SOME TRAFFIC FLOW RELATIONSHIPS ON TWO-LANE URBAN STREETS

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by

R.C. SONNTAG

Joint Highway Research Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Final Report

SOME TRAFFIC FLOW RELATIONSHIPS ON TWO-LANE URBAN STREETS

TO:	K. B. Woods, Director Joint Highway Research Project	January 30, 1963
FROM:	H. L. Michael, Associate Director Joint Highway Research Project	File: 8-4-24 Project: C-36-17K

Attached is a final report entitled "Some Traffic Flow Relationships on Two-Lane Urban Streets" which has been authored by Mr. Ronald C. Sonntag, graduate assistant on our staff. The research was also used by Mr. Sonntag as the basis for his M.S.C.E. thesis requirement and was performed under the supervision of Professor H. L. Michael.

The report includes data taken at six sites in two cities on two-lane urban arterial streets. The analysis indicates that there is a slight decrease in mean speed with an increase in traffic volume and average density and that these relationships are approximately linear for free-flowing traffic conditions. An analysis of the observed distribution of headways was also made and is compared with two proposed theoretical distributions.

The report is presented for the record.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Secretary

HLM:kmc

Attachment

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by

Ronald C. Sonntag
Graduate Assistant

Joint Highway Research Project
File No: 8-4-24
Project No: C-36-17K

Purdue University
Lafayette, Indiana

January 30, 1963

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The author also wishes to express his appreciation to the Joint Highway Research Project whose financial assistance made his attendance at Purdue University possible.

Special appreciation is also due my wife whose encouragement and patience made this work possible.

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ABSTRACT

Sonntag, Ronald C., MSCE Purdue University, January 1963. Some Traffic Flow Relationships on Two-Lane Urban Streets. Major Professor: Harold L. Michael.

The purpose of this study was to investigate some of the interrelationships of traffic flow variables on two-way two-lane urban arterial streets. The major variables considered were the relationships of speed, volume and density, and the distribution of observed headways. The study was performed on several types of urban arterial streets relative to the parking conditions.

Six sites, selected in Lafayette and Indianapolis, were as identical as possible regarding roadway geometry and adjacent land use. The data were collected at each of the six sites by means of a continuously operating, traffic-actuated recording device. The traffic flow relationships investigated were analyzed by means of a multiple linear regression analysis for each individual site, each type of site and for each group of sites located within a particular city where the data were collected.

The analysis indicated a slight decrease in mean speed with an increase in volume and average density and also an increase in average density with an increase in volume. These relationships appear to be linear for the range of data obtained for free-flowing traffic conditions.

When other variables-commercial vehicles, directional distribution, and turning movements at nearby intersections - were considered with the speed-volume relationship, volume was the only significant variable in

explaining the variability in mean speed. In general, the proportion of explained variability was low. The relationships were not the same among site types located in different cities or among different site types located within the same city; however, the relationship was consistent among similar site types within the same city.

A study of the observed headway distributions indicated that the volume of opposing traffic had very little influence upon the distribution of headways in a traffic stream, and then only when opposing volumes were very low. The observed distribution of headways for all volume levels indicated that more vehicles travel at shorter headways on rural highways than on the urban streets studied. The observed headway distributions were found to fit well in two theoretical distributions proposed in the literature.

INTRODUCTION

The planning, design and operation of the future and existing highway plant are the primary endeavors of the state highway departments and their highway and traffic engineers. The immediate effect of their efforts is felt most strongly by the public in the area of traffic control, which concerns itself with the safe and efficient movement of traffic on the existing highway facilities.

The control of traffic, however, is hampered by the inability to predict the precise movements of the traffic stream. This is largely due to the fact that the motor vehicle, the basic element of the traffic stream, is a mechanical tool subjected to the desires, motivations, and inhibitions of its human driver. This human factor has been largely responsible for the difficulties in attempts to express traffic flow in precise and reliable mathematical models.

Many studies have been made in the area of the quality and theory of traffic flow. The quality of the flow is concerned with expressing the efficiency of flow with regard to the degree of congestion, driver satisfaction or other similar parameters, while the theory of traffic flow concerns itself with the relationships of speed, volume, density and headway.

A measure of the quality of traffic flow may be used to indicate the efficiency of operation and possible trouble spots, but a better understanding of the theory of traffic flow would permit more satisfactory improvements or solutions of traffic problems. A thorough understanding of traffic flow has become increasingly essential as the use of traffic actuated control devices and centrally located monitoring systems have developed. The more recent traffic flow studies have concerned themselves with freeway-type facilities but the majority of road and street mileage is of urban arterial character with little or no control of access. Regardless of the extent of freeway development in an urban area, some portion of all vehicle trips will occur on this type of facility thereby necessitating some type of control measure and knowledge of the traffic flow characteristics of this type of roadway.

In order to proceed further in a discussion of the theory of traffic flow a clear understanding of the variables and their relationship is necessary. The basic equation relating the fundamental variables is Volume = Density x Average Speed. This equation has been used to calculate one of the variables knowing the other two because the equation is dimensionally correct, and J. G. Wardrop (32)* has verified the relationship mathematically when the variables are used in the proper form. The variables of volume and density usually do not present a problem of understanding or usage as they are well defined as follows: (a) volume is the number of vehicles passing a fixed point on a roadway in a given period of time, and (b) density is the number of vehicles on a fixed length of roadway at a given instant.

Confusion may result when using the term average speed since it may be either time-mean speed or space-mean speed, depending upon the method of collecting field data. Time-mean speed is the average of the spot speeds, or instantaneous speed, of vehicles at a specific location while space-mean

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* Numbers in parentheses refer to entries in the List of References.

speed is the length of a given route divided by the average time of travel over the route. Wardrop has shown that the appropriate speed to be used in the basic equation is the space-mean speed and that it is equal to the harmonic mean of the spot speeds (32). The term average speed, however, most often refers to time-mean speed and it will be used with this meaning in this report unless otherwise indicated.

Two other terms to be used frequently and requiring definition are headway and traffic stream. Headway is the time interval, at a fixed point on the roadway between the passage of successive vehicles traveling in the same direction, measured from head to head of the vehicles (19). The distance between successive vehicles is also often referred to as headway but this measure is more correctly termed the gap interval. The distribution of headways may be expressed as a functional distribution and can be used to predict arrivals of vehicles at a point. The traffic stream is made up of lanes of traffic flow which are not physically separated and may be two-way or one-way but the term as used in this text will be for two-way traffic (19).

PREVIOUS INVESTIGATIONS

The interrelationships of the basic traffic flow variables of speed, volume, density and headway have been investigated by various authors on several types of roadway. These investigations have ranged from a purely theoretical analysis with a minimum of actual field data (6, 8, 11, 17), to studies which have included prodigious amounts of data from many locations (23, 24, 25, 33). In general, the speed-volume relationship has received the most intense study of the four fundamental variables, not necessarily because this relationship is considered to be the most important but because the variables are easy to measure directly. In fact, it has been suggested that the speed-volume relationship is not a good indicator of traffic flow because it is relatively insensitive to changes in traffic conditions (21).

The relationship of density to the other variables of traffic flow has received some study but more or less indirectly (9, 21, 28, 30). The difficulty of actually measuring vehicle density over a section of roadway, particularly in urban areas, has led to the practice of calculating density from speed and volume and subsequently investigating its relationship to these variables.

Vehicle headway characteristics have been investigated as a part of some traffic flow studies but often have been analyzed for two entirely different purposes. Some investigators have studied the variation and relationship of minimum headways and speeds, while others have analyzed the distribution of headways in a traffic stream.

In 1934, B. D. Greenshields reported on the speed, volume, and density relationships for two-lane rural roads in Ohio (9). He reported finding that as volume increased beyond 400-600 vehicles per hour, the average speed of all vehicles decreased. He also calculated density from the speed-volume variables. Then assuming a linear relationship between speed and density, he derived a curvilinear expression relating speed and volume. The shape of the true speed-volume curve, whether it is linear or curvilinear and whether it is significant or not, has since been the subject of much research and controversy.

A comprehensive study of traffic flow was performed in the late 1930's and early 1940's and led directly to the publication of the Highway Capacity Manual (3). This investigation indicated that there is a linear relationship between speed and volume on rural roads and urban streets, provided the "critical density" is not exceeded and that other influencing factors remain constant (3, 26). When considering the speed - volume relationship in one direction of a two-way highway, some other factor or factors appeared to influence the relationship. O. K. Normann determined this factor to be the volume of opposing traffic (24).

A recent study was also performed in Chicago to determine the relation between speed and volume on urban streets (13). The moving car method was used to estimate speeds and volumes on several study sections. A linear multiple regression analysis was performed and equations were developed for various types of roadways. The speed-volume relationship was the prime consideration but other variables describing traffic and roadway conditions were included, "to better isolate the relation between speed and volume by holding these other variables constant, and to use as speed predictors should that speed-volume relation prove too week" (13). Some conclusions obtained from this study were: (a) the simple correlation between speed and volume on urban or rural roads was low although the multiple coefficient of correlation ranged from 0.49 to 0.84 for various types of roadways; and (b) the method of multiple regression and correlation analysis lends itself readily to this type of study.

Several other large scale traffic flow studies have been made but these have emphasized freeway type facilities (5, 18, 21, 33). The results of these studies have shown a trend of decreasing speed with increasing volume, but T. W. Forbes concluded that "the desires of the drivers in each of the three lanes as to speed of travel and the anticipation of hazards had more influence on speeds and on maximum capacity than did the physical relationships between mean speed and volume of traffic" (5).

As noted previously the density factor as a variable in traffic flow has been studied but usually as a calculated quantity. In reviewing the available literature, this researcher was unable to find a study specifically pertaining to a two-lane, two-way, urban arterial street. Several studies performed on multilane facilities have indicated a curvilinear relationship of speed-density and volume-density when periods of high density flow were encountered (21, 28, 30). R. T. Underwood (30) suggested the possibility of an exponential speed-density relationship and subsequently derived a speed-volume curve. He states that, "For volumes and densities up to, and well beyond, the practical capacity, the speed-volume and speed-density curves closely approximate straight lines and, for all practical purposes, could be taken as such" (30).

Several authors have attempted to theoretically describe the functional relationships between speed, volume, and density (6, 8, 11, 17). H. Greenberg, considering traffic to be analogous to fluid flow, and D. C. Gazis, et al., using the equations of motion, independently developed a basic formula describing traffic flow (6, 8). M. J. Lighthill and G. B. Whitham developed the same general shape of volume-density curve as Greenberg by also using a fluid flow analogy (17). Some field studies indicate that these theories describe fairly accurately traffic flow at special locations (4, 28, 30). These study locations, on a parkway and in a tunnel, conformed to the theory requirements of one-way single lane flow and no intermediate access points.

The headway variable of a traffic stream is a function of vehicle and driver characteristics and is not necessarily an indicator of traffic flow, although minimum acceptable headways to a large degree will determine the maximum capacity of a highway (24). Several field studies have been performed to investigate minimum headway and speed relationships (5, 24, 25) and the distribution of headways in a traffic stream (21, 24, 25). O. K. Normann concluded that the frequency distribution of headways is practically the same for one highway as another regardless of the speeds of the vehicles and that the distribution changes fairly uniformly with changes in volume (24). He also stated in 1959, that vehicle headways have remained essentially unchanged during the past two decades despite improvements in vehicle performance (27).

Due to the similarity of headway distributions on different roadways, it is desirable to express this characteristic in a functional form for the prediction of vehicle distributions and arrival times. The Poisson distribution was initially used (1, 15), but it has been shown to deviate from observed data at high volumes (10). A. Schul proposed a composite exponential distribution which recognized the existance of a minimum headway and the platooning phenomenon in traffic flow (29). A California Study fitted the composite exponential distribution to data obtained on two-lane urban arterial highways in that state (14). R. M. Lewis has also attempted to account for the platooning effect of traffic and the existence of minimum headways in his proposal of a modified binomial distribution to describe vehicle arrivals (16). These distributions have shown good agreement with field data.

PURPOSE AND SCOPE

The purpose of this study was to investigate some of the interrelationships of speed, volume, density and headway on two-lane urban arterial streets. The study was divided into two parts. The first of these consisted of an investigation of the speed, volume, and density variables, while the second was concerned with the headway distribution of vehicles in the traffic stream. In the first part of this study, the object was to determine some functional relationship of the variables and to see if this relationship was consistent among different locations and types of roadways. The second part of the study concerned an investigation of the headway distribution in a stream of traffic relative to mathematical equations which have been proposed to describe this distribution.

The scope of this study included two-lane urban arterial streets of several types, located in Lafayette and Indianapolis, Indiana. The particular locations selected, discussed in detail in the following section, were roadways with two-way traffic flow.

STUDY LOCATIONS

The selection of suitable study sites presented a problem due to several restrictions imposed on the final selection of these sites. The object of imposing these restrictions was to eliminate variables which were not of immediate concern in this study. The criteria used in the final selection were intended to hold roadside development, speed limits, traffic control and road alignment as consistent as possible from site to site so that the influence of these variables could be assumed to be constant for all sites. It was also desired that the data obtained should be representative of a type of roadway and that the character of traffic should be that which normally uses such a roadway. In addition, the sites selected had to be within a reasonable distance of Lafayette in order to facilitate the collection of data. The following criteria, therefore, were used in determining the acceptability of a location:

1. There should be approximately one-half mile to traffic control devices from the location, with no obstructions to through traffic such as railroad crossings. This restriction was imposed so that there would be no adverse effect upon the movement of traffic other than that imposed by prevailing traffic and roadway conditions.
2. The roadway should be a level tangent section so that passing sight restrictions would not effect a driver's opportunity to maneuver.

3. The one-mile section in which the location existed should have development as consistent as possible along its entire length with no high-volume traffic generators located on it. Commercial development at the ends of the section was not considered objectionable.

4. The section should have a period of high volume flow so that a wide range of traffic volumes to represent high density and low density flow could be obtained.

Considerable difficulty was encountered in locating sections which fulfilled these requirements. Each criteria was adhered to individually by a number of locations, but roadway sections which fulfilled all the requirements were more difficult to find. The particular restriction most difficult to comply with was the one mile length between traffic control devices as high-volume urban arterials generally have traffic signals located at least at one-half mile intervals.

Study sites were also selected so that three types of two-lane roadway sections would be represented. Each of the roadway types was represented by two sites with similar roadway geometry and development. The three types of roadway selected, with all being two-lane, two-way urban arterials, were sections with (I) parking permitted on one side only, represented by sites A and B; (II) parking permitted on both sides, represented by sites C and D; and (III) a two-lane roadway with no parking lanes provided, represented by sites E and F. The study sites are pictured in Figures 1-3. A brief description of each site is given in Table 1.

TABLE 1

SUMMARY OF STUDY LOCATIONS

Location	City	Street	Direction	Parking
Site Type I				
A	Indianapolis	30th Street	East-West	South side only
B	Lafayette	South Street	East-West	South side only
Site Type II				
C	Lafayette	9th Street	North-South	Both sides
D	Indianapolis	Washington Blvd.	North-South	Both sides
Site Type III				
E	Indianapolis	21st Street	East-West	No Parking Lanes
F	Indianapolis	Arlington Street	North-South	No Parking Lanes

Site Type I - Parking on One Side Only

Site A

The section of 30th Street in Indianapolis used as a study site is located between Keystone Avenue and Sherman Drive. At this point the east-west arterial passes through an intermediate area approximately three and one-half miles northeast of the central business district. The development along the arterial is primarily residential with a park fronting for a few blocks on the north side about midway between Keystone and Sherman. The roadway is 30 feet from curb to curb with parking allowed on the south

SITE A - 30 TH. STREET, INDIANAPOLIS

SITE B - SOUTH STREET, LAFAYETTE

FIGURE I SITE TYPE I - PARKING ON ONE SIDE ONLY side only, leaving two 11-foot traffic lanes. The vertical alignment is generally flat with a few gently rolling grades. The total length of this section is approximately one mile between traffic signals located at Keystone and Sherman. Data were collected at a point 0.53 miles east of Keystone Avenue.

Site B

South Street in Lafayette is the major east-west route between the central business district and the U. S. 52 bypass. The section investigated is 0.93 miles in length and is located between traffic signals at Main Street and Earl Avenue. It is approximately 1.5 miles from the central business district. Development along this level section is primarily residential with a few small commercial establishments and professional offices scattered along its length. A large hospital is situated at the midpoint of the section, so the study site was located between the signals at Earl Avenue and the hospital. The point of data collection was located 0.33 miles west of Earl Avenue. The roadway is 32-feet wide at this point with parking allowed on the south side, thus allowing two 12-foot lanes for traffic movement.

<u>Site Type II - Parking on Both Sides</u>

Site C

The section of 9th Street in Lafayette studied is located between Kossuth Street and Teal Road approximately one mile southeast of the central business district. The development along 9th Street at this point is purely residential with a golf course located a few blocks south of the study site. The vertical alignment of this 0.95-mile section is generally

SITE C – 9 TH. STREET, LAFAYETTE

SITE D – WASHINGTON BLVD., INDIANAPOLIS

FIGURE 2. SITE TYPE II – PARKING ON BOTH SIDES

level with a small valley located approximately two-thirds of a mile south of Kossuth Street. The roadway is 40 feet wide from curb to curb. Parking is allowed on both sides of the street leaving room for two 12-foot traffic lanes. The intersection of 9th Street and Teal Road is signalized while that at Kossuth is a four-way stop. The data collection site was located midway between Kossuth and the crest of the downgrade and is approximately 0.27 miles south of Kossuth.

Site D

The study site on Washington Boulevard in Indianapolis was located between signalized intersections at 38th Street and 46th Street approximately 4.3 miles north of the central business district. At this point Washington Boulevard passes through a level intermediate area with purely residential development. The roadway is 40 feet wide with parking allowed on both sides, leaving room for two 12-foot traffic lanes. The data collection site was 0.50 miles north of 38th Street on this section which is one mile long.

Site Type III - No Parking Lanes Provided

Site E

The section of roadway selected as a study site on 21st Street in Indianapolis is between Arlington Street and Emerson Street. This street is an east-west arterial located approximately five miles from the central business district in a fringe area of the city. The development along the right-of-way is primarily residential with slight commercial development at each end of the section and a small neighborhood shopping center located midway between Arlington and Emerson. The roadway is 18-feet wide with

SITE E— 21 ST. STREET, INDIANAPOLIS

SITE F— ARLINGTON STREET, INDIANAPOLIS

FIGURE 3. SITE TYPE III— NO PARKING LANES PROVIDED

four to six-foot gravel shoulders at each side. There is a no parking on these shoulders. The total length of this section is 1.05 miles between traffic lights located at Arlington and Emerson. The point of data collection was located approximately 0.35 miles west of Arlington Street.

