoon coefficients} \]
\[ K = \text{traffic density} \]

This equation shows velocity to be a linear function of density. It is then shown
that the coefficients C and B have different values in different density subregions thus supporting the concept of a multi-linear speed-density relationship.
CHAPTER 1

THE MULTI-LINEAR SPEED-DENSITY RELATIONSHIP:

EMPIRICAL APPROACH

1.1 DEVELOPMENT OF THE MULTI-LINEAR MODEL

Previous attempts to establish the relationship between average velocity and traffic density have been hampered by the lack of data free of undeterminable influences. Among these influences are the effects of different drivers, different vehicles and differences in traffic performance before and after a traffic disturbance. The aerial photogrammetric data collection technique developed by the Transportation Research Center permits the collection of data in which these various effects can be recognized and analyzed. This is done by following the same vehicles (with the same drivers) through a variety of different operating conditions over a period of time. It is felt that this method of data collection provides data superior to that which can be collected by ground-based techniques and, hence, provides an opportunity for a fresh look at the relation between speed and density.

1.1.1 DESCRIPTION OF THE DATA

The data used for this study was selected from the vehicle trajectory plot shown in Figure I-2. These vehicle trajectories were obtained by aerial
survey of Interstate 71 in Columbus, Ohio. The trajectories show that the vehicles initially moved in a relatively low density region, then were compressed through a kinematic wave, and finally were released to a uniform flow condition.

Two platoons, Platoon A and Platoon B, were chosen from these vehicle trajectories for detailed analyses. The two platoons are shown in Figures I-3 and I-4, respectively. The main reason for selecting these two platoons was to try to keep lane changing to a minimum. In this case, Platoon A had no vehicles entering or leaving while Platoon B had one vehicle leaving long before the platoon entered the kinematic wave and two vehicles entering near the end of the study period. The selected sizes of the platoons were chosen primarily to provide the highest possible maximum densities. It can be seen from Figure I-2 that the larger the size of the platoon, the smaller its observed maximum density. High maximum densities were desired so that as much of the density domain as possible would be covered by the data.

1.1.2 QUEUE FORMING AND RELEASING CHARACTERISTICS

When a freeway operates at or beyond capacity, fluctuations in traffic movement occur. As demand increases, such fluctuations may cause traffic flow to deteriorate to a stop-and-go type of operation. Temporary stop-and-go operations may also be caused by accidents or disabled vehicles. This stop-and-go phenomenon is somewhat similar to the situation in which queues are formed and released by traffic signals. Intuitively, one would imagine that
Figure I-3  Identification of Platoon A Vehicles
(Enclosed Area)
Figure I-4 Identification of Platoon B Vehicles
(Enclosed Area)
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drivers caught in this type of flow condition would have to be more alert in a queue-forming situation than in a queue-releasing situation to avoid possible rear-end collisions. Several studies have confirmed that drivers tend to react more quickly to traffic changes in a queue-generating condition than in a queue-releasing condition (3, 4, 5). This difference in driver reaction tends to result in a less efficient movement of vehicles after a traffic disturbance than before such a disturbance.

It seems that research in the direction of understanding the queue phenomenon might make a very significant contribution to the field of traffic flow theory; however, there has been no direct investigation of this phenomenon to date. In this study, the use of aerial techniques provides data on vehicular platoons during the generation and dissipation stage of a complete traffic stoppage. This data provides an ideal opportunity for studying the queue phenomenon.

**Observed Hysteresis Phenomenon**

For the selected platoons, average velocities and densities were calculated at one-second time intervals. The relationships between these two parameters are shown in Figures 1-5 and 1-6. Data points before and after the kinematic disturbance are differentiated in the following manner: small circles represent "before" conditions and crosses represent "after" conditions. Arrowheads are used to show the time sequence of the data points. By following the time sequence it can be observed that retarded traffic movement occurred
Figure I-5 Observed Speed-Density Relationship for Platoon A
Figure I-6 Observed Speed-Density Relationship for Platoon B
Immediately after the maximum density, i.e., the traffic moved at a lower speed in the queue-releasing condition than the queue-forming condition for the same density values. The retarded behavior of traffic in the "after" condition forms a loop with the "before" data similar to the hysteresis loop formed by the loading and unloading of a plastic material. Unlike the plastic material, however, the retarded performance of the traffic platoons was temporary and each platoon recovered to its "before" performance level at a certain density. For Platoon A, the traffic recovered at a density of 73 vpm, while Platoon B recovered at 59 vpm. These density values are termed the point of recovery (P.O.R.) for the platoons. An interesting phenomenon is that when traffic, after going through the hysteresis loop, reached its point of recovery, traffic density stopped decreasing but the average speed of the platoon kept increasing. Traffic for some time stayed at the P.O.R. density before a further reduction in density occurred. Eventually, Platoon A can be seen regaining the "before" condition at a much lower density of about 33 vpm. This density value is termed the Point of Return to Normalcy (P.O.R.N.). For Platoon B, this return to the "before" condition was not shown due to the lack of data observed below the density of 43 vpm on its recovery phase. However, the trend of the data indicates that Platoon B would return to the "before" condition at a traffic density of about 33 vpm.

The hysteresis phenomenon can also be seen on the volume-density plots presented in Figures 1-7 and 1-8 and the speed-volume plots presented in Figures 1-9 and 1-10.
Figure I-7 Observed Volume-Density Relationship for Platoon A
Figure I-8 Observed Volume-Density Relationship for Platoon B
Figure I-9  Observed Speed-Volume Relationship for Platoon A
Figure I-10  Observed Speed-Volume Relationship for Platoon B
From these graphs, two loops formed by data "before" and "after" the platoon passed through a kinematic disturbance can be observed.

(a) The hysteresis loop.

Between the point of observed maximum density and the point of recovery, the normal behavior of traffic before maximum density and the retarded behavior of traffic immediately following maximum density form a loop. This is termed the hysteresis loop to emphasize the retarded performance of traffic flow during its recovery phase.

(b) The energy-gain loop.

Between the point of recovery and the point of return to normalcy, the behavior of traffic before the maximum density and the higher performance of traffic on its recovery path also form a loop. Since the hysteresis phenomenon of plastic materials is caused by energy dissipation, it is thought that the performance of traffic in this portion of its recovery phase is a result of regaining the energy lost in the hysteresis loop. In the following analysis of the traffic energy variations before and after maximum density, this hypothesis is partially verified. Therefore, this loop is termed the energy-gain loop to emphasize the superior performance of traffic on this portion of its releasing path.

Energy Analysis

A platoon of vehicles traveling along a roadway possesses two kinds of energy: kinetic energy and internal energy. The kinetic energy of a traffic flow is defined as the product of density and the square of the corresponding
mean speed. In reference (2) the coefficient of variation of speed (CVS) obtained by dividing the standard deviation of speed by the average speed has been shown to be a good indication of "internal energy." In Figure I-11, the kinetic energy and the coefficient of variation of speed for Platoon A are plotted against density. In Figure I-12 the same parameters are presented for Platoon B.

Following the time sequence of the data, it can be seen that as traffic recovers from maximum density, a decrease of kinetic energy results accompanied by an increase of the coefficient of variation of speed. At the point of recovery, the coefficient of variation of speed crosses its inbound path and keeps on decreasing. This is coupled with a significant build-up of kinetic energy. When traffic flow acquires further momentum from the point of recovery, the "after" condition shows higher kinetic energy and lower coefficient of variation of speed when compared with the "before" condition. This pattern seems to indicate that the hysteresis phenomenon of a traffic flow is caused by the transfer of kinetic energy to internal energy as traffic releases from a disturbance. Once the point of recovery is reached, a reverse transfer occurs with internal energy converted back into energy of movement.

Supporting Analyses

In order to obtain more insight into the hysteresis phenomenon of traffic so that proper interpretation of this phenomenon can be made, several other traffic parameters were analyzed.
Figure I-11 Kinetic Energy and Coefficient of Variation of Speed Versus Density for Platoon A
Figure I-12 Kinetic Energy and Coefficient of Variation of Speed Versus Density for Platoon B
Speed Dispersion Analysis

In Figures 1-13 and I-14 the standard deviation of vehicular speeds for Platoon A and Platoon B are plotted against density. By following the time sequence for Platoon A it can be seen that the standard deviation of speeds initially displayed low values, proceeded to a maximum at 55 vpm and then dropped to an absolute minimum at 92 vpm. It rose again to a local maximum at 125 vpm and then decreased as the density approached its maximum value. Immediately after the platoon emerged from the disturbance, the standard deviation assumed values lower than the "before" values at respective densities. It soon increased and surpassed the "before" values, however, reaching a local maximum at about 100 vpm. It then decreased to a value similar to the "before" value at about 78 vpm and continued to decrease without substantial changes in density. The density level remained at around 73 vpm for some time and the standard deviation reached a local minimum before density declined further. When traffic began to gain momentum, the standard deviation of speeds increased slowly with detectable oscillations.

For Platoon B the variation of the standard deviation of speeds exhibits a similar pattern, although the density values corresponding to the local minimums and local maximums are not the same as those of Platoon A.

Spacing Dispersion Analysis

In Figures I-15 and I-16, the standard deviation of vehicle spacings for Platoon A and Platoon B are plotted against density. Different variational patterns of the standard deviation of spacings can be clearly identified before
Figure I-13  Standard Deviation of Speed Versus Density for Platoon A
Figure I-15 Standard Deviation of Spacing Versus Density for Platoon A
Figure I-16 Standard Deviation of Spacing Versus Density for Platoon B
and after the maximum density.

Average Headway Analysis

In Figures I-17 and I-18 the variational patterns of average headway versus density for Platoon A and Platoon B are presented. By following the time sequence of Platoon A data, it can be seen that the traffic maintained an average headway of about two seconds in the density region between 33 vpm and 110 vpm. The average time headway then increased, first slowly and then sharply, as traffic flow approached its maximum density. When the platoon was released from the maximum density, the average headway decreased but maintained higher values at respective densities when compared to "before" conditions. The average headway finally reached the "before" value of two seconds at 73 vpm and kept decreasing while density remained nearly constant. After reaching a local minimum of about one second, the average headway then increased with a reduction in density and finally returned to the "before" condition at 33 vpm.

For Platoon B, the density region in which a nearly constant average headway was maintained is from 33 vpm to 90 vpm. The point where the recovery path intersected the "before" path is around 59 vpm. Other than these values, the average headway of Platoon B exhibits a similar pattern with density to that of Platoon A.

General Interpretation of the Experimental Analyses

Based on the information presented in the foregoing sections, the
**Figure I-17** Average Headway Versus Density for Platoon A
Figure I-18 Average Headway Versus Density for Platoon B
observed hysteresis phenomenon of a traffic stream can be interpreted as follows:

Consider a platoon of vehicles starting from a low density such that drivers are little affected by other vehicles and intervehicular spacings are large. When density increases, headways should be reduced due to the shortened spacings between vehicles. However, since a headway represents the time available for a following driver to respond to changes by the lead car, it seems logical to assume that a driver would choose to maintain a headway which is greater than, or at least equal to, his response time. The result is that faster moving vehicles tend to slow down, causing the entire platoon to decrease its speed and reducing the variation of spacing within the platoon, while the average headway remains at a constant value of about two seconds. As the process continues, the dispersion of speeds and spacings eventually reach minimum values where vehicles in the platoon are equally spaced and are moving at about the same speed. Driving at a constant speed, however, requires continuous attention in dense traffic and places a considerable strain on the drivers. Hence the dispersions of speed and spacing slowly begin to increase. As the density continues to rise, a further reduction of mean speed and a resulting increase in the average headway results. The traffic stream finally slows to a stop and a maximum density is obtained. Drivers obviously take care in the process to avoid possible conflicts.

When the queue begins to release from a stopped position, drivers tend to respond to changing traffic conditions more slowly than in the queue forming.
condition. This is because there is no immediate danger of rear end collision. The slow response by the drivers results in larger headways at the same densities than in the queue generating condition. This results in a less efficient traffic flow than that exhibited before the kinematic disturbance. In other words, the average speed of the traffic stream is lower than the speed of the queue generating condition at the same density level. As density decreases, with relatively large headways between vehicles, trailing vehicles tend to catch up; the result is increased mean speeds, reduced speed and spacing dispersions, and decreased average headways. As this process continues, the traffic eventually recovers from its retarded performance and reaches its "before" performance level with average headway reduced to the two second value. Since the dispersion of speed is relatively low at the point of recovery, the first vehicle of the platoon moves at about the same speed as the last vehicle; therefore, the length of the platoon remains unchanged. However, since the momentum of vehicles adjusting their speeds and spacings still prevails, vehicles tend to increase their speeds with reduced speed and spacing dispersions and without expanding the platoon length. The result is that the average speed of the platoon keeps increasing at the point of recovery without a change in density. As this process continues and the average headway reaches a value below two seconds, drivers experience headways which are not sufficient for them to respond to possible traffic changes. Therefore, trailing vehicles tend to slow their acceleration. This results in a decrease in density and an increase in average headways, spacing dispersion and speed
dispersion. Finally, traffic flow regains the "before" condition.

Resulting Effect of the Hysteresis Phenomenon

The above interpretation of the hysteresis phenomenon is only an attempt to describe and explain observed traffic behavior. Further work is necessary to confirm some of the assumptions made in the interpretation. However, there are two major conclusions which can be drawn at this stage of the investigation.

1. The movement patterns exhibited by traffic streams before entering a kinematic disturbance are different from the movement patterns exhibited after departing from a kinematic disturbance. Traffic performance is retarded when released from the area of maximum density and does not recover until a much lower density value is reached. As a result, a hysteresis loop occurs between maximum density and the point of recovery. When traffic density is further decreased below that at the point of recovery, traffic performance is superior to the condition prior to the congestion. The traffic finally returns to the original condition at a density of about 33 vpm. An energy-gain loop exists between the point of recovery and the point of return to normalcy.

2. At respective density levels, the average speed of the vehicles in a traffic stream is affected by the dispersion of vehicle speeds. A low coefficient of variation of speeds is associated with a high average speed.

As a result of the hysteresis analysis, it is felt that the whole traffic density domain can be divided into three different operating regions defined by
zero density, the point of return to normalcy, the point of recovery, and jam density. As far as the development of a suitable speed-density relationship is concerned, the discussed hysteresis phenomenon provides two major recommendations. First, since speed-density exhibits different relationships between a queue generating and a queue releasing condition, it seems appropriate that the speed-density relationship of queue generating conditions be treated separately from the speed-density relationship of queue releasing conditions. Second, since the hysteresis phenomenon tends to suggest that traffic can be divided into three different operating regions with density changes having different effects on speed changes in each of the regions, it seems logical that the speed-density relationship should be analyzed on a region by region basis.

Consequently, it was decided to concentrate efforts at this time on developing a speed-density relationship for the queue generating condition. The speed-density relationship for queue releasing conditions is left for future investigation.

1.1.3 ANALYSIS OF QUEUE FORMING BEHAVIOR

Although the hysteresis analysis suggests that traffic flow can be divided into three different operating regions, it is felt that detailed analyses of several traffic parameters during queue generating conditions are needed to confirm these regions and possibly to identify subregions.

Absolute and Marginal Safety Considerations

Absolute and marginal safety concepts in car-following have been intro-
duced in previous research work conducted at the Transportation Research Center (6). The absolute safety concept is defined as follows: "The leading vehicle is brought to a sudden stop by some object in the roadway (running into a suddenly appearing obstacle, heavy truck or container). The driver of the trailing vehicle reacts on the incidence of the collision and is able to stop his car without hitting the leading car in a rear-end collision. No space is left between the vehicles after the stopping maneuver."

The marginal safety concept is defined as follows: "The driver of the leading car is forced to bring his car to a standstill in an emergency and tries to stop his vehicle in the shortest possible distance. After some delay caused by reacting to the maneuver of the leading car, the driver of the trailing vehicle duplicates the braking maneuver of the leading car, and both vehicles come to a safe stop. No rear-end collision will occur although no space remains between the vehicles after stopping."

The absolute safety concept requires a spacing, $S_a$, between vehicles equal to the safe stopping distance and is given by

$$S_a = 1.47 VT + \frac{V^2}{30f}$$

where $V = \text{vehicle speed in mph}$

$T = \text{the reaction time of the driver in seconds, and}$

$f = \text{the coefficient of friction between tires and roadway}.$

The marginal safety concept requires less distance between successive vehicles and is mainly dependent on the reaction time of the driver. The spacing, $S_m$, required can be expressed as:
\[ S_m = 1.47 VT \]

where the terms are as defined above. Since speed is a function of traffic density, both of these safe spacings can be related to density.

Based on his driving experience, an alert driver can judge the risk of a rear end collision in accordance with his speed and the spacing available between his vehicle and the leading vehicle and will respond to changing traffic conditions accordingly. Therefore, a comparison between required safe spacings and actual observed spacings should provide some insight into different traffic operating conditions.

For Platoon A and Platoon B, absolutely safe spacings and marginally safe spacings were calculated based on a response time of two seconds and the appropriate velocity-dependent coefficient of friction for a dry pavement. The calculated marginally safe spacings versus density are shown in Figures I-19 and I-20. Absolutely safe spacings versus density can be seen in Figures I-21 and I-22. Solid curves shown in the graphs represent the observed average spacings at the respective densities. Dotted lines depict the estimated safe spacings in the low density region. The estimated values were obtained by extrapolation from the existing data points and by assuming a free flow speed of 70 mph.

From the graphs, it can be seen that the required safe spacing data for both safe spacing concepts intersects the average spacing curve at two points thus separating the density domain into three regions. However, the points of intersection are not the same for the two safe spacing concepts. In
Figure I-19 Marginally Safe Spacings Versus Density for Platoon A
(Prior-Queue Conditions)
Figure I-20  Marginally Safe Spacings Versus Density for Platoon B
(Prior-Queue Conditions)
Figure I-21 Absolutely Safe Spacings Versus Density for Platoon A (Prior-Queue Conditions)
Figure I-22 Absolutely Safe Spacings Versus Density for Platoon B (Prior-Queue Conditions)
the plots of marginally safe spacing versus density, the three regions identified can be seen to correspond to the three operating regions suggested by the hysteresis analysis, (i.e., the two intersecting points of marginally safe spacing and average spacing are coincident with the Point of Recovery and the Point of Return to Normalcy identified in the hysteresis analysis.) In the case of the absolutely safe spacings, however, Figure I-21 shows that Platoon A data intersects the average spacing at 14 vpm and 112 vpm. Figure I-22 shows that the two intersecting points are at 12 vpm and 99 vpm for Platoon B. The regions defined by these points are substantially different than those previously identified.

To summarize, the combined considerations of marginally safe spacings and absolutely safe spacings divide the traffic domain into five different operating regions. Consider Platoon A as an example. The first region is from zero density to 14 vpm; in this region average spacing is greater than both the required absolutely safe spacing and the required marginally safe spacing. The second identified region is between 14 vpm and 33 vpm. In this region average spacing is less than the required absolutely safe spacing but greater than the required marginally safe spacing. In the third region, between 33 vpm and 73 vpm, the average spacing is less than both the absolutely safe spacing and the marginally safe spacing. This density region offers the lowest degree of safety as far as safe spacings are concerned. The fourth region, from 73 vpm to 112 vpm, has a safe spacing relationship similar to that of the second region; however, the operating speed and actual spacings are considerably lower than 52
in the second region. The last region, from 112 vpm to jam density, has
average spacings greater than both the required absolutely safe spacing and
marginally safe spacing. This condition is similar to the first region; however,
actual spacings in this region are very low and vehicles are moving at very
low speeds. In addition to the five operating regions that can be identified by
the safe spacing analysis, several pertinent findings can be extracted from the
graphs.

1. The result of the marginally safe spacing analysis tends to confirm
the hypothesis that the Point of Recovery and the Point of Return to Normalcy
identified in the hysteresis analysis do separate different traffic operating
conditions.

2. It appears that drivers are more concerned about marginally safe
spacings than absolutely safe spacings. Marginal safety was maintained nearly
all of the time.

3. From the absolutely safe spacing graphs, it can be seen that only
at very low densities (less than 10 vpm) can drivers drive at speeds exceeding
70 mph and be considered absolutely safe. This seems to be the region with no
interaction between vehicles, that is, the region of free flow.

Average Headway Analysis

It is logical that when a driver drives on a crowded highway he tries to
maintain a headway which allows him enough time to respond to any possible
traffic changes. Therefore, the headway values obtained from a traffic stream
can be expected to provide information on prevailing traffic conditions
and consequently it should be possible to use these values to establish different operating regions.

Since no data was available for low density conditions, a theoretical analysis of headways at low densities was conducted. According to definition, the average headway (in seconds) of a traffic stream can be expressed as

\[ h = \frac{3600}{Q} = \frac{3600}{KV} \]

where \( Q \) is the volume of the traffic stream in vph, \( K \) is the traffic density in vpm and \( V \) is the average speed of the stream in mph. For different values of speed, the theoretical relationship between \( h \) and \( K \) is plotted in Figure I-23. It can be seen from the graph that for any speed, the average headway drops very quickly at first and then tapers off. In reality, since the average speed of the vehicles in a freeway platoon can be expected to be 60–70 mph in the low density region, it can be expected that the actual headway–density curve would be well confined in the narrow band bounded by the 50 mph and 80 mph curves. With the help of this theoretical analysis, headway–density relationships were estimated in the low density region for the two groups of vehicles. This information was added to the actual data collected and is shown in Figures I-24 and I-25. Five division lines separating different traffic operating regions are also shown on these graphs. The positions of the first four division lines were determined by the previously discussed hysteresis phenomenon and safe spacing analyses. The position of the last division line is based on the average headway analysis and was determined as follows.
Figure I-23  Theoretical Relationships Between Headway and Density for Different Speed Values

Headway = $\frac{3600}{KV}$
Figure I-24  Average Headway Versus Density for Platoon A
(Prior-Queue Conditions)
Figure I-25 Average Headway Versus Density for Platoon B (Prior-Queue Conditions)
Considering Platoon A, the variance of average headway with density can be described on a region-by-region basis in the following manner. In the region between zero density and 14 vpm, average headway drops sharply from infinity to about 5 seconds. In the region between 14 vpm and 33 vpm, average headway drops from 5 seconds to 2 seconds at a moderate rate. After the average headway reaches the 2 second value, a further increase of density no longer causes a reduction in average headway. In the region between 33 vpm and 73 vpm, average headway remains constant at 2 seconds. In the region between 73 vpm and 112 vpm, average headway assumes a value of 2.2 seconds most of the time, but increases slowly when density approaches 112 vpm. When density further increases, average headway also increases, first slowly and then sharply, as the density approaches its maximum value. The important point of the above description is that each of the operating regions defined previously characterizes a different average headway pattern.

Since a larger headway means less urgency in responding to traffic changes than a smaller headway, changes of density should have less effect on speed changes in a region with longer headways than in a region with smaller headways. In other words, traffic operating under different average headway conditions should perform differently. The observation presented in this analysis seems not only to confirm that the operating regions defined in previous analyses are reasonable, it also indicates that average headway itself can be used to define different operating regions.