Site F

The final site selected was on Arlington Street in Indianapolis between 38th Street and 46th Street. The study site on this north-south arterial was approximately 6.5 miles northeast of the central business district and was located in a fringe area with suburban type residential development. This roadway is 24-feet wide with six-foot gravel shoulders. Vehicles are occasionally parked well off these shoulders. The length of this section is 1.05 miles between traffic lights at 38th Street and 46th Street. Data were collected at a point approximately 0.45 miles north of 38th Street.

In addition to the above site descriptions a posted speed limit of 30 miles per hour was common to all sites. Due to the similarities and consistency of the roadway features from site to site, each site was directly comparable to any other on the basis of physical characteristics. The primary recognizable physical differences were in the parking conditions.

EQUIPMENT

Three methods are generally available to record field data for traffic studies. These are personal observation, photographic techniques and traffic-actuated recording devices. Collecting the data by using several observers at each location was eliminated due to the fact that this method would probably break down at high volumes and the accuracy of the data would be questionable. Photographic methods were also excluded because of the high costs of data collection and the lack of appropriate photographic equipment in Project laboratories. It was felt that data collected by a traffic-actuated device would produce information quickly and accurately at all volume levels with a minimum of confusion, and that these data would be easily transferable to a usable form in the office.

All speed, volume and headway data were obtained with the use of an Esterline Angus event recorder. The particular model used was a Model A-W heavy duty 20-pen recorder with a Type 2 spring-wound drive and a common return circuit. Electrical power to the pens was provided by a 12-volt storage battery.

The basic principle involved in utilizing this equipment to record speed and headway data is to create an impulse for each vehicle entering and leaving a measured distance and to record these impulses on a chart moving at a constant rate so that a time increment may be obtained. A system of pneumatic roadtubes as shown in Figure 4 was laid out on the roadway to provide an individual impulse for each vehicle entering and leaving the trap in each direction. The air impulse created by a vehicle passing over a roadtube was converted to an electrical impulse to actuate the corresponding pen by means of an air switch. These air switches were purchased from the Streeter-Amet Corporation and are pictured in Figure 5. The free ends of the pneumatic roadtubes were connected to the air switches which were mounted in aluminum boxes. Each switch was connected to the recorder with its own circuit. The impulses received from the roadtubes were recorded on the moving chart by means of continuously operating pens.

After the road tubes were in place and connected to the recorder, the mechanism operated in the following manner. A vehicle traveling in the direction of the upper arrow as indicated in Figure 4 contacted tube A and sent an initial impulse to the recorder. As the vehicle passed through the trap its front wheels contacted tube B and another impulse was recorded. Knowing the chart speed of the recorder, the length between these two impulses was converted to a time increment which in turn was used to calculate speed. In a similar manner the impulses received from two successive vehicles crossing the same point were used to obtain the time headway between the two vehicles. Volumes were readily obtainable since each vehicle provided four impulses and all vehicles were recorded.

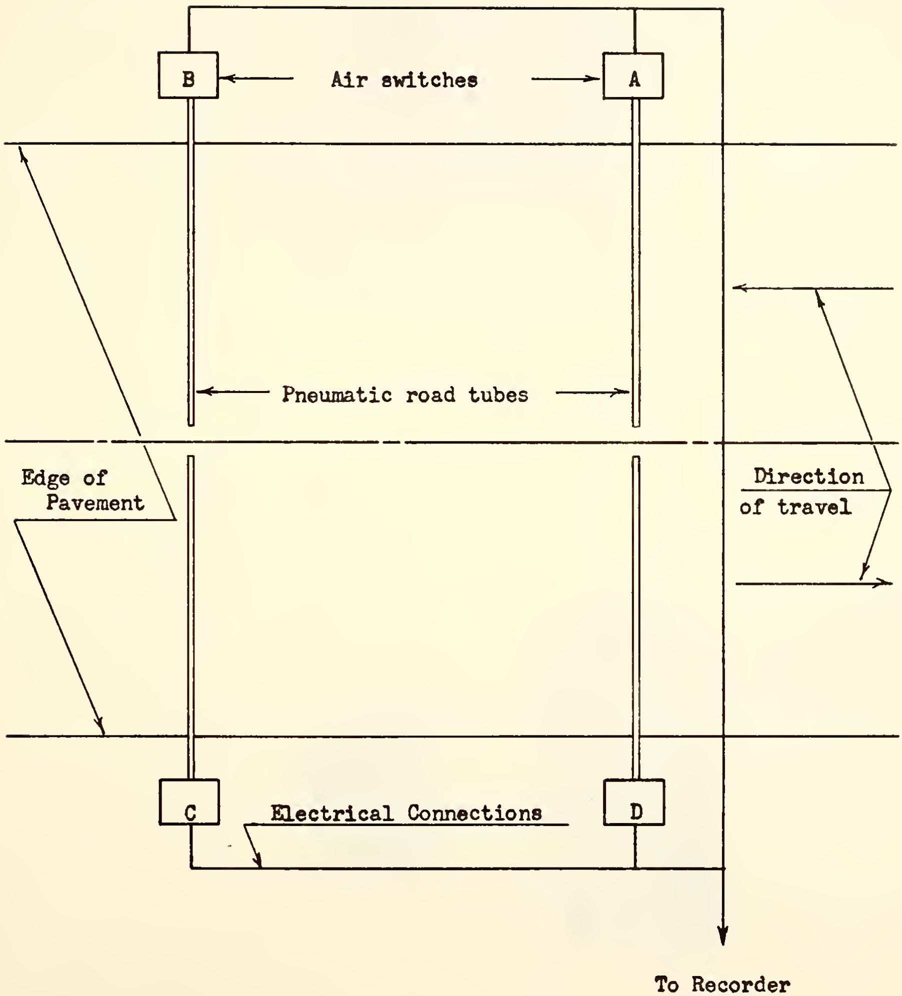


FIGURE 4. TYPICAL LAYOUT OF PNEUMATIC TUBES

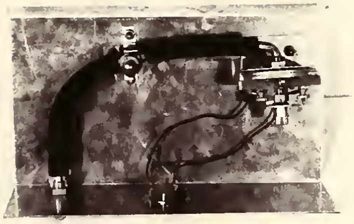


FIGURE 5. AIR SWITCH AS USED

FIGURE 6. ESTERLINE ANGUS 20-PEN RECORDER

PROCEDURE

Field

Data collection was performed using the same procedure at all of the study sites. One week prior to the time when data were to be collected, the pneumatic tubes were placed at the particular location so that their adverse effect on vehicle speeds would be minimized. It was assumed that if the tubes were in place several days prior to data collection the drivers would become accustomed to their presence and would not associate them with law enforcement.

Just prior to collecting data on a particular day the air switches, electrical leads, and pen recorder were connected to the road tubes which were already in place. These preliminaries were usually completed about one-half hour before data collection commenced. Enough wire was used to enable the pen recorder to be placed as far as 100 feet off the edge of the roadway. This allowed enough room to conceal adequately the recording equipment and the observer without much difficulty. The equipment was placed behind bushes or on porches of abutting residences, whichever was available, and provided adequate concealment as well as a good vantage point at each particular site.

The recorder was operated at a chart speed of twelve inches per minute and the road tubes were placed at a spacing of 100 feet. This chart speed allowed the time increments through the trap to be scaled directly to the nearest one-tenth of a second or estimated to five one-hundredths of a second. This chart speed in conjunction with the 100-foot spacing of the road tubes allowed calculation of vehicle speeds in approximately one mile per hour increments within the range of normally expected speeds; that is, 25 to 35 miles per hour. The chart speed was checked with a stop watch about every five minutes to insure that the rate of twelve inches per minute was maintained as there was a tendency for the speed to slow down when the spring drive become unwound. Adjustments of the chart speed were made when necessary.

Data were collected during a peak and off-peak period in the morning and afternoon of two days at each site so that a wide range of volume levels could be obtained. At sites located in Indianapolis these data were taken on successive days. At sites located in Lafayette, data were collected in the mornings and afternoons as allowed by the class schedule of the observer, but all data were usually collected for each site within a one week period. The chart speed and length of chart paper used allowed data to be collected continuously for one hour and 40 minutes, the usual time used for data collection during each period. All commercial vehicles were indicated by marking their impulses at the time of data collection. Except for the first site (Site A), vehicles which performed a turning movement at intersections adjacent to the site were also noted by marking the impulses. At Site A only, which was used as a preliminary study, a test car was driven through the study section to obtain the space-mean speed of the traffic stream.

Office

Data from the field tapes were summarized in 15-minute increments so that six samples were obtained from each chart when the chart ran for the full 100 minutes. The data obtained from each chart were the time increment through the measured distance, the arrival time and the direction of travel for each vehicle and the volume of traffic for each 15-minute period. The basic data of arrival time and time increment through the trap were measured directly on the chart. An example of the chart is shown in Figure 7. Pens 6 and 7 were used for one direction of travel and pens 9 and 10 were used for vehicles in the other direction. Because pens 7 and 9 were directly opposite each other on the graph and were connected to tubes placed on a line across the roadway, arrival times were scaled to these points. The difference in arrival times for two vehicles traveling in the same direction is the headway between the two vehicles. The additional information of turning movements and vehicle type were taken from the chart and tabulated with the corresponding vehicle in 15-minute increments.

The LGP-30 computer available in the School of Civil Engineering was used to calculate desired information. A program was written to produce average time through the trap, space-mean speed, time-mean speed, equivalent hourly volume, and the average density for each 15-minute period from the basic input of time increment through the trap for each vehicle and the total number of vehicles for the 15 minutes. These results were then tabulated and statistically analyzed.

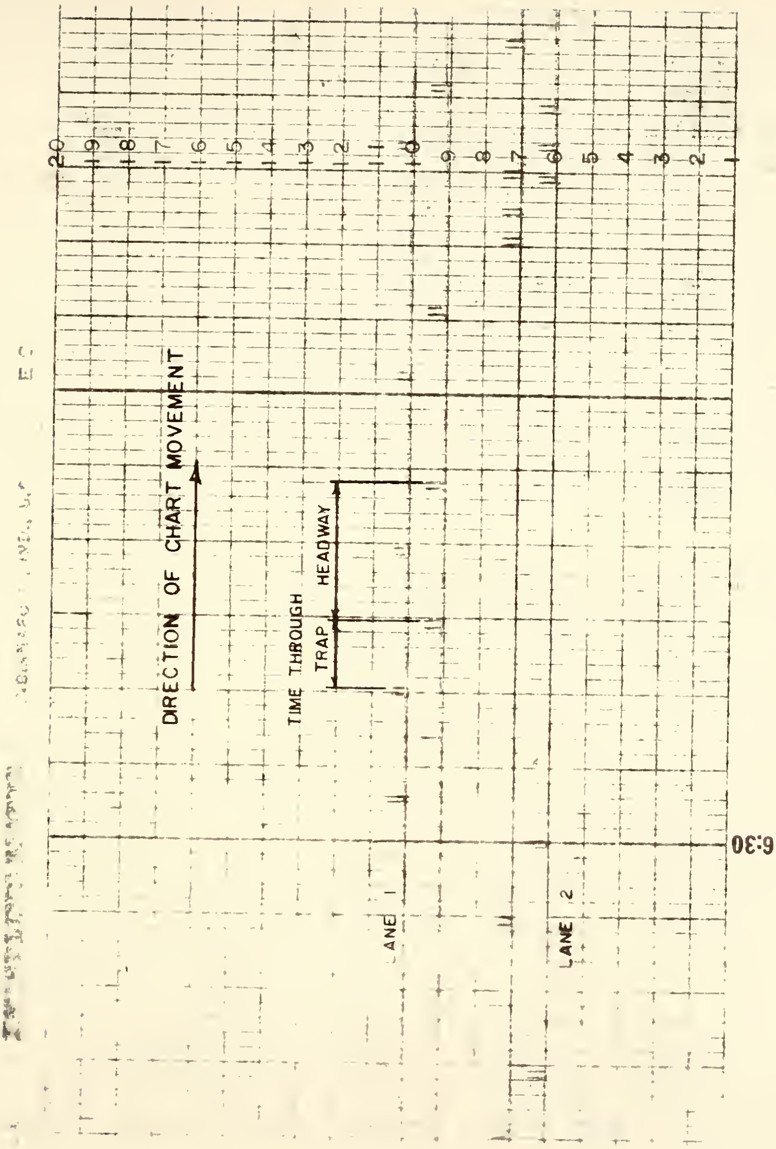


FIGURE 7. EXAMPLE OF CHART FROM PEN RECORDER

ANALYSIS

Initially the relationships of speed, volume and density were studied graphically to determine which relationship(s) would warrant further analysis. The first problem resolved was whether to calculate density from the basic equation relating the variables by using the space-mean speed as measured by a test car or to calculate density by using the harmonic mean of the spot speeds.

A test car was run at Site A to measure the travel time through the section using the "average" car method, where the driver attempts to drive at what, in his opinion, is the average speed of traffic. The average travel time was then used to calculate the space-mean speed for the period of time considered. The number of runs by the test car which could be made during each 15-minute increment varied from one to four, but D. S. Berry and F. H. Green have shown that to estimate the average travel time within ten percent of the true mean with 95 percent confidence, a minimum of eight to twelve runs are necessary (2). In addition, the densities obtained at Site A using measured space-mean speed and then calculated space-mean speed were for all practical purposes the same. On the basis of these findings density was calculated using the harmonic mean of the spot speeds as an estimate of space-mean speed and test car runs for space-mean speed were not obtained for the other sites.

The relationships of average density to average speed and to volume obtained at Site C are shown in Figure 8 and are typical of the data

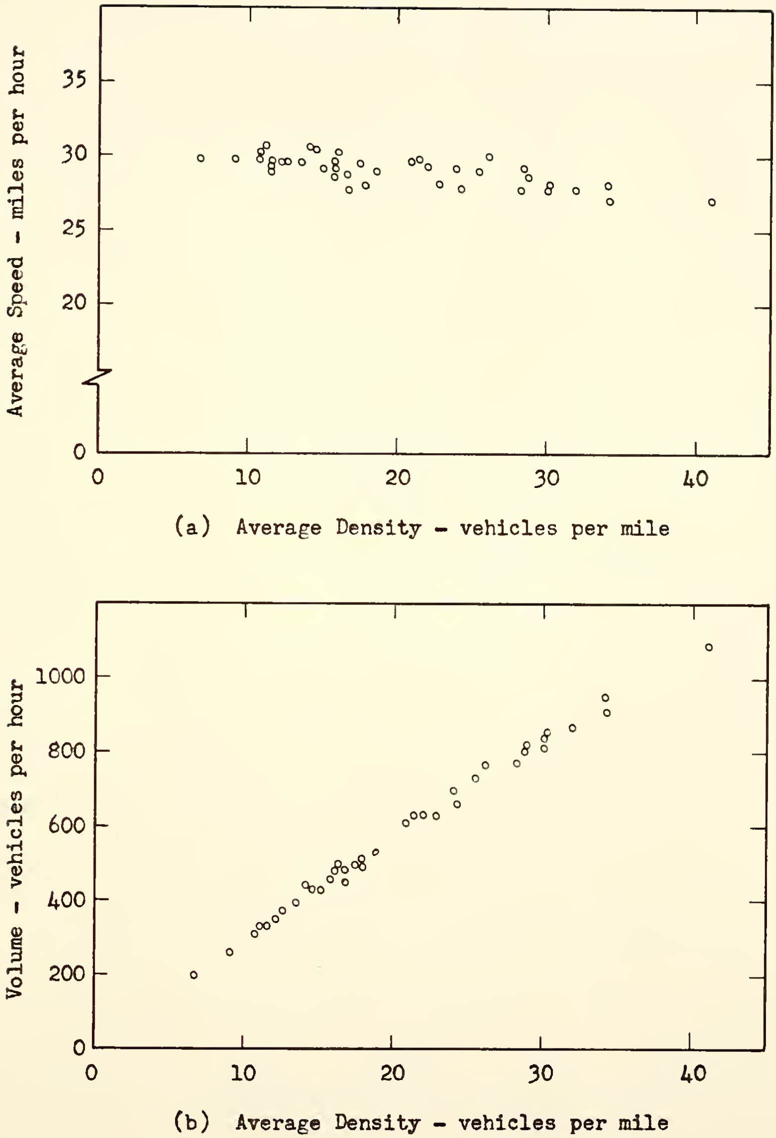


FIGURE 8. DENSITY RELATIONSHIPS AT SITE C

obtained at all sites. These graphs indicate; (a) a slight downward trend of mean speed with an increase in average density, and (b) an increase in average density with an increase in volume. The data obtained, however, are for only relatively low values of density, less than 50 vehicles per mile. The conditions of higher densities and critical density and jam density were not obtained at the study sites during the study periods. The volume-density graph obtained in this study suggests an almost linear trend at these relatively low densities. Other researches have shown that this relationship is curvilinear, especially at higher densities. They have shown that density increases as volume increases until a critical density is reached. Then, as density continues to increase the volume decreases until the point of jam density is reached. At this point the flow of vehicles is zero.

The densities obtained in this study, however, were calculated using measured volumes and as a result densities were dependent on measured volumes. More complete analysis of the density-volume and speed-density relationships, therefore, was not felt to be fruitful and a thorough analysis of the relationship between speed and volume, which were independently measured, was initiated.

Statistical Analysis of the Speed-Volume Relationship

A multiple linear correlation and regression analysis was used to investigate the relationship of mean speed to volume and to other variables of the traffic stream that could be measured at each site. In this analysis the mean speed was taken as the dependent variable with the independent variables designated as follows:

X_1 volume (in hundreds of vehicles per hour).

X_2 percent of vehicles performing turning movements within one block of the site.

X_3 percent of commercial vehicles in the traffic stream.

X_4 percent of the total volume distributed in the major direction of flow

These variables were selected to describe the differences in the composition of the traffic stream and were measureable quantities at each site studied. The observations were based upon the 15-minute samples and volume was expressed as an equivalent hourly volume (4 times the 15-minute volume).

The computational procedures for the correlation and regression analysis were accomplished by use of the IBM 7090 computer available through the Computer Science Center. The computer program used produced the necessary information for an analysis of the significance of the regression equations and the variables included in the equations. The output listed the variables in the regression equation in the order of their contribution to the amount of variability in the mean speed explained by the independent variables. The program also computed and printed the necessary information to test the significance of the addition of each variable. An analysis of variance table for the final form of the equation was computed by the program in the form shown in Appendix A (Table 6).