By further examining the average headway data of Platoon A, it seems
that an additional density break-point that separates different operating regions can be positioned at 156 vpm. It can be seen that in the region between 112 and 156 vpm, average headway exhibits a variational pattern similar to that of the region between 73 vpm and 112 vpm; i.e., average headway remains constant at first but increases as traffic approaches another operating region. Once 156 vpm is passed, however, average headway values begin to increase sharply and continue to do so until the maximum density is reached. For Platoon B, the additional density break-point defined from average headway considerations is estimated to be at 128 vpm.

In addition to the above discussion, it is noted that the observed average headway seems to have a minimum value of 2 seconds when the hysteresis effect is not considered.

**Speed Dispersion Analysis**

In Figure I-26 the standard deviation of speeds for the vehicles in Platoon A is plotted against density. The corresponding information for Platoon B is shown in Figure I-27. The previously defined operating regions are separated by dividing lines as shown on the graphs. Since no data was observed in the low density regions, the dividing line in this region is not shown.

By examining the graphs, the pattern displayed by speed dispersions is evidently different in each of the operating regions. Since speed dispersion contains information on vehicle interactions which affects the performance of traffic, the different speed dispersion patterns observed in each region tend to confirm that the previously defined operating regions do indeed exist.
Figure 1-26 Standard Deviation of Speed Versus Density for Platoon A
(Prior-Queue Conditions)
Figure I-27 Standard Deviation of Speed Versus Density for Platoon B
(Prior-Queue Conditions)
1.1.4 SUMMARY OF OBSERVED TRAFFIC CHARACTERISTICS AND THEIR INFLUENCE ON SPEED-DENSITY RELATIONSHIPS

From the previously observed traffic characteristics, the traffic density domain seems to contain six different traffic operating regions. In order to provide an all-in-one reference to this conclusion, previously discussed parameters and their defined operating regions are shown collectively in Figures I-28 and I-29 for Platoon A and Platoon B, respectively. The presented parameters are average speed ($\bar{v}$), absolutely safe spacing ($S_a$), average headway ($\bar{H}$), and the standard deviation of speeds ($\sigma_v$). Marginally safe spacing is not included due to the limited graph size. It should be remembered, however, that the region break-points defined by the marginal safety concept correspond with those of the hysteresis analysis. The variation of average speed before and after a kinematic disturbance is presented to illustrate the hysteresis effect on the different operating regions. All other parameters represent only the conditions prior to a kinematic disturbance.

As far as the form of the speed-density relationship in the various operating regions is concerned, an interpretation based on driving responsibility can be made. This discussion is mainly a collective description of traffic behavior as exhibited by the analyzed traffic parameters.

Considering Platoon A, when vehicles travel in the density region from zero to 14 vpm, drivers enjoy an average headway of more than 5 seconds and "absolutely safe spacings" are provided. There is no urgency for any driver to respond to the maneuvers of other drivers. Therefore, changes in density will have little or no effect on drivers' speeds.
Figure I-28 Summary of Studied Platoon Parameters and Defined Operating Regions for Platoon A
Figure I-29 Summary of Studied Platoon Parameters and Defined Operating Regions for Platoon B
When the vehicles travel in the density region from 14 vpm to around 33 vpm, the average headway is reduced from 5 seconds to 2 seconds and a transition from absolutely safe car-following conditions to marginally safe car-following conditions occurs. Considering that the observed minimum average headway under normal operating conditions is 2 seconds, drivers operating in this region still maintain considerable freedom in their maneuverability. Therefore, changes of density will have only a minimal effect on speeds.

When the vehicles travel in the density region between 33 vpm and 73 vpm, drivers are operating in a condition where average headways are at two seconds and the average spacing is less than the spacing required for maintaining marginal safety. Changes of density should have the greatest effect on speeds in this region. Since the average headway value remains constant and the speed distribution does not change very much throughout this region, the unit change of average speed due to a unit change of density also remains relatively constant.

In the operating region between 73 vpm and 112 vpm, drivers face similar driving conditions as in the previous region but with a slightly increased average headway. The most important difference is that vehicles are moving at more uniform speeds than in the previous region. Therefore, the effect of a change in density on speed should be less pronounced in this region than in the previous region.

For the two remaining operating regions which are defined by 112 vpm to 156 vpm and 156 vpm to jam density, the previously stated arguments on average headway, safe spacings and speed dispersions can be applied and the result is that changes of density have only a small effect on speeds.
1.1.5 PROPOSED MULTI-LINEAR SPEED-DENSITY HYPOTHESIS

In general, the relationship between speed ($V$) and density ($K$) can be represented by a differential equation

$$\frac{dV}{dK} = -C \quad (0 \leq K \leq k_j)$$

$$k_j = \text{jam density}$$

where the minus sign indicates that an increase in density causes a decrease in speed and "C" indicates the particular effect of density changes on changes in speed. By assuming different "C" functions, different speed-density relationships can be presented. For example, a constant "C" value indicates that speed and density have a linear relationship.

From the previous discussion, if only prior queue conditions of traffic are to be considered, it seems that changes in density have different effects on changes in speed in six mutually exclusive operating regions. In other words, the speed-density differential equation can be rewritten as:

$$\frac{dV}{dK} = \begin{bmatrix} -b_1 \\ -b_2 \\ -b_3 \\ -b_4 \\ -b_5 \\ -b_6 \end{bmatrix} \quad (0 \leq K \leq k_j)$$

(for the first region)

(for the second region)

(for the third region)

(for the fourth region)

(for the fifth region)

(for the sixth region)

In the above expression, $b_i$ represents the specific effect of density...
changes on changes of speed in the i-th operating region. From the material presented in Section 1.1.4, it seems that two assumptions with regard to the properties of the $b_i$'s can be made. The first one is that the $b_i$'s are constants. The second one is that the $b_i$'s of neighboring operating regions are different. It also can be claimed that $b_1 < b_2 < b_3$ and $b_4 > b_5 > b_6$.

Based on the first assumption, a speed-density hypothesis can be obtained by integration. The resulting relationships are:

$$V = \begin{bmatrix}
    a_1 - b_1 K \\
    a_2 - b_2 K \\
    a_3 - b_3 K \\
    a_4 - b_4 K \\
    a_5 - b_5 K \\
    a_6 - b_6 K
\end{bmatrix}
\quad \text{(for the first region)}$$

$$V = \begin{bmatrix}
    a_2 - b_2 K \\
    a_3 - b_3 K \\
    a_4 - b_4 K \\
    a_5 - b_5 K \\
    a_6 - b_6 K
\end{bmatrix}
\quad \text{(for the second region)}$$

$$V = \begin{bmatrix}
    a_3 - b_3 K \\
    a_4 - b_4 K \\
    a_5 - b_5 K \\
    a_6 - b_6 K
\end{bmatrix}
\quad \text{(for the third region)}$$

$$V = \begin{bmatrix}
    a_4 - b_4 K \\
    a_5 - b_5 K \\
    a_6 - b_6 K
\end{bmatrix}
\quad \text{(for the fourth region)}$$

$$V = \begin{bmatrix}
    a_5 - b_5 K \\
    a_6 - b_6 K
\end{bmatrix}
\quad \text{(for the fifth region)}$$

$$V = \begin{bmatrix}
    a_6 - b_6 K
\end{bmatrix}
\quad \text{(for the sixth region)}$$

where $a_i$'s $(i = 1, \ldots, 6)$ are integration constants.

Physically, this proposed speed-density relationship is a multi-linear function which consists of six linear segments. It can also be stated that the segment in the third region exhibits the steepest slope. The very nature of traffic requires that speed and density vary in a continuous manner with respect to time. Therefore, if this relationship is to be a valid speed-density model, the six linear segments should present a continuous function with no discontinuities as traffic moves from one operating region to another.
1.2 VERIFICATION OF THE MULTI-LINEAR MODEL

Since the proposed multi-linear model hypothesizes that a linear relationship exists between speed and density in each of the previously defined operating regions, a statistical verification of the model must rest mainly upon the proof of the linearity of the speed-density data in each of these regions. This problem is approached by applying linear regression techniques to the speed-density data in each region to obtain the corresponding linear correlation coefficient. Due to the nature of a regression analysis, however, fluctuations in the data will cause discontinuity of the least-square lines obtained at the junction of every two adjacent operating regions. Since continuity is essential for the model to be a valid representation of the speed-density relationship, it must be proved that the gaps created between neighboring least-square lines are due to errors and not due to any inherent weakness of the model. In addition to establishing the linearity and continuity of the model, it is also necessary to prove that any two neighboring least-square lines have different slopes. This is required to confirm the statement that density changes have different effects on speeds in different operating regions.

Thus, the verification of the multi-linear speed-density model will be performed in three stages.

1. Linear regression analysis will be applied to the prior-queue speed-density data in each operating region. For every region, the output of the analysis will include a linear correlation coefficient, a least-square line describing the speed-density relationship, and a 90% confidence interval for the least-square line.
2. As a result of the regression, gaps will be created at the density break-points separating adjoining operating regions. In order to prove that these gaps are due to chance (so that continuity of the model can be maintained), a test of the null hypothesis that the expected size of each gap is zero will be performed. A t-test is employed for this purpose.

3. A t-test also will be used to test the hypothesis that a significant difference exists between the slopes of any two adjacent least-square lines.

1.2.1 DESCRIPTION OF THE STATISTICAL ANALYSIS

The three part statistical analysis described above was conducted using the speed-density data for the queue forming condition presented for Platoon A and Platoon B in the previous section.

**Linear Regression Analysis**

Theoretically, for a random sample of \( n \) pairs of values of \( x \) and \( y \) represented by \((x_i, y_i), (i = 1, 2, \ldots, n)\), the linear correlation coefficient \( r \) can be estimated by

\[
r = \frac{\sum_i (x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\left[\sum_i (x_i - \bar{x})^2\right] \left[\sum_i (y_i - \bar{y})^2\right]}}
\]

where

\[
\bar{x} = \frac{\sum_i x_i}{n}, \quad \bar{y} = \frac{\sum_i y_i}{n}
\]
The least-square line, \( y = a + bx \), for these \( n \) pairs of \( x \) and \( y \) values can be constructed by calculating its slope "\( b \)" according to the equation

\[
b = \frac{n \sum_i x_i y_i - \sum_i x_i \sum_i y_i}{n \sum_i x_i^2 - (\sum_i x_i)^2}
\]

and its \( y \)-intercept "\( a \)" can be calculated by

\[
a = \bar{y} - b \bar{x}
\]

According to statistical theory, the coefficients "\( a \)" and "\( b \)" have normal distributions. Thus confidence intervals can be readily calculated for each coefficient. For "\( b \)" the 100 \(( 1 - \alpha \) per cent confidence interval is between

\[
b + (t_{n-2}, 1 - \alpha/2 \cdot \hat{\sigma}_b) \quad \text{and} \quad b - (t_{n-2}, 1 - \alpha/2 \cdot \hat{\sigma}_b)
\]

where \( t_{n-2} \), \( 1 - \alpha/2 \) is the \( t \) value obtained for \((n-2)\) degrees of freedom and a cumulative probability of \( 1 - \alpha/2 \). \( \hat{\sigma}_b \) is the estimated standard deviation of "\( b \)" and can be calculated from

\[
\hat{\sigma}_b = \left[ \frac{\sum_i(y_i - a - bx_i)^2}{(n-2)\sum(x_i - \bar{x})^2} \right]^{1/2}
\]

For the \( y \)-intercept, "\( a \)", the 100 \(( 1 - \alpha \) per cent confidence interval is between

\[
a + (t_{n-2}, 1 - \alpha/2 \cdot \hat{\sigma}_a) \quad \text{and} \quad a - (t_{n-2}, 1 - \alpha/2 \cdot \hat{\sigma}_a)
\]

where \( \hat{\sigma}_a \) is the estimated standard deviation of "\( a \)" and can be calculated from

\[
\hat{\sigma}_a = \left[ \frac{\sum_i(y_i - a - bx_i)^2}{n-2} \cdot \left( \frac{1}{n} + \frac{\bar{x}^2}{\sum_i(x_i - \bar{x})^2} \right) \right]^{1/2}
\]

Linear regression analyses were applied to both Platoon A and Platoon B data.
Due to the lack of observations in the low density regions, only four out of the six previously defined operating regions could be analyzed. Figures I-30 and I-31 show the speed-density data and the fitted least-square lines for each region. The resulting correlation coefficients, the equations of the least-square lines, and the 90% confidence intervals for slopes and y-intercepts are given in Table I-1. From the graphs, it can be seen that Platoon A has an estimated jam density of 222 vpm and Platoon B has an estimated jam density of 214 vpm.

Test for Continuity of the Multi-Linear Model

In Figures I-30 and I-31 gaps can be seen between the two neighboring least-square lines at every break-point separating two adjacent operating regions. Since speed and density of a platoon of vehicles should vary continuously, it must be proved that the discontinuation of neighboring least-square lines is due entirely to errors of observation. Otherwise, the multi-linear speed-density model cannot be accepted as a valid representation of the actual speed-density relationship.

The proof is conducted as follows. At any density break-point, a least-square estimate of speed, \( V_1 \), can be obtained from the data in the operating region to the left of the break-point. According to statistical theory, if a least-square line \( y = a + bx \) is constructed from \( n \) pairs of \( x \) and \( y \) values, the \( y \) of any point \( (x_0, y_0) \) on this line has an estimated variance, \( \hat{\sigma}_{y_0}^2 \), which is given by

\[
\hat{\sigma}_{y_0}^2 = \sum_i \left[ (x_i - \bar{x}) + (\bar{x} - x_0) \right]^2 \quad (i=1,2,\ldots,n)
\]
Figure I-30 Least-Square Multi-Linear Speed-Density Function for Platoon A
(Prior-Queue Conditions)
Figure I-31 Least-Square Multi-Linear Speed-Density Function for Platoon B
(Prior-Queue Conditions)
### TABLE I-1

RESULTS OF REGRESSION ANALYSES FOR SPEED-DENSITY DATA IN EACH OPERATING REGION

<table>
<thead>
<tr>
<th>Platoon</th>
<th>Operating Regions</th>
<th>Least Square Line ( V = a + bK )</th>
<th>( r ) (%)</th>
<th>Stai Deviation</th>
<th>90% Confidence Interval</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \hat{a} )</td>
<td>( \hat{b} )</td>
</tr>
<tr>
<td><strong>A</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>33 - 73</td>
<td>( V = 71.24 - 0.651K )</td>
<td>99.52</td>
<td>0.77</td>
<td>0.015</td>
<td>69.90 - 72.58</td>
</tr>
<tr>
<td>73 - 112</td>
<td>( V = 46.10 - 0.096K )</td>
<td>94.27</td>
<td>2.16</td>
<td>0.024</td>
<td>42.36 - 49.84</td>
</tr>
<tr>
<td>112 - 156</td>
<td>( V = 2362 - 0.139K )</td>
<td>97.35</td>
<td>3.03</td>
<td>0.023</td>
<td>19.77 - 37.47</td>
</tr>
<tr>
<td>156 - 222</td>
<td>( V = 24.09 - 0.109K )</td>
<td>99.24</td>
<td>1.38</td>
<td>0.008</td>
<td>20.84 - 27.34</td>
</tr>
<tr>
<td><strong>B</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>33 - 59</td>
<td>( V = 93.33 - 1.09K )</td>
<td>98.34</td>
<td>2.15</td>
<td>0.051</td>
<td>89.60 - 97.06</td>
</tr>
<tr>
<td>59 - 100</td>
<td>( V = 54.33 - 0.115K )</td>
<td>99.48</td>
<td>0.99</td>
<td>0.012</td>
<td>52.57 - 56.09</td>
</tr>
<tr>
<td>100 - 128</td>
<td>( V = 24.45 - 0.122K )</td>
<td>98.96</td>
<td>0.98</td>
<td>0.009</td>
<td>22.36 - 26.54</td>
</tr>
<tr>
<td>128 - 214</td>
<td>( V = 20.26 - 0.091K )</td>
<td>97.60</td>
<td>1.64</td>
<td>0.010</td>
<td>16.77 - 23.75</td>
</tr>
</tbody>
</table>
where $\bar{x} = \frac{1}{n} \sum x_i$ and $\hat{\sigma}_{y_0}^2$ has degrees of freedom equal to (n-2). From this formula, the variance $\hat{\sigma}_1^2$ of $V_1$ can be obtained. Then, considering the operating region to the right of the break-point, similar calculations can be made and a speed value $V_r$ and its variance $\hat{\sigma}_r^2$ can be obtained.

It is obvious that $(V_1 - V_r)$ represents the gap between the two adjacent least-square lines at the density break-point. Since continuity of the multi-linear model requires that no gap exist between any two line segments, the null hypothesis that the expected value of $(V_1 - V_r)$ equals zero can be made. From the nature of observation errors, $V_1$ and $V_r$ can be assumed to have normal distributions. Therefore, $(V_1 - V_r)$ is also normally distributed with a variance which equals the sum of $\hat{\sigma}_1^2$ and $\hat{\sigma}_r^2$. The aforementioned null hypothesis can be tested using a t-test. In this case the t value is obtained from

$$\quad t = \frac{V_1 - V_r}{\sqrt{\hat{\sigma}_1^2 + \hat{\sigma}_r^2}} \quad \text{(degree of freedom} = \text{n}_1 + \text{n}_r - 4)$$

where $n_1$ is the number of data points in the region to the left of the break-point and $n_r$ is the number of data points in the region to the right of the break-point.

This t-test was applied to Platoon A and Platoon B data and the results of the tests are presented in Table I-2.

**Test of Differences of Slopes**

Since the slope of the speed-density line represents the effect of changing density on changes in speed in a given region, failure to prove that slopes are significantly different for any two adjacent lines would mean that no real
TABLE I-2

T-TEST RESULTS OF CONTINUITY OF THE MULTI-LINEAR SPEED DENSITY MODEL

<table>
<thead>
<tr>
<th>Platoon</th>
<th>Density Break Points (vpm)</th>
<th>$V_0$ (mph)</th>
<th>$\hat{\sigma}_0^2$ (mph)</th>
<th>$V_r$ (mph)</th>
<th>$\hat{\sigma}_r^2$ (mph)</th>
<th>$t = \frac{V_0 - V_r}{\sqrt{\frac{\hat{\sigma}_0^2}{n} + \frac{\hat{\sigma}_r^2}{n}}}$</th>
<th>Degree of Freedom</th>
<th>5% Critical t value</th>
<th>Null Hypothesis $V_0 - V_r = 0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>73</td>
<td>23.72</td>
<td>.1586</td>
<td>24.59</td>
<td>.1903</td>
<td>1.471</td>
<td>27</td>
<td>2.052</td>
<td>Accepted</td>
</tr>
<tr>
<td></td>
<td>112</td>
<td>12.95</td>
<td>.3233</td>
<td>13.15</td>
<td>.2263</td>
<td>0.279</td>
<td>21</td>
<td>2.080</td>
<td>Accepted</td>
</tr>
<tr>
<td></td>
<td>156</td>
<td>6.94</td>
<td>.4532</td>
<td>7.24</td>
<td>.0403</td>
<td>0.427</td>
<td>5</td>
<td>2.571</td>
<td>Accepted</td>
</tr>
<tr>
<td>B</td>
<td>59</td>
<td>27.90</td>
<td>.9410</td>
<td>29.84</td>
<td>.0766</td>
<td>1.840</td>
<td>28</td>
<td>2.048</td>
<td>Accepted</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>12.83</td>
<td>.0751</td>
<td>12.25</td>
<td>.0122</td>
<td>1.970</td>
<td>16</td>
<td>2.120</td>
<td>Accepted</td>
</tr>
<tr>
<td></td>
<td>128</td>
<td>8.83</td>
<td>.0327</td>
<td>8.52</td>
<td>.1452</td>
<td>0.736</td>
<td>8</td>
<td>2.306</td>
<td>Accepted</td>
</tr>
</tbody>
</table>
operating differences exist between the two regions involved. It is therefore essential to show that the least-square lines in adjacent regions have significantly different slopes. From the linear regression analyses, slopes and their variances for the least-square line in each operating region have been calculated. For two slopes, \( b \) and \( b' \), and their variances, \( \sigma_b^2 \) and \( \sigma_{b'}^2 \), a t-statistic can be obtained by using the following formula:

\[
t = \frac{b - b'}{\sqrt{\sigma_b^2 + \sigma_{b'}^2}}
\]

(degree of freedom = \( n + n' - 4 \))

where \( n \) is the number of data points used to derive \( b \) and \( \sigma_b^2 \), and \( n' \) is the number of data points used to derive \( b' \) and \( \sigma_{b'}^2 \).

From the calculated t value, the null hypothesis that \( b \) is not significantly different from \( b' \) can be tested. This test was applied to Platoon A and Platoon B data and the results are given in Table I-3.

1.2.2 RESULTS OF THE STATISTICAL ANALYSIS

Shortcomings of the Data

The major weakness of the analysis for verification of the multi-linear model is that insufficient data points have been observed. Even though the material presented in Section 1.1 indicated that the whole traffic domain can be divided into six different operating regions, no data was observed in the first two regions. Consequently, only four regions can be verified. It can also be seen that data points in the fifth and sixth regions are rather scarce.
<table>
<thead>
<tr>
<th>Platoon</th>
<th>Regions Compared (rpm)</th>
<th>$b$</th>
<th>$\hat{\sigma}_b^2$</th>
<th>$b'$</th>
<th>$\hat{\sigma}_b'^2$</th>
<th>$t = \frac{b - b'}{\sqrt{\frac{\hat{\sigma}_b^2}{n} + \frac{\hat{\sigma}_b'^2}{n}}}$</th>
<th>Degree of Freedom</th>
<th>5% Critical t value</th>
<th>Null Hypothesis $b = b'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>33 - 73 to 73 - 112</td>
<td>0.651</td>
<td>0.000225</td>
<td></td>
<td>0.296</td>
<td>0.000576</td>
<td>12.54</td>
<td>27</td>
<td>2.052</td>
</tr>
<tr>
<td></td>
<td>73 - 112 to 112 - 156</td>
<td>0.296</td>
<td>0.000576</td>
<td></td>
<td>0.139</td>
<td>0.000529</td>
<td>4.76</td>
<td>21</td>
<td>2.080</td>
</tr>
<tr>
<td></td>
<td>112 - 156 to 156 - 222</td>
<td>0.139</td>
<td>0.000529</td>
<td></td>
<td>0.108</td>
<td>0.000064</td>
<td>1.28</td>
<td>5</td>
<td>2.571</td>
</tr>
<tr>
<td>B</td>
<td>33 - 59 to 59 - 100</td>
<td>1.109</td>
<td>0.002600</td>
<td></td>
<td>0.415</td>
<td>0.000144</td>
<td>13.33</td>
<td>28</td>
<td>2.048</td>
</tr>
<tr>
<td></td>
<td>59 - 100 to 100 - 128</td>
<td>0.415</td>
<td>0.000144</td>
<td></td>
<td>0.122</td>
<td>0.000081</td>
<td>13.96</td>
<td>16</td>
<td>2.120</td>
</tr>
<tr>
<td></td>
<td>100 - 128 to 128 - 214</td>
<td>0.122</td>
<td>0.000081</td>
<td></td>
<td>0.091</td>
<td>0.000100</td>
<td>2.38</td>
<td>8</td>
<td>2.306</td>
</tr>
</tbody>
</table>
Discussion of the Experimental Results

In the linear regression analysis, high linear correlation coefficients were found for speed-density data in every operating region analyzed. The lowest value found was 94.27% and all other values exceeded 97%. These high linear correlation coefficients indicate that speed and density have a strong tendency to linearity in each of the operating regions tested.