In testing the significance of the regression, the F-test was used to determine, (a) the overall significance of the variation in average speed explained by the variables in the equation, and (b) the significance of the increase in the explained variation attributed to each variable added to the equation. The latter F-test was actually performed on the increase in R^2, where R^2 is the proportion of the sum of squares of the dependent variable explained by the multiple regression equation (37), or more simply it is the fraction of variation in the dependent variable explained by the regression equation. A disadvantage of this type of analysis is that a variable may be determined to be significant when actually it is not, and vice-versa, depending upon the order in which the variables are entered into the equation and the order in which they are tested. The increase in R^2 due to a particular variable may not be significant although the variable may explain a significant amount of variability in the dependent variable in a simple regression or if the order of entry into the multiple regression equation were different. This would probably be due to the intercorrelation of the variables. This problem may have been alleviated somewhat since the variables were tested in the order of their decreasing significance as determined by the computer program.

Another question investigated was whether or not the regression line for a type of site or for all locations within a city regardless of site type was the same as the regression line for a particular site. If the variability between individual site regression equations is not significantly different from the within site variability about the regression line, then a site type equation may be assumed to be the same as the individual site equations within a site type. In this case, the comparisons had to be made with equations containing the same variables. As a result several sub-problems of each line of regression had to be run so that the appropriate equations would be available for the several comparisons.

At this point it should be noted that some of these tests were not wholly independent since several tests were performed on the same data as in the case of the regressions for the individual sites. For the most part however, the tests performed in the comparisons of the several lines of regression were independent. Due to the large number of F-tests performed the level of significance had a tendency to decrease. For this reason an α level of 0.01 was selected in an attempt to keep the resulting level of significance at a reasonable value. The test procedures and notations used are shown in Appendix A.

Distribution of Headways

Several distribution functions have been proposed to describe headways in a traffic stream. One of the purposes of this study was to compare the distribution of observed headways with some of these theorized distributions. The theories of particular interest to this researcher were those proposed by R. M. Lewis (16) and by A. Schul (29). Another point which arose during the course of this research warranted some investigation. This was to determine, if possible, the effect of opposing volume on the distribution of headways in the traffic stream as this factor had not been considered in any of the theoretical distributions.

The distribution proposed by Lewis is a modified binomial distribution which attempts to account for the platooning effect of vehicles in the traffic stream. To account for this platooning effect two levels of probability of a car arrival are utilized. The appropriate level of probability is determined by whether a vehicle is considered to be in a platoon or not to be in a platoon. To be considered part of a platoon a vehicle must be within the influence of the preceeding vehicle. In effect what these two levels of probability say is "the occurrence of a recent arrival enhances the probability of an arrival at the present instant. Likewise, if the previous arrival was some time ago, the probability of an arrival at this instant is diminished" (16). In developing this distribution Lewis defined several parameters. The headway at or below which it was assumed that a platoon exists was taken to be 4.75 seconds, while the minimum headway permitted was 1.5 seconds.

The distribution proposed by A. Schul is a composite exponential distribution which is actually the sum of two sub-distributions. The sub-distributions express the probability of arrivals for free-moving and restricted vehicles. The unknown parameters necessary for the solution of this distribution at a particular volume level are the proportion of restrained vehicles in the traffic stream, the average headways of restrained and free-moving vehicles and a minimum allowable headway. J. H. Kell (14) has rewritten the equation of this distribution to obtain four constants which are functions of the unknown parameters proposed by Schul, but the equation retains the form of two sub-distributions, one for the free-moving and one for the restrained vehicles. The resulting equation was fitted by Kell to 585 samples with volume rates ranging from 100 vehicles per hour to 1200 vehicles per hour, and the constants were evaluated in terms of volume only. The final equation obtained by Kell is applicable to two-lane urban streets (14).

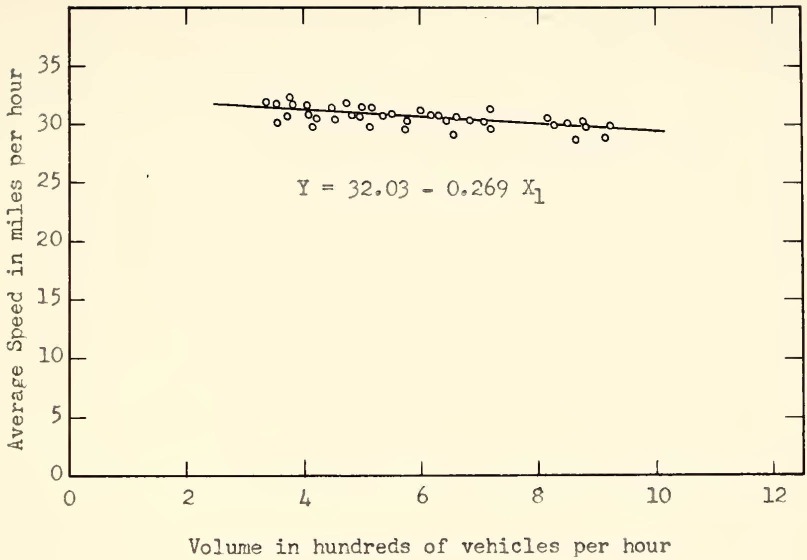
To facilitate analysis of the effect of opposing volumes of traffic on the headway distribution, all of the data were summarized in half-hour increments and classified into groups of 100 vehicles per hour with the limits ranging from 50 vehicles per hour to 549 vehicles per hour according to inbound and outbound direction. Two samples were randomly selected from each cell where possible. This resulted in 84 samples used in the analysis. A summary of these data is tabulated in Appendix C. The data are expressed in terms of the percent of headways less than a given time for various levels of opposing volume. In order to express these data graphically an exponential curve of the form $P = 1-e^{cV}$ was fitted by the method of least squares, where P represents the proportion of headways less than a given value of time, c is a constant, e is the natural base of logarithms, and V is the volume level at which this proportion was measured. The equation was forced through the origin since the proportion of headways less than a given time must be zero when the volume level is zero. The calculated c values for the several time increments considered are also tabulated in Appendix C.

RESULTS

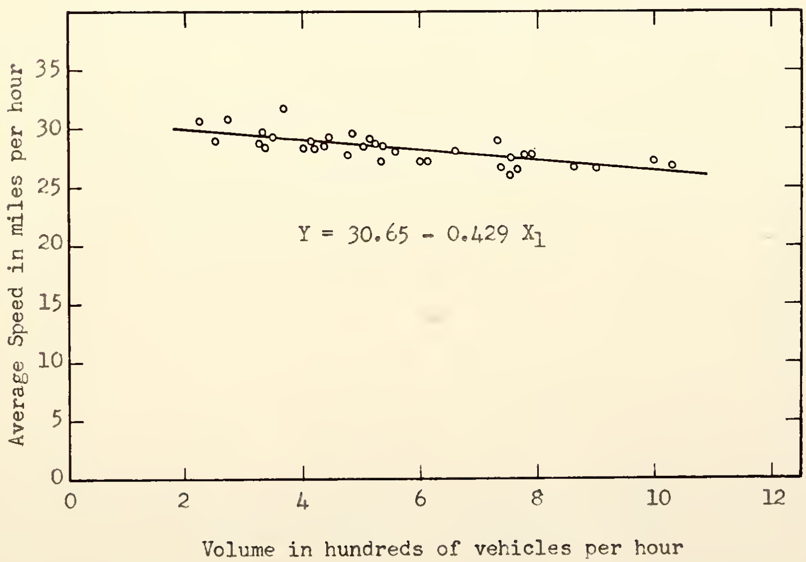
Speed-Volume Relationships

The speed-volume data for each site are graphically shown in Figures 9-11. The best fitting straight line was fitted to the data for each of the sites and is shown on each of the graphs. In all cases a very slight decrease in speed occurred with an increase in volume. The slopes of the regression lines indicate a decrease in mean speed of approximately 0.1-0.4 miles per hour for each 100 vehicles per hour increase in volume. The values of mean speed for the same volume also differed from site to site, approximately 1-3 miles per hour, especially between sites in Lafayette and Indianapolis. This mean speed difference existed between sites of the same type as well as between sites of different types.

In order to evaluate the effects of differing quantities of commercial vehicles, turning movements at adjacent intersections, and directional distribution of the traffic volume as well as volume of traffic on mean speed, a multiple regression analysis was performed. Only data obtained for volumes of 350 vehicles per hour and higher were included in this analysis. At two of the sites, only one observation at volumes below 350 vehicles per hour was made, and it was believed desirable to make comparisons between sites over a similar volume range. In addition, Normann has reported an apparent change in the speed-volume relationship at low volumes (27). Other researchers have also found a change in the relationship of speed and volume for volumes

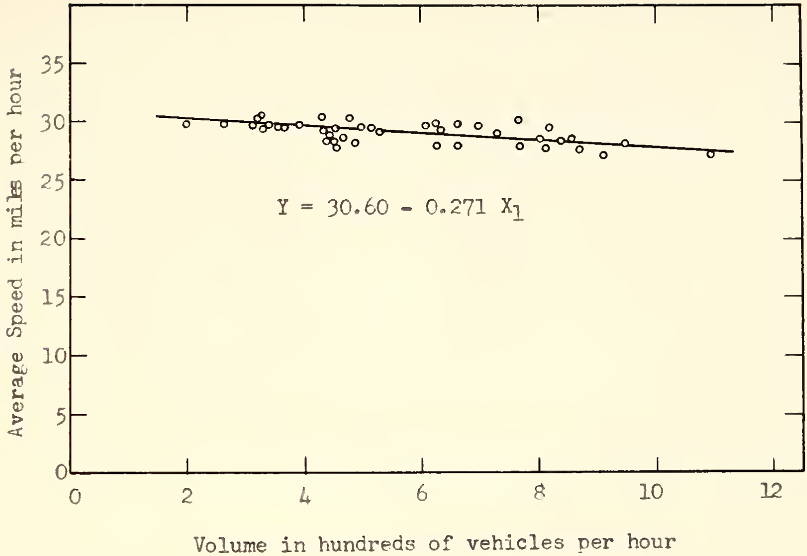


(a) Site A

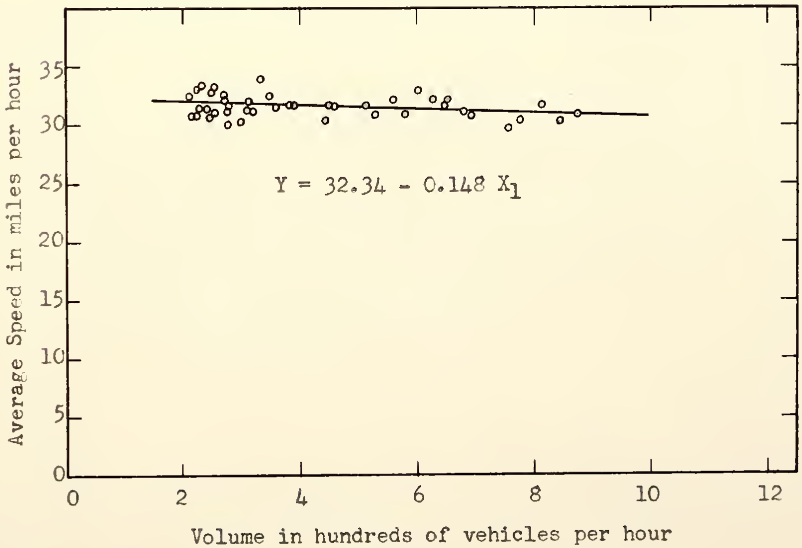


(b) Site B

FIGURE 9. SPEED-VOLUME RELATIONSHIPS FOR SITE TYPE I

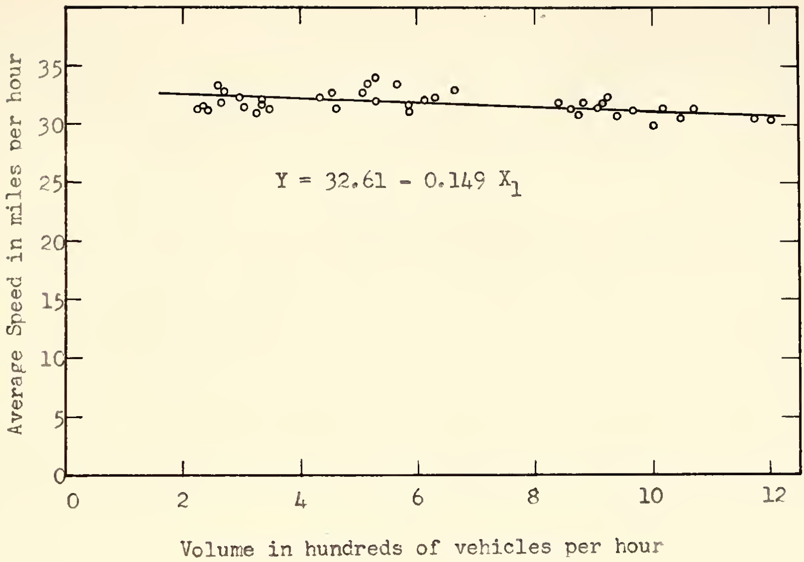


(a) Site C

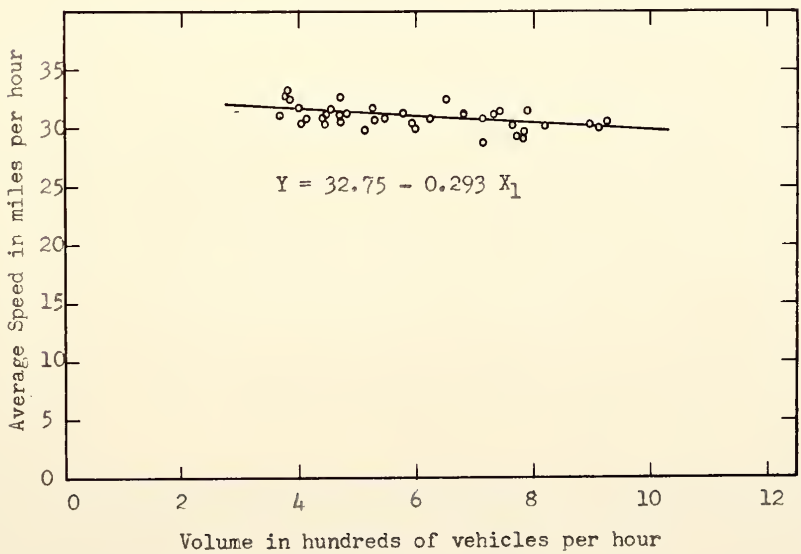


(b) Site D

FIGURE 10. SPEED-VOLUME RELATIONSHIPS FOR SITE TYPE II



(a) Site E



(b) Site F

FIGURE 11. SPEED-VOLUME RELATIONSHIP FOR SITE TYPE III

near roadway capacity. All observations made in this study, however, were well below capacity conditions and all traffic was free-flowing. As a result no restrictions were placed on the higher volumes obtained in this study.

The equations obtained from the multiple regression analysis for each site are indicated in Table 2 along with the multiple R^2 and the simple r^2 for the regression of speed on volume only. The variables are indicated in the equations in the order of their significance in contributing to the variability of the average speed in the samples obtained at each site. The regression equations obtained were significant at the one percent level of significance for all sites except D where the total variability explained by the independent variables was only 0.18. The R^2 varied from 0.30 to 0.66 for the other five study sites. In each case the most significant single variable was the volume and in no case did the addition of the other independent variables increase the explained variability significantly at the one percent level of significance. The improvement in the R^2 value, however, was sizeable for sites B, C, and E when the variables other than volume were included. The turning movement effect at sites B and E and the commercial vehicles at sites C and E contributed most to the additional explained variability. The turning movement variable was not included at sites A and D. This variable was not applicable for site D as there was no nearby adjacent intersection. The information was not obtained at site A, the first site observed.

In the multiple regression analysis, when data over the same range of volumes (350 and over) and other variables of the traffic stream were considered, a more consistent relationship between mean-speed and volume was shown. At

TABLE 2

REGRESSION EQUATIONS FOR INDIVIDUAL SITES

Site	Equation	R^2	r^2
A	$Y = 31.55 - 0.334\ X_1 + 0.017\ X_4 - 0.021\ X_3$	0.37*	0.35
B	$Y = 28.90 - 0.430\ X_1 - 0.110\ X_2 + 0.061\ X_4 - 0.041\ X_3$	0.66*	0.51
C	$Y = 31.75 - 0.332\ X_1 - 0.146\ X_3 - 0.039\ X_2 + 0.008\ X_4$	0.41*	0.30
D	$Y = 32.71 - 0.234\ X_1 - 0.111\ X_3 + 0.003\ X_4$	0.18	0.17
E	$Y = 35.81 - 0.354\ X_1 - 0.352\ X_2 - 0.096\ X_3 - 0.007\ X_4$	0.62*	0.52
F	$Y = 32.28 - 0.355\ X_1 - 0.038\ X_3 - 0.041\ X_2 + 0.026\ X_4$	0.30*	0.27

* Significant at the one percent level of significance

Where:

Y is mean speed (in miles per hour)

X_1 is volume (in hundreds of vehicles per hour)

X_2 is the percent of vehicles performing turning movements within one block of the site

X_3 is the percent of commercial vehicles in the traffic stream

X_4 is the percent of the total volume distributed in the major direction of flow

four of the six sites studied the regression analysis indicated a decrease in mean-speed of about 0.35 miles per hour for each 100 vehicles per hour increase in volume. This relationship differed at sites B and D, however, by approximately 0.1 mile per hour for each increase in volume of 100 vehicles per hour.

The coefficients for three of the independent variables (X_1 , X_2 , and X_3) were consistent regarding sign among all of the sites and indicate a decrease in average speed with increased volume, turning movements, and commercial vehicles. The coefficient for directional distribution (X_4) in all cases except site E was positive and indicates an increase in average speed with an increasing percentage of vehicles in the major direction. With the exception of volume there was no consistent trend as to which variables were most important in explaining the variability in average speed. Although the total variability explained by the regression equations was significant at five of the six sites studied, the proportion explained was greater than 50 percent at only two sites. These two sites were different types located in different cities. No reason is apparent as to why the R^2 at these two sites was much larger than at the other four.

A regression analysis was performed on the data grouped by site types. Initial analysis included the use of a lane width factor and the number of intersections within one-quarter mile each way from the speed station. These two factors were intended to measure between site differences. The regression equations which resulted were not reasonable and were abandoned. For example, for one type of site the regression equation indicated that as the number of intersections increased the speed increased, and no difference in lane width existed for one type of site. It was concluded, therefore, that lane width and number of intersections were not adequate measures of between site variability.

The regression equations obtained by grouping the data by site types using the variables of volume (X_1), turning movements (X_2), commercial vehicles (X_3), and directional distribution (X_4) are shown in Table 3. The turning movement factor (X_2) could not be included for site types I and II because these data were not available for one site in each type.

All of the regression equations were significant at the one percent level of significance with the proportion of explained variability ranging from 0.28 to 0.67. The most significant variable was different in the case of each site type, and was volume (X_1) for site type I, percent commercial vehicles (X_3) for site type II, and turning movements (X_2) for site type III. In each case where volume (X_1) was not the most significant variable, including it in the regression equation did significantly increase the explained variability in average speed. In the case of site type II, each of the three variables (X_1 , X_3 , and X_4) increased the R^2 significantly while for types I and III, including X_3 and X_4 in the regression did not significantly increase the explained variability. The sign of each variable was consistent with those obtained for the individual sites except for one case with X_3, where its contribution to the variability was negligible.

These site-type equations were tested using the procedure outlined in Appendix A to determine whether the regression equation obtained for the site type could be considered to be the same as the regressions for the individual sites. The results of this test were that for sites type I and II, the site-type equation could not be considered equivalent to the individual site equations. However, for site type III, the site-type equation could be considered the same as the individual equations.

TABLE 3

REGRESSION EQUATIONS BY SITE TYPE

Type	Equation	R^2
I	$Y = 28.53 - 0.443\ X_1 + 0.060\ X_4 + 0.010\ X_3$	0.28*
II	$Y = 30.26 - 0.298\ X_3 - 0.474\ X_1 + 0.050\ X_4$	0.67*
III	$Y = 33.34 - 0.111\ X_2 - 0.361\ X_1 - 0.071\ X_3 + 0.025\ X_4$	0.42*

*Significant at the one percent level of significance

Where:

Y is mean speed (in miles per hour)

X_1 is volume (in hundreds of vehicles per hour)

X_2 is the percent of vehicles performing turning movements within one block of the site

X_3 is the percent of commercial vehicles in the traffic stream

X_4 is the percent of the total volume distributed in the major direction of flow

In the latter case, each site was located within the same city while for types I and II, each site within a type was located in a different city. This may indicate that similar types of facilities located within the same city have a similar relationship among the variables affecting speed, but that the variables affecting speed have different relationships on similar type facilities in different cities. The possibility exists, however, that if the differences in sites could be adequately considered a site-type regression equation could describe the relationships in different cities.

These data were also analyzed by grouping the data by city without regard to site type. The resulting equations obtained from that analysis for the sites in Lafayette and in Indianapolis are shown in Table 4. The total explained variability in each of the regression equations is significant at the 0.01 level, although the proportion explained is less than 50 percent in both cases. In both cases the volume is the most significant variable although the commercial vehicle variable (X_3) does significantly increase the explained variability for the sites in Lafayette. In neither of the cases do the other variables significantly increase the explained variability of the mean speeds.

These equations were also tested to determine if the regression equation for a city, regardless of site type, could be considered the same as the individual site equations. In both cases, neither of these equations could be considered equivalent to the individual site equations at the one percent level of significance.

TABLE 4

REGRESSION EQUATIONS BY LOCATION

Location	Equation	R^2
Lafayette	$Y = 31.07 - 0.463 X_1 - 0.149 X_3 + 0.030 X_4 - 0.045 X_2$	0.45*
Indianapolis	$Y = 32.05 - 0.224 X_1 + 0.009 X_4 - 0.025 X_3$	0.16*

* Significant at the one percent level of significance

Where:

Y is mean speed (in miles per hour)

X_1 is volume (in hundreds of vehicles per hour)

X_2 is the percent of vehicles performing turning movement within one block of the site

X_3 is the percent of commercial vehicles in the traffic stream

X_4 is the percent of the total volume distributed in the major direction of flow

Headway Distributions

The results obtained by fitting the exponential curves ($P = 1-e^{cV}$) to the headway distributions at various levels of opposing volume did not agree with what had been expected. Analyzing what the probable effect of opposing volume would have on the headway distribution of a one-way stream of traffic, it was expected that as the number of opposing vehicles increased more drivers would be forced to accept shorter headways. In other words, as the volume of opposing traffic increases the fraction of headways less than a given time would increase. This would be the result of the inability of more drivers to maintain their desired speed because of fewer adequate gaps in the opposing lane of traffic for desired passing maneuvers. The availability of this adequate gap decreases as the volume of opposing traffic increases.

The results of this analysis are shown in Figures 12 and 13. They indicate that the percentage of vehicles traveling at shorter headways on two-lane, two-way urban arterials is greatest when the volume of opposing traffic is lowest (approximately 100 vehicles per hour). As this opposing volume increases the fraction of drivers accepting short headways decreases until the distribution seems to stabilize at 300-500 vehicles per hour of opposing traffic.

These results may occur because at low volumes of opposing traffic the opportunity to pass is almost always available, therefore more vehicles are accepting shorter headways waiting for this opportunity. As the volume of opposing traffic increases, the drivers realize that their passing opportunities are practically nonexistent. Consequently fewer drivers travel at a short headway anticipating a passing maneuver. These results

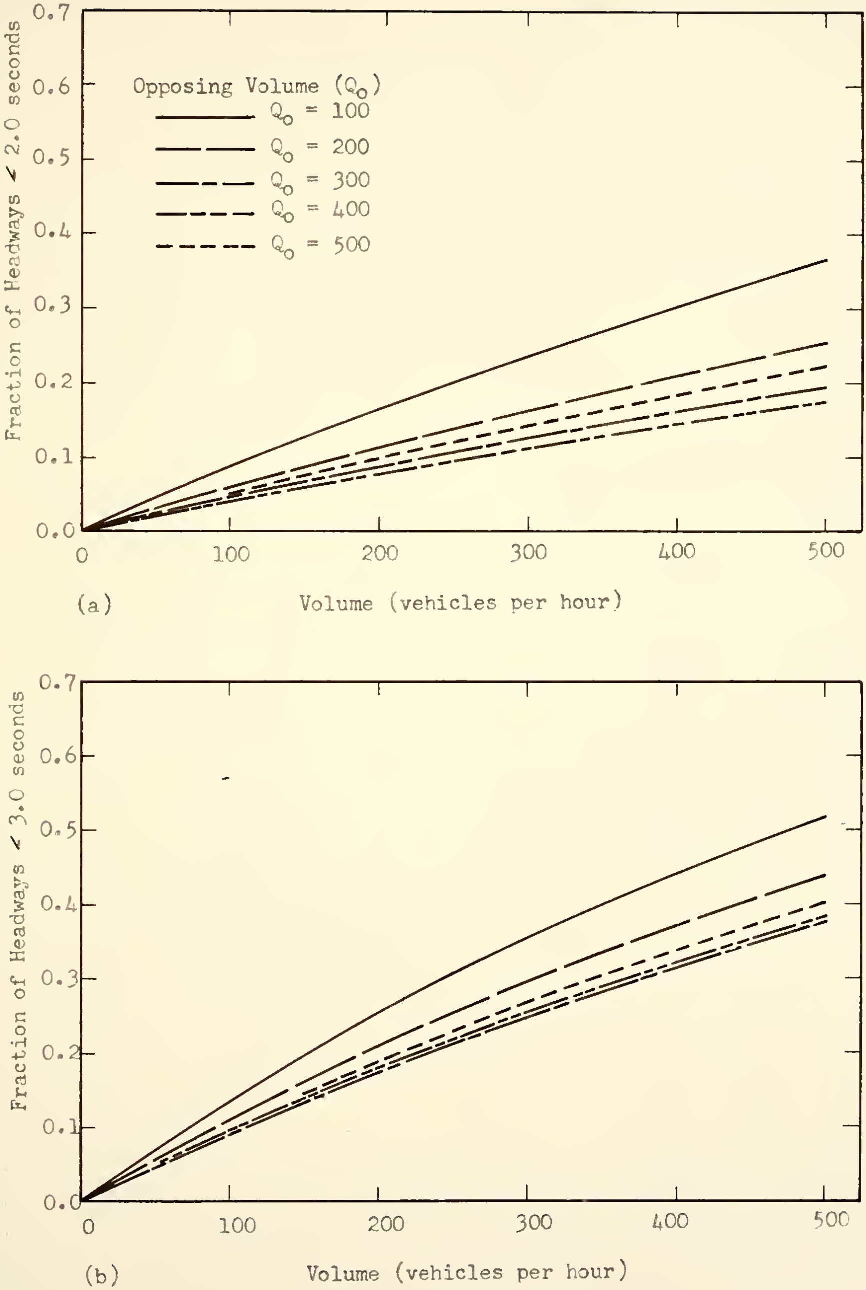


FIGURE 12. EFFECT OF OPPOSING VOLUME ON HEADWAY DISTRIBUTIONS

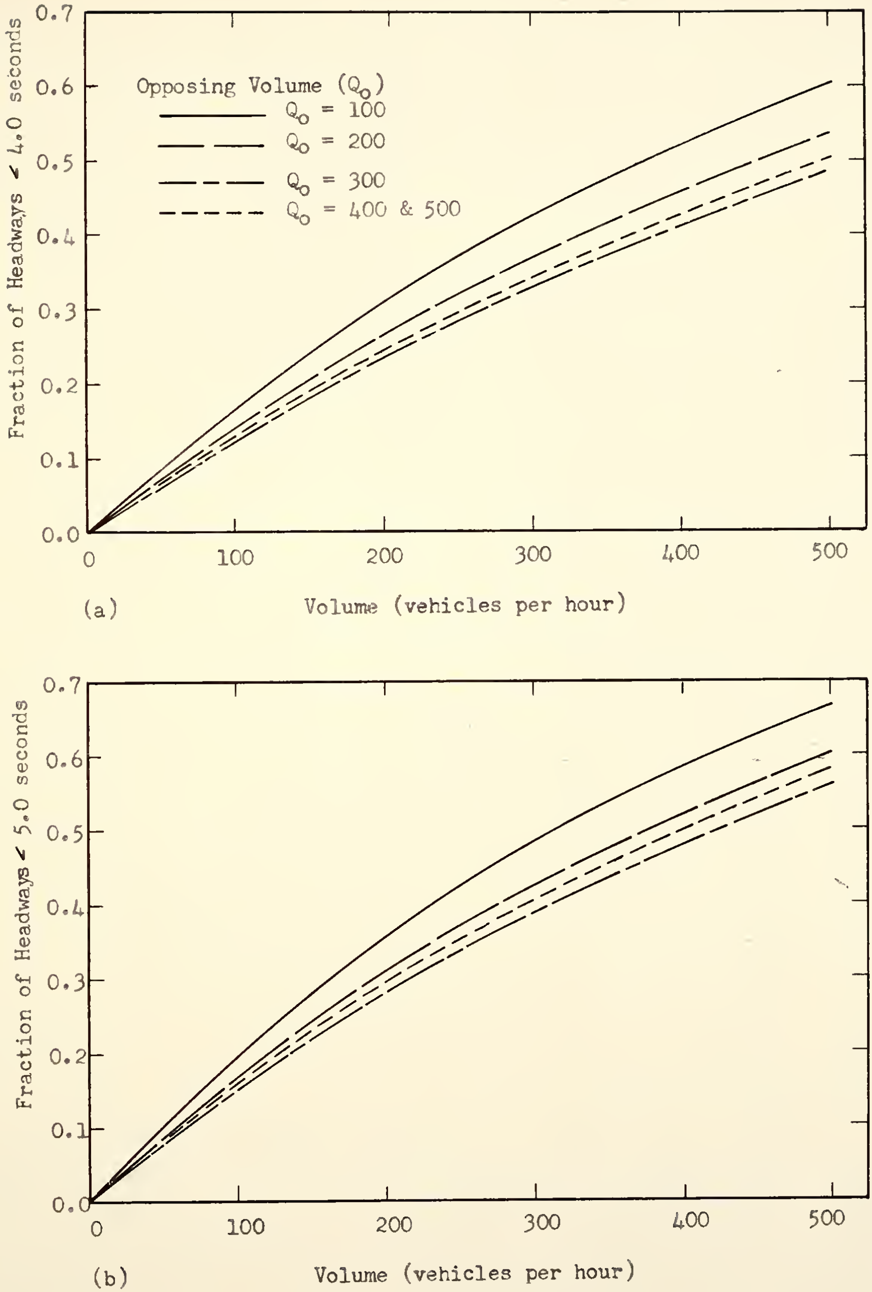


FIGURE 13. EFFECT OF OPPOSING VOLUME ON HEADWAY DISTRIBUTIONS

may have occurred because of a possible bias introduced by the method of data collection. Although the opportunity to pass was available, some drivers might have followed the preceeding vehicle through the sites because the pneumatic tubes placed on the roadway presented the possibility of a law enforcement "speed-trap".

Further analysis of the figures presented indicates that as the headway time increment increases the effect of the opposing volume seems to decrease. This is shown by the fact that the curves begin to come together, and was even more noticeable when time increments of 10, 20 or 30 seconds were considered. This effect is not unexpected since vehicles traveling at larger headways probably would not be affected by the opposing volume.

The distributions of headways on the two-way two-lane urban arterial streets were also calculated from the field data for several headway time increments. Thirty half-hour samples, selected at random, were chosen from the field data for this analysis. Exponential curves of the form $P = 1 - e^{cV}$ were fitted to the field data by the method of least squares after transformation. Three of these fitted curves with the plots of the field data are shown in Figure 14. The curves fit the data quite well. The fitted curves without the field plots for fourteen headway time increments are shown in Figure 15. The data used in this analysis and the calculated c values for the various time increments are tabulated in Appendix C.

A figure similar to Figure 15 is included for two-way two-lane rural roads in the Highway Capacity Manual (3). A comparison of these two analyses indicates that on an urban street there are fewer vehicles traveling at short headways than on rural roads at the same volume level. This condition may be the result of the different operating speeds for rural

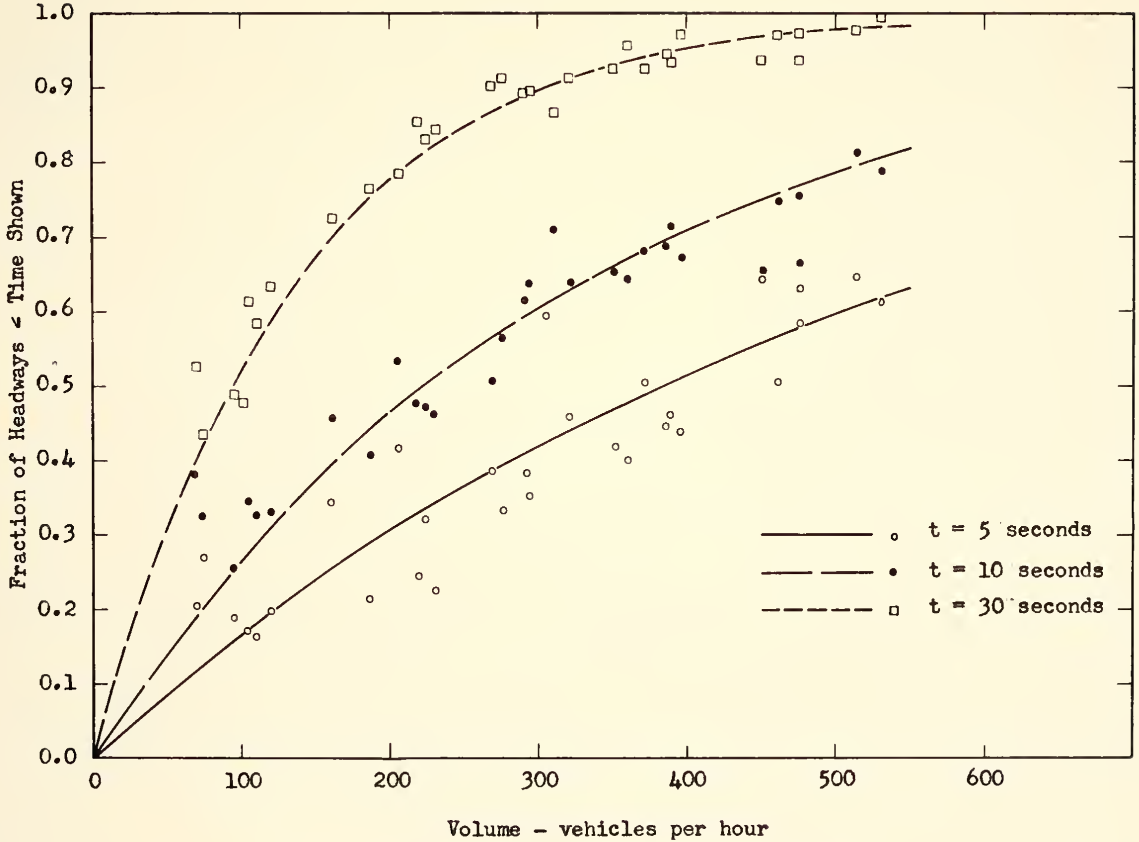


FIGURE 14. EXPONENTIAL CURVE FITTED TO FIELD OBSERVATIONS

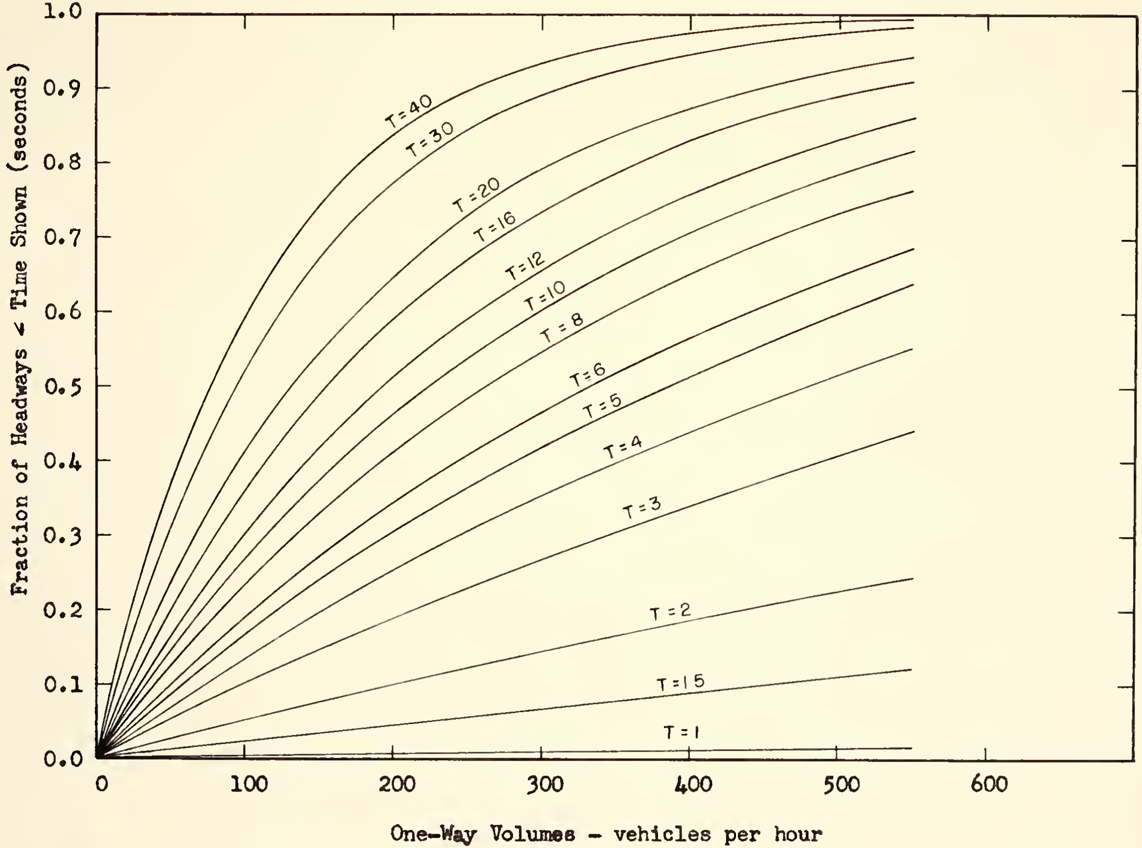


FIGURE 15. DISTRIBUTION OF HEADWAYS ON TWO-WAY, TWO-LANE URBAN ARTERIAL STREETS

and urban conditions. In an urban area a headway of two seconds at a speed of 30 miles per hour is a spacing of 88 feet while at a speed of 60 miles per hour for the same headway the distance spacing is 176 feet. Drivers probably select their vehicle spacing in terms of distance rather than time and this distance spacing does not increase in direct proportion to their speed.