In the test of the continuity of the multi-linear model, all gaps which appeared at the density break-points were found to be a result of chance. This result indicates that the proposed multi-linear model is essentially continuous over the entire density domain.

In the test for differences of slopes, the slopes of all neighboring least-square lines, with the exception of one pair, were found to be significantly different from one another. This result favors the conclusion that changing density has different effects on speed in the different operating regions.

Judging from the results of all of the statistical tests made, it appears that the proposed multi-linear speed-density model is statistically acceptable as a representation of the speed-density relationship for the density range analyzed.

Development of Derived Volume-Density Relationship

Volume-density plots for Platoon A and Platoon B are presented in Figures I-32 and I-33. In the low density regions, three lines are drawn to represent volume-density relationships for constant speeds of 50, 60 and 70
Figure I-33 Derived Volume-Density Function for Platoon B
(Prior-Queue Conditions)
mph. Since speeds in the low density region would most likely vary within the range from 50 mph to 70 mph, the actual volume-density curve is expected to fall within the narrow band bounded by the 50 mph and the 70 mph lines. Guided by these two lines and based on the assumption that vehicles travel at 70 mph when densities are below 10 vpm (free flow region), the volume-density curves in the low density region were estimated and are shown by the dotted lines. For the remaining portion of the graphs, actual volume-density data and theoretical curves derived from the multi-linear speed-density relationship are presented. From these graphs, it can be seen that the analyzed density range covers both the high density, low volume regions where most traffic problems occur, and the region where traffic flow reaches its maximum value. These are the regions which are most important for purposes of freeway traffic control. Therefore, the multi-linear speed-density model should lend itself well to the application of freeway control strategies.

1.3 APPLICATION OF THE MULTI-LINEAR MODEL: THE LEVEL OF SERVICE CONCEPT AND DEVELOPMENT OF CONTROL STANDARDS

The traffic carrying capability of a roadway facility can be increased either by physical expansion of the facility or by application of properly chosen traffic control techniques. In many cases, however, expansion is not possible due to physical or financial limitations. Therefore, traffic control becomes the only available tool for improving traffic flow conditions. Effective control requires the identification of the characteristics of different levels of traffic
operation so that a decision can be made as to what type of control is needed and when it should be applied. Attempting to identify and label the various regions of traffic operation has thus been a subject to which many researchers have devoted their efforts. Examples of research in this area can be found in References 8, 9, 10 and 11.

The most widely used parameter in previous attempts to develop a level of traffic service framework has been the average operating speed of the traffic stream. Although speed has an intuitive appeal since it is a parameter to which the driver is sensitive, it is felt that density provides an improved indication of differing regions of traffic operation. In general, speeds range from zero to 70 mph on most freeways while densities range from zero to a jam value of about 220 vpm. Consequently, density will provide a more sensitive measure of changing conditions than will average speed. Density can be used in all cases while the upper limit of speed is restricted by the geometric and control conditions on a given facility. In addition, a density value presents a clearer picture of actual traffic conditions than does speed. For instance, let us consider a traffic stream moving at an average speed of 30 mph and at a density of 60 vpm. Only one of these two parameters is to be used to describe conditions in the traffic stream. The density value of 60 vpm suggests that the average vehicle spacing is at 88 feet (which is about 4 to 5 vehicle lengths). This traffic condition can be visualized in an imaginary manner and the associated traffic speed can often be estimated. On the other hand, merely mentioning the fact that traffic moves at 30 mph will not directly lead to such an imaginary picture of the actual condition.
1.3.1 Delineating Traffic Operating Regions

From the previous analyses, it appears that the entire traffic domain can be divided into six mutually exclusive operating regions; however, two problems have yet to be solved before the obtained information can be applied for control purposes.

The first problem is the determination of acceptable density break-point values that can be used to describe traffic operating regions for every day traffic conditions. For Platoon A, the five density break-points that separate the six operating regions were found to be 14 vpm, 33 vpm, 73 vpm, 112 vpm, and 156 vpm. For Platoon B, the values were 12 vpm, 33 vpm, 59 vpm, 100 vpm, and 128 vpm. In the previous study of platoon behavior conducted during the first phase of this research project, the behavior of eight platoons were studied using aerial data. From the analysis of the data of these platoons in conjunction with that of Platoons A and B it was felt that 30 vpm, 60 vpm, 105 vpm, and 145 vpm represented acceptable dividing lines for the second, third, fourth, and fifth density subregions, respectively. The first density break-point is positioned at 10 vpm based on considerations that are explained later.

The second problem is the determination of the effect of the hysteresis phenomenon on the characteristics of the operating regions. From previous analyses it is known that the hysteresis phenomenon can affect the value of the maximum traffic volume, the position of density where this maximum volume occurs, and the behavior of traffic at a given density level; however, the
hysteresis phenomenon does not affect the positions of the density break-points that separate operating regions. It is also true that the hysteresis phenomenon does not change the relative performances of the operating regions. For example, the third region is always the region where the maximum volume of a traffic stream occurs.

The operating characteristics of each density region can now be generally described in the manner presented below. Those traffic characteristics which are affected by the hysteresis phenomena are omitted from the discussion.

1. Free flow region. This region has a density span from zero to 10 vpm. From the absolutely safe spacing analyses of Figure I-22, it can be seen that for a speed of 70 mph and a reaction time of 2 seconds, the provided average spacing between vehicles at 12 vehicles per mile met the absolute safety requirements. It seems that driving at 70 mph and being able to maintain an absolute safe spacing are good criteria for free flow conditions. Therefore, the rounded figure of 10 vehicles per mile is thought to be a reasonable choice as the upper limit of the free flow region. From another viewpoint, since a small increase of density in this low density region would cause a tremendous decrease of average spacing between vehicles, a conservative choice of 10 vpm as the maximum free flow density seems logical.

2. Semi-free flow region. This region covers the density span from 10 vehicles per mile to 30 vehicles per mile. In this region the safe spacing analysis applies. Absolutely safe spacing is not maintained, but marginally safe spacing is provided. The average speed of vehicles in this range varies

85
from 50 mph to 70 mph. This high operating speed indicates that vehicles in this region still have a great deal of freedom in their movement. Accordingly, this region is referred to as a semi-free flow region.

3. Capacity flow region. This region covers the density span from 30 vpm to 60 vpm. In this region the average headway varies in the narrow range of 1.8 seconds to 2 seconds. The 1.8 second average headway is thought to be the minimum average headway value that is acceptable to most drivers in normal operating conditions. This average headway corresponds to a flow rate of 2000 vph per lane which is the accepted nominal capacity of a freeway lane under ideal conditions. Marginally safe spacing is maintained in this region only when response times are less than 1.8 seconds requiring considerable alertness on the part of the drivers. The nearly constant but low value of the average headway in this region indicates the tremendous influence of changing density on average speed. The average speed has a wide range of variation from 60 mph to 25 mph as motorists continually vary their speeds to maintain the nearly constant headway value. The relatively wide range of speed variations (individual vehicles) as shown by the vehicle interaction study indicates that the choice of individual speed is not severely restricted which tends to suggest that this region is tolerable to most drivers. Since this is the region where the capacity flow (maximum volume) of traffic is obtained, it is termed the capacity flow region.

4. Restricted flow region. This region is bounded by 60 vpm and 105 vpm. In this region, the average headway value is somewhat higher than 2
seconds but remains in a narrow range. Marginally safe spacing (for a response time of 2 seconds) is barely satisfied in this region. This indicates that a high degree of driver alertness is still required. The average speed does not have as wide a range as in the previous region indicating that the influence of density changes on speed is not as great as in the capacity flow region. This is because vehicles are moving with approximately uniform speeds and spacings, (low \( \sigma_v \) and \( \sigma_s \)) thus the shortening of total platoon length due to increased density is absorbed by every vehicle instead of by one or two vehicles. Since the vehicles operating in this region tend to float with the traffic stream having little choice about their own speed, this region is called the restricted flow region. The volume in this region is quite high and according to some speed-density hypotheses it is in this region that maximum volume occurs.

5. Disturbed flow region. This region is bounded by densities of 105 vpm and 145 vpm. In this region the average headway varies little in the range from 2.5 seconds to 3.5 seconds. The absolute and marginal safety spacings are provided in this region because of the low average speed. Despite the low average speed, the standard deviation of speeds is relatively high (much higher than in the previous region) which indicates the vehicles in this region no longer move in a uniform manner. It is termed the disturbed flow region to describe its non-uniform flow condition.

6. Forced flow region. This is the region where density exceeds the value of 145 vpm. The average headway value begins around 3.4 seconds when
traffic enters this region and increases sharply as the density approaches $k_j$. Both absolutely safe and marginally safe spacings are provided because of the low average speed. No freedom of movement is available to drivers in this region and they are forced to move as dictated by the stream as a whole.

The above traffic operating regions and their operating characteristics are summarized in Table I-4.

1.3.2 LEVEL OF SERVICE CONCEPT FOR UNINTERRUPTED FLOW

The late Dr. Johannes F. Schwar, when he was the chairman of the Highway Research Board Committee on Quality of Traffic Service, wrote an article entitled, "Quality of Traffic Service" (12). The basic qualitative considerations involved in the problem of defining levels of traffic service were well summarized in this article. The tentative definition of level of service suggested was "The quality of level of traffic service is a measure of the adequacy of a road, street, highway or system when compared to desirable and practical standards... A meaningful and easily measured parameter is average over-all speed. This measure of traffic service is influenced by and affects other parameters." The last sentence in the above quoted statement recognizes the fact that participating parameters involved in determining the level of service are not independent of each other. Thus, any parameter chosen as an index of level of service should be representative of all parameters considered. Apparently this philosophy was adapted by the authors of the 1965 Highway Capacity Manual (HCM) and level of service standards for uninterrupted flow were accordingly suggested. The resultant tabular form of the HCM
<table>
<thead>
<tr>
<th>TRAFFIC FLOW CONDITIONS</th>
<th>DENSITY RANGE (VPH)</th>
<th>OPERATING SPEED RANGE (mph)</th>
<th>AVERAGE headway</th>
<th>STANDARD deviation of VEHICULAR SPEEDS</th>
<th>SAFETY SPACINGS CONSIDERATION</th>
<th>GENERAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free Flow</td>
<td>0-10</td>
<td>50-70</td>
<td>&gt; 5 secs, drops quickly when density increases.</td>
<td>VARY</td>
<td>Absolute safety spacing provided at all times.</td>
<td>Complete freedom of movement of individual vehicle.</td>
</tr>
<tr>
<td>Semi-Free Flow</td>
<td>10-30</td>
<td>60-70</td>
<td>drops slowly from 5 secs to 2 secs as density increases.</td>
<td>VARY</td>
<td>Marginal safety spacing provided at all times.</td>
<td>Freedom of movement of individual vehicle is partially restricted.</td>
</tr>
<tr>
<td>Capacity Flow</td>
<td>30-60</td>
<td>25-60</td>
<td>≤ 2.0 secs, most values stay around 1.8 secs.</td>
<td>Relatively high</td>
<td>Marginal safety spacing provided only with driver awareness.</td>
<td>Average speed is restricted, but freedom of choice is not seriously restricted. (Max. kinetic energy and capacity attained)</td>
</tr>
<tr>
<td>Restricted Flow</td>
<td>60-105</td>
<td>15-30</td>
<td>≥ 2.0 secs, most values stay around 2.1 secs.</td>
<td>Relatively low (reaches a legal min.)</td>
<td>Marginal safety spacing barely provided.</td>
<td>Average speed as well as individual speed are highly restricted; vehicles &quot;flow&quot; in the traffic stream.</td>
</tr>
<tr>
<td>Disturbed Flow</td>
<td>105-145</td>
<td>8-15</td>
<td>most values remain constant in the range between 2.5 secs.</td>
<td>Relatively high</td>
<td>Absolute safety spacing provided at all times</td>
<td>High variation of individual speed associated with low average speed indicates the traffic flow is disturbed.</td>
</tr>
<tr>
<td>Forced Flow</td>
<td>145</td>
<td>&lt; 8</td>
<td>&gt; 3.6 secs, increases sharply as density approaches Kj.</td>
<td>Relatively low</td>
<td>Absolute safety spacing provided at all times</td>
<td>Vehicle maneuverability is severely restricted.</td>
</tr>
</tbody>
</table>
recommendation is duplicated and shown in Table I-5.

From the material contained in Section 1.3.1, it should be possible to establish a framework for level of service based on density considerations. However, before such a system is introduced, further study of the HCM system will be of value. This system is based on average speed as the fundamental parameter. An equivalent density-based system can be defined, however, by employing the deterministic relationship between speed, density and volume. In Column 3 of Table I-5, the minimum operating speeds for each service level are listed. Their corresponding upper density limits can be calculated by using the maximum volume values appearing in Column 9 of the same table divided by the speed values in Column 3. Such calculations have been made and the results are summarized in Table I-6. The density regions obtained in this manner can now be compared with those of Table I-4. This comparison leads to the following observations:

1. The region of Level of Service A suggested by the Highway Capacity Manual is the free flow region defined in Section 1.3.1

2. The combined region of Levels of Service B and C specified in the HCM as the stable flow region covers the same density range suggested as the semi-free flow region in Section 1.3.1

3. The combined region of Levels of Service D and E specified in the HCM as the unstable flow region covers the same range suggested as the capacity flow region in Section 1.3.1

4. The region of Level of Service F suggested by the HCM is further
# Table I-5

**Levels of Service and Maximum Service Volumes for Freeways and Expressways Under Uninterrupted Flow Conditions (HCM 1965)**

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>Description</th>
<th>Operating Speed (kph)</th>
<th>Basic Limiting Value for Average Highway Speed (Lane) of 70 MPH, for:</th>
<th>Approximate Working Value For Any Number of Lanes For Instructed Average Highway Speed of:</th>
<th>Maximum Service Volume Under Ideal Conditions, Including 70 MPH Average Highway Speed (Total Passenger Cars Per Hour, One Direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A</strong></td>
<td>Free flow</td>
<td>560</td>
<td>20.35</td>
<td>20.40</td>
<td>20.43</td>
</tr>
<tr>
<td><strong>B</strong></td>
<td>Stable flow (upper speed range)</td>
<td>555</td>
<td>20.50</td>
<td>20.58</td>
<td>20.63</td>
</tr>
</tbody>
</table>

**Peak-Hour Factor (PHF)**

| **C** | Stable flow | 550 | 20.75xPHF | 20.80xPHF | 20.83xPHF | 20.45xPHF | - | 1350 | 1500 | 1500 | 1500 | 1500 | 1500 | 1500 | 1500 | 1500 |
| **D** | Approaching unstable flow | 540 | 20.90xPHF | 20.80xPHF | 20.45xPHF | - | 1500 | 1500 | 1500 | 1500 | 1500 |
| **E** | Unstable flow | 30-35* | 21.00 | 21.00 * | 21.00 * | 21.00 * | 21.00 * | 21.00 * | 21.00 * | 21.00 * |
| **F** | Forced flow | <30* | Not meaningful | Widiy variable (0 to capacity) |

---

*Operating speed and headway are independent measures of level of service; both limits must be satisfied to any designation of level.

Operating speed applied for this level is any available even at lower volumes.

4-lane freeway in the ratio of half the available volume to the highest rate of flow occurring during a 5-mi interval within the peak hour.

4-lane freeway based on the volume of the available volume to the highest rate of flow occurring during a 5-mi interval within the peak hour.

A peak-hour factor of 1.00 is assumed, but the working limit flow should be considered as maximum average flow rates likely to be obtained during the peak 5-mi interval within the peak hour.

Approximately.

Capacity.
### Table I-6

**Upper-Density Limits for Different Service Levels Suggested by HCM (1965)**

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>Maximum Service Volume (Column 9 of Table 5) VPH / 2 Lanes</th>
<th>Min. Operating Speed (Column 3 of Table 5) MPH</th>
<th>Upper Density Limit VPM / Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1400</td>
<td>60</td>
<td>11.7</td>
</tr>
<tr>
<td>B</td>
<td>2000</td>
<td>55</td>
<td>18.5</td>
</tr>
<tr>
<td>C</td>
<td>3000</td>
<td>50</td>
<td>30.0</td>
</tr>
<tr>
<td>D</td>
<td>3600</td>
<td>40</td>
<td>45.0</td>
</tr>
<tr>
<td>E</td>
<td>4000</td>
<td>30</td>
<td>60.0</td>
</tr>
<tr>
<td>F</td>
<td>&lt;30</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
subdivided into a restricted flow region, a disturbed flow region, and a forced flow region in Section 1.3.1.

The above described close association between the traffic regions suggested by the Highway Capacity Manual and the traffic regions defined in this study is not the only common ground found from the comparison. The Highway Capacity Manual suggests that the capacity under ideal conditions is 2000 vph per lane which occurs when the average speed of the traffic is around 30 mph and within the density range of 45 vpm - 60 vpm (Level of Service E). The aerial photographic data collected in this study confirms these figures. The comparison between the HCM system and the operating regions established in this study is summarized in Figure I-34.

From the findings of Section 1.1.3 and the previous section, a new concept of level of service for uninterrupted flow is suggested. This system is illustrated in Table I-7. The operating conditions for these designated levels of service can be described as follows:

Level of Service A describes a free flow condition. The speeds of vehicles are controlled by drivers' desires and/or the imposed speed limits and physical roadway conditions. A spacious average headway of greater than 5 seconds is provided so that even sudden speed changes of lead vehicles do not result in serious problems for the following driver. Drivers have complete freedom to maneuver and driving is a relatively tension-free task.

Level of Service B describes a semi-free flow condition. An average headway of more than 2 seconds is provided so that marginal safety is main-
<table>
<thead>
<tr>
<th>Traffic Conditions suggested by HCM</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free Flow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Forced Flow</td>
</tr>
<tr>
<td>Stable Flow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Stable Flow</td>
</tr>
<tr>
<td>Upper speed range</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Approaching Unstable</td>
</tr>
<tr>
<td>Stable Flow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Unstable Flow</td>
</tr>
<tr>
<td>Free Flow</td>
<td></td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td>Capacity Flow</td>
</tr>
<tr>
<td>Semi-Free Flow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Restricted Flow</td>
</tr>
<tr>
<td>Free Flow</td>
<td></td>
<td>45</td>
<td></td>
<td></td>
<td></td>
<td>Disturbed Flow</td>
</tr>
<tr>
<td>Capacity Flow</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Forced Flow</td>
</tr>
</tbody>
</table>

Figure I-34 Comparison of Recommended Traffic Operating Regions With Those of the Highway Capacity Manual (1965)
### Table I-7

**Recommended Levels of Service for Uninterrupted Flow Conditions**

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>Traffic Flow Conditions</th>
<th>Density Range (VPM)</th>
<th>Speed Range (MPH)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Free flow</td>
<td>0 - 10</td>
<td>&gt; 70</td>
</tr>
<tr>
<td>B</td>
<td>Semi-free flow</td>
<td>10 - 30</td>
<td>50 - 70</td>
</tr>
<tr>
<td>C</td>
<td>Capacity flow</td>
<td>30 - 60</td>
<td>25 - 60</td>
</tr>
<tr>
<td>D</td>
<td>Restricted flow</td>
<td>60 - 105</td>
<td>13 - 30</td>
</tr>
<tr>
<td></td>
<td>Disturbed flow</td>
<td>105 - 145</td>
<td>8 - 15</td>
</tr>
<tr>
<td></td>
<td>Forced flow</td>
<td>&gt; 145</td>
<td>&lt; 8</td>
</tr>
</tbody>
</table>
tained (see Section 1.1.3 for marginal safety concept). Even though the average speed of the traffic is somewhat restricted, the freedom of selecting one's individual speed is reasonably maintained. It is realized that driving is less restricted in the lower density portion than in the higher density portion of this region; however, no clear-cut dividing point can be found by which this region could be further subdivided.

Level of Service C describes the capacity flow condition. The average headway maintains a constant value of about 2 seconds. An increase in density causes a reduction of spacings between vehicles which immediately results in a reduction of average speed. Marginal safety is provided only if drivers are extremely alert (reaction times of drivers must be less than 1.8 seconds to assure safety at all times). Vehicles are tied together as a continuous flow but individual movement is not really seriously restricted (evidenced from a large value of standard deviation of speeds). Average speeds vary widely from a high value of 60 mph to a minimum of only 20 mph. However, since the average headway is the same (2 seconds) throughout this region, the alertness needed in driving and the maneuverability (freedom of choosing lanes and speed) enjoyed in driving do not vary greatly. Subdivision of this region into more than one level of service according to operating speed is possible but merely picking any speed value to serve as an additional boundary hardly seems logical. Therefore, the Level of Service C suggested here is not further divided. It can be noted that the maximum traffic volume occurs in the density range of this service level.
Level of Service D includes all the undesirable traffic conditions defined in this study. It includes the restricted flow region, the disturbed flow region and the forced flow region. Vehicle speeds are low and the maneuverability available to drivers is severely restricted in each of these regions. The chance of collision is also high in each case. Even though previous studies have defined three different traffic conditions as described above, it is felt that the disturbed flow condition and the forced flow condition are the consequences of the restricted flow state instead of being independent of it. Since all of these conditions are undesirable, it does not appear worthwhile to further subdivide this service level.

1.3.3 PROPOSED CONTROL STANDARDS FOR UNINTERRUPTED FLOW

Traffic changes occur rapidly in the field. In order for any proposed control system to be effective, it must be based on a control parameter which:

1. Possesses a sound theoretical basis to assure its workability,
2. Lends itself to easy and accurate field measurements, and
3. Provides a sensitive indication of traffic changes.

A traffic density oriented control system seems promising in this regard. Density, as a parameter of traffic flow, is both theoretically sound and sensitive to traffic changes. However, measurement of density in the field is not easy since it requires the counting of the number of vehicles in a section of roadway during an instant of time. Photogrammetry is a relatively simple way to obtain density values but the time required for the processing of photographic data makes traffic control based on real-time density information
impossible. Thus, density cannot be used directly for control purposes.

Fortunately, there is a parameter, lane occupancy, which can be related to density and fulfills all the requirements described above. For any kind of vehicle detector, lane occupancy is defined as the ratio (in per cent of time) that the detector is occupied by vehicles to the total time period considered. According to Athol (13), the relationship between lane occupancy and density can be expressed as:

\[
\text{Lane occupancy (decimal)} = \text{average vehicle length (feet) \times density (vehicles per foot)}.
\]

Accepting 17.6 feet as the average vehicle length, this relationship can be reduced to an even simpler form:

\[
\text{Lane occupancy (\%)} = \frac{1}{3} \text{ density (vpm)}.
\]

From this relationship, standards for controlling traffic at different levels of service (based on a density definition) can be established by using lane occupancy as the control parameter. The appropriate numerical values are listed in Table 1-8.