The distribution of headways as given by Figure 15 for a given volume was assumed to be the distribution of field data for the urban arterial streets studied. To facilitate comparison of the distribution of these field data with the theoretical distributions of Lewis and Kell, it was desirable to express the headways as the fraction equal to or greater than a given time increment for selected volume of one-way traffic volume. The necessary values were easily obtained by subtracting the known value of the fraction of headways less than a given time increment from one. The shape of this curve for a volume of 300 vehicles per hour is indicated in Figure 16.

The original distribution suggested by Lewis allows a minimum permissible headway of 1.5 seconds. He further states that minimum headways have been found to range from 0.5 to 2.0 seconds, but that the average minimum headway is 1.5 seconds (16). In analyzing the data collected, the occurrence of headways between one and two seconds was not unusual but headways less than one second rarely occurred. This is also shown in Figure 15. The parameter of minimum permissible headway as used by Lewis was redefined as the minimum acceptable headway and was assumed to be equal to one second. A second modification of the Lewis distribution was the calculation of fraction of headways equal to or less

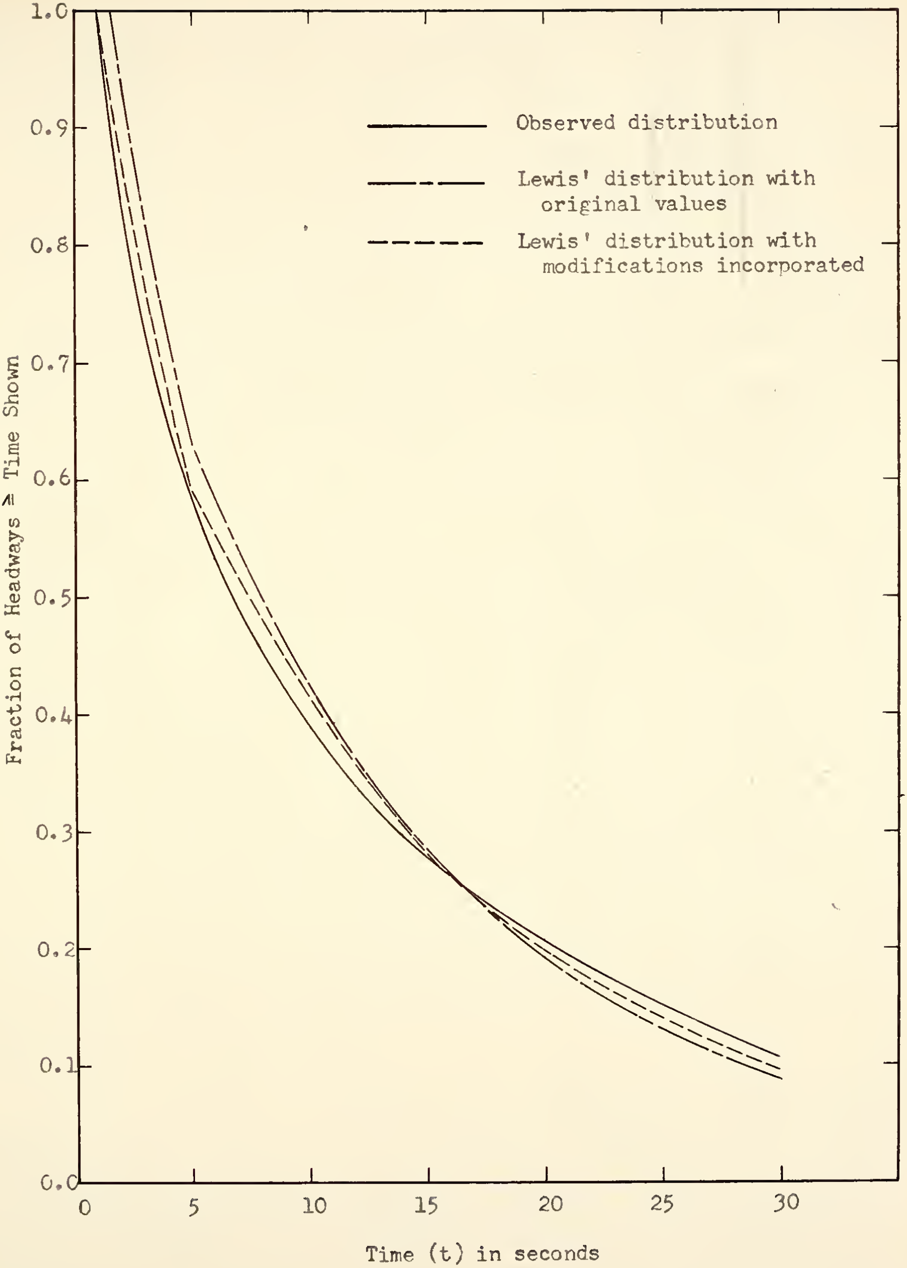


FIGURE 16. COMPARISON OF LEWIS' DISTRIBUTION WITH ORIGINAL VALUES AND WITH THE MODIFICATIONS INCORPORATED AT 300 VPH

than 4.75 seconds. Lewis fitted an exponential curve of the form $P = 1-e^{cV}$ to field data and determined the value of c to be - 0.00151. This same type curve was fitted to the data of this study and a c value of -0.00171 was obtained. The result of these modifications is shown in Figure 16 where Lewis' original distribution and the distribution with the modifications are plotted with the empirical data from this study at a volume of 300 vehicles per hour. The distribution with the modifications incorporated follows the field data closer at all time increments. Similar results were obtained for other volume levels although the differences between Lewis' distributions, with and without the modifications, were less at lower volumes than at higher volumes.

The headway distribution as determined from Figure 15 is shown graphically in Figures 17 through 21 for five different levels of volume. Also pictured are the composite distribution proposed by Schul using the constants as determined by Kell, and Lewis' distribution with the modifications. In general all three curves appear to follow each other fairly closely with only a few consistent differences. The largest discrepancies appear to be at the lower volumes in the area of the longer headways. This may be the result of traffic control devices located at either end of the study sections, where because of the control exercised, some long headways would have a tendency to become longer.

The most consistent difference at all but the lowest volume level was in the area where headways were less than 15-20 seconds. The distribution of field data indicates that there were slightly more vehicles traveling at shorter headways than are predicted by either of the two theoretical distributions. Lewis' attempt to account for the platooning effect of

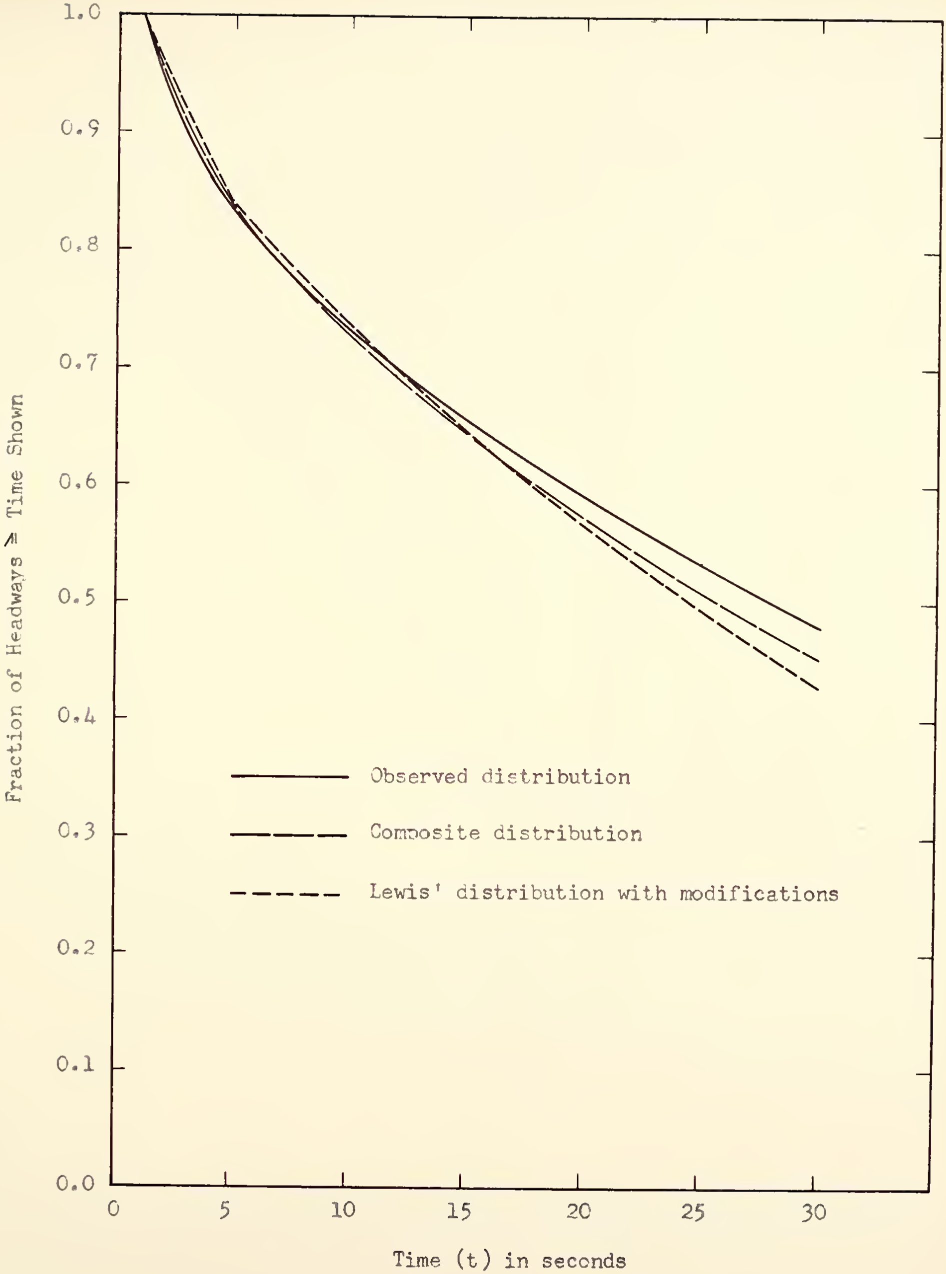


FIGURE 17. HEADWAY DISTRIBUTIONS AT 100 VPH

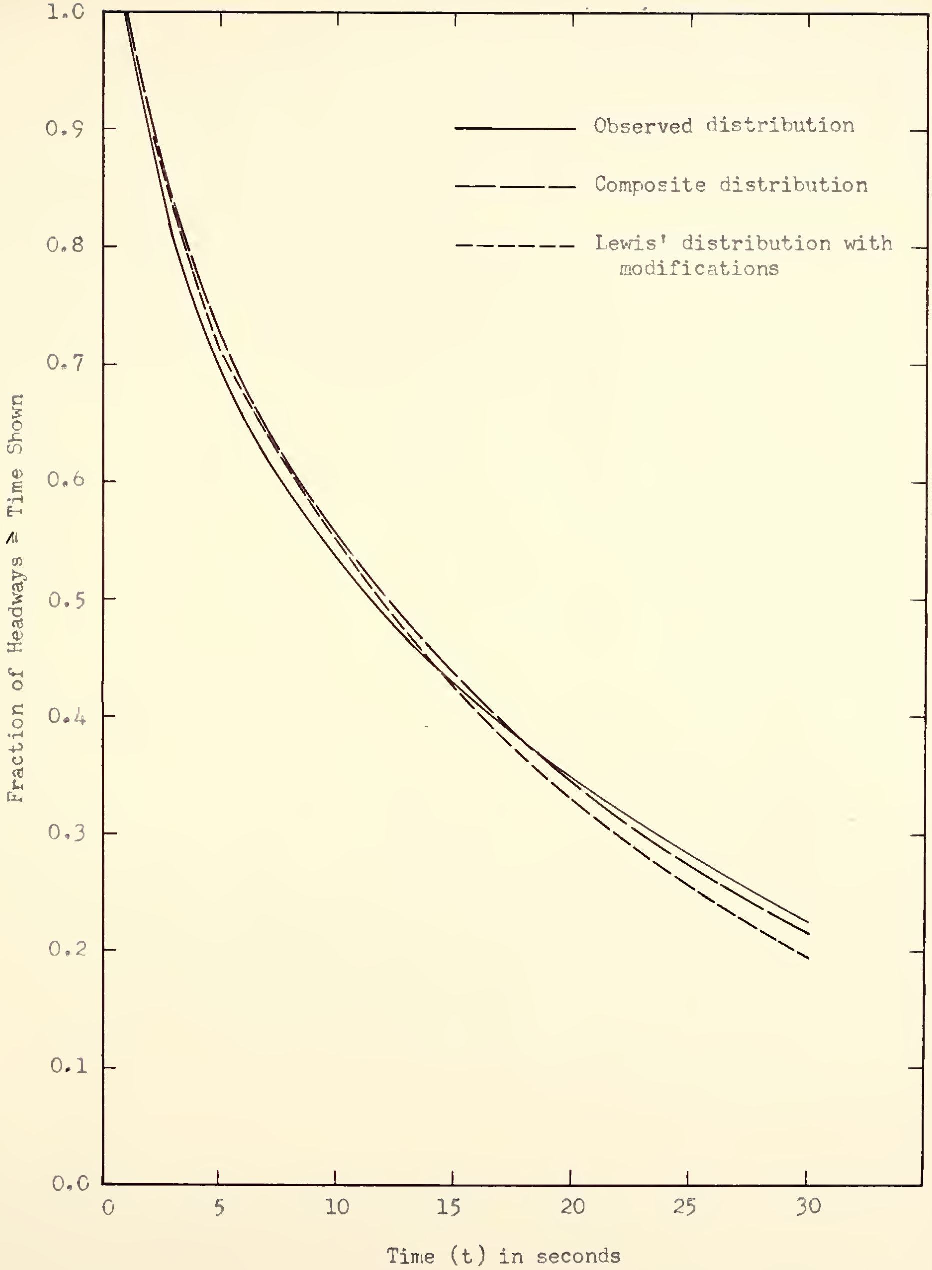


FIGURE 18. HEADWAY DISTRIBUTIONS AT 200 VPH

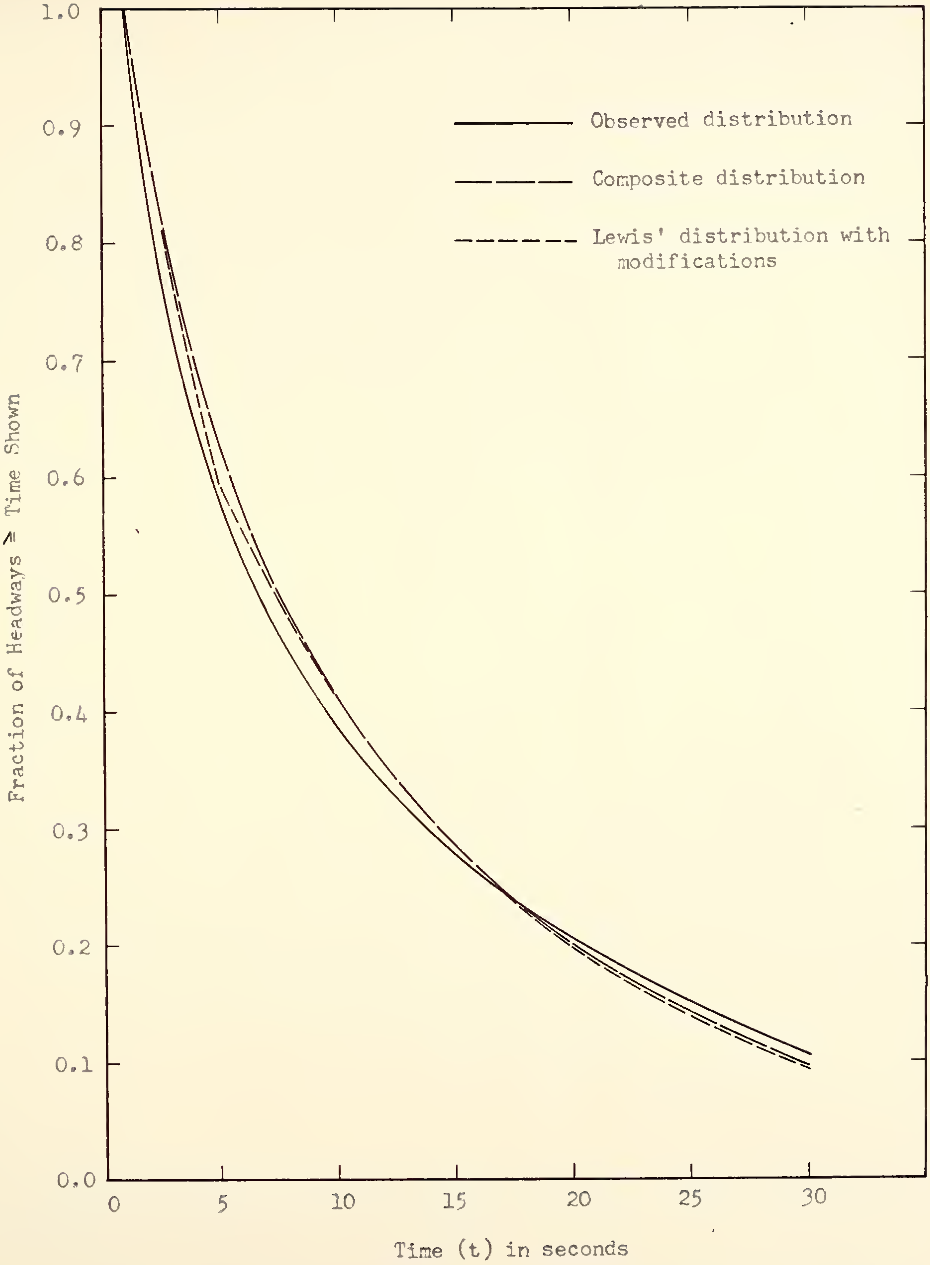


FIGURE 19. HEADWAY DISTRIBUTIONS AT 300 VPH

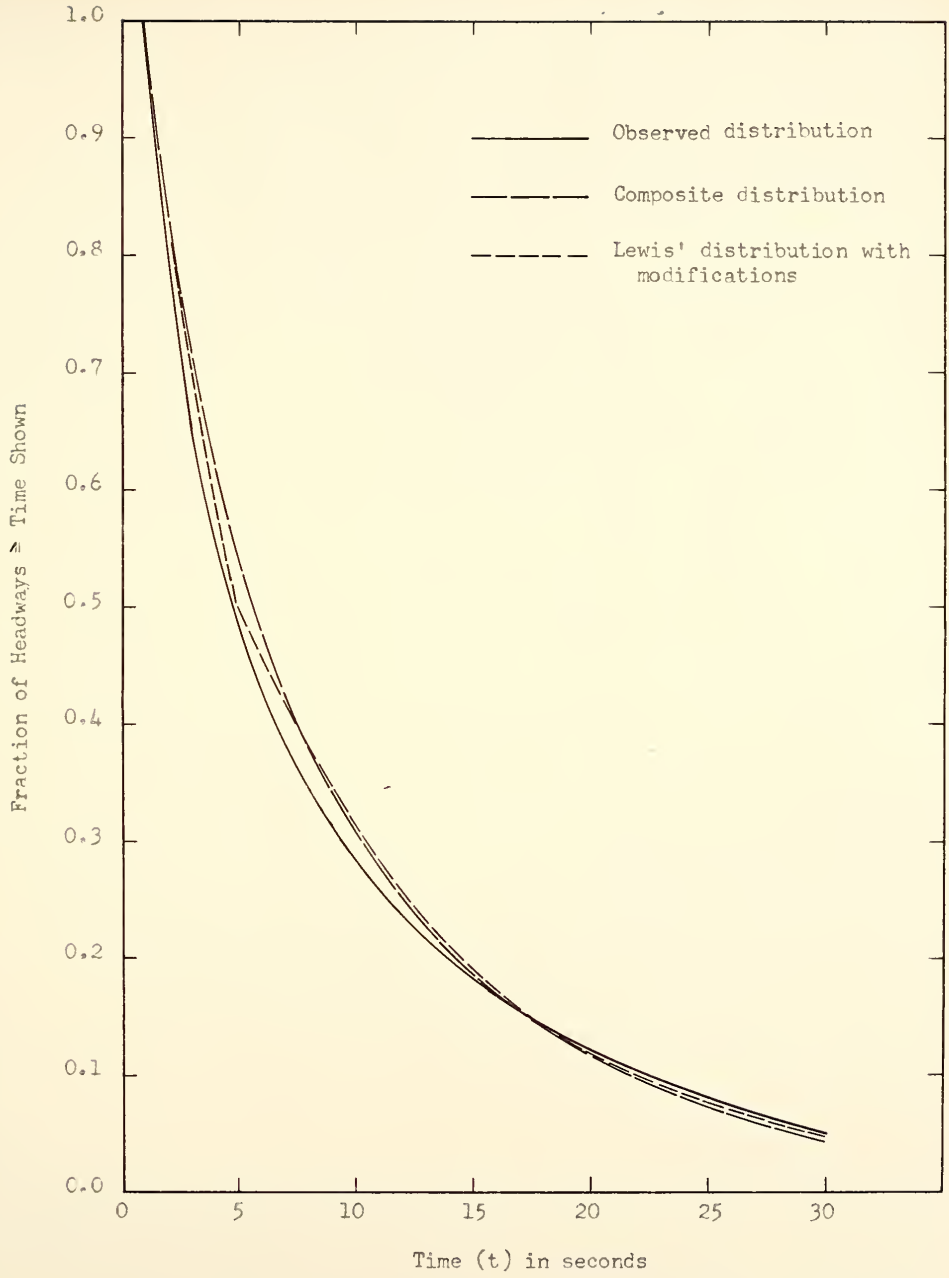


FIGURE 20. HEADWAY DISTRIBUTIONS AT 400 VPH

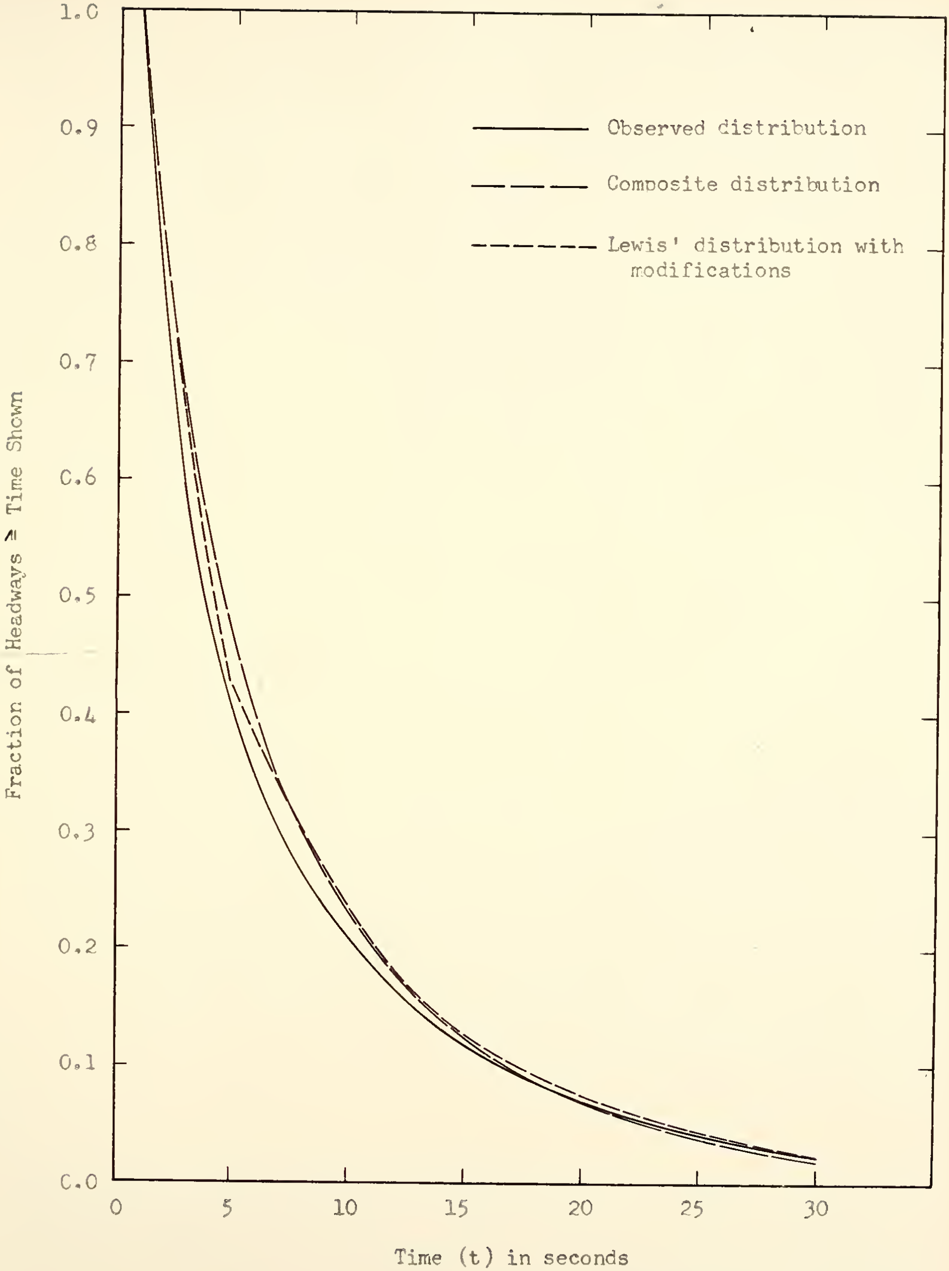


FIGURE 21. HEADWAY DISTRIBUTIONS AT 500 VPH

vehicles by utilizing two levels of probability and forcing a break in the curve at 4 3/4 seconds does bring his curve closer to the field data in this area at a volume level of 300 vehicles per hour and greater. The two theoretical distributions, however, are quite close to the empirical distribution of this study. Both fit quite well and it is doubtful if one can be claimed better than the other.

SUMMARY OF RESULTS

The results which follow are for two-way, two-lane urban arterial streets where free-flowing traffic conditions prevailed, speed limit was 30 miles per hour, and no major traffic generators existed. Three types of arterials were investigated: Type I, arterials with parking on one side; Type II, arterials with parking on both sides; and Type III, arterials with no parking.

1. For all three types of arterials and for average densities over 15-minute time periods of less than 50 vehicles per mile, mean speed decreased slightly as density increased and volume increased substantially. The relationships of speed to density and volume to density appeared for this range of density to be linear.
2. For traffic volumes within the range of 200-1200 vehicles per hour, a slight decrease (approximately 0.1 to 0.4 miles per hour per 100 vehicles) in speed occurred with increasing traffic volumes on all three types of arterials. This relationship for this volume range was essentially linear.
3. The relationship of speed to volume was not consistent on streets of the same type when they were in different cities. For one case where streets of the same type were in the same city, there was a similar relationship of speed to volume.
4. Mean speed for the same volume on the same type of street differed by 1-3 miles per hour when measured at sites in different cities.

Mean speeds differed approximately the same amount from one type of street to another, again with no consistency of difference.

5. When the effects on mean speed of volume of traffic, percent of commercial vehicles, percent of vehicles turning at nearby intersections, and percent of traffic traveling in the major direction were analyzed by multiple regression for the volume range of 350 to 1200 vehicles per hour, volume was found to provide the only significant effect. In this range and under these conditions of considering these other variables, speed decreased by about 0.35 miles per hour for each volume increase of 100 vehicles per hour for all types of streets studied.
6. Although the effect on speed was not significant, increasing percentages of turning movements at nearby intersections and increasing percentages of commercial vehicles consistently had negative regression coefficients.
7. The factors of volume, turning movements, commercial vehicles, and directional distribution as measured in this study, explained in most cases less than 50 percent of the variability in mean speed. This suggests that other factors, probably not characteristics of flow, are of major influence in a driver's selection of vehicle speed.
8. The distribution of headways in the traffic stream was not influenced appreciably by opposing traffic volumes. Slight affects, however, occurred for short headways when opposing traffic volumes were very low. The percentage of vehicles traveling at short headways, in fact, was largest when the volume of opposing traffic was very low (100 vehicles per hour or less).

9. Exponential curves of the type $P = 1-e^{cV}$ (where P is the proportion of headways less than a time increment, c is a constant, e is the natural base of logarithms, and V is the volume at which the proportion was measured) fit well the data on headways collected in this study. This was particularly true for the headways greater than 5 seconds.

10. A comparison with a similar analysis of headway distributions made in the Highway Capacity Manual (3) for rural roads indicated that for the same volume the urban streets studied had fewer vehicles traveling at short headways than did rural roads.

11. The empirical distribution of headways obtained in this study was fitted quite well by the theoretical distribution proposed by Lewis (16) with the modified parameters used in this study and the theoretical distribution proposed by Schul and adapted by Kell (14).

12. The method of data collection, using a constant speed impulse recorder, activated by vehicles passing over pneumatic tubes placed on the roadway at known distances, proved satisfactory for the types of streets and the volumes of traffic studied in this research. It is possible that the air tubes, which had the appearance of a "speed trap", had some effect on speeds and on restraining passing maneuvers in the study sections. It is believed that the measures taken to minimize these effects were successful.

LIST OF REFERENCES

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1. Adams, W. F., "Road Traffic Considered as a Random Series," Journal of the Institution of Civil Engineers, Vol. 4, pp. 121-130, November 1936.

2. Berry, D. S. and Green F. H., "Techniques for Measuring Over-all Speeds in Urban Areas," Proceedings, Highway Research Board, Vol. 20, pp. 311-318, 1949.

3. Committee on Highway Capacity, Highway Capacity Manual, U. S. Dept. of Commerce, Bureau of Public Roads, Washington D. C., 1950.

4. Edie, L. C. and Foote, R. S., "Traffic Flow in Tunnels," Proceedings, Highway Research Board, Vol. 37, pp. 334-344, 1958.

5. Forbes, T. W., "Speed, Headway and Volume Relationships on a Freeway," Proceedings, Institute of Traffic Engineers, pp. 103-126, 1951.

6. Gazis, D. C., Herman, R., and Potts, R. B., "Car-Following Theory of Steady-State Traffic Flow," Operations Research, Vol. 7, No. 4, pp. 499-505, July-Augst 1959.

7. George, H. P., "Measurement and Evaluation of Traffic Congestion," Quality and Theory of Traffic Flow, Bureau of Highway Traffic, Yale University, pp. 41-68, 1961.

8. Greenberg, H., "An Analysis of Traffic Flow," Operations Research, Vol. 7, No. 1, pp. 79-85, January-February 1959.

9. Greenshields, B. D., "A Study of Traffic Capacity," Proceedings, Highway Research Board, Vol. 14, pp. 448-477, 1934.

10. Greenshields, B. D. and Weida, F. M., Statistics with Application to Highway Traffic Analysis, Eno Foundation for Highway Traffic Control, Saugatuck, Conn., 1952.

11. Haight, F. A., "Towards a Unified Theory of Road Traffic," Operations Research, Vol. 6, pp. 813-826, 1958.

12. Haight, F. A., "The Volume Density Relation in the Theory of Road Traffic," Operations Research, Vol. 8, pp. 572-573, 1960.

13. Keefer, L. B., "The Relation Between Speed and Volume on Urban Streets," Paper presented at the 37th Annual Meeting of the Highway Research Board, January 6-10, 1958.

14. Kell, J. H., "A Theory of Traffic Flow on Urban Streets," Proceedings, of the 13th Annual Western Section Meeting of the Institute of Traffic Engineers, San Francisco, California, pp. 66-70, July 17-20, 1960.

15. Kinzer, J. P., "Applications of the Theory of Probability to Problems in Highway Traffic," Proceedings, Institute of Traffic Engineers, pp. 118-123, 1934.

16. Lewis, R. M., "The Simulation of Traffic Flow to Obtain Volume Warrants for the Intersection Control," Doctor of Philosophy Thesis, Purdue University, August 1962.

17. Lighthill, M. J. and Whitham, G. B., "A Theory of Traffic Flow on Long Crowded Roads," Proceedings, Royal Society of London, Series A, Vol. 229, pp. 317-345, 1955.

18. Malo, A. F., Mika, H. S., and Walbridge, V. P., "Traffic Behavior on an Urban Expressway," Traffic Behavior on Freeways, Highway Research Board Bulletin No. 235, pp. 19-37, 1960.

19. Matson, T. M., Smith, W. S., and Hurd, F. W., Traffic Engineering, New York: McGraw-Hill Book Company, Inc., 1955.

20. May, A. D. Jr., "Characteristics of Traffic Flow on Freeways," Proceedings, American Society of Civil Engineers, Vol. 85, No. 4, Journal of the Highway Division, December 1959.

21. May, A. D. Jr. and Wagner, F. A., "Headway Characteristics and Interrelationships of Fundamental Characteristics of Traffic Flow," Proceedings, Highway Research Board, Vol. 39, pp. 524-547, 1960.

22. May, A. D. Jr., "Traffic Characteristics and Phenomena on High Density Controlled Access Facilities," Traffic Engineering, Vol. 31, No. 6, pp. 11-19, 56, March 1961.

23. Maskowitz, K., "Research on Operating Characteristics of Freeways," Proceedings, Institute of Traffic Engineers, pp. 85-110, 1956.

24. Normann, O. K., "Preliminary Results of Highway Capacity Studies," U. S. Dept. of Commerce, Bureau of Public Roads, Washington D. C., Public Roads, Vol. 19, No. 12, pp. 225-ff, February 1939.

25. Normann, O. K., "Results of Highway Capacity Studies," U. S. Dept. of Commerce, Bureau of Public Roads, Washington D. C., Public Roads, Vol. 23, No. 4, pp. 57-ff, June 1942.

26. Normann, O. K. and Walker, W. P., "Highway Capacity: Practical Applications of Research," Public Roads, Vol. 24, No. 10, pp. 201-234, October 1949, Vol. 25, No. 11, pp. 237-277, December 1949.

27. Normann, O. K., "Research to Improve Tomorrow's Traffic," Traffic Engineering, Vol. 29, No. 7, pp. 11-21, April 1959.

28. Palmer, M. R., "The Development of Traffic Congestion," Quality and Theory of Traffic Flow, Bureau of Highway Traffic, Yale University, pp. 105-140, 1961.

29. Schul, A., "The Probability Theory Applied to Distribution of Vehicles on Two-Lane Highways," Poisson and Traffic, Eno Foundation for Highway Traffic Control, Saugatuck, Conn., pp. 59-75, 1955.

30. Underwood, R. T., "Speed, Volume, and Density Relationships," Quality and Theory of Traffic Flow, Bureau of Highway Traffic, Yale University, pp. 141-188, 1961.

31. Walker, W. P., "Speed and Travel Time Measurement in Urban Areas," Traffic Speed and Volume Measurements, Highway Research Board Bulletin No. 156, pp. 27-44, 1957.

32. Wardrop, J. G., "Some Theoretical Aspects of Road Traffic Research," Proceedings, Institution of Civil Engineers, Part II, Vol. 1, pp. 325-362, 1952.

33. Webb, G. M. and Maskowitz, K., "California Freeway Capacity Studies - 1956," Proceedings, Highway Research Board, Vol. 36, pp. 587-642, 1957.