1.4 SUMMARY OF THE STUDY AND RECOMMENDATIONS FOR FURTHER RESEARCH

The study described in this chapter was undertaken with the purpose of developing a new speed-density hypothesis based on data collected using aerial photogrammetric techniques. The basic premise of the study was that different traffic operating conditions can be described in terms of mutual exclusive
<table>
<thead>
<tr>
<th>Level of service desired</th>
<th>Upper density limit (vpm)</th>
<th>Lane occupancy cannot exceed</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>10</td>
<td>3.3</td>
</tr>
<tr>
<td>B</td>
<td>30</td>
<td>10.0</td>
</tr>
<tr>
<td>C</td>
<td>60</td>
<td>20.0</td>
</tr>
</tbody>
</table>

In order that undesirable traffic conditions be avoided, the traffic should not be allowed to exhibit a lane occupancy value of more than 20%.
density subregions. Thus, a speed-density relationship was sought which would allow the boundaries of these subregions to be identified. The relationship selected was made up of a series of linear segments with each segment representing the speed-density function of a particular density subregion. Statistical tests were then employed to establish that this relationship provided a continuous, statistically acceptable description of the experimental data and that a significant difference in operating conditions did indeed exist between adjacent density subregions. A new level of service concept and a set of control standards were then proposed based on the identified density subregions.

1.4.1 SPECIFIC ACHIEVEMENTS OF THE STUDY

The specific accomplishments of the study can be summarized as follows:

1. A multi-linear speed-density relationship was proposed and verified. This relationship divides the density domain into six separate subregions with speed and density displaying a linear relation in each subregion. Verification of the relationship was based on the results of three statistical tests:

a) A test of linearity of the speed-density data in each subregion

b) A test of continuity of the relationship across the break-points subdividing subregions

c) A test to determine whether the slopes of adjacent linear segments were significantly different.

100
Verification was achieved for four of the six subregions covering the density range from 33 vpm to jam density. No data was available for the two subregions between 0 vpm and 33 vpm.

2. The operational characteristics of traffic flow were determined for each of the subregions through an analysis of the patterns displayed by selected traffic parameters. Parameters analyzed included volume, standard deviation of speed, kinetic energy, headway and spacing. The six subregions classified according to their operating characteristics are as follows:

- Free Flow Region
  0-10 vehicles per mile
- Semi-free Flow Region
  10-30 vehicles per mile
- Capacity Flow Region
  30-60 vehicles per mile
- Restricted Flow Region
  60-105 vehicles per mile
- Disturbed Flow Region
  105-145 vehicles per mile
- Forced Flow Region
  145-\( K_j \) vehicles per mile

3. A new concept of level of service was proposed based on density as the fundamental parameter. Four levels of service were designated: Level A (0-10 vpm), Level B (10-30 vpm), Level C (30-60 vpm), and Level D (60-\( K_j \)).

4. Standards for controlling traffic at different levels of service were developed using lane occupancy as the control parameter. Lane occupancy is a one to one function of traffic density.

5. The phenomenon of traffic hysteresis was discovered and described. Hysteresis is the name given to the retarded behavior of traffic following emergence from a traffic disturbance as compared to its before disturbance behavior.
The hysteresis phenomenon is thought to hold important implications for traffic control in terms of preventing the delays and hazards of stop and go traffic movement.

1.4.2 RECOMMENDATIONS FOR FURTHER RESEARCH

As a result of this study the following recommendations are offered for continued research.

1. Since the present study dealt with data collected on a freeway, the findings (especially the quantitative results) will not be applicable to traffic flows under other geometric and control conditions. Additional research should be conducted for uninterrupted flow under non-ideal geometric conditions and for traffic flows on signalized roadways.

2. Since it has been shown that the traffic domain is composed of mutually exclusive subregions displaying different operating conditions, it is proposed that existing models of traffic flow be evaluated on a regional basis. For example, car-following models certainly have limited application in the free flow region but might provide a quite accurate description of flow patterns in the capacity and restricted flow regions. A determination of the region of application of the various models and the form of the sensitivity parameter in each applicable region would be a valuable contribution to the theory of traffic flow.

3. This study was devoted entirely to an analysis of traffic behavior prior to a traffic disturbance. Realizing, however, that the hysteresis phenomenon does exist and that traffic behaves differently after release from a
disturbance than before entering a disturbance, it is proposed that a similar analysis be made of after disturbance behavior. Such a study should provide valuable information concerning control of traffic once a disturbance has formed.

4. This study was based entirely on an empirical approach utilizing experimental data as a basis for developing the proposed speed-density hypothesis. It would be interesting to study the possibilities of a mathematical basis for the multi-linear speed-density model.
CHAPTER 2

THE MULTI-LINEAR SPEED-DENSITY RELATIONSHIP:

MATHEMATICAL APPROACH

2.1 BACKGROUND OF THE MATHEMATICAL STUDY

In recent years numerous mathematical models of traffic flow have been developed. These models have been both deterministic and probabilistic in nature and have been developed either empirically using experimental data or through analogy with related phenomena in other fields of study. The most common analogy has compared traffic flow to the flow of a compressible fluid (hydrodynamic approach). This is the approach used by Lighthill and Whitham in their classic paper on the theory of traffic flow on long, crowded roads (14). Other important contributions to the description of the macroscopic movement of traffic have been made by Herman and Potts (15), Edie and Foote (16), Wardrop (17), Prigogine, Herman and Anderson (18), and Kometani and Sasaki (19) to name but a few.

In spite of this concentrated research effort there remains much work to be done in building a complete understanding of the mechanism of traffic flow. Further research is required concerning both the microscopic and macroscopic behavior of the traffic stream for both uninterrupted and interrupted flow situations. Existing traffic models must be tested to determine their applica-
bility under differing conditions of traffic operations and on different roadway facilities. New models must be developed which provide improved descriptions of the fundamental nature of traffic movement. In the preceding chapter an empirical approach was used to derive a relationship relating the average speed of the traffic stream to traffic density. The resulting relationship was multi-linear in form with a different linear segment describing the speed-density function in each of six different traffic operating regions. Using experimental data it was shown that this relationship provided a continuous, statistically acceptable description of macroscopic flow behavior.

This chapter reports the results of a parallel phase of study undertaken using a mathematical approach. The objective is once again to obtain a model relating the average speed of traffic to density. The underlying assumption of this phase of study is that the behavior of a platoon of vehicles can be analyzed using the concepts of force and work from applied physics. The force and work functions are defined and expressed in terms of average speed, density and other selected traffic parameters. These expressions are then manipulated to provide the desired speed-density relationship.

2.2 ANALYSIS OF PLATOON BEHAVIOR

In order to analyze the mechanisms of platoon movement, it is first necessary to define certain terms. These definitions are as follows:

\[ n = \text{number of vehicles forming a platoon} \]

\[ u_i = \text{velocity of the ith vehicle of the platoon} \]
\[ a_i = \text{acceleration of the } i\text{th vehicle of the platoon} \]

\[ y = \text{platoon length} \]

Using these four variables it is possible to define three important platoon parameters. These parameters are average platoon velocity, platoon density, and average acceleration of the platoon and are given by

\[ u = \frac{\sum_{i=1}^{n} u_i}{n} \]  \hspace{1cm} (I-1)

\[ K = \frac{n - 1}{y} \]  \hspace{1cm} (I-2)

\[ a = \frac{\sum_{i=1}^{n} a_i}{n} \]  \hspace{1cm} (I-3)

respectively.

2.2.1 PLATOON CHARACTERISTICS

Consider a group of \( n \) vehicles moving along a roadway. At time \( t_0 \) this platoon has an average velocity \( u_0 \) and occupies a length of roadway \( y_0 \) as shown in Figure I-35. This platoon continues to move freely until it encounters a region of high density, low velocity traffic. At this point the lead vehicle must slow down to the speed of the downstream traffic and the trailing vehicles begin to pack up behind it. As a result the average velocity of the platoon is reduced and the platoon length contracts so that a shorter section of roadway is occupied. This condition is shown in the middle section of Figure I-35 as the condition at time \( t_1 \). As traffic density continues to increase the velocity of the platoon will continue to decrease until jam density is reached.
Figure I-35 Summary of Platoon Characteristics

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At jam density the platoon will come to a complete stop \((u_j = 0)\) and the length of the platoon will be reduced to the minimum possible length for accommodating \(n\) vehicles \((y = y_j = y_{minimum})\). The condition at jam density \((time = t_j)\) is depicted at the bottom of Figure I-35. The platoon will then be released from the jam and free flow will eventually be reattained.

Since the speed-density model developed in Chapter 1 was based only on prior-jam movement, the analysis of platoon behavior herein will be limited to the conditions occurring up to jam density. Considering any two points in time, such as \(t_1\) and \(t_2\), the change in platoon movement can be theoretically divided into two distinct stages. These stages are illustrated in Figure I-36. In the first stage all vehicles in the platoon are initially considered to be traveling at the average velocity of the platoon as if the platoon was a train moving at a velocity of \(u_1\). As the lead vehicle slows to \(u_2\) all following vehicles reduce their speeds accordingly and no change in the length of the platoon takes place. The platoon is now a train traveling at a velocity of \(u_2\) with length \(y_1\). In the second stage the individual vehicles are released to adjust their speeds and spacings in correspondence with the desires of their drivers. The average velocity of the platoon remains at \(u_2\) but the spacing between vehicles is reduced thus resulting in a decrease in the length of the platoon to \(y_2\) \((y_2 < y_1)\). The combination of these two stages results in a platoon of decreased length \((y_2)\) moving along the roadway at a reduced velocity \((u_2)\).
Figure I-36  Two Stage Mechanism for Changes in Platoon Movement
2.2.2 CONCEPT OF AN IMAGINARY FORCE

In physics a force is defined as a push or a pull that tends to produce a change in the motion of a body. Considering the platoon of vehicles described above, a change of motion is produced in each of the two stages of movement occurring between time $t_1$ and time $t_2$. In stage 1 the platoon is moved forward an infinitesimal distance $dx$ (see Figure I-36) while its average velocity is reduced from $u_1$ to $u_2$. In stage 2 the individual vehicles in the platoon are moved relative to each other resulting in an infinitesimal decrease in platoon length of $dy$. It is logical, therefore, that some force is acting in each stage to produce these changes in the movement of the platoon. Since there is no physical force present, the force at work has been termed an imaginary force. The stage 1 component is called an external imaginary force because it works on the platoon as a whole while the stage 2 component is termed an internal imaginary force since it works within the platoon.

2.2.3 CONCEPT OF PLATOON WORK

Referring once more to physics, work is defined as the product of the force acting on a body times the distance through which the point of application of the force moves in the direction the force acts. Since there is platoon movement involved as a consequence of the action of both the external and internal imaginary force components, it follows that these force components perform work on the platoon. In accordance with the definition given above, this work can be expressed as:
\[
\begin{align*}
\text{d}W_E &= F_E \text{d}x \\
\text{d}W_I &= F_I \text{d}y
\end{align*}
\]  
\text{(I-4)}  
\text{(I-5)}

where:

\text{d}W_E = \text{infinitesimal external work done} \\
\text{d}W_I = \text{infinitesimal internal work done} \\
F_E = \text{imaginary external force component} \\
F_I = \text{imaginary internal force component} \\
\text{d}x = \text{infinitesimal distance platoon is moved} \\
\text{d}y = \text{infinitesimal length change for platoon}

These two kinds of work can be visualized in the following manner. \text{d}W_E is the work required to reduce the average velocity of the platoon from \( u_1 \) to \( u_2 \) and \text{d}W_I is the work necessary to compact the platoon from length \( y_1 \) to length \( y_2 \).

2.3 FORMULATION OF THE MATHEMATICAL MODEL

According to the study of microscopic platoon behavior described above, there is an imaginary force acting upon the platoon which influences its movement. During an infinitesimal interval of time this force performs two infinitesimal types of work on the platoon. It is hypothesized that these two types of work are proportional to one another. This hypothesis seems logical since the more platoon velocity is reduced, the more the vehicles in the platoon tend to pack up and the shorter the platoon length becomes. Mathematically this relationship can be expressed as:

\[
-\text{d}W_E = \alpha \text{d}W_I
\]  
\text{(I-6)}

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where $\alpha$ is the proportionality constant and the negative sign shows that the movement of the platoon is in the opposite direction from the action of the imaginary external force.

2.3.1 DEVELOPMENT OF WORK EXPRESSIONS

In order to utilize the concept of platoon work to develop a mathematical model relating average speed and traffic density, it is necessary to express the two types of work in terms of common platoon parameters. This can be done as follows.

External Work

Consider a group of $N$ molecules of a fluid each having a mass of $m$. Now if a force acts on this fluid mass it will cause the mass to move with some average acceleration $a$. By virtue of Newton's Second Law of Motion the magnitude of the accelerating force can be expressed as the product of the fluid mass times the average acceleration of the mass. However, since each molecule of the fluid has the same mass, an equally valid indication of the amount of force applied to a given number of molecules can be obtained by simply multiplying the number of molecules by their average acceleration. Analogously, consider the aforementioned platoon of $n$ vehicles. If this platoon is acted upon by a force causing a change in its average velocity (and hence an acceleration), this force can also be expressed in the form of number of included units times acceleration. Thus,

$$F_E = n \times a$$  \hspace{1cm} (I-7)
Since this force acts through a distance \( \text{dx} \) as shown in Figure I-36, the work it performs can be expressed as:

\[
dW_E = n \times a \times \text{dx}
\]

(I-8)

**Internal Work**

An expression for internal work is somewhat more difficult to derive but such an expression can be obtained using dimensional analysis. Since a model relating average platoon speed and traffic density is the desired result of this study these two parameters will be the primary variables in the dimensional analysis. In addition a new term will be included which is a function of both speed and density. Let \( C \) be defined as the ratio of the change of average velocity to the change in density during a given time interval. Mathematically \( C \) can be written as:

\[
C = \frac{u_{j+1} - u_j}{k_{j+1} - k_j}
\]

where the subscript \( j \) denotes the time sequence.

The objective of dimensional analysis is to obtain expressions relating a number of selected variables. This is done by forming a group of dimensionless products and then finding relationships between these products thus relating the variables which comprise the products. The variables of interest in this case are average velocity \( (u) \), traffic density \( (K) \), the speed-density ratio \( (C) \) and the imaginary internal force \( (F_I) \). The desired expression will contain \( F_I \) as a function of the other three variables.

Let \( \pi \) be any product of the variables \( u, K, C, \) and \( F_I \). Hence
\[ \pi = C^{k_1} F_x^{k_2} u^{k_3} K^{k_4} \]  \hspace{1cm} (I-10)

By substituting the dimensions for the variables into this expression, the dimensions of \( \pi \) can be obtained.

\[ [\pi] = \left[ \begin{array}{c} N^1 L^{l_2} T^{r_2} \\ N L T^{l_3} \end{array} \right]^{k_1} \left[ \begin{array}{c} L^2 T^{r_2} \\ N T^{l_3} \end{array} \right]^{k_2} \left[ \begin{array}{c} L T^{r_2} \\ N L \end{array} \right]^{k_3} \left[ \begin{array}{c} N L \end{array} \right]^{k_4} \]  \hspace{1cm} (I-11)

where \( N, L \) and \( T \) represent vehicles, length and time dimensions respectively.

However, \( \pi \) by definition is dimensionless. Therefore the combined exponents of \( N, L \) and \( T \) must each be zero. Thus it can be written that

\[-k_1 + k_2 + k_4 = 0 \]  \hspace{1cm} (I-12)

\[2k_1 + k_2 + k_3 - k_4 = 0 \]

\[-k_1 - 2k_2 - k_3 = 0 \]

Equations I-12 can be written in matrix form as follows:

\[
\begin{bmatrix}
-1 & 1 & 0 & 1 \\
2 & 1 & 1 & -1 \\
-1 & -2 & -1 & 0
\end{bmatrix}
\begin{bmatrix}
k_1 \\
k_2 \\
k_3 \\
k_4
\end{bmatrix} =
\begin{bmatrix}
0 \\
0 \\
0
\end{bmatrix}
\]  \hspace{1cm} (I-13)

There are four variables in the above system of equations. The rules of dimensional analysis state that the number of independent dimensionless products which can be formed from any group of variables is equal to the number of variables minus the rank of their dimensional matrix. Since the rank of the above matrix is 2, a total of \( 4 - 2 = 2 \) dimensionless products can be formed. These products can be obtained by selecting values for any two of the variables \( (k_1, k_2, k_3, \text{ or } k_4) \) and solving the equation system for the
remaining two. Thus \( k_3 \) and \( k_4 \) can be expressed as:

\[
\begin{align*}
  k_3 &= -k_1 - 2k_2 \\
  k_4 &= k_1 - k_2
\end{align*}
\] (I-14)

Now if we let \( k_1 = 1 \) and \( k_2 = 0 \) the corresponding values for \( k_3 \) and \( k_4 \) are \(-1\) and \( 1 \) respectively. Another solution is obtained by letting \( k_1 = 0 \) and \( k_2 = 1 \). In this case \( k_3 \) is found to be \(-2\) and \( k_4 \) is equal to \(-1\). Hence the corresponding dimensionless products are:

\[
\begin{align*}
  \pi_1 &= C' F_1^0 u^{-1} K^1 = \frac{Ck}{u} \\
  \pi_2 &= C^0 F_1^1 u^{-2} K^{-1} = \frac{F}{ku^2}
\end{align*}
\] (I-15)

The general relationship between any pair of dimensionless products can be written as

\[
\pi_1 = B \pi_2^r + M
\] (I-16)

where \( r \) is any real value and \( M \) and \( B \) are constants. Observing the boundary conditions for the variables contained in \( \pi_1 \) and \( \pi_2 \), it is noted that at jam density both \( C \) and \( F_1 \) approach zero. This is true since velocity changes vary little with density in the vicinity of jam density (\( C \Rightarrow 0 \)) and because no further contraction of the platoon is possible (\( F_1 \Rightarrow 0 \)). From the definition of \( \pi_1 \) and \( \pi_2 \) if \( C \) and \( F_1 \) approach zero at jam density so do both dimensionless products. Hence it follows that \( M \) has a value of zero and can be dropped from Equation I-16. Then assuming \( r \) is equal to 1 to produce the simplest possible relation between \( \pi_1 \) and \( \pi_2 \), Equation I-16 can be rewritten as

\[
\pi_1 = B \pi_2
\] (I-17)
Substituting the values for $\pi_1$ and $\pi_2$ shown in Equation I-15 the following relation is obtained.

$$\frac{C k}{u} = B \frac{F_I}{k u^2}$$  \hspace{1cm} (I-18)

Rearranging this equation into a more suitable form results in

$$F_I = (C/B)K^2u$$  \hspace{1cm} (I-19)

Hence the imaginary internal force is expressed as a function of platoon velocity and density.

Now since this force acts through a net distance of $dy$ the internal work done can be written as

$$dW_I = (C/B)K^2u \, dy$$  \hspace{1cm} (I-20)

2.3.2 DERIVATION OF SPEED-DENSITY RELATIONSHIP

Now replacing $dW_E$ and $dW_I$ in Equation I-6 with the expressions obtained in the preceding section the following result is achieved

$$- n \alpha \, dx = \alpha (C/B)K^2u \, dy$$  \hspace{1cm} (I-21)

Differentiating Equation I-2 to get an expression for $dy$ and substituting the result above yields

$$n \alpha \, dx = \alpha(C/B)K^2u \frac{y^2}{n-1} \, dK$$

which can be manipulated as follows

$$\frac{du}{dt} \, dx = \frac{\alpha C}{n B} K^2u \left( \frac{n-1}{y^2} \right) \, dK$$

$$u \, du = (n-1) \frac{\alpha C}{n B} K^2u \frac{1}{K^2} \, dK$$

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\[ du = \frac{(n-1) \alpha C}{n B} \ dK \]  \hspace{1cm} (I-22)

Finally integrating Equation I-22 the desired speed-density relationship is obtained. This relationship is

\[ u = A + \beta (C/B) K \]  \hspace{1cm} (I-23)

where \( A \) is a constant of integration and \( \beta = \frac{(n-1) \alpha}{n} \) is a constant for a given platoon.

This result shows that speed is indeed a linear function of density with a slope determined by the values of the coefficients \( C \) and \( B \). If these coefficients can be shown to have different values in different density subregions, then a mathematical basis for the multi-linear speed-density relationship of Chapter 1 will be established. The evaluation of these coefficients is undertaken using experimental data in the next section. The platoons chosen for analysis are the same platoons used in the study described in Chapter 1.

2.3.3 EVALUATION OF COEFFICIENT VALUES

For the convenience of the reader Platoons A and B as used in Chapter 1 are redefined in Figures I-37 and I-38. It should be noted that only that portion of the platoon movement occurring prior to jam density is of interest in the evaluation of the coefficient values. For each platoon the following parameter values were calculated at one second intervals: density, average velocity, incremental change in density, incremental change in velocity and the two
Figure I-37 Identification of Platoon A Vehicles
(Enclosed Area).

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Figure I-38 Identification of Platoon B Vehicles
(Enclosed Area)
dimensionless products, $\pi_1$ and $\pi_2$. A summary of these values is given in Table I-9.

According to Equation I-9 the C value can be calculated as the ratio of the incremental change in velocity to the incremental change in density during any time interval. Thus if $(u_{j+1} - u_j)$ is plotted versus $(K_{j+1} - K_j)$ the value of C would be represented by the slope of the plot at each point along the curve. However, since $(u_{j+1} - u_j)$ and $(K_{j+1} - K_j)$ tend to be rather small numbers, a more consistent pattern for C can be obtained by plotting the parsum of $(u_{j+1} - u_j)$ versus the parsum of $(K_{j+1} - K_j)$. In this plot C is still represented by the slope of the curve. Such a plot has been prepared for both Platoons A and B. These plots are shown in Figures I-39 and I-40.

In a like manner Equation I-17 states that the B value can be obtained by calculating the ratio of $\pi_1$ and $\pi_2$ at each time interval. Once again, due to smallness of the $\pi_1$ and $\pi_2$ values, a plot was prepared for each platoon relating the parsum of $\pi_1$ to the parsum of $\pi_2$. The B value is represented by the slope of these plots. The B value curves are shown as Figures I-41 and I-42.

Now utilizing the B and C value curves a value for the ratio of C/B can be calculated at one second intervals. Each C/B value can be related to its corresponding traffic density level using the data of Table I-9. Plots of C/B versus density for Platoons A and B are shown as Figures I-43 and I-44. These plots indicate that the C/B values are different at different density levels. This agrees with the concept of a multi-linear speed-density model.

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Platoon B

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Figure I-39 "C" Curve for Platoon A
Figure I-42 "B" Curve for Platoon B
Figure I-43 Ratio of C over B Versus Density for Platoon A
lines have been placed on each of these figures indicating the break-points separating the various density subregions as determined in the analysis of Chapter 1. Table I-10 contains a summary of average C, B and C/B values by density subregions for each of the two platoons. Only subregions 3, 4, 5 and 6 are included as no data was available for the platoons at traffic densities below 30 vehicles per mile.

Table I-10 Summary of Average Platoon Coefficient Values

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<th>No. of Values</th>
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From observation of Table I-10 it appears that a definite difference in C/B values does exist among the four subregions for Platoon B. It is also interesting to note that the value of C/B is the highest in the subregion between 30 and 60 vehicles per mile confirming the observation of the last chapter that platoon velocity is most sensitive to changes in density in this subregion. C/B
values then decrease steadily as density increases indicating that unit changes of density result in increasingly smaller changes in speed as jam density is approached. This result is also consistent with the conclusions of Chapter 1.

No real conclusion can be drawn from the data of Platoon A other than the general tendency of C/B to decrease with increasing density. Although some difference does exist between the C/B values of subregions 3, 4, and 5, the difference is not large enough to state that operating conditions in these subregions actually differ from one another. Subregion 6 exhibits values almost identical to those of subregion 5.

2.4 SUMMARY OF THE MATHEMATICAL STUDY

This study was undertaken with the objective of developing a mathematical model relating the average speed of a platoon of vehicles to traffic density. A mathematical approach was taken in which the microscopic behavior of the platoon was analyzed using the concepts of force and work from applied physics. Two types of work were identified as resulting from the action of an imaginary force on the platoon as it moves along the roadway. Expressions were written for these two work components in which each type of work was defined in terms of common traffic parameters—among which were average speed and density. These expressions were then manipulated to produce the desired speed-density model.

The model obtained establishes average speed as a linear function of traffic density with the slope of the function determined by two coefficients,
C and B. These coefficients were then evaluated using experimental data. As a result of this analysis it was shown that the slope of the speed-density curve tends to exhibit different values for different parts of the density domain. Specific attention was then turned to the density subregions defined in Chapter 1 and an attempt was made to show that the average slope (C/B value) of each subregion varied markedly from the slopes of adjacent subregions. If this could be done it would provide further evidence of the existence of these subregions.

Unfortunately the results of the C/B analysis proved somewhat inconclusive. First, data was available only for traffic density conditions exceeding 30 vehicles per mile thus prohibiting study of the first two subregions. Second, while Platoon B data showed a definite difference between C/B values in the remaining four subregions, Platoon A data did not. As a result, although the existence of a multi-linear speed-density relationship has been supported, no statement can be made at this time regarding the validity of the six operating subregions. Further analysis is presently underway in which additional platoons are being studied in an attempt to provide more conclusive evidence regarding the existence of traffic operating subregions. In addition to the C/B values, the patterns displayed by other traffic parameters with density are also being studied. The results of this study will be reported as they become available.
PART II

APPLICATION STUDIES
OVERVIEW

Primary emphasis during the second phase of the subject research program was placed on the application of the aerial photogrammetric data collection technique to solve practical problems of highway design and operation. One such problem is the congestion which plagues urban freeways during the morning and evening rush hours thereby greatly diminishing the traffic carrying ability of these high investment facilities. Once a problem common only to cities such as New York, Chicago and Los Angeles, the curse of peak period traffic congestion and its accompanying delay and accident hazard has spread to cities large and small throughout the country. With continuing urbanization and an ever increasing number of motor vehicles competing for space on the urban roadway system, the outlook for efficient traffic movement within high density population areas in the years to come is bleak. The treatment of this problem thus becomes one of the highest priority tasks facing the transportation engineer be he researcher or practitioner.

In order to successfully deal with problems of traffic operation within an urban freeway corridor, a vast amount of data concerning the characteristics of traffic movement within that corridor is required. In previous phases of the research program the aerial photogrammetric technique developed by the Transportation Research Center has proven to be an extremely powerful tool for providing traffic data which would be difficult if not impossible to collect.
using standard ground-based methods. Examples of such data include information on weaving maneuvers, traffic stoppages, vehicle spacing and headways and vehicle acceleration patterns. It seems logical, therefore, that such a tool could be a valuable asset in the battle against peak period traffic congestion.

Accordingly, in consultation with representatives of the Ohio Division of Highways and the City of Columbus Division of Traffic, the research group selected a site on which a pilot operational study emphasizing aerial photogrammetric data collection techniques could be conducted. The site chosen was a section of Interstate 71 located in the north side of Columbus, Ohio (see Figure II-1). Interstate 71 is the primary north-south highway in Ohio connecting the state's three largest cities—Cleveland, Columbus and Cincinnati. The section chosen for study varies from six to nine lanes in width and stretches between the interchange with Interstate 270, the Columbus Outerbelt, to the north and the Fort Hayes Interchange, a primary interchange on the Columbus Innerbelt, to the south. Due to its strategic location, I-71 is the principal link connecting the highly populated northern suburbs to the CBD and as a result is heavily trafficked at all times with especially bad congestion occurring during the morning and evening rush hour periods.

The procedure to be used in conducting the pilot operational study was structured into five stages as follows:

1. Conduct several general survey flights over I-71 between the I-270 and Fort Hayes Interchanges in an attempt to locate regions of high
Figure II-1 Location of Site for Operational Pilot Studies
congestion symptomatic of operational problems

2. Conduct a series of ground-based studies concentrated in the regions identified in Stage 1 for the purpose of isolating existing traffic bottlenecks—study techniques to be used included travel time runs, density trap counts and volume counts.

3. Conduct a series of intensive aerial surveys of identified bottleneck areas to determine the following information:
   a. Specific nature of the problem
   b. Parameters which can be used to describe the traffic situation at the bottleneck
   c. Causative factors leading to flow breakdowns and congestion at the problem points.

4. Formulate a control strategy for I-71 based on the data collected in stages 1, 2 and 3; inform appropriate State of Ohio and City of Columbus personnel of findings and supply available data for their analysis.

5. Conduct a series of "after" aerial surveys and field studies to determine the effectiveness of the proposed control strategy in improving the operational efficiency of I-71 and provide recommendations concerning needed strategy modifications (contingent on implementation of proposed strategy by city and state).

A separate study was undertaken relative to the southbound morning peak period and the northbound evening peak period as traffic problems during
these two time periods were thought to be independent of one another. At the
time of this report the southbound study has progressed through all five stages
and is considered complete. Details of this study are contained in Chapter 1
of this part of the report. The study of northbound traffic operation has
progressed through the first four stages and a tentative evening peak period
control plan has been proposed. No progress has yet been made, however,
concerning the implementation of this plan in the field. Details of the north-
bound study are reported in Part II - Chapter 2.
CHAPTER 1

STUDIES OF SOUTHBOUND INTERSTATE 71

1.1 ORIGINAL STUDY: 1969 - 1970

The material contained in this section describes the data collection, data reduction and data analysis phases of the study of operational problems on southbound Interstate 71. The information reported herein was collected during the period from October 1, 1969, to August 31, 1970.

1.1.1 DATA COLLECTION

The data collection phase of the study is comprised of the first three steps of the procedure for the pilot operational study; namely, general survey flights, ground-based field studies and intensive survey flights.

General Survey Flights

A series of four general survey flights were flown over the study section between October 29 and November 13, 1969. The flights were flown at an altitude of approximately 3000 feet above ground allowing for a longitudinal coverage per photograph of nearly one mile and were about 20 to 30 minutes each in duration. A range of exposure intervals were used varying from a minimum of 0.2 seconds to a maximum of three seconds between photographs. A summary of these flights is given below.

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<table>
<thead>
<tr>
<th>Flight Identification</th>
<th>Date</th>
<th>Time of Coverage</th>
<th>Exposure Interval</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flight #10</td>
<td>Thurs. Oct. 30, 1969</td>
<td>7:40-8:00 AM</td>
<td>0.2 &amp; 1 sec.</td>
</tr>
<tr>
<td>Flight #11</td>
<td>Friday, Nov. 7, 1969</td>
<td>7:20-7:50 AM</td>
<td>1 sec.</td>
</tr>
</tbody>
</table>

In the course of each flight an average of three runs were made along the study corridor and photographs were taken beginning with the first sign of restricted traffic flow and continuing until free flow was reestablished. During each run approximately 85 feet of film (200 frames) was shot of the congested area giving a total of 1020 feet (2400 frames) for the four flights. In addition, the observer in the helicopter was instructed to record the position of all areas of traffic congestion observed in the study section as well as the time at which the congestion was sighted. Furthermore, the observer was urged to use his aerial vantage point to identify any and all possible causative factors which might be responsible for traffic flow breakdowns. These causative factors would then be investigated in detail during the ground-based studies and the intensive survey flights which were to follow.

Preliminary analysis of the 2400 frames of photography taken combined with the debriefing of the helicopter observer revealed that the congested area during the morning peak period extends from midway between the Morse Road and Cooke Road overpasses to just south of the southbound on-ramp at Hudson
Street in the vicinity of the Ohio Historical Society Museum. In this area traffic disturbances were frequent and complete breakdowns of flow were not uncommon. Especially high concentrations were observed near the on-ramps at East North Broadway, Weber Road and Hudson Street. It was decided therefore that further data collection would be concentrated in the region bounded by Morse Road on the north and Seventeenth Avenue on the south with special emphasis placed on the merging areas of the three on-ramps mentioned above. (See Figure II-2)

**Ground-Based Studies**

A group of ground-based studies was conducted between February 5, 1970, and August 31, 1970, for the purpose of further defining operational problem areas on SB I-71 and investigating probable causative factors leading to traffic congestion. Three types of standard traffic engineering studies were used: travel time runs, density traps, and volume counts. The details of the procedures followed in each set of studies are described below.

**Travel Time Runs**

Travel time runs offer one of the most easily accomplished methods for obtaining a preliminary indication of the location of operational trouble spots. Since the velocity of a traffic stream is inversely related to traffic density, regions of high density will be reflected as spots of low velocity in a velocity profile of the study section. Such a profile can be obtained through a series of travel time runs.
Figure II-2 Region of Concentrated Data Collection
(Southbound Morning Peak)
Three sets of runs were conducted. On Thursday, February 5, 1970, a series of nine runs were made by vehicles between the Morse Road on-ramp and the off-ramp at Seventeenth Avenue. The first car was on the freeway at Morse Road at 7:10 AM and the final run began at 7:58 AM and ended at 8:06 AM, giving a coverage of some 56 minutes of freeway operation during the peak hour. The cars traveled in the middle lane of the three lane section and the observer was instructed to record the times at which his car passed certain predetermined points. These times were then converted to average velocities for the sections between reading points. The resulting velocity profile could be plotted for each of the runs. A second set of runs was conducted on Tuesday, March 24, 1970. Twelve runs were made this time using a similar procedure. These runs were all continued to the off-ramp at Cleveland Avenue in order to provide a check on the observation made during the general survey flights that congestion ended at the Hudson Street on-ramp. The last set of runs was made on Thursday, June 18, 1970, as part of an Input-Output study conducted during the week of June 15 to 19. The runs were made between Cooke Road and Fifth Avenue for the purpose of checking velocity patterns against those previously obtained. This was done to determine whether the data collected on densities and volumes during the Input-Output study could be taken as representative of an average traffic week on Interstate 71.

Density Trap Study (Input-Output)

An Input-Output analysis of traffic behavior was conducted between the
East North Broadway overpass and the Hudson Street on-ramp in an attempt to further pinpoint the trouble spots on southbound I-71. The analysis was so designed as to provide both volume values for traffic in all freeway lanes and on all ramps in the chosen region and sufficient data to compute average traffic densities for the Broadway to Weber and Weber to Hudson subsections.

The Input-Output study was conducted in cooperation with city and state forces in order to have sufficient manpower available. It consisted of sequential 5 minute volume counts at various locations within the study section during the period 6:30 AM to 8:30 AM throughout a five day study period. Counts were made at fifteen different counting stations located as shown in Figure II-3.

To begin the volume counts each morning the study section was "cleared" of vehicles by driving a signal car down the middle lane. Each counting station commenced counting with the passage of the signal vehicle and counted until the end of the first five minute interval. Volumes were then recorded and were recorded at the end of every five minute interval thereafter until 8:30 AM. Separate volumes were recorded for each of the three freeway lanes at the East North Broadway, the Weber Road and the Hudson Street overpasses and for all ramps beginning with the East North Broadway off-ramp and ending at the Hudson Street on-ramp.

Using the data collected, it should have been possible to calculate the number of vehicles in each subsection between overpasses at 5 minute intervals throughout the two hour counting period. Analysis of the data in the office after the study revealed, however, that the density calculations obtained in

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Figure H-3: Location of Input-Output Counting Stations

- Hudson Street
- Weber Road
- East North Broadway

+ - Position at which a count is required

Direction of travel

North
this manner were not accurate. Small counting errors on the order of 0.5% to 1.0% made at the different counting stations combined to yield density values which were not realistic. The density values obtained jumped randomly about and were sometimes negative. Thus it was apparent that unless very accurate electronic equipment could be obtained another method would have to be found to obtain traffic density information. As no such equipment was available it was decided to postpone the density study until the data from the intensive aerial survey flights of the area became available and to obtain density values from the photographs. The description of the procedure used to obtain density from photographs will be described later in this chapter in the section on intensive survey flights.

Supplementary Volume Counts

In spite of the difficulty incurred in calculating traffic densities using the volume data of the Input-Output study, it is thought that the body of volume data itself is sufficiently accurate to provide valuable information regarding traffic composition, traffic lane distribution, ramp usage and volume patterns over time. Since the study was conducted for a full week the volume pattern could be studied both for its variation during the period 6:30 to 8:30 AM and its variation from day to day.

In addition to these volume counts, supplementary volume data was collected for each on-ramp merge area between East North Broadway and Hudson Street. Of interest was a volume value known as the merge area
capacity. This capacity parameter is measured just downstream from an on-ramp and is a measure of the maximum number of vehicles which can pass through this downstream section. The merge area capacity is thus an indication of the amount of traffic which an on-ramp can effectively feed into a freeway stream. Specifically, no more traffic can be efficiently served by an on-ramp than that volume given by subtracting the freeway volume upstream of the ramp from the merge area capacity. (See Figure II-4) Each merge area has a unique capacity depending upon the horizontal and vertical alignment of the area, the number of lanes, the lateral clearance, the nature of weaving in the area and the angle which the on-ramp makes with the freeway mainline.

Merge area capacity data was collected at 5 minute intervals over a period of three days for each of the three on-ramps being studied. The data was then averaged and a graphical procedure was employed to determine the capacity of each merge area.

**Intensive Survey Flights**

As a result of the ground-based studies the region of most serious congestion during the morning peak period was found to lie between the off-ramp at East North Broadway and the on-ramp at Hudson Street. Lesser congestion was located as far north as Cooke Road and as far south as Seventeenth Avenue.

A series of three intensive survey flights were conducted over this area (Cooke Road to Seventeenth Avenue) during the week of June 15 to 19, 1970.
Figure II-4 Schematic for Merge Area Capacity Study

To reduce congestion \( Q_f + Q_r \leq Q_{op} \)
Since the ground-based attempt to collect traffic density data in the subsections between overpasses had failed, a procedure was designed to obtain traffic densities from the air. The procedure chosen involved continuous aerial circuits over the study section with photographs taken at three second intervals. As each circuit required approximately 5 minutes, density values could be obtained at 5 minute intervals by simply counting the vehicles on the photographs in each subsection. The only difficulty to be overcome was that an accurate time base had to be maintained so that the time at which each circuit was begun could be determined. This problem was solved by adding a time keeper with a stop watch to the aerial survey crew. The density values obtained in this manner could then be plotted versus time to provide a continuous record of the density pattern existing in each freeway subsection.

A summary of the intensive survey flights is given below.

<table>
<thead>
<tr>
<th>Flight Identification</th>
<th>Date</th>
<th>Number of Circuits</th>
<th>Time of Coverage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flight #13</td>
<td>Monday, June 15, 1970</td>
<td>6</td>
<td>7:15-7:59 AM</td>
</tr>
<tr>
<td>Flight #14</td>
<td>Thurs. June 18, 1970</td>
<td>10</td>
<td>7:11-8:10 AM</td>
</tr>
<tr>
<td>Flight #15</td>
<td>Friday, June 19, 1970</td>
<td>10</td>
<td>7:02-8:03 AM</td>
</tr>
</tbody>
</table>

In addition to the density data, the intensive survey flights also provided information on the following characteristics of traffic operation:

1. Weaving maneuvers

2. Traffic stoppages
3. Lane usage

4. Traffic composition

5. Speed patterns

6. Vehicle spacings and headways

This information when combined with that obtained through the general survey flights and the ground-based studies provides a reasonably complete picture of traffic operation on SB I-71 during the morning peak hour. A summary of all pertinent data collected and an analysis of existing operational problems on SB I-71 is presented in the next section.

1.1.2 DATA REDUCTION AND ANALYSIS

Reduction of data was conducted concurrently with the data collection phase and proceeded as data became available. Data reduction was accomplished using standard techniques for the most part but where extremely accurate values were required use was made of the special aerial data reduction system featuring the Mann 829D Comparator developed in an earlier phase of the subject research project. This new system functioned smoothly and no problems were encountered in the data reduction process.

The resulting data was then summarized and analyzed toward the end of identifying operational problem areas on SB I-71 and determining the underlying reasons for these problems. The results of the analyses are presented below in a structure corresponding to that used for describing the data collection procedure.
General Survey Flights

Observations and photography from the general survey flights were used to define the general location of traffic congestion both in terms of position on the freeway and time of day. The first two flights covered the entire study section between the Interstate 270 Interchange and the Fort Hayes Interchange and were concentrated in the time period between 7:40 and 8:10 AM. These flights revealed that relatively free flow was maintained until traffic progressed south of the Morse Road overpass and was regained in the vicinity of the Seventeenth Avenue overpass. In addition it was discovered that the region that exhibited high density traffic flow was already heavily congested by the time the first run of each flight was made. The remaining two flights, therefore, were limited to the congested section and were begun earlier in an attempt to define when traffic congestion first developed during the morning peak hour. From photographs taken on these flights the critical section of freeway exhibiting the worst congestion was further limited to that region extending from midway between Morse and Cooke Roads to just south of the on-ramp at Hudson Street. The time of worst congestion was the period between 7:20 and 7:50 AM.

Figure II-5 is a plot of traffic density (total density for three lanes) versus distance constructed from data collected by aerial survey on Thursday, November 13, 1969. This plot clearly illustrates the high density conditions commonly existing in the above mentioned section. Especially significant are the high density peaks located at the merge areas of the East North Broadway,
Figure II-5  Traffic Density Versus Distance for Data of Thursday, November 13, 1969
Weber Road and Hudson Street on-ramps. Densities in these merging areas were consistently greater than 60 vehicles per mile per lane and were found to reach values as high as 120 vehicles per mile per lane at certain times during the peak period.

Within this area of traffic congestion regions of stop and go driving characteristically associated with the passage of a kinematic wave through the traffic stream were observed by the helicopter observer. Traffic stoppages numbering 10 to 15 cars in magnitude were frequent and complete breakdowns with all three traffic lanes stationary for periods of 5 to 30 seconds were not uncommon. Merging of on-ramp traffic with the mainline stream was extremely difficult in this region and many instances of ramp vehicles queuing at the ramp nose unable to enter the freeway were noted. Frequent use of the ramp and freeway shoulders as well as hazardous maneuvers by ramp drivers forcing their way into the mainline flow were also observed.

It became apparent that operating conditions in the Morse Road to Seventeenth Avenue section were the major source of delay to motorists on SB I-71 during the morning peak period and that this section represented a dangerous accident hazard as well. It was to further define the problems occurring in this region that the ground-based studies were undertaken.

Ground-Based Studies

Travel Time Runs

The results of the three sets of travel time runs conducted are
conveniently summarized in a series of velocity-distance profiles in which the average velocity of the test vehicle between each pair of reading points is plotted against the corresponding position on the freeway for each run. The profiles for representative runs from February 5, 1970, March 24, 1970, and June 18, 1970, are shown in Figures II-6, II-7, and II-8 respectively. Although the magnitudes of the average section speeds vary for the three days surveyed, the same general pattern is evident in each case. During the period when congestion is the heaviest (7:20-7:50 AM) the lowest average speeds are found in the region between the East North Broadway off-ramp and the Hudson Street overpass. Speeds in this region are consistently below 40 mph and can drop as low as 10 mph at certain points.

In addition to the low speeds, observers in the test vehicles noted a large number of weaving maneuvers taking place in this region as motorists jockeyed for position trying to find a path of least resistance through the high density traffic. Many of these maneuvers were directed into gaps that were too small to allow for smooth merging and, hence, other motorists were forced to slow down to avoid rear end collisions. As a result traffic flow became increasingly unstable. Especially high numbers of weaving maneuvers were recorded in the East North Broadway to Weber and Weber to Hudson subsections. A similar problem was caused by on-ramp drivers in these same two subsections who forced their way into the mainline stream. This phenomenon was observed earlier from the air during the general survey flights.

Another interpretation of the data from the travel time runs is
Figure II-6 Velocity-Distance Profiles for Data of Thursday, February 5, 1970
Figure II-7 Velocity-Distance Profiles for Data of Tuesday, March 24, 1970
Figure II-8 Velocity-Distance Profiles for Data of Thursday, June 18, 1970
presented in Figure II-9. In this figure the total travel times for the individual runs (Morse Road on-ramp to Seventeenth Avenue off-ramp) is plotted versus the time of day at the start of the run for the data of February 5 and March 24. Once again the magnitudes of the values are different for the different days but the pattern is generally the same in each case. Congestion begins between 7:15 and 7:20 AM and lasts until about 7:50 AM with the most severe congestion each day occurring at about 7:30 AM. During this period of heavy congestion motorist travel times are anywhere from 25% to 100% greater than the free flow values for the same section. The time lost by motorists during this 30 minute interval indicates that a significant operational problem does exist on I-71 during the southbound morning peak period.

Density Trap Study (Input-Output)

Although no meaningful density data was obtained from the Input-Output study due to the small counting errors discussed earlier, certain valuable information was collected. This information included:

1. Composition of traffic (total and by lanes)
2. Lane usage (total and by section of freeway)
3. Ramp usage (total and pattern over time)
4. Traffic volume (total and pattern over time)

This data is thought to be representative of conditions occurring during each week on Interstate 71. No accidents or inclement weather disturbed the flow of traffic nor were any other unusual traffic events noted.
Figure II-9  Pattern of Travel Time with Time of Day for Travel Time Runs of February 5 and March 24, 1970
In addition, the pattern of travel times and the velocity profiles collected on Thursday, June 18, while the study was in progress were similar to those collected during February and March providing further evidence that the study week was a typical week.

The volume data collected was summarized in the form of histograms for each of the five days of data collection. A separate group of histograms was constructed for each traffic lane at each of the three overpass observation points and for each ramp starting with the East North Broadway on-ramp and continuing through the Hudson Street on-ramp. In addition a summary plot was prepared showing the variation of volume at each counting station with the day of the week. The histograms for the five ramps are shown as Figures II-10 to II-14 and the summary for all counting points is presented as Figure II-15. These figures reveal that there is marked volume variation within the two hour counting period during each day as well as variation in the pattern exhibited by volume from day to day. At the same time, however, the total two hour volumes recorded at each counting station remain relatively consistent throughout the week with a maximum variation of only about 10%. Similarly, the peak hour volume values calculated for each counting station do not vary greatly from day to day as shown in the following table (Table II-1). Thus it should be possible to obtain a reasonably accurate estimate of the volumes which can be expected at each counting station during either the two hour period from 6:30 to 8:30 AM or the peak hour by using the five day average values from the Input-Output study. The peak hour varies somewhat from station to
Figure II-10 Volume Histogram for East North Broadway On-ramp
Figure II-11 Volume Histogram for Weber Road Off-ramp
Figure II-12 Volume Histogram for Weber Road On-ramp
Figure II-13 Volume Histogram for Hudson Street Off-ramp
Figure II-14 Volume Histogram for Hudson Street On-ramp
Figure II-15 Summary of Day to Day Volume Variation at Various Counting Stations for Period June 15 to June 19, 1970
(Study Interval - 6:30 to 8:30 AM)
station but generally speaking lasts from 7:00 to 8:00 AM.