Statistical References

34. Anderson, V. L., Statistical Analysis, Unpublished book, p. 44, September 1962.

35. Burr, I. W., Engineering Statistics and Quality Control, New York: McGraw-Hill Book Company, Inc., 1953.

36. Duncan, A. J., Quality Control and Industrial Statistics, Homewood, Illinois: Richard I. Irwin, Inc., 1959.

37. Ostle, B., Statistics in Research, Ames, Iowa: Iowa State College Press, 1954.

38. Snedecor, G. W., Statistical Methods, Ames, Iowa: Iowa State College Press, 1956.

APPENDIX A

DEFINITION OF STATISTICAL TERMS

Correlation	A statistical method to determine the extent of the relationship between two variables.
Level of Significance	The probability of rejecting a hypothesis when it is true.
Partial regression coefficient	The measure of change in the dependent variable with a unit change in the independent variable, when all other independent variables remain constant.
Population	The whole collection of measurements or elements of a particular characteristic under consideration.
Regression equation	The function that indicates how the average value of one variable for given values of one or more other variables varies with these other variables (36).
Sample	A set of results taken from the population.
Sum of squares	The sum of squares of the deviations of a set of observations from their mean.

NOTATIONS USED

α	Level of significance
b	Partial regression coefficient
d. f.	Degrees of freedom
g	Number of individual regressions included within a general regression equation
i, j, l	Index of summation
k, r, s	Number of independent variables within a regression equation $r < s \leq k$
n	Number of observations within a regression equation
SS	Sum of squares
SP	Sum of the cross products
X_i	ith observation of the X variable
Y_i	ith observation of the Y variable
x_i	X_i minus the mean of the X_i's
y_i	Y_i minus the mean of the Y_i's

STATISTICAL TESTS

Three tests were used in determining the statistical significance of the several regressions. The standard F-test was used to test the significance of each line of regression. To answer the more specific questions pertaining to the significance of increased R^2 due to the addition of a specific variable and whether two separate regressions may be considered equivalent to one regression combining the data, special test techiques were required.

The test used to determine whether R_s^2 for the equation with s independent variables is significantly greater than R_r^2 for r independent variables, where s is greater than r, and r is included in s is given as follows: (34)

$$F = \frac{\frac{R_s^2 - R_r^2}{s - r}}{\frac{1 - R_s^2}{n - s - 1}}$$

The value computed has an F- distribution with (s - r) and (n - s - 1) degrees of freedom. The hypothesis being tested in this case is that there is no increase in true R^2 when additional independent variables are included in the line of regression.

To test whether one general regression line can be used rather than several individual regressions, an extension of the test for the case of a single variable as given by Ostle (37) is necessary. The test procedure used is shown below.

Let k = number of independent variables

$j = 1, \ldots\ldots, k$

g = number of individual regressions with n_i observations in each

$i = 1, \ldots\ldots, g$

$l = 1, \ldots\ldots, n_i$

The sum of the analysis of variance tables for the individual regressions is obtained as follows:

TABLE 5

SUM OF ANALYSIS OF VARIANCE FOR g INDIVIDUAL REGRESSIONS

Source of Variation	d. f.	S.S.
Due to regression	gk	$\Sigma_i \Sigma_j b_{ij} \Sigma_l x_{ijl} y_{ijl}$
Deviations about regression	$\Sigma_i n_i - gk - g$	$\Sigma_i \Sigma_l (Y_{il} - \hat{Y}_{il})^2 = S_1$
Total	$\Sigma_i n_i - g$	$\Sigma_i \Sigma_l y_{il}^2$

Fitting one line of regression to the g groups of data, the analysis of variance table is:

TABLE 6

ANALYSIS OF VARIANCE FOR MULTIPLE LINEAR REGRESSION

Source of Variance	d. f.	S. S.
Due to regression	k	$\Sigma_j b_j \Sigma_i x_i y_i$
Deviations about regression	$\Sigma n_i - k - 1$	$\Sigma_i (Y_i - \hat{Y}_i)^2 = S_t$
Total	$\Sigma n_i - 1$	$\Sigma_i y_i^2$

The test for aggregation is:

$$F = \frac{\dfrac{(S_t - S_1)}{(k+1)(g-1)}}{\dfrac{S_1}{\Sigma n - g(k-1)}}$$

which has an F-distribution with (k-1)(g-1) and $\Sigma n_i - g(k-1)$ degrees of freedom. The hypothesis being tested in this case is that the variability between individual regressions is zero, or more generally, whether one regression line can be used for all of the observations.

APPENDIX B

COMPUTATIONAL PROCEDURES OF THE THEORETICAL DISTRIBUTIONS INVESTIGATED

The derivations of the previously discussed distributions as developed by the authors are not presented here since they are readily available in the references cited. The computational equations and the necessary definitions which were used in the calculation of the curves illustrated in the text are merely presented.

Lewis' Distribution

Since vehicles in a traffic stream have a tendency to form platoons the probability of a vehicle arrival is not constant over the full range of time. Lewis utilizes two levels of probability to account for this platooning effect where a diminished probability of arrivals is incorporated when a platoon no longer exists. The modified binomial distribution used to describe this phenomenon is a discrete distribution. In order to obtain the probability of arrivals at one-half second intervals the time values used in calculating the probability of arrivals are expressed in units of one-half second increments. The definitions of symbols used in the equations to be presented are as follows:

h = headway or time increment considered

$\bar{h}$ = mean of all headways

e = minimum headway permitted

r = the maximum headway for which the probability of an arrival is enhanced

n = any value of h

P_a = the enhanced probability of an arrival at a time increment when $e \leq h \leq r$

$q_a = 1 - P_a$; the probability of no arrivals when $e \leq h \leq r$

P_b = the diminished probability of an arrival at a time increment when $h > r$

$q_b = 1 - P_b$; the probability of no arrivals when $h > r$

K = the fraction of all headways which are $\leq r$

F = volume of traffic in vehicles per hour

The expression derived for the fraction of vehicles at a particular volume level that are traveling at or less than r is as follows:

$$K = 1 - e^{-0.00151F}$$

The derived equations for the two levels of probability are:

$$P_a = 1 - (1-K)^{\frac{1}{r - e + 1}}$$

$$P_b = \frac{1-K}{\bar{h} - e - \sum_{n=1}^{r-e} (1 - P_a)^n}$$

The final equations used to calculate the probability of no arrivals as shown in the text are:

for $e \leq h \leq r$, $\quad P(h \geq n) = q_a^{\,n-e}$

for $h > r$, $\quad P(h \geq n) = q_a^{\,r-e+1} \; q_b^{\,n-r-1}$

Composite Distribution

The composite distribution, originally proposed by Schul, has been simplified by Kell and was used in that form to calculate the curves illustrated in this study.

The equation for the composite exponential distribution is,

$$P\,(h \geq t) = (1 - \alpha)\, e^{-\frac{t - \lambda}{T_1 - \lambda}} + \alpha\, e^{-\frac{t - \tau}{T_2 - \tau}}$$

where: $P\,(h \geq t)$ = the probability of a headway (h) equal to or greater than the time (t)

α = the proportion of the traffic stream in the restrained group

$1 - \alpha$ = the proportion of the traffic stream in the free-moving group

T_1 = average headway of the free-moving vehicles

T_2 = average headway of the restrained vehicles

λ = the minimum headway of the free-moving vehicles

τ = the minimum headway of the restrained vehicles

e = natural base of logarithms

J. H. Kell has rewritten this equation in the form,

$$P\,(h \geq t) = e^{a - \frac{t}{K_1}} + e^{c - \frac{t}{K_2}}$$

where:

$$a = \frac{\lambda}{T_1 - \lambda} + \ln\,(1 - \alpha)$$

$$K_1 = T_1 - \lambda$$

$$C = \frac{\tau}{T_2 - \tau} + \ln\,\alpha$$

$$K_2 = T_2 - \tau$$

The resulting values of these constants obtained by fitting this curve to 585 samples of field data are:

$$a = -0.046 - 0.0448 \left(\frac{V}{100}\right)$$

$$K_1 = e^{8.48 - 1.024 \ln V}$$

$$c = e^{-10.503 + 2.829 (\ln V) - 0.173 (\ln V)^2} - 2.0$$

$$K_2 = 2.659 - 0.120 \left(\frac{V}{100}\right)$$

The equation using these constants is applicable to two-lane urban streets and was used to calculate the curves shown in the text.

APPENDIX C

TABLE 7
Summary of Traffic Flow Data
Site A - 30th Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
3/26/62							
7:30 - 7:45	816	30.58	29.92	27.27		5.9	60.8
7:45 - 8:00	704	30.09	29.19	24.12		5.1	64.8
8:00 - 8:15	516	29.74	29.04	17.76		8.5	62.8
8:15 - 8:30	416	29.74	28.82	14.43		6.7	64.4
8:30 - 8:45	356	30.14	28.90	12.32		9.0	53.9
8:45 - 9:00	356	29.94	29.37	12.12		13.5	57.3
11:30 - 11:45	352	31.58	30.67	11.48		8.0	57.9
11:45 - 12:00	384	31.49	30.58	12.56		11.4	56.2
12:00 - 12:15	408	30.66	29.49	13.84		3.9	60.8
12:15 - 12:30	404	31.55	30.23	13.36		6.9	51.5
12:40 - 12:55	472	31.77	31.07	15.18		7.6	56.8
12:55 - 1:10	372	30.83	29.89	12.44		6.4	52.1
3:00 - 3:15	516	31.43	30.73	16.78		5.4	55.0

TABLE 7 (continued)
Summary of Traffic Flow Data
Site A - 30th Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
3/26/62 continued							
3:15 - 3:30	500	31.50	30.70	16.29		8.0	53.6
3:30 - 3:45	576	30.17	29.04	19.83		6.9	50.0
3:45 - 4:00	656	29.10	28.33	23.15		2.4	50.6
4:00 - 4:15	572	29.56	28.71	19.91		6.3	55.2
4:15 - 4:30	684	30.17	29.29	23.35		4.1	52.6
4:50 - 5:05	912	28.76	28.14	32.40		4.4	57.9
5:05 - 5:20	864	28.57	27.80	31.07		2.8	69.0
5:20 - 5:35	716	29.48	28.45	25.16		5.0	64.2
5:35 - 5:50	616	30.71	29.93	20.58		5.2	70.8
3/27/62							
6:45 - 7:00	448	31.37	30.27	14.80		3.6	75.9
7:00 - 7:15	660	30.56	30.04	21.97		9.1	67.9
7:15 - 7:30	596	31.26	30.86	19.31		8.7	70.5

TABLE 7 (continued)
Summary of Traffic Flow Data
Site A - 30th Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
3/27/62 continued							
7:30 - 7:45	876	30.29	29.55	29.64		4.6	61.2
7:45 - 8:00	716	31.30	30.40	21.96		5.6	62.0
8:00 - 8:15	508	29.82	28.97	17.54		16.5	63.6
10:30 - 10:45	380	31.64	30.68	12.38		12.6	57.9
10:45 - 11:00	372	30.61	29.62	12.69		14.9	60.6
11:00 - 11:15	452	30.26	29.24	15.46		9.7	51.3
11:15 - 11:30	336	31.80	30.94	10.86		9.5	58.3
11:30 - 11:45	376	32.29	31.25	12.03		13.8	58.5
11:45 - 12:00	420	30.37	29.27	14.34		2.8	51.4
2:35 - 2:50	496	30.58	29.70	16.69		9.7	54.0
2:50 - 3:05	480	30.74	29.60	16.22		10.0	55.0
3:05 - 3:20	552	30.79	30.09	18.34		8.0	50.0
3:20 - 3:35	536	30.71	30.11	17.79		6.0	50.0

TABLE 7 (continued)
Summary of Traffic Flow Data
Site A - 30th Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
3/27/62 continued							
3:35 - 3:50	644	30.21	29.22	21.90		5.6	51.5
3:50 - 4:05	568	29.01	27.64	20.55		8.4	55.6
4:20 - 4:35	600	31.07	30.32	19.79		3.3	52.0
4:35 - 4:50	824	29.87	29.21	28.20		1.9	52.9
4:50 - 5:05	920	29.85	29.22	31.48		3.9	64.8
5:05 - 5:20	848	29.94	29.29	28.95		1.9	68.8
5:20 - 5:35	880	29.82	28.80	30.40		2.3	66.4
5:35 - 5:50	628	30.71	29.93	20.31		4.5	61.8

TABLE 8
Summary of Traffic Flow Data
Site B - South Street, Lafayette

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
4/23/62							
1:50 - 2:05	484	29.13	28.59	16.92	8.3	7.4	55.4
2:05 - 2:20	512	29.02	28.18	18.16	7.0	11.7	50.8
2:20 - 2:35	536	28.47	27.82	19.26	8.2	4.5	53.7
2:35 - 2:50	520	28.57	27.90	18.63	6.2	11.5	56.2
2:50 - 3:05	524	28.87	28.14	18.61	9.9	4.6	57.2
3:15 - 3:30	756	25.83	24.81	30.46	11.6	7.4	52.4
4:00 - 4:15	764	26.51	25.47	29.99	17.8	4.2	50.3
4:15 - 4:30	756	27.47	26.84	28.15	14.3	5.8	53.4
4:30 - 4:45	776	27.66	27.00	28.73	11.3	6.7	56.2
4:45 - 5:00	660	28.00	27.57	23.93	13.3	6.1	55.2
5:00 - 5:15	1028	26.87	26.25	39.15	10.5	2.7	52.5
5:15 - 5:30	752	27.67	26.82	28.03	12.8	3.7	60.6

TABLE 8 (continued)

Summary of Traffic Flow Data

Site B - South Street, Lafayette

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
4/24/62							
6:25 - 6:40	328	28.66	28.12	11.66	13.4	7.3	67.1
6:40 - 6:55	356	29.22	28.68	12.41	11.2	5.6	60.7
6:55 - 7:10	224	30.46	29.68	7.54	10.7	14.3	64.3
7:10 - 7:25	344	29.09	27.94	12.31	12.8	7.0	60.5
7:25 - 7:40	612	27.34	26.62	22.98	26.1	4.6	65.4
7:40 - 7:55	896	26.59	25.96	34.50	15.2	1.8	63.8
8:25 - 8:40	352	29.12	28.21	12.47	11.4	10.2	53.4
8:40 - 8:55	444	29.36	28.74	15.44	15.3	9.0	55.8
8:55 - 9:10	368	31.64	30.42	12.09	9.8	9.8	58.7
9:10 - 9:25	412	28.78	27.75	14.84	13.6	9.7	54.4
9:25 - 9:40	420	28.25	27.49	15.27	19.0	5.7	55.2
4/25/62							
1:45 - 2:00	436	28.33	27.63	15.77	7.3	13.8	53.2

TABLE 8 (continued)

Summary of Traffic Flow Data

Site B - South Street, Lafayette

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
4/25/62 continued							
2:00 - 2:15	400	28.34	27.62	14.47	12.0	9.0	58.0
2:15 - 2:30	556	27.97	27.44	20.25	11.5	5.0	52.5
2:40 - 2:55	476	27.71	27.00	17.62	9.2	9.2	50.4
2:55 - 3:10	532	27.16	25.09	21.19	15.8	7.5	58.6
3:10 - 3:25	600	27.03	26.43	22.69	18.7	4.0	56.0
4:00 - 4:15	736	26.68	26.07	28.23	20.6	5.4	51.1
4:15 - 4:30	760	27.43	26.96	28.18	10.5	3.2	56.8
4:30 - 4:45	788	27.63	27.05	29.12	15.7	3.0	55.3
4:45 - 5:00	784	27.91	27.31	28.70	11.7	4.6	56.6
5:00 - 5:15	996	27.11	26.23	37.96	8.4	3.6	50.2
5:15 - 5:30	732	28.84	28.05	26.08	9.8	3.8	59.6
4/26/62							
6:25 - 6:40	252	28.79	28.04	8.98	14.3	9.5	74.6

TABLE 8 (continued)

Summary of Traffic Flow Data

Site B - South Street, Lafayette

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
4/26/62 continued							
6:40 - 6:55	332	2[illegible].50	28.74	11.54	14.4	6.0	56.6
6:55 - 7:10	272	30.65	29.74	9.14	10.3	4.4	58.8
7:10 - 7:25	336	28.27	26.92	12.48	17.8	9.5	57.1
7:25 - 7:40	500	28.41	27.69	18.05	22.4	4.8	68.8
7:40 - 7:55	860	26.63	26.00	33.06	22.3	3.7	68.4

TABLE 9
Summary of Traffic Flow Data
Site C - 9th Street, Lafayette

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
5/15/62							
2:00 - 2:15	328	29.03	28.22	11.61	14.6	7.3	54.8
2:15 - 2:30	332	29.36	28.65	11.58	19.3	4.8	53.0
2:30 - 2:45	440	28.69	27.64	15.91	12.7	8.2	52.7
2:45 - 3:00	532	29.02	28.41	18.72	17.3	2.3	52.6
3:00 - 3:15	772	27.80	27.19	28.39	20.2	5.7	54.4
4:00 - 4:15	700	29.71	29.19	23.97	15.4	1.1	58.3
4:15 - 4:30	664	27.98	27.11	24.49	28.9	3.6	50.6
4:30 - 4:45	816	27.75	27.12	30.08	26.5	4.9	61.3
4:45 - 5:00	628	28.09	27.43	22.89	26.1	1.9	51.0
5:00 - 5:15	908	27.18	26.43	34.34	25.6	1.8	72.7
5:15 - 5:30	872	27.82	27.24	32.00	22.0	0.5	75.7
5/28/62							
1:45 - 2:00	444	29.10	28.14	15.77	20.7	4.5	54.1

TABLE 9 (continued)
Summary of Traffic Flow Data
Site C - 9th Street, Lafayette

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
5/28/62 continued							
2:00 - 2:15	456	29.66	28.81	15.82	10.5	3.5	57.9
2:15 - 2:30	436	30.52	29.82	14.61	11.0	3.7	50.4
2:30 - 2:45	500	29.49	28.59	17.48	16.8	7.2	52.8
2:45 - 3:00	608	29.66	29.05	20.92	10.5	5.9	55.9
3:00 - 3:15	728	29.06	28.55	25.49	11.5	4.9	52.2
4:00 - 4:15	768	30.01	29.75	26.16	18.2	1.0	53.6
4:15 - 4:30	628	29.91	29.35	21.39	21.7	1.9	58.0
4:30 - 4:45	804	28.60	27.99	28.71	12.4	2.5	50.2
4:45 - 5:00	820	29.34	28.77	28.49	21.5	0.5	62.4
5:00 - 5:15	840	28.43	27.87	30.13	14.3	2.9	69.5
5:15 - 5:30	856	28.78	28.22	30.33	15.9	2.8	68.2
5/29/62							
6:25 - 6:40	328	30.64	29.22	11.22	24.4	2.4	69.5