Table II-1 Peak Hour Volumes at Counting Stations for Various Days of the Week

<table>
<thead>
<tr>
<th>Counting Station</th>
<th>Peak Hour Volume</th>
<th>5 Day Avg</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mon</td>
<td>Tues</td>
</tr>
<tr>
<td>East North Broadway O/P</td>
<td>4376</td>
<td>4295</td>
</tr>
<tr>
<td>Weber Road Overpass</td>
<td>5029</td>
<td>5119</td>
</tr>
<tr>
<td>Hudson Street Overpass</td>
<td>5442</td>
<td>5434</td>
</tr>
<tr>
<td>East North Broadway on-ramp</td>
<td>1050</td>
<td>913</td>
</tr>
<tr>
<td>Weber Road Off-ramp</td>
<td>130</td>
<td>135</td>
</tr>
<tr>
<td>Weber Road On-ramp</td>
<td>759</td>
<td>758</td>
</tr>
<tr>
<td>Hudson Street Off-ramp</td>
<td>403</td>
<td>420</td>
</tr>
<tr>
<td>Hudson Street On-ramp</td>
<td>418</td>
<td>405</td>
</tr>
</tbody>
</table>

Table II-2 summarizes the lane usage data collected at the three overpass observation points. The average lane usage values for SB I-71 agree closely with those reported for six lane freeways by May (20) in his study of traffic characteristics on high density controlled access highways.

Combining the data of Tables II-1 and II-2 the peak hour volumes traveling in the shoulder lane at each overpass can be calculated. The five day average values obtained are 1007 vph at East North Broadway, 1395 vph at Weber Road and 1442 vph at Hudson Street. By adding to these shoulder lane volumes the peak hour volume at the three on-ramps the peak hour demand
on the respective ramp merging areas can be established: East North Broadway - 1945 vph, Weber Road - 2135 vph, Hudson Street - 1862 vph.

These high demand volumes provide an explanation for the extreme difficulties experienced by on-ramp drivers in merging into the freeway stream which were noted earlier during the general survey flights and the travel time runs.

<table>
<thead>
<tr>
<th>Day</th>
<th>E.N. Broadway Lane Usage (%)</th>
<th>Weber Road Lane Usage (%)</th>
<th>Hudson Street Lane Usage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>S</td>
<td>C</td>
<td>M</td>
</tr>
<tr>
<td>Monday</td>
<td>25.1</td>
<td>32.3</td>
<td>40.6</td>
</tr>
<tr>
<td>Tuesday</td>
<td>22.9</td>
<td>35.2</td>
<td>41.9</td>
</tr>
<tr>
<td>Wednesday</td>
<td>23.4</td>
<td>35.6</td>
<td>41.0</td>
</tr>
<tr>
<td>Thursday</td>
<td>23.5</td>
<td>34.8</td>
<td>41.7</td>
</tr>
<tr>
<td>Friday</td>
<td>23.2</td>
<td>35.9</td>
<td>40.9</td>
</tr>
<tr>
<td>5 Day Average</td>
<td>23.6</td>
<td>35.2</td>
<td>41.2</td>
</tr>
</tbody>
</table>

S = Shoulder Lane  C = Center Lane  M = Median Lane

Table II-3 is a summary of traffic composition by lane in the Broadway to Weber and Weber to Hudson subsections. For purposes of this analysis a truck was considered to be any vehicle of intermediate or large size having significantly different operating characteristics than a passenger car. Thus buses were combined with the truck percentage while pick-ups and light vans were considered as passenger cars.
Table II-3 Traffic Composition by Lane

<table>
<thead>
<tr>
<th>Day</th>
<th>% Trucks in Indicated Lane</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Median Lane</td>
<td>Center Lane</td>
<td>Shoulder Lane</td>
<td>Average</td>
</tr>
<tr>
<td>Wedn. May 6</td>
<td>0.00</td>
<td>4.46</td>
<td>3.66</td>
<td>2.62</td>
</tr>
<tr>
<td>Tues. May 19</td>
<td>0.32</td>
<td>4.30</td>
<td>3.74</td>
<td>2.87</td>
</tr>
<tr>
<td>Tues. May 26</td>
<td>0.09</td>
<td>4.30</td>
<td>2.80</td>
<td>2.30</td>
</tr>
<tr>
<td>Average</td>
<td>0.21</td>
<td>4.44</td>
<td>3.40</td>
<td>2.60</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Day</th>
<th>% Trucks in Indicated Lane</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Median Lane</td>
<td>Center Lane</td>
<td>Shoulder Lane</td>
<td>Average</td>
</tr>
<tr>
<td>Tues. May 19</td>
<td>0.05</td>
<td>4.80</td>
<td>3.49</td>
<td>2.74</td>
</tr>
<tr>
<td>Wedn. May 20</td>
<td>0.00</td>
<td>5.94</td>
<td>2.84</td>
<td>2.84</td>
</tr>
<tr>
<td>Mon. May 25</td>
<td>0.05</td>
<td>2.73</td>
<td>3.07</td>
<td>2.02</td>
</tr>
<tr>
<td>Tues. June 2</td>
<td>0.72</td>
<td>3.54</td>
<td>3.02</td>
<td>2.24</td>
</tr>
<tr>
<td>Average</td>
<td>0.21</td>
<td>4.25</td>
<td>3.11</td>
<td>2.46</td>
</tr>
</tbody>
</table>

The average percentage of trucks for all lanes was approximately 2.5% for both subsections with no value higher than 6% for any individual lane on any day. In addition, almost all trucks travel in the center and shoulder lane leaving the higher speed median lane to passenger car traffic. Trucks during the morning peak hour do not seem to be a problem.

Supplementary Volume Counts

Supplementary volume counts were conducted at the merge areas at East North Broadway, Weber Road and Hudson Street on three different days.
Data was collected at 5 minute intervals and a plot of volume versus time of day was prepared for each merge area based on a three day average. These plots are shown as Figures II-16 to II-18. A graphical procedure was then used to provide an empirical estimate of the maximum volume which could consistently pass through each merge area without causing a breakdown in traffic flow. It is important that traffic flow be kept stable and not allowed to peak because a flow peak is characteristically followed by deteriorating flow which results in delay to motorists and increases the hazard of rear end collisions.

In each case the volume value chosen was between 91% and 96% of the peak volume for any individual 5 minute interval. This value was defined as the desirable capacity of the merge area and a summary of these values is presented in Table II-4.

<table>
<thead>
<tr>
<th>Merge Area</th>
<th>Desirable Capacity (5 minute volume for three lanes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>East North Broadway</td>
<td>460</td>
</tr>
<tr>
<td>Weber Road</td>
<td>505</td>
</tr>
<tr>
<td>Hudson Street</td>
<td>505</td>
</tr>
</tbody>
</table>

It is significant to note that the capacity value for the East North Broadway merge area is about 10% lower than the other two. This explains why the Broadway to Weber subsection is heavily congested even though volumes in
Figure II-16 Merge Area Capacity Plot for East North Broadway Merge Area
Figure II-17 Merge Area Capacity Plot for Weber Road Merge Area
Figure II-18  Merge Area Capacity Plot for Hudson Street Merge Area
this region are 10% to 15% lower than volumes found downstream at Weber Road and Hudson Street. (See Table II-1.)

**Intensive Survey Flights**

One important parameter of freeway operation obtained from the intensive survey flights was traffic density. For the purpose of obtaining density data by aerial survey the freeway study section was divided into five subsections as listed below.

<table>
<thead>
<tr>
<th>Subsection Number</th>
<th>Description and Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cooke Rd. overpass-East North Broadway overpass (5597 ft.)</td>
</tr>
<tr>
<td>2</td>
<td>East North Broadway overpass-Weber Rd. overpass (2530 ft.)</td>
</tr>
<tr>
<td>3</td>
<td>Weber Rd. overpass-Hudson St. overpass (3390 ft.)</td>
</tr>
<tr>
<td>4</td>
<td>Hudson St. overpass-Hiawatha Park overpass (2780 ft.)</td>
</tr>
<tr>
<td>5</td>
<td>Hiawatha Park overpass-17th Ave. off-ramp nose (2520 ft.)</td>
</tr>
</tbody>
</table>

Figures II-19 to II-23 present summaries of the traffic density in each subsection as a function of time of day for each of the three days surveyed.

The following general observations can be noted from these density plots:

1. Traffic densities are consistently higher on Monday than on the other two days surveyed in every subsection

2. The density values for Thursday and Friday exhibit clear similarities
Figure II-20  Traffic Density Versus Time of Day for East North Broadway Overpass—Weber Road Overpass Subsection
Figure II-21  Traffic Density Versus Time of Day for Weber Road Overpass—Hudson Street Overpass Subsection
Figure II-23  Traffic Density Versus Time of Day for Hiawatha Park Overpass—Seventeenth Avenue Off-ramp Nose Subsection
in both magnitude and pattern over time. The higher density Monday values follow a somewhat different pattern.

3. The time of day at which the highest densities occur is not the same for the entire study section. Considering Thursday and Friday only, the peak density occurs at approximately 7:30 AM in subsections 1 and 2. This peak then moves southward occurring at approximately 7:35 AM in subsection 3, 7:40-7:45 AM in subsection 4 and 7:50 AM in subsection 5. The Monday data follows a more irregular pattern but once again the peak density occurs in the various subsections at different times.

It is difficult to make specific conclusions regarding the relative congestion existing in the various subsections from Figures II-19 to II-23 due to the variability of the density patterns from subsection to subsection. A parameter suitable for comparing subsections can be obtained, however, by integrating the density curves to determine the area contained underneath. This area has dimensions of vehicle-time per distance and is a measure of the amount of motorist travel time spent per unit distance in each subsection. This parameter is referred to herein as the Congestion Index and a summary of values for the data of Figures II-19 to II-23 is shown in Table II-5.

The data of Table II-5 reveals that on days with lower traffic volumes (Thursday and Friday) the entire region between East North Broadway and the Seventeenth Avenue off-ramp operates at about equal efficiency (or inefficiency) with the heaviest congestion experienced in Section 5 on Friday. On days with
high volumes (Monday), however, there is more variability between subsections with Section 2 exhibiting greater congestion than any other section. In this section (East North Broadway-Weber Road) approximately 232 vehicle-hours of motorist travel time are spent per mile during the morning peak hour. This high value represents significant delay to motorists and is indicative of definite operational problems existing in the Broadway to Weber subsection.

Table II-5 Summary of Congestion Indices

<table>
<thead>
<tr>
<th>Day of Week</th>
<th>Section 1</th>
<th>Section 2</th>
<th>Section 3</th>
<th>Section 4</th>
<th>Section 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monday, June 15</td>
<td>89.8</td>
<td>132.5</td>
<td>113.0</td>
<td>99.4</td>
<td>103.0</td>
</tr>
<tr>
<td>Thursday, June 16</td>
<td>78.2</td>
<td>133.8</td>
<td>134.5</td>
<td>135.2</td>
<td>130.5</td>
</tr>
<tr>
<td>Friday, June 19</td>
<td>75.1</td>
<td>112.0</td>
<td>125.0</td>
<td>123.0</td>
<td>152.5</td>
</tr>
</tbody>
</table>

Note: Values for Monday, June 15, are based on only 30 minutes of data while those for Thursday, June 16, and Friday, June 19, represent 60 minutes of data. To convert Monday data to a 60 minute base multiply each value by approximately 1.75.

In addition to the density data described above, the intensive survey flights provided information on weaving maneuvers and traffic stoppages. Especially high weaving volumes were recorded in the East North Broadway-Weber and Weber-Hudson subsections with the high entrance ramp volumes at East North Broadway (938 vph) and Weber Road (740 vph) contributing significantly to the total. Many traffic stoppages were observed to occur as a direct result of this difficult weaving situation. Stoppages were noted in all three lanes but the greatest number consistently occurred in the shoulder lane and
were caused primarily by ramp drivers forcing their way into the freeway stream. Stoppages caused in this manner affected in some cases as many as 30 vehicles and lasted for as long as three minutes before movement was restored.

The data on lane usage and traffic composition obtained from the intensive survey flights was used to check that collected during the Input-Output study. Close enough agreement (± 5%) was achieved in each case to support the accuracy of the Input-Output data. No analysis was performed to obtain speed information from the aerial films as it was felt that the spot speed values resulting would not be meaningful enough to justify the large amount of data reduction time required to produce them.

Summary of Findings

On the basis of the data collected, it seems clear that significant congestion is experienced by motorists traveling on Interstate 71 during the morning peak period. The critical section exhibiting the worst operational problems extends from the off-ramp at East North Broadway to the on-ramp at Hudson Street and consists primarily of the two subsections between the Broadway, Weber Road and Hudson Street overpasses. Low average velocities, high densities, high weaving volumes and frequent traffic breakdowns characterize both subsections. High entrance ramp volumes on the East North Broadway and Weber Road on-ramps result in both stoppages in the mainline stream and queues at the respective ramp noses. The danger of collision in the
vicinity of the ramp merge points is quite high and several accidents were observed during the course of data collection. Each accident results in extensive delay to the heavily loaded mainline stream.

Both freeway geometrics and traffic demand contribute to the problems experienced in the critical section. The relatively close spacing of the East North Broadway, Weber Road and Hudson Street Interchanges does not provide adequate weaving length for the high weaving volumes found in the area. Although an auxiliary lane is provided between ramps in each subsection this lane is not used efficiently by motorists and appears to be more of a traffic hazard than an aid to smoother traffic flow. The close interchange spacing also results in a roller coaster-like vertical alignment which increases the difficulty of the driver's task especially under high density operating conditions. The sweeping horizontal curve just south of Hudson Street also seems to have a retarding effect on peak hour traffic flow especially those vehicles in the median lane. Median lane drivers approaching this curve can be observed applying their brakes for no apparent reason other than the psychological fear of being squeezed between the center lane of traffic and the glare screen on the roadway median. High traffic demands prevail throughout the critical section with demand frequently exceeding the capacity of the freeway in the on-ramp merging areas.

It was thought that a freeway control system so designed as to limit the traffic volumes entering the Broadway to Hudson Street section could be of great benefit in alleviating congestion and improving traffic flow during the
morning peak hour. Such a system featuring fixed-time ramp metering and utilizing simple and inexpensive equipment is proposed and described in the next section.

1.2 FORMULATION OF PEAK PERIOD CONTROL STRATEGY

Control can be exerted on freeway traffic in any of three locations; namely, when traffic leaves the freeway at exit ramps, when traffic is on the freeway and when traffic enters the freeway at entrance ramps. Experience to date gained in freeway surveillance and control projects conducted in Chicago, Houston and Los Angeles indicates that of these three approaches control of traffic entering the freeway at entrance ramps is the most effective technique for improving freeway traffic flow.

1.2.1 REVIEW OF SELECTED FREEWAY CONTROL PROJECTS

It is not the purpose of this section to provide an all-inclusive review of the experience accrued in the approximately seventeen years since the first freeway surveillance project was initiated on the John C. Lodge Freeway in Detroit, Michigan. Rather, it includes only those projects whose results are thought to have a pertinent bearing on the control problem existing in Columbus, Ohio.

Freeway Ramp Control Techniques

There are several methods which can be used to control traffic on freeway entrance ramps. These methods are listed and described briefly below.
Ramp Closure

Ramp closure is the most restrictive of the ramp control techniques. It consists of closing off an entrance ramp during all or part of the peak period through either the posting of warning signs or the use of physical barriers or both. This technique can be used to reduce demand on a freeway section or, in certain cases, to improve poor geometrics. For example, ramp closure could be used to increase the effective interchange spacing in a section where closely spaced interchanges generate operating conditions which result in reduced traffic flow. Use of ramp closure requires that a suitable alternative route be available capable of absorbing the diverted freeway traffic.

Fixed-Time Ramp Metering

Fixed-time ramp metering is a type of ramp control in which vehicles are released onto the freeway at predetermined time intervals based on an estimate of expected freeway flow conditions. This estimate is generally based on an analysis of historical data collected in the control section. Several different metering rates may be used during the peak period but these rates do not respond to unexpected changes in traffic conditions and, hence, system breakdowns may occur due to accidents or other traffic incidents. Though less restrictive than ramp closure, this type of control also may result in extensive traffic diversion and, thus, requires the existence of a suitable alternative route.
Capacity-Demand Ramp Metering

A ramp metering system using the capacity-demand mode of control is capable of responding to traffic flow changes on the freeway as they occur. The metering rate is variable and is determined as a function of one or more flow parameters which are continuously monitored on the freeway through the use of surveillance equipment. Parameters which might be chosen include lane occupancy, volume, density or average speed in one or all lanes of the freeway stream. The choice of parameters for use and the form of the metering function is once again determined from analysis of historical data. This type of system is obviously more expensive than the previous two due to the use of freeway surveillance equipment. The availability of an alternate route for diverted traffic is also a requirement for this mode of control.

Gap Acceptance Ramp Metering

Gap acceptance metering is another form of responsive ramp control. Whereas demand-capacity metering is based on average stream characteristics this mode of control utilizes a more microscopic approach. Freeway surveillance equipment is used to locate and measure existing gaps in the shoulder lane flow. When an "acceptable" gap is found a ramp vehicle is released to fill this gap. The ramp driver may be left to perform the merge on his own or further equipment may be added to guide or pace him into the gap. The underlying philosophy of this approach is that by helping the driver with the ramp merging maneuver one of the primary causes of freeway flow breakdowns can
be eliminated. In addition, since all acceptable freeway gaps are filled optimal use is made of existing shoulder lane capacity.

This mode of control is more expensive than the demand-capacity mode as it requires more sophisticated equipment. It will also result in very high ramp diversion rates when freeway shoulder lane volumes are high and, hence, acceptable gaps few in number.

System Demand-Capacity Ramp Metering

The previous four metering approaches treat each ramp as an isolated entity to be controlled without regard to traffic conditions in the remainder of the system. In system ramp metering a series of ramps are controlled in a group and the type of control to be employed at each ramp is dependent on conditions in the system considered as a whole. This mode of control requires local controllers located on each ramp as well as a central controller which coordinates system control activities. Extensive surveillance equipment is required to monitor traffic conditions throughout the control section and an on-line computer is needed to perform the control logic decision-making. System metering thus becomes the most expensive form of freeway ramp control.

Pertinent Ramp Control Experience

Each of the above control techniques has been field tested at one time or another during one of the major freeway control projects. A summary of the experience gained from a selected group of projects is given below.
Harbour Freeway, Los Angeles (21)

The Harbour Freeway is a major 8-lane facility extending 22 miles from downtown Los Angeles to San Pedro. The control project involves limiting on-ramp traffic in a 5-mile section of the southbound lanes during the evening peak period. Prior to ramp control heavy congestion was caused by demand exceeding capacity in two bottleneck areas for much of the peak hour. Six ramps are controlled in total using a combination of ramp closure and fixed-time metering. One ramp is closed during the period between 3:30 and 5:30 PM, 2 ramps release platoons of vehicles at fixed intervals and 3 ramps are controlled on a single car fixed-time basis. The fixed-time signals use a variety of metering rates during the peak period with the choice of rate determined by a time clock.

As a result of ramp control a minimum average speed of 40 mph is maintained throughout the control period even though occasional breakdowns still occur. This compares with speeds of 15 to 25 mph prior to control. Individual motorists save an average of 4 to 5 minutes per trip with a maximum saving of 9 minutes on some days. It is estimated that about 1,000 vehicle-hours per day are saved on the freeway against an increase of about 130 vehicle-hours for diverted or delayed ramp traffic. The average travel time increase for diverted traffic is estimated to be 2 minutes.

Hollywood Freeway, Los Angeles (22)

A one mile section of this freeway, three lanes wide, is fed by a four-lane roadway and two on-ramps (Sunset Boulevard and Hollywood Boulevard).
In this section demand exceeded capacity during the evening peak period and ramp control was initiated on the two on-ramps. The Sunset Boulevard ramp was metered at a fixed rate of 1 vehicle per fifteen seconds with control provided by a standard three lens traffic signal. The Hollywood Boulevard ramp was closed for the peak period (4:15-6:00 PM).

Even with this extremely simple freeway control strategy, freeway traffic conditions improved tremendously. Speeds on the freeway improved from stop and go to an average of about 40 mph and delay per vehicle was reduced by 3 to 5 minutes depending upon the time of the journey within the peak period. Total travel time savings to freeway drivers was estimated at 485 vehicle-hours for the two-hour peak period. No serious increase in travel times was noted on the adjacent surface streets.

Gulf Freeway, Houston (22)

Eight on-ramps in a six mile section of this six lane freeway are being controlled during the morning peak period. Control is exerted using both the demand-capacity and gap acceptance metering modes. The metering device is a standard three lens traffic signal.

Freeway volume upstream of each ramp is used as the control parameter for the demand-capacity metering. Mainline flow is monitored in all three inbound lanes and converted to an equivalent hourly volume which is used to determine the metering rate between a minimum of 240 vph and a maximum of 900 vph. The ramp flow rate is chosen so that it in combination with the mainline flow will not exceed the capacity of the on-ramp merging area. Shoulder
lane gaps required for gap acceptance metering are measured by a detector located several hundred feet upstream of the ramp nose. The size of the "acceptable" gap was determined from historical data. An analog computer is used to calculate the proper time of release to allow smooth merging of ramp vehicles with the mainline stream.

Total travel and total travel times were used to evaluate and compare the effectiveness of the two control techniques. Both techniques significantly increased the total throughput of the freeway corridor while producing a marked reduction in total travel time. Estimated daily travel time savings for demand-capacity metering was 330 to 340 vehicle-hours. Gap acceptance metering resulted in a lesser savings but was thought to produce a more stable traffic flow.

Eisenhower Expressway, Chicago (23, 24)

Four entrance ramps over a 2.5 mile three-lane section of this freeway were metered to reduce traffic demand during the westbound evening peak period. The demand-capacity mode of control was used with center lane occupancy employed as the control parameter. The variable metering rate ranged from a minimum of 234 vph to a maximum of 570 vph and was determined at specified intervals by comparing the actual occupancy to a predetermined occupancy representing capacity flow.

After control, speeds on the mainline ranged from 30 to 40 mph as compared to 20 to 35 mph before control. Net travel time savings for freeway
motorists was estimated at 256 vehicle-hours with an estimated travel time increase for diverted traffic of only 2 minutes per vehicle. A significant reduction in the number of peak period accidents was also attributed to the control system.

Dan Ryan Expressway, Chicago (25)

A short term study of entrance ramp control was conducted during the morning peak period on the northbound lanes of the Dan Ryan Expressway. Four successive entrance ramps were metered using a fixed-rate of 1 vehicle per 5 seconds exerted by manually operated, portable control equipment. Before metering volumes at one of the ramps had been as high as 1370 vph.

Although ramp control did not eliminate congestion during the peak period the duration and extent of the congestion was significantly reduced. There was a reduction in total travel time and a 5.2% increase in vehicle-miles of travel between 6:00 and 8:30 AM. No noticeable change was detected on the surface streets even though 1330 vehicles were diverted from the freeway. Ramp delays were relatively large running as high as 7 minutes for certain vehicles.

Summary of Findings from Control Project Review

Entrance ramp control has consistently proven to be a viable method for decreasing delay and increasing throughput on congested urban freeway systems. Significant flow improvements have been achieved using traffic responsive systems capable of instantaneous response to changing traffic
conditions. Unfortunately such systems often require both extensive freeway surveillance equipment to monitor flow conditions and a computer of some sort to convert the observations of this equipment into a suitable control decision. Equipment of this kind is expensive to acquire and maintain.

Experience from the California studies and the Dan Ryan control project indicates that very marked improvements in flow can also be achieved using fixed-time metering and ramp closure. These modes of control can be implemented using relatively inexpensive equipment which can be installed in a short period of time. Since Columbus, Ohio, is just beginning to experiment with freeway control methods and since the critical section of freeway to be controlled is relatively short in length (about 2.0 miles), it seems wise to begin with a simple system which can be modified as experience with its use is gained. As such a system could be constructed using equipment already owned by the City of Columbus or the Ohio Division of Highways, it could be put to use immediately at little cost. Consequently it was decided to design the control system for southbound I-71 based on a fixed-time metering approach to ramp control.

1.2.2 PROPOSED CONTROL SYSTEM FOR SOUTHBOUND INTERSTATE 71

The control strategy proposed for southbound I-71 is based on a system used during the early stages of the Gulf Freeway Control Project. The strategy aims at keeping the merging volumes just downstream of freeway on-ramps at or below an optimum merging volume such that stable flow will be maintained.
and traffic breakdowns will not occur. This is accomplished by limiting on-ramp volumes when necessary using fixed-time metering techniques.

**Definition of Control Area**

The studies conducted on southbound I-71 revealed that the critical section during the morning peak hour extends from the off-ramp at East North Broadway to just south of the on-ramp at Hudson Street. Three entrance ramps are located in this section at the East North Broadway, Weber Road and Hudson Street interchanges. At each of these ramps, merging area volumes were found to exceed during a considerable portion of the peak hour the volume determined to be the maximum allowable for maintenance of stable flow by the merge area capacity analysis. It was decided, therefore, to institute fixed-time ramp metering at these three ramp locations.

**Determination of the Metering Policy**

The selection of the proper metering rate and the duration of the metering period at each ramp can be established using the desirable merge area capacity values determined by the merge area capacity analysis and the volume data collected during the Input-Output study.

Experience gained from the control projects conducted in Los Angeles and Houston has shown that the maximum allowable metering rate is 1 vehicle per 4 seconds (900 vph). Above this value the effect on merging traffic is the same as if no metering was being done at all. Since this rate defines the threshold of successful metering, it was used to determine the duration of the
metering period at each ramp. Figure II-24 shows in histogram form the average allowable ramp volumes for each on-ramp calculated at five minute intervals. The allowable volume was determined for each interval by subtracting the total upstream volume for all freeway lanes counted at the respective overpasses from the merge area capacity for the corresponding ramp. A line has been drawn on each histogram at the 75 vehicle level (1 vehicle/4 seconds) signifying the limit of ramp metering. In those intervals where the allowable ramp volume is less than 75 vehicles per 5 minutes metering must be used. Observation of Figure II-24 reveals the following times when metering should be in force.

- East North Broadway 7:10 to 7:35 AM
- Weber Road 7:10 to 7:40 AM
- Hudson Street 7:05 to 7:45 AM

Figure II-24 can also be used to determine the proper metering rate. This rate is calculated using the allowable ramp volumes shown on this figure and the metering period duration shown above. For each interval the allowable ramp volume can be converted to an equivalent rate of 1 vehicle per every X seconds. A metering strategy can then be established which allows passage of a single vehicle at the appropriate time intervals.

Following this procedure for each of the three on-ramps in the critical section and averaging over all 5 minute intervals within the metering period for a period of one week, the following constant metering rates were derived.
Figure II-24  Average Allowable Ramp Volume Histograms for East North Broadway, Weber Road and Hudson Street On-ramps
East North Broadway 1 vehicle per 5.0 seconds
Weber Road 1 vehicle per 5.3 seconds
Hudson Street 1 vehicle per 10.0 seconds

These metering rates fall comfortably below the 1 vehicle per 4 seconds guideline and indicate that metering is indeed necessary.

Other Control Considerations

Metering of the three on-ramps at the rates and for the periods discussed above will cause vehicle queues to form on each ramp. The proper treatment of these queued vehicles so that they do not disturb traffic flow on the adjacent surface streets is an important part of the overall control problem. Utilizing the selected metering policy and the historical ramp demand data collected during the Input-Output study, the length of queue for each ramp can be calculated at 5 minute intervals. Figure II-25 shows the theoretical variation of queue length with time of day calculated in this manner. These queue lengths are based on the assumption that no significant variation in travel patterns will occur due to the freeway control project.

Observation of Figure II-25 reveals that the maximum queue lengths which can be expected are about 85 vehicles at East North Broadway, 55 vehicles at Weber Road and 45 vehicles at Hudson Street. These queue lengths greatly exceed the available storage areas on the ramps which provide room for only 22, 26 and 42 vehicles respectively. Thus it becomes apparent that a certain number of ramp-bound vehicles must be diverted to the surface street.
Figure II-25 Variation of Ramp Queue Length with Time of Day for Metered Southbound Entrance Ramps
network. The volume of diverted traffic is estimated to be 63 vehicles at East North Broadway, 29 vehicles at Weber Road and 3 vehicles at Hudson Street during the respective metering periods. These vehicles will have to be absorbed by the surface street network.

The diverted traffic at East North Broadway and Weber Road should not cause serious problems due to the presence of a service road, Silver Drive, which parallels I-71 on the west side between East North Broadway and Hudson Street. Special volume counts conducted on this road recorded a peak hour volume in the southbound direction of only 125 vph at the East North Broadway intersection and 226 vph at the Weber Road intersection. Sufficient unused capacity exists on this roadway to easily absorb the added traffic load. The small number of vehicles diverted from the Hudson Street ramp should have no significant effect on surface street operations. These vehicles can proceed westward to Summit Street and follow this one-way arterial to their downtown destinations. Summit Street should also be able to handle the increased traffic load from Silver Drive without undue difficulty.

Proposed Improvements to Surface Street Network

Although the routes discussed in the above section provide adequate capacity for diverted ramp traffic at present, this condition is not expected to prevail for an extended period of time. Increased traffic loads expected on I-71 during the next five years will soon overload Silver Drive and Summit Street. It is recommended, therefore, that in addition to the freeway control
project immediate steps be taken to provide increased surface street capacity for southbound traffic in the section between Morse Road and Seventeenth Avenue. Toward this end two specific improvements are proposed.

Proposed Improvement of Silver Drive

Silver Drive is presently designed to permit two way operation between East North Broadway and Hudson Street. As a consequence, it must cross the East North Broadway and Weber Road on-ramps to ensure that traffic waiting to make left turns from these streets onto the service road does not block traffic wishing to turn onto the ramp and so that traffic can leave the service road eastbound without interference from ramp traffic. This crossing creates a dangerous accident hazard and decreases the storage area available on the ramps for queued vehicles.

It is recommended that Silver Drive be converted to one way southbound and that the service road-ramp crossing be eliminated in favor of a combined design such as the one shown for East North Broadway in Figure II-26. This modification would increase available ramp storage area, eliminate the hazardous crossing maneuver and increase the capacity of Silver Drive southbound thus facilitating the diversion of traffic from I-71. The effect of this modification on the accessibility of the commercial establishments located on the west side of Silver Drive is not thought to be great due to the relatively short distance between Broadway and Weber and Weber and Hudson. Motorists wishing to reach these establishments from the south will simply have to go around the block.
To further facilitate the use of Silver Drive for the diversion of traffic from the freeway, it is recommended that fixed-time traffic signals be installed at its intersection with East North Broadway and Weber Road. A signal is already in operation at the Hudson Street intersection. These three signals should then be inter-connected to allow progressive traffic movement from north to south along Silver Drive.

Proposed North-South Arterial

In addition to the Silver Drive improvement it is recommended that a new north-south arterial be developed along the right-of-way presently occupied by Karl Road and either McGuffey or Hamilton Avenues. At present there are several good alternative routes for freeway drivers living west of the freeway. These include Summit Street via Indianola Avenue, High Street and the Olentangy River Road; each of which provides access to the CBD. On the east side of I-71, however, only Cleveland Avenue extends between Morse Road and downtown. This street is some distance from the freeway and has limited capacity due to bad geometrics in some sections. As a result it is observed that the majority of the traffic using the freeway at the metered ramps during the morning peak period comes from the east side of the freeway. This trend can be expected to continue and intensify with the rapid growth of the northeast quadrant of Columbus. The proposed north-south arterial would serve to relieve I-71 of some of this load. The route to be followed by this arterial is shown in Figure II-27.
Further southbound capacity east of the freeway could be provided by improvement of bottleneck sections on Cleveland Avenue especially the one located between Seventeenth Avenue and the Penn-Central Railroad bridge. This improvement is also indicated on Figure II-27 and would serve those drivers living in the far northeast area.

Summary of the Control Plan

The traffic control plan proposed for the Interstate 71 corridor in North Columbus can be summarized in five steps:

1. Fixed rate metering of the on-ramps at East North Broadway, Weber Road and Hudson Street according to the following Schedule:

<table>
<thead>
<tr>
<th>Ramp</th>
<th>Metering Rate</th>
<th>Metering Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>East North Broadway</td>
<td>1 vehicle/ 5.0 seconds</td>
<td>7:10 to 7:35 AM</td>
</tr>
<tr>
<td>Weber Road</td>
<td>1 vehicle/ 5.3 seconds</td>
<td>7:10 to 7:40 AM</td>
</tr>
<tr>
<td>Hudson Street</td>
<td>1 vehicle/10.0 seconds</td>
<td>7:05 to 7:45 AM</td>
</tr>
</tbody>
</table>

2. Conversion of the western service road, Silver Drive, from two-way to one-way southbound

3. Installation of a coordinated signal system on Silver Drive from East North Broadway to Hudson Street to include new signals at East North Broadway and Weber Road

4. Improvement of Cleveland Avenue to provide a major arterial route to the CBD from the northeast quadrant of Columbus
Administration to work out the details of a field test of the control plan. These negotiations were concluded in September, 1971.

1.3.1 ORGANIZATION OF THE FIELD TEST

It was decided that the control plan field test would be carried out as a joint project of The Ohio State University, the State of Ohio Department of Transportation, and the City of Columbus. The city would be responsible for obtaining and installing the required control equipment at its expense and would collect the necessary data for evaluating the effect of the control plan on traffic flow on the arterial streets in the Interstate 71 corridor. The university would be responsible for collecting the required data to evaluate the effect on traffic flow on the freeway itself and would aid the city in the evaluation of the effect on traffic movement in the corridor as a whole. The state would represent the research project sponsors and would act in an advisory capacity providing aid where needed.

The implementation schedule chosen was the latter one suggested in the previous section. For the purpose of the field test the control steps taken would include only the fixed-time metering of the three on-ramps and the installation of traffic signals at the intersections of Silver Drive with East North Broadway and Weber Road. Wednesday, October 13, 1971, was established as the target date for the beginning of the field test.

1.3.2 IMPLEMENTATION OF THE FIELD TEST

The implementation of the field test required effort in three specific
giving the duration of the metering period and suggesting an alternative route, Silver Drive, for use when the ramp storage area was full. A typical control installation for the East North Broadway ramp is shown in Figures II-28 and II-29. The entire control system is summarized in Figure II-30.

The signs and signals used for the control installation were obtained from stock materials on hand at the signal shop of the City of Columbus. Total cost for materials and labor required to control the three ramps was estimated at $2,000. In addition, the traffic signals installed on Silver Drive were obtained at a cost of $6,000 each. Thus the total cost of the control installation amounted to less than $15,000.

**Pre-Test Publicity**

Motorist understanding and cooperation are important prerequisites of any successful traffic control endeavor. This is especially true when an unfamiliar control technique such as freeway ramp metering is introduced into a community. If the full potential of this technique is to be realized motorists have to be informed as to why the technique is necessary, what it is expected to do and how they as individuals are expected to respond to it.

For this reason before the ramp signals were activated articles appeared in both the Columbus Citizen-Journal (morning paper) and the Columbus Dispatch (evening paper) explaining the purpose of the proposed system and how it was designed to operate. A portion of the Citizen-Journal article is included herein as Figure II-31. In addition spot announcements
Traffic Test
Plan Revealed

A test system of traffic lights intended to limit the number of cars entering the North Freeway at three southbound entrance ramps is to go into operation next month, Acting Traffic Engineer Joseph Ridgeway said Tuesday.

Ridgeway explained the lights, which use only red and yellow lenses, will be operated during the morning rush hour from 7 to 8 a.m. at E. North Broadway, E. Weber Rd., and E. Hudson St.

THE LIGHTS will be placed sufficiently back from freeway traffic lanes to allow motorists starting from a dead stop to attain freeway speed before emerging, Ridgeway said.

He explained there will be some stackup behind the lights which will release one vehicle at a time, but the ramps at E. North Broadway and E. Weber Rd. will allow storage of 10 cars or more.

At E. Hudson St., where Silver Dr. provides extra space, even more storage is available, Ridgeway stated.

HOWEVER, HE pointed out some backup is desired since it will encourage motorists to use alternate routes such as Summit St., thereby relieving the load on the freeway.

Signs directing traffic to alternate routes will be installed in conjunction with the lights, Ridgeway said.

"We don't expect the storage to be a problem if people will accept the alternate routing," he added.

Figure II-31 Pre-Test Publicity Article from Columbus Citizen-Journal

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under the original metering schedule. It was not thought that this slight increase in load (1 vehicle per 1.5 minutes) would cause any real problems.

Once adjusted the signals became a completely automatic installation requiring no human intervention. They were treated the same as all other traffic signals in the Columbus system with no special maintenance personnel standing by and no special police personnel assigned to guarantee motorist obedience. An observer from the research group was placed at each ramp during the first few days of automatic signal operation to observe motorist behavior. He was instructed to remain inconspicuous, however, and to give no indication that he was connected with the ramp metering project.

Once the complete system was operational it was allowed to run continuously Monday through Friday during the prescribed time period and effort was concentrated on the collection of the "after" data on traffic flow required to evaluate the effectiveness of the system. Both university and city personnel participated in this data collection effort.

1.4 EVALUATION STUDY: 1971-1972

In order to evaluate the effectiveness of the control steps taken on Interstate 71 an "after" study of traffic operations was conducted with data collection concentrated in the region between the Morse Road overpass and the Seventeenth Avenue overpass. The evaluation study was a joint effort of the university and the City of Columbus and covered the period between October 13, 1971, and July 31, 1972.
and 30) provide photographic coverage of a substantial part of the peak hour.
The data from these four flights was analyzed for comparison with the data
collected during the original SB I-71 study of 1970. The remaining flights
provide coverage varying from 25 to 38 minutes and were used to provide a
check on the corresponding portions of the completed flights.

Ground-Based Studies

A series of ground-based studies was conducted during the early part
of 1972 on Interstate 71 by university forces. Study procedures and locations
were so chosen as to allow comparison of the results of these studies with
data obtained during 1970. A separate series of studies was conducted by city
personnel with emphasis placed on traffic operations on the adjacent surface
streets in the I-71 corridor. The results of these studies were compared
with "before" data collected by the city during September and October of 1971.

Travel Time Runs - Ohio State University Study

Three sets of runs were conducted with coverage in each case extending
from the Morse Road on-ramp to the off-ramp at Cleveland Avenue. A total
of 26 runs were made broken down as follows:

<table>
<thead>
<tr>
<th>Date</th>
<th>Number of Runs</th>
<th>Time of Coverage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thursday, May 11, 1972</td>
<td>9</td>
<td>7:00–8:09 AM</td>
</tr>
<tr>
<td>Friday, May 26, 1972</td>
<td>8</td>
<td>7:00–8:10 AM</td>
</tr>
<tr>
<td>Monday, June 5, 1972</td>
<td>9</td>
<td>7:00–8:04 AM</td>
</tr>
</tbody>
</table>
volume passing under this overpass at five minute intervals during the period
6:45 to 8:15 AM for each of the five weekdays.

Volume Counts - Columbus Study

Automatic volume counts were made at nine locations in the freeway
corridor as follows:

1. SB I-71 just north of SB off-ramp at Hudson Street
2. SB High Street just south of Hudson Street
3. SB Indianola just north of Hudson Street
4. SB Silver Drive just south of East North Broadway
5. SB Silver Drive just south of Weber Road
6. SB Summit Street just south of Hudson Street
7. SB Cleveland Avenue just south of Hudson Street
8. WB East North Broadway just west of Silver Drive
9. WB Weber Road just west of Silver Drive.

The counts were conducted continuously 24 hours per day for a period of one
week (November 12 to 19, 1971).

The southbound counts were made to determine if a change in travel
patterns had occurred which could be attributed to the freeway control project.
The westbound counts were made for the same purpose as well as to determine
if the control project had resulted in an increased load being placed on the
already overloaded intersections of East North Broadway and Weber Road with
Indianola Avenue.
analysis was undertaken to determine if the number of accidents and the type of accidents found in the control section after control was initiated differed significantly from the accident pattern before control. Accident data was procured from the files of the Columbus Police Department and, hence, includes only those accidents on which police reports were filed. The section of freeway studied extended from the East North Broadway overpass to the on-ramp at Hudson Street.

1.4.2 DATA REDUCTION AND ANALYSIS

The data collected during the evaluation study was reduced, summarized and compared to the "before" data on freeway and surface street operation. The results of this "before" and "after" analysis are presented below. For ease of presentation the university and Columbus studies are discussed concurrently.

Aerial Survey Study

The data of the four completed aerial survey flights was summarized in the form of density versus time of day plots such as those shown in Section 1.1.2. One plot was prepared for each of the first four subsections. No plot could be made for the subsection between Hiawatha Park and the Seventeenth Avenue off-ramp as this section was not included on a sufficient number of the survey runs. These plots were then integrated to obtain values for the Congestion Index for comparison with the data of the original study. In each case the time values chosen as the limits of integration were selected to correspond
the highest volume of traffic is normally found on the freeway during the morning peak hour. It appears that under high volume conditions, with the ramp metering system in operation, subsection 1 (Cooke Road Overpass—East North Broadway Overpass) is challenging and perhaps has replaced subsection 2 as the region of greatest congestion.

An attempt was made to obtain an indication of the efficiency of traffic operation in the entire critical section on a "before" and "after" basis by calculating a combined Congestion Index. This combined Index was computed by adding the individual subsection indices for those subsections where data was available for all days (subsections 1 through 4). When this was done for the Monday data, however, the following results were obtained.

<table>
<thead>
<tr>
<th>Date</th>
<th>Combined Index ((\frac{\text{vehicle-hrs.}}{\text{mile}}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monday, June 15, 1970 (before control)</td>
<td>434.7</td>
</tr>
<tr>
<td>Monday, November 15, 1971 (after control)</td>
<td>492.1</td>
</tr>
<tr>
<td>Monday, July 31, 1972 (after control)</td>
<td>403.1</td>
</tr>
</tbody>
</table>

No real conclusion can be drawn from these values except to state the obvious; freeway operation can be both better and worse with ramp control in effect than it was during 1970 when no control existed. No more will be said about this apparent paradox at this time. An attempt will be made, however, to interpret this result later on in this chapter.
Figure II-32 Composite Speed-Distance Profiles for "Before" and "After" Travel Time Data
Volume Counts

Table II-8 summarizes the results of the "before" and "after" volume counts conducted by the city on Interstate 71, the adjacent parallel streets in the I-71 corridor and the important cross-streets in the control section. All volumes reported are for the two-hour period between 7:00 and 9:00 AM. Counts were made at the locations listed in Section 1.4.1.

In addition to this data a special volume count was undertaken on I-71 at the Hudson Street overpass to determine the peak hour volume being carried in 1972 for comparison with data collected during the 1970 Input-Output study. Based on five days of data collection (Monday through Friday) a peak hour volume of 5600 was obtained as compared to 5362 for March, 1970.

Analysis of the data of Table II-8 reveals the following relevant facts about Interstate 71 and the associated surface streets in the I-71 corridor.

1. Traffic volumes on I-71 measured at the southern end of the metered section at Hudson Street have increased by an average of 615 vehicles (6%) since ramp metering was initiated (7:00-9:00 AM).

2. Traffic volumes on southbound Indianola Avenue and Summit Street remain basically unchanged. Indianola Avenue shows an average increase of 57 vehicles during the two-hour counting period while Summit Street shows a decrease of 41 vehicles.

3. Volumes on southbound Silver Drive have increased significantly. An average increase of 61 vehicles (32%) was measured between East North
4. Volumes on southbound High Street have also increased significantly with a net increase of 208 vehicles recorded (11.3%).

5. Volumes on westbound East North Broadway and Weber Road measured just west of I-71 remain basically unchanged. East North Broadway shows an average decrease of 52 vehicles while Weber Road shows an increase of 39 vehicles.

It appears, therefore, that a substantial increase in traffic volume has been attained on Interstate 71 without any adverse effect on surface street flows. Only southbound Silver Drive and High Street show appreciable increases in traffic volume during the counting period. The increase on Silver Drive was expected since Silver Drive was the recommended alternate route for diverted freeway drivers. Sufficient excess capacity was available to absorb this load without difficulty. The increased volume on High Street was not expected and although no real evidence is available on the nature of this load increase, it is doubtful that it was due to the freeway control project. It is difficult to believe that freeway drivers would divert to High Street rather than Indianola Avenue and Summit Street which are much closer to I-71. These streets exhibited no significant volume changes.

The data from the special peak hour count can be combined with the average velocity from the travel time study to determine the change in peak hour throughput for the control section in 1972 as compared to 1970. If this is done, a net increase of 7983 vehicle-miles per hour\(^2\) is attained. It is thought that this throughput increase can be attributed to the freeway control project.
There was substantial delay, however, to motorists waiting in line at the metered on-ramps for entry to the freeway. This delay was computed using the sampling procedure described previously. Table II-9 summarizes the results of this delay analysis. The data presented is an average for four days of data collection. The delay at the ramps as shown in Table II-9 amounts to 78,058 vehicle-seconds or, equivalently, 21.7 vehicle hours. Combining this value with the delay estimated from sources 2 and 3 yields an average total delay caused by the freeway control project of 31.7 vehicle-hours maximum per day. This amount of delay, although irritating to the drivers involved, is substantially less than that caused by the single two-car accident mentioned earlier.

<table>
<thead>
<tr>
<th></th>
<th>Average Queue Length (veh)</th>
<th>Metering Interval (sec)</th>
<th>Average Delay per Vehicle (sec)</th>
<th>Metering Period Demand Volume (veh)</th>
<th>Total Delay (veh-sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E.N. Broadway</td>
<td>9.4</td>
<td>5.0</td>
<td>47.0</td>
<td>410</td>
<td>19,270</td>
</tr>
<tr>
<td>Weber Road</td>
<td>9.9</td>
<td>5.0</td>
<td>49.5</td>
<td>385</td>
<td>19,058</td>
</tr>
<tr>
<td>Hudson Street</td>
<td>13.7</td>
<td>10.0</td>
<td>137.0</td>
<td>290</td>
<td>39,730</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>78,058</strong></td>
</tr>
</tbody>
</table>

Accident Analysis

Accident data was collected for both the freeway itself and all of the on- and off-ramps between East North Broadway and Hudson Street. Informa-
conducted to provide specific information about freeway operation under control and the performance of the control system in general. The results of these studies are listed below.

1. Motorist observance of the ramp signals has been quite good in spite of the fact that no special effort has been made to apprehend violators. Field observations reveal a violation rate of less than 5% at each of the three on-ramps.

2. Vehicle queuing at the ramp noses has been virtually eliminated. Occasional queues result due to overcautious ramp drivers but these are short in length (2 to 4 vehicles) and are quickly dissipated.

3. Flow through the control section is more uniform than it was in 1970 with less platooning and, hence, less unused space between platoons.

4. Freeway stoppages have been reduced in number. When a stoppage does occur, however, it is likely to affect more vehicles in 1972 than in 1970. This is thought to be due to the increase in the volume being carried on the freeway and the decrease in platoon type movement. The region in which stoppages are most frequent is the subsection between the Cooke Road and East North Broadway overpasses.

5. Although queues waiting at the metered on-ramps completely filled the available ramp storage areas at certain times during the metering period, no serious operational difficulties resulted on the adjacent cross-streets due to ramp overflow.
A significant increase in traffic volume has occurred on Silver Drive but this was expected since Silver Drive was the suggested alternative route for motorists diverted from the East North Broadway and Weber Road entrance ramps. No serious congestion has resulted on Silver Drive due to the increased volumes and average speeds remain basically unchanged. Delay to motorists estimated at approximately 32 vehicle-hours per day was caused by the entrance ramp control. This delay is less than that which can be caused by a minor traffic accident and seems a small price to pay for the increase in both throughput and the stability of traffic flow.

Although the flow on I-71 has become more stable it is still extremely susceptible to flow breakdowns. Once a breakdown is initiated it may affect many vehicles since traffic densities remain high and several minutes may be consumed before the breakdown is completely dissipated. A major breakdown caused by a stalled vehicle or a traffic accident may affect flow for 30 to 45 minutes before stability is restored. During such a disturbance many motorists suffer delay and the risk of accident, especially rear-end collision, is high. The freeway subsection between Cooke Road and East North Broadway seems particularly susceptible to such breakdowns.

It is this enduring vulnerability to traffic breakdowns which is thought to have caused the apparent paradox which prevented the evaluation of traffic flow efficiency using the Combined Congestion Index. Although flow efficiency has been generally increased by ramp control, a sufficient number of breakdowns occurring on a single day can result in congestion greater than that
CHAPTER 2

STUDIES OF NORTHBOUND INTERSTATE 71

2.1 ORIGINAL STUDY: 1970 - 1972

The study of traffic operations on northbound Interstate 71 was conducted using procedures and techniques identical to those described in Chapter 1 for the southbound study. The northbound study was begun on August 17, 1970, and continued through August 31, 1972.

2.1.1 DATA COLLECTION

Data collection was performed using both aerial and ground-based techniques. General and intensive survey flights were made over the study corridor to identify regions of heavy congestion and to determine the causative factors leading to the congestion. Supplementary data was supplied through travel time runs and volume counts at selected locations. No density trap study was made due to the problems encountered during the southbound study and previously described in Chapter 1. Density data was obtained from the intensive survey flights.

General Survey Flights

Two general survey flights were conducted along the study section. Each flight began just south of the Fort Hayes Interchange and continued north-
Figure II-33 Region of Concentrated Data Collection
(Northbound Evening Peak)
for the Main Street-Seventeenth Avenue runs. Observers in the test vehicles were instructed to be alert for geometric or traffic conditions which could potentially lead to traffic congestion. Included in this category were such things as long upgrades, short weaving sections, sight distance limitations and heavy ramp volumes. The location, time of day and duration of all observed traffic stoppages were also recorded.

Volume Counts

Volume counts were conducted at several different locations within the section identified from the general survey flights as being congested during the evening peak period. The counting locations are given below.

1. Fifth Avenue overpass - 4 lanes of traffic
2. Eleventh Avenue overpass - 3 lanes of traffic
3. Fifth Avenue off-ramp
4. Fifth Avenue on-ramp
5. Eleventh Avenue off-ramp
6. Eleventh Avenue on-ramp
7. Seventeenth Avenue off-ramp
8. Seventeenth Avenue on-ramp
9. Hudson Street off-ramp
10. Hudson Street on-ramp
11. Weber Road off-ramp

Two sets of counts were made. The first set was conducted during May, 1971,
<table>
<thead>
<tr>
<th>Flight Identification</th>
<th>Date</th>
<th>Number of Circuits</th>
<th>Time of Coverage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flight #17</td>
<td>Fri. Sept. 10, 1971</td>
<td>10</td>
<td>5:02-6:00 PM</td>
</tr>
<tr>
<td>Flight #25</td>
<td>Wed. May 24, 1972</td>
<td>9</td>
<td>5:05-5:45 PM</td>
</tr>
<tr>
<td>Flight #26</td>
<td>Thu. June 8, 1972</td>
<td>11</td>
<td>4:47-5:38 PM</td>
</tr>
<tr>
<td>Flight #29</td>
<td>Mon. July 24, 1972</td>
<td>8</td>
<td>4:43-5:24 PM</td>
</tr>
</tbody>
</table>

All flights were made from an altitude of approximately 2500 feet above ground and photography was taken at 1 and 3 second intervals. Flights # 17, 26, 27 and 29 were made on days when the weather was good and no traffic incidents occurred to disturb the flow of traffic. The data of these flights is thought to represent normal operating conditions during the evening peak hour on northbound I-71. Flight #25 was made on a day when a major traffic disturbance was caused by a vehicle which stalled in the center traffic lane under the Eleventh Avenue overpass. The data from this flight has been included to demonstrate the effect of such an incident on peak period traffic flow.

2.1.2 DATA REDUCTION AND ANALYSIS

The data provided by the general survey flights, the ground-based studies and the intensive survey flights was reduced and summarized as it became available. The reduced data was then analyzed to determine the nature of the specific operational problems occurring during the evening peak period and to identify the probable causative factors from which these problems arise. The results of this analysis are given below.
southward until it reached the point where I-71 passes over the railroad yard just south of First Avenue. It remained in this position until the survey flights were terminated at 5:20 PM. Smaller traffic disturbances were noted in the immediate vicinity of the on-ramps at Eleventh Avenue, Seventeenth Avenue, and Hudson Street. These disturbances did not begin until near the end of the survey period at about 5:10 PM.

Traffic stoppages were observed in all lanes throughout the Fort Hayes-Hudson Street region. Stoppages were especially prevalent, however, in the section between the Fifth Avenue off-ramp and the Eleventh Avenue on-ramp. In this section a stoppage frequently involved as many as 25 to 30 cars and could last for well over a minute. Queues numbering as many as 5 vehicles were also frequent on the Eleventh Avenue on-ramp itself. Very high weaving volumes were noted in the vicinity of the Eleventh Avenue off-ramp and between the Eleventh Avenue and Seventeenth Avenue overpasses.

As a result of these observations it was decided to concentrate the ground-based data collection efforts in the region between the Fort Hayes Interchange and the Hudson Street on-ramp. A data collection interval extending from 4:30 to 6:00 PM was chosen as being sufficient to cover the entire peak period.

Ground-Based Studies

Travel Time Runs

Figures II-34 through II-38 present velocity-distance profiles
Figure II-37  Velocity-Distance Profiles for Data of Wednesday, June 21, 1972
constructed using the data from the five sets of travel time runs. Each figure depicts the variation of average speed with both time of day and location along the study section during the evening peak period. Figure II-39 summarizes the results for all five days of data collection. This figure consists of five velocity-distance profiles, one profile for each day of data collection. Each profile represents the average speed prevailing at various locations along the study section during the entire peak period.

Observation of Figures II-34 through II-38 reveals that speeds vary widely throughout the study section during the peak period. On each day surveyed, however, the minimum speeds are consistently found in the region between the Fort Hayes Interchange and the Eleventh Avenue overpass. Speeds in this region average between 20 mph and 25 mph (see Figure II-39) during the entire peak period and can drop as low as 10 mph during an individual run. A second region of low speed is located in the vicinity of the on-ramp at Hudson Street. Although speeds in this region are not as low as those found in the Fort Hayes-Eleventh Avenue region, they are sufficiently low (20-30 mph) to cause substantial delay to northbound motorists especially during the later part of the peak period. This second region also poses a dangerous accident hazard since speeds upstream of the Hudson Street overpass are significantly higher in many cases than speeds at the overpass itself. Thus motorists approaching the overpass must decelerate rapidly to avoid rear end collisions. This problem is complicated by the sweeping horizontal curve located just south of Hudson Street which hides the low speed traffic from view until the
northbound motorist is nearly upon it. Small disturbances were also noted in
the vicinity of the on-ramps at Eleventh and Seventeenth Avenues. These
disturbances were generally quickly dissipated.

The primary cause of the low speeds in the Fort Hayes-Eleventh Avenue
region is thought to be the drop lane exit at Eleventh Avenue where northbound
I-71 narrows from four lanes to three. Observers in the test vehicles noted
extremely high weaving volumes in this area as well as a high incidence of
motorists illegally traveling along the right shoulder of the freeway to avoid
exiting at Eleventh Avenue. Other motorists trapped in the exit lane would
come to a complete stop at the ramp nose (thus blocking the exit lane) and
attempt to force their way into the adjacent through traffic lane. Many stop-
pages in this through lane came as a direct result of such maneuvers by exit
lane drivers. Once a stoppage had occurred, it would often precipitate related
disturbances in the other two through lanes as stopped traffic attempted to
change lanes to escape from the forming queue. Due to the extremely high de-
mand upstream of Eleventh Avenue, these disturbances would then propagate
southward resulting in the region of low speed, high density traffic observed
during the general survey flights.

The low speeds in the Hudson Street area are caused by a combination
of traffic and geometric conditions. The Hudson Street on-ramp serves a high
volume of traffic during the evening peak period. This traffic must enter a
freeway stream which is both heavily loaded and is initially moving at a rela-
tively high speed (40 to 50 mph). In addition, the next downstream exit ramp
Figure II-40 Pattern of Travel Time with Time of Day for Travel Time Runs of August 26, 1970 and June 13, June 21, and July 10, 1972
Figure II-41 Summary of Day to Day Volume Variation at Various Counting Stations for May 1971 Data Collection
(Study Interval - 4:30 to 6:00 PM)

255
peak hour volumes for each counting station are summarized in Tables II-11 (May, 1971) and II-12 (July-August, 1972). Although there was some variation among counting stations, the peak hour was generally found to extend from 4:30 to 5:30 PM.

Comparison of the data of Figures II-41 and II-42 with that of Tables II-11 and II-12 reveals an interesting result. At each counting station the 90 minute volumes from the 1972 counts exceed the corresponding 1971 volumes by 2% to 5%. This volume increase is in line with the expected traffic growth over a one year period. The same increase is seen in the 1972 peak hour volumes for the nine ramp counting stations as compared to the 1971 volumes for the same locations. The peak hour volumes for the two overpass counting stations, however, exhibit a different trend. At these stations the 1972 volumes are approximately 2% lower than the corresponding 1971 volumes. Since overall demand has increased as shown by the 90 minute counts, this peak hour decrease must be attributed to increasing congestion on the freeway mainline. As demand continues to increase this congestion in the Fifth Avenue-Eleventh Avenue region can also be expected to increase resulting in a further reduction in volume and throughput.

Table II-13 shows the lane distribution of traffic at the two overpass counting stations. The data in this table is from the 1972 counts. The average lane distribution values from the 1971 counts are nearly identical and, therefore, no need was seen to present them.
Table II-12  Peak Hour Volumes at Counting Stations
for Various Days of the Week
(July-August, 1972)

<table>
<thead>
<tr>
<th>Counting Station</th>
<th>Peak Hour Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mon</td>
</tr>
<tr>
<td>5th Ave. overpass</td>
<td>5393</td>
</tr>
<tr>
<td>11th Ave. overpass</td>
<td>5300</td>
</tr>
<tr>
<td>5th Ave. off-ramp</td>
<td>381</td>
</tr>
<tr>
<td>5th Ave. on-ramp</td>
<td>390</td>
</tr>
<tr>
<td>11th Ave. off-ramp</td>
<td>538</td>
</tr>
<tr>
<td>11th Ave. on-ramp</td>
<td>623</td>
</tr>
<tr>
<td>17th Ave. off-ramp</td>
<td>369</td>
</tr>
<tr>
<td>17th Ave. on-ramp</td>
<td>393</td>
</tr>
<tr>
<td>Hudson St. off-ramp</td>
<td>375</td>
</tr>
<tr>
<td>Hudson St. on-ramp</td>
<td>728</td>
</tr>
<tr>
<td>Weber Rd. off-ramp</td>
<td>386</td>
</tr>
</tbody>
</table>
lanes remains fairly constant at both counting stations. This total composition value (2.5% - 3.5% trucks) represents a relatively low percentage of trucks for an urban freeway such as the section of Interstate 71 being studied and indicates that commercial vehicles do not represent a serious problem during the evening peak period. This conclusion is supported by observations made during the general survey flights and the travel time runs in which no operational problems directly attributable to commercial traffic were noted.

Table II-14 Traffic Composition by Lane

<table>
<thead>
<tr>
<th>Day</th>
<th>Location</th>
<th>5th Ave. - % trucks</th>
<th>11th Ave. - % trucks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1 2 3 4 Avg.</td>
<td>1 2 3 Avg.</td>
</tr>
<tr>
<td>Mon.</td>
<td></td>
<td>0.28 3.04 3.62 7.41 3.13</td>
<td>0.40 3.50 4.65 2.76</td>
</tr>
<tr>
<td>Tue.</td>
<td></td>
<td>0.16 3.36 3.66 6.02 2.80</td>
<td>0.28 4.02 4.10 2.73</td>
</tr>
<tr>
<td>Wed.</td>
<td></td>
<td>0.22 3.08 4.74 4.13 2.83</td>
<td>0.19 3.70 5.10 2.90</td>
</tr>
<tr>
<td>Thu.</td>
<td></td>
<td>0.48 3.89 4.20 5.86 3.30</td>
<td>0.32 3.92 4.48 2.81</td>
</tr>
<tr>
<td>Fri.</td>
<td></td>
<td>0.31 2.96 4.44 5.91 3.10</td>
<td>0.24 3.27 5.03 2.71</td>
</tr>
<tr>
<td>5 Day</td>
<td></td>
<td>Avg. 0.29 3.27 4.13 5.87 3.03</td>
<td>Avg. 0.29 3.68 4.67 2.78</td>
</tr>
</tbody>
</table>

* Lane 1 is the median lane

Merge Area Capacity Counts

In addition to the volume studies described above, a supplementary set of counts was made at the merge areas of the on-ramps at Fifth, Eleventh, and Seventeenth Avenues and Hudson Street in order to determine the controlling freeway capacity value in the vicinity of these ramps. The counts at Eleventh Avenue, Seventeenth Avenue, and Hudson Street were made at a point
It is significant to note that the lowest capacity is found in the section between Fifth and Eleventh Avenues. This fact agrees with the observation from the general survey flights and the travel time runs that the greatest congestion is found in this section.

**Intensive Survey Flights**

Five intensive survey flights were made over the region between the Fort Hayes Interchange and the Seventeenth Avenue overpass. For the purposes of analyzing the traffic density pattern in this region using the aerial data, the section was divided into five subsections. These subsections are described below.

<table>
<thead>
<tr>
<th>Subsection Number</th>
<th>Description and Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fort Hayes Interchange – South End of Railroad Yard (1124 ft.) Five lanes</td>
</tr>
<tr>
<td>2</td>
<td>South End of Railroad Yard – Fifth Avenue Overpass (3004 ft.) Four lanes</td>
</tr>
<tr>
<td>3</td>
<td>Fifth Avenue Overpass – Railroad Tunnel (1752 ft.) Four lanes</td>
</tr>
<tr>
<td>4</td>
<td>Railroad Tunnel – Eleventh Avenue Overpass (692 ft.) Three lanes</td>
</tr>
<tr>
<td>5</td>
<td>Eleventh Avenue Overpass – Seventeenth Avenue Overpass (2512 ft.) Three lanes</td>
</tr>
</tbody>
</table>

The variation of the density pattern in each subsection with time of day for each of the five days surveyed is shown in Figures II-43 through II-47. It should be remembered when studying these patterns that the data of Wednesday,
Figure II-44  Traffic Density Versus Time of Day for South End of Railroad Yard - Fifth Avenue Overpass Subsection
May 24, 1972, reflects the effect of a stalled vehicle under the Eleventh Avenue overpass. This vehicle blocked traffic in lane 2 for a period of about 8 minutes. No major traffic incidents occurred on the other four days surveyed.

In order to facilitate comparison of the density patterns among the various subsections, the total density curves were integrated to obtain values of the Congestion Index. Since freeway width varies between subsections with three, four and five lane sections represented, the Congestion Index values for each subsection were then divided by the number of lanes to obtain a per lane measure of congestion. These values are summarized in Table II-16.

Table II-16 Summary of Per Lane Congestion Indices

<table>
<thead>
<tr>
<th>Date</th>
<th>Section 1</th>
<th>Section 2</th>
<th>Section 3</th>
<th>Section 4</th>
<th>Section 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fri. Sept. 10, 1971</td>
<td>28.1</td>
<td>36.2</td>
<td>46.0</td>
<td>61.0</td>
<td>39.9</td>
</tr>
<tr>
<td>Thu. June 8, 1972</td>
<td>41.9</td>
<td>54.8</td>
<td>58.1</td>
<td>54.5</td>
<td>48.4</td>
</tr>
<tr>
<td>Fri. July 7, 1972</td>
<td>50.9</td>
<td>54.2</td>
<td>58.4</td>
<td>51.7</td>
<td>44.3</td>
</tr>
<tr>
<td>Mon. July 24, 1972</td>
<td>47.4</td>
<td>47.8</td>
<td>48.1</td>
<td>52.1</td>
<td>43.9</td>
</tr>
<tr>
<td>4 Day Average</td>
<td>42.1</td>
<td>48.3</td>
<td>52.7</td>
<td>54.8</td>
<td>44.1</td>
</tr>
<tr>
<td>Wed. May 24, 1972</td>
<td>64.3</td>
<td>70.3</td>
<td>62.9</td>
<td>65.0</td>
<td>52.0</td>
</tr>
</tbody>
</table>

*Values are based on 48 minutes of data collected during the peak hour.

Table II-16 is organized in the following manner. The first four rows contain the per lane Congestion Index values for the four days during which "normal" traffic conditions prevailed. The fifth row presents average Index values.
gested during the evening peak period. Within this section the worst congestion is found between the Fifth and Eleventh Avenue overpasses. This region is characterized by low speeds (20 - 25 mph) and high densities (60 - 120 vehicles per mile per lane) during most of the peak period. Traffic stoppages are frequent in all traffic lanes with some stoppages involving as many as 30 vehicles. These stoppages then propagate upstream causing congestion as far south as the Fort Hayes Interchange. Weaving volumes are also quite high in the Fifth - Eleventh section with the majority of the weaves taking place in the immediate vicinity of the Eleventh Avenue exit ramp. This drop lane exit which results in a reduction in freeway width from four to three lanes is thought to be the primary cause of evening peak period congestion on northbound I-71.

A second region of low speed, high density traffic is located just upstream of the Hudson Street on-ramp. The problem here occurs late in the peak period and is caused by a heavy surge of on-ramp traffic on the Hudson Street entrance ramp attempting to enter a freeway traffic stream which is both heavily loaded and moving at a relatively high speed (40 - 50 mph). The result is a series of disturbances which eventually cause stoppages in all three freeway traffic lanes. The problem is further complicated by a high number of weaves occurring in this same region as freeway drivers wishing to exit at the Weber Road exit ramp attempt to get into the shoulder lane in conflict with the entering Hudson Street ramp traffic. The Weber Road exit ramp is located only 1500 feet downstream from the Hudson Street on-ramp.
such as those described in Section 1.2.1 of the preceding chapter. A preliminary control strategy based on these techniques is outlined below.

Definition of Control Area

Four northbound entrance ramps are located in the congested section described above. Proceeding from south to north these ramps are located at Fifth Avenue, Eleventh Avenue, Seventeenth Avenue and Hudson Street. During the data collection phase of the study, traffic disturbances were frequently noted in the merge areas of each of these ramps. These disturbances contributed to the overall instability of the traffic stream and many times led to serious traffic stoppages. It seems logical, therefore, that some type of ramp control applied at each of these ramps would result in substantial improvement in evening peak period traffic flow. Fixed-time metering offers a relatively simple means of exerting such control.

Determination of the Metering Policy

The metering rate and the duration of the metering period at each on-ramp location were determined using a procedure similar to that described in Section 1.2.2 of Chapter 1. The basic premise of the procedure is that the sum of the on-ramp volume plus the mainline volume during any interval of time should not exceed the desirable capacity of the merge area of the corresponding ramp. Since the merge area capacities have previously been determined, the volumes which can be allowed to enter at each on-ramp each time interval can be calculated using these capacity values and the historical volume
<table>
<thead>
<tr>
<th>Time of Day, PM</th>
<th>4:30</th>
<th>5:00</th>
<th>5:30</th>
<th>6:00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hudson St.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Queue Length</td>
<td>102</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storage</td>
<td>24</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diverted</td>
<td>78</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17th Ave.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Queue Length</td>
<td>10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storage</td>
<td>34</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diverted</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11th Ave.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Queue Length</td>
<td>118</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storage</td>
<td>26</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diverted</td>
<td>92</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5th Ave.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max Queue Length</td>
<td>118</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storage</td>
<td>27</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diverted</td>
<td>91</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure II-48 Variation of Ramp Queue Length with Time of Day for Metered Northbound Entrance Ramps
this control strategy would significantly improve peak period traffic operations on the freeway. Implementation of the plan, however, would result in 261 vehicles being diverted from I-71 to the surface street network. No suitable alternative route is presently available to serve this traffic. The construction of a new north-south arterial is proposed in this report and would provide the required diversion route when completed. No action is expected on this proposal, however, for several years. If the control plan is to be used before this time, therefore, the diverted drivers will have to be depended upon to find their own way home. The public outcry which will result from this circumstance is duly noted and must be weighed against any improvement in freeway traffic movement by those deciding whether or not immediate implementation of northbound freeway control is advisable.
REFERENCES


2. Transportation Research Center, The Ohio State University, "Investigation of Traffic Dynamics by Aerial Photogrammetry Techniques," Interim Report EES 278-3, Columbus, Ohio, 1970.


6. Transportation Research Center, The Ohio State University, "Investigation of Traffic Dynamics by Aerial Photogrammetry Techniques," Interim Report EES 278-1, Columbus, Ohio 1967.