TABLE 9 (continued)
Summary of Traffic Flow Data
Site C - 9th Street, Lafayette

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
5/29/62 continued							
6:40 - 6:55	340	29.80	29.14	11.66	23.5	1.2	77.6
6:55 - 7:10	200	29.85	29.03	6.88	26.0	8.0	68.0
7:10 - 7:25	368	29.60	28.92	12.72	21.7	9.8	77.2
7:25 - 7:40	664	27.84	27.38	24.24	23.5	6.0	72.9
7:40 - 7:55	1092	27.11	26.57	41.09	19.8	4.4	70.7
8:30 - 8:45	480	30.38	29.88	16.05	17.5	3.3	58.3
8:45 - 9:00	452	28.47	27.77	16.27	17.7	3.5	59.3
9:00 - 9:15	464	28.75	27.86	16.64	14.7	5.2	63.8
9:15 - 9:30	428	30.46	29.53	14.49	17.8	4.7	53.3
9:30 - 9:45	356	29.61	28.87	12.32	22.5	5.6	55.1
9:45 - 10:00	392	29.61	28.88	13.57	11.2	5.1	53.1
5/31/62							
6:25 - 6:40	312	29.73	28.89	10.79	20.5	5.1	78.2

TABLE 9 (continued)

Summary of Traffic Flow Data

Site C - 9th Street, Lafayette

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
5/31/62 continued							
6:40 - 6:55	424	30.64	29.89	14.18	24.5	1.9	72.6
6:55 - 7:10	264	29.83	28.94	9.12	30.3	6.1	62.1
7:10 - 7:25	320	30.32	29.57	10.82	20.0	7.5	68.8
7:25 - 7:40	516	29.61	28.94	17.82	13.2	3.1	69.0
7:40 - 7:55	948	28.20	27.70	34.21	12.7	3.4	78.1
8:30 - 8:45	456	27.83	27.13	16.80	13.2	4.4	57.0
8:45 - 9:00	636	29.42	28.86	22.03	8.2	5.7	59.1
9:00 - 9:15	472	28.88	28.40	16.61	10.2	5.9	59.3
9:15 - 9:30	432	29.21	28.57	15.11	10.2	10.2	66.7
9:30 - 9:45	440	28.29	27.53	15.98	13.6	5.5	53.6
9:45 - 10:00	492	28.10	27.37	17.96	13.0	3.3	50.4

TABLE 10
Summary of Traffic Flow Data
Site D - Washington Boulevard, Indianapolis

Time Period	Equiv. Hourly Volume (vph.)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/11/62							
7:00 - 7:15	360	32.77	32.24	11.16		0.0	84.4
7:15 - 7:30	556	32.26	31.97	17.38		0.7	85.6
7:30 - 7:45	876	31.00	30.77	28.46		0.0	85.4
7:45 - 8:00	652	31.96	31.68	20.57		0.0	81.6
8:00 - 8:15	688	31.27	31.00	22.18		0.0	76.7
8:15 - 8:30	528	30.74	30.25	17.44		0.0	86.9
6/12/62							
7:00 - 7:15	332	34.03	33.50	9.90		0.0	79.5
7:15 - 7:30	600	33.02	32.73	18.32		0.7	86.6
7:30 - 7:45	812	31.81	31.39	25.86		0.5	84.7
7:45 - 8:00	692	30.90	30.28	22.84		0.6	84.4
8:00 - 8:15	652	32.35	31.88	20.44		0.6	75.5
8:15 - 8:30	444	30.47	29.09	15.26		0.8	84.3

TABLE 10 (continued)
Summary of Traffic Flow Data
Site D - Washington Boulevard, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/12/62 continued							
9:30 - 9:45	224	31.09	30.68	7.29		1.8	55.4
9:45 - 10:00	300	30.35	29.64	10.11		1.3	53.3
10:00 - 10:15	252	31.02	30.34	8.30		1.6	66.7
10:15 - 10:30	244	30.85	30.52	7.99		0.0	52.5
10:30 - 10:45	248	30.85	30.27	8.19		0.0	51.6
10:45 - 11:00	224	31.53	30.98	7.22		0.0	51.8
2:00 - 2:15	320	31.38	30.96	10.33		2.5	62.5
2:15 - 2:30	316	31.47	30.97	10.20		1.3	57.0
2:30 - 2:45	240	31.55	31.16	7.69		1.7	56.7
2:45 - 3:00	384	31.90	31.33	12.25		1.0	60.4
3:00 - 3:15	360	31.61	31.18	11.54		0.0	67.8
3:15 - 3:30	312	32.07	31.65	9.85		0.0	66.7
4:00 - 4:15	460	31.60	31.22	14.73		0.9	70.4

TABLE 10 (continued)

Summary of Traffic Flow Data

Site D - Washington Boulevard, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/12/62 continued							
4:15 - 4:30	512	31.73	31.20	16.40		0.0	77.3
4:30 - 4:45	624	32.20	31.90	19.55		0.0	67.9
4:45 - 5:00	680	31.47	31.17	21.80		0.0	81.8
5:00 - 5:15	840	30.26	29.99	28.00		0.0	88.6
5:15 - 5:30	776	30.55	30.21	25.68		0.5	82.0
6/13/62							
9:30 - 9:45	212	32.49	31.99	6.62		1.9	62.3
9:45 - 10:00	248	32.96	32.36	7.66		3.2	61.3
10:00 - 10:15	256	33.22	32.85	7.79		1.6	59.4
10:15 - 10:30	232	33.42	32.92	7.04		0.0	50.0
10:30 - 10:45	224	33.12	32.64	6.86		1.8	51.8
10:45 - 11:00	268	32.67	32.19	8.32		1.5	52.2
2:00 - 2:15	216	30.91	30.47	7.08		1.9	55.6

TABLE 10 (continued)

Summary of Traffic Flow Data

Site D - Washington Boulevard, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/13/62 continued							
2:15 - 2:30	272	32.41	31.70	8.57		1.5	72.1
2:30 - 2:45	308	31.45	30.82	9.99		2.6	67.5
2:45 - 3:00	280	31.33	30.92	9.05		1.4	68.6
3:00 - 3:15	280	31.48	31.06	9.01		0.0	67.1
3:15 - 3:30	316	32.43	32.04	9.86		1.3	70.9
3:55 - 4:10	392	31.81	31.51	12.44		2.0	75.5
4:10 - 4:25	448	31.83	31.52	14.20		1.8	72.3
4:25 - 4:40	580	31.03	30.59	18.95		1.4	71.7
4:40 - 4:55	756	29.88	29.48	25.63		0.5	76.2

TABLE 11
Summary of Traffic Flow Data
Site E - 21st Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/5/62							
6:50 - 7:05	584	31.49	30.94	18.87	1.4	4.8	57.5
7:05 - 7:20	964	31.10	30.56	31.53	1.2	3.7	54.4
7:20 - 7:35	1064	31.26	30.80	34.54	2.3	4.1	59.8
9:30 - 9:45	264	31.73	30.63	8.61	1.5	12.1	54.5
9:45 - 10:00	324	30.85	29.89	10.83	4.9	11.1	54.3
10:00 - 10:15	236	31.47	30.59	7.71	3.4	10.2	54.2
10:15 - 10:30	244	31.20	30.04	8.12	4.9	13.1	54.1
10:30 - 10:45	304	31.29	30.86	9.84	0.0	11.8	56.6
10:45 - 11:00	336	32.01	31.01	10.83	1.2	11.9	54.7
2:00 - 2:15	428	32.41	31.63	13.52	1.9	11.2	51.4
2:15 - 2:30	460	31.16	30.36	15.15	1.7	10.4	50.4
2:30 - 2:45	524	33.98	33.30	15.73	0.0	7.6	57.3
2:45 - 3:00	504	32.65	31.92	15.78	0.8	8.7	58.7

TABLE 11 (continued)

Summary of Traffic Flow Data

Site E - 21st Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/5/62 continued							
3:00 - 3:15	664	32.91	32.17	20.63	0.6	6.0	56.6
3:15 - 3:30	608	31.82	31.37	19.37	0.7	6.6	50.0
4:00 - 4:15	872	30.76	30.09	28.97	1.4	1.8	72.9
4:15 - 4:30	532	31.89	31.42	16.92	2.3	6.8	54.1
4:30 - 4:45	904	31.33	30.79	29.35	1.8	4.9	60.6
4:45 - 5:00	880	31.43	30.92	28.45	0.9	0.5	54.5
5:00 - 5:15	1208	30.13	29.36	41.13	1.3	1.7	65.6
5:15 - 5:30	928	32.04	31.33	29.61	2.2	2.6	62.5
6/6/62							
7:00 - 7:15	912	31.72	31.33	29.10	0.9	3.5	59.6
7:15 - 7:30	1016	31.36	30.83	32.94	2.4	3.9	51.6
7:30 - 7:45	1044	30.13	27.49	37.97	2.3	4.2	65.9
7:45 - 8:00	860	31.32	30.54	28.15	2.3	2.8	55.3

TABLE 11 (continued)
Summary of Traffic Flow Data
Site E - 21st Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/6/62 continued							
8:00 - 8:15	524	32.03	31.40	16.68	0.8	9.9	71.0
9:30 - 9:45	260	33.36	32.72	7.94	1.5	12.3	50.8
9:45 - 10:00	268	32.64	31.79	8.42	4.5	9.0	55.2
10:00 - 10:15	296	32.25	31.60	9.36	4.1	11.9	50.0
10:15 - 10:30	224	31.31	30.79	7.27	1.8	11.9	53.6
10:30 - 10:45	344	31.05	30.10	11.42	0.0	7.0	51.2
10:45 - 11:00	332	31.56	30.80	10.77	1.2	15.7	56.6
2:00 - 2:15	432	32.01	31.12	13.87	2.8	10.2	53.7
2:15 - 2:30	436	32.35	31.65	13.77	0.9	11.9	57.8
2:30 - 2:45	452	32.55	31.72	14.24	2.7	7.1	52.2
2:45 - 3:00	516	33.17	32.55	15.84	0.8	9.3	55.0
3:00 - 3:15	564	33.35	32.59	17.30	0.7	5.0	51.8
3:15 - 3:30	584	30.98	30.00	19.46	1.4	8.2	50.7

TABLE 11 (continued)

Summary of Traffic Flow Data

Site E - 21st Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/6/62 continued							
4:00 - 4:15	860	31.32	30.73	27.97	1.4	3.3	72.1
4:15 - 4:30	628	32.27	31.73	19.78	1.9	4.5	62.4
4:30 - 4:45	840	31.71	31.31	26.82	2.9	3.3	65.2
4:45 - 5:00	996	29.91	28.76	34.62	2.4	4.4	54.6
5:00 - 5:15	1172	30.23	29.68	39.48	1.4	3.1	67.2
5:15 - 5:30	936	30.65	29.84	31.35	2.6	3.0	61.1

TABLE 12
Summary of Traffic Flow Data
Site F - Arlington Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/7/62							
6:55 - 7:10	468	32.69	32.01	14.61	17.1	5.1	68.4
7:10 - 7:25	552	31.63	31.08	17.75	12.3	6.5	63.8
7:25 - 7:40	740	31.56	31.18	23.72	17.3	2.2	68.6
7:45 - 8:00	732	31.43	30.87	23.70	9.8	5.5	55.2
8:00 - 8:15	648	32.43	31.86	20.33	4.3	8.6	51.2
8:15 - 8:30	376	32.65	31.89	11.79	5.3	13.8	66.0
9:30 - 9:45	380	33.27	32.63	11.64	7.3	11.5	53.1
9:45 - 10:00	376	32.64	32.05	11.73	6.4	11.7	52.1
10:00 - 10:15	384	32.39	31.78	12.08	9.4	13.5	54.2
10:15 - 10:30	400	31.72	31.18	12.82	2.0	11.0	54.0
10:30 - 10:45	440	30.90	30.38	14.47	7.3	10.0	50.9
10:45 - 11:00	408	30.75	30.00	13.59	7.8	7.8	54.9
2:00 - 2:15	528	30.70	30.01	17.58	7.6	4.6	51.9

TABLE 12 (continued)
Summary of Traffic Flow Data
Site F - Arlington Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/7/62 continued							
2:15 - 2:30	444	30.37	29.91	14.84	9.9	12.6	55.9
2:30 - 2:45	528	31.33	30.25	17.46	3.9	12.9	50.0
2:45 - 3:00	592	30.43	29.73	19.90	4.7	9.5	51.4
3:00 - 3:15	596	30.04	29.35	20.30	6.0	7.4	51.6
3:15 - 3:30	620	30.84	30.09	20.60	10.3	7.1	51.6
4:00 - 4:15	788	31.43	30.81	25.56	7.1	2.5	56.3
4:15 - 4:30	820	30.12	28.96	28.31	6.3	2.9	58.5
4:30 - 4:45	740	31.31	30.87	23.96	9.2	2.7	50.2
4:45 - 5:00	784	29.61	28.94	27.08	8.7	3.1	64.7
5:00 - 5:15	920	30.53	30.17	30.48	9.1	3.0	56.0
5:15 - 5:30	892	30.33	29.91	29.82	11.7	0.9	61.4
6/8/62							
6:55 - 7:10	480	31.31	30.64	15.66	13.3	5.0	75.0

TABLE 12 (continued)

Summary of Traffic Flow Data

Site F - Arlington Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph.)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm.)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/8/62 continued							
7:10 - 7:25	516	31.23	30.48	18.89	18.1	6.2	65.9
7:25 - 7:40	760	30.22	29.77	25.52	13.1	1.6	72.1
7:45 - 8:00	780	29.26	28.75	27.12	11.8	10.8	54.8
8:00 - 8:15	544	30.84	30.37	17.90	8.1	10.3	56.6
8:15 - 8:30	368	31.02	30.32	12.13	5.4	9.8	60.0
9:25 - 9:40	376	32.80	32.18	11.68	11.7	8.5	57.4
9:45 - 9:55	468	31.13	30.29	14.12	7.5	11.2	52.3
9:55 - 10:10	400	30.27	29.48	13.56	15.0	9.0	60.0
10:10 - 10:25	412	30.98	30.41	13.54	7.8	6.8	62.1
10:25 - 10:40	472	30.67	29.99	15.73	10.2	10.2	55.9
10:40 - 10:55	452	31.45	30.99	14.58	8.8	15.0	52.2
1:50 - 2:05	448	31.22	30.71	14.58	13.4	6.2	54.5
2:05 - 2:20	456	31.67	31.03	14.69	4.4	4.4	52.6

TABLE 12 (continued)
Summary of Traffic Flow Data
Site F - Arlington Street, Indianapolis

Time Period	Equiv. Hourly Volume (vph)	Time-mean Speed (mph)	Space-mean Speed (mph)	Ave. Density (vpm)	Turns (%)	Comm. Veh. (%)	Dist. by Dir. (%)
6/8/62 continued							
2:20 - 2:35	516	31.44	30.83	16.73	7.7	7.7	57.4
2:35 - 2:50	708	28.80	26.70	26.52	9.0	12.4	52.0
2:50 - 3:05	508	29.81	29.21	17.38	11.8	4.7	50.4
4:00 - 4:15	732	31.16	30.64	23.88	8.2	3.8	54.1
4:15 - 4:30	712	30.82	30.46	23.37	12.4	3.9	64.6
4:30 - 4:45	672	31.24	30.78	21.83	13.1	3.6	61.9
4:45 - 5:00	772	29.13	28.31	27.26	10.9	3.1	57.5
5:00 - 5:15	908	30.12	29.61	30.66	14.5	1.3	66.1
5:15 - 5:30	924	30.62	30.13	30.66	8.2	1.3	60.2

TABLE 13
Observed Headway Distributions for Opposing Volume of 100 vph
Percent Headways Less than Time Shown

Observed Volume (vph)	Time in seconds						
	2.0	3.0	4.0	5.0	10.0	20.0	30.0
146	12.3	20.5	26.0	28.8	39.7	54.8	68.5
120	3.3	8.3	15.0	20.0	33.3	48.3	63.3
148	5.4	14.9	17.6	21.6	39.2	55.4	68.9
120	8.3	13.3	16.7	18.3	33.3	51.7	63.3
188	6.4	9.6	17.0	23.4	53.2	64.9	77.6
176	9.1	15.9	26.1	28.4	45.4	60.2	72.7
200	18.0	27.0	35.0	37.0	52.0	71.0	74.0
206	16.5	29.1	38.8	41.7	53.4	67.0	78.6
276	10.1	16.7	24.7	33.3	56.2	76.1	91.3
310	32.3	46.5	55.5	59.4	71.0	81.3	86.5
392	31.6	43.9	53.6	58.2	75.0	86.2	90.8
390	28.2	44.1	50.8	58.5	75.4	86.7	92.3
360	35.6	49.4	52.8	57.8	71.7	84.4	92.2
476	37.0	50.0	57.6	63.0	76.5	88.6	93.7

TABLE 13 (continued)

Observed Headway Distributions for Opposing Volume of 100 vph

Percent Headways Less than Time Shown

Observed Volume (vph)	Time in seconds						
	2.0	3.0	4.0	5.0	10.0	20.0	30.0
452	34.1	48.2	54.9	64.1	75.7	88.0	93.8

TABLE 14
Observed Headway Distributions for Opposing Volume of 200 vph
Percent Headways Less than Time Shown

Observed Volume (vph)	Time in seconds						
	2.0	3.0	4.0	5.0	10.0	20.0	30.0
94	2.1	10.6	12.8	19.1	25.5	44.7	48.9
92	6.5	6.5	10.9	13.0	21.7	41.3	52.2
104	3.8	5.8	9.7	17.3	34.6	46.2	61.5
128	7.8	7.8	9.4	12.5	26.6	57.6	73.4
180	4.4	11.1	15.6	17.8	36.7	63.3	76.7
186	5.4	9.7	16.1	21.5	40.9	65.6	76.3
218	9.2	14.7	19.2	24.8	47.7	69.7	85.3
192	8.3	18.8	27.1	34.4	46.9	63.5	76.0
274	9.6	18.4	25.0	29.4	56.6	75.7	93.4
342	10.6	24.7	32.4	36.5	63.5	82.9	92.9
298	14.8	22.1	31.5	36.2	58.4	78.5	89.9
262	11.5	22.9	31.3	34.4	53.4	76.3	84.0
386	19.7	31.6	37.8	44.6	68.9	88.6	94.8
368	16.3	29.9	39.7	47.8	71.2	88.6	93.5

TABLE 14 (continued)
Observed Headway Distributions for Opposing Volume of 200 vph
Percent Headways Less than Time Shown

Observed Volume (vph)	Time in seconds						
	2.0	3.0	4.0	5.0	10.0	20.0	30.0
448	21.9	35.7	42.0	47.3	73.7	92.4	97.3
514	22.6	47.1	56.4	62.2	78.2	92.2	97.3
460	18.8	38.0	48.5	56.8	78.6	89.1	94.8
490	32.2	57.6	66.5	71.8	82.9	88.6	91.8
496	41.1	57.3	66.9	71.0	81.9	87.5	92.3

TABLE 15
Observed Headway Distributions for Opposing Volume of 300 vph
Percent Headways Less than Time Shown

Observed Volume (vph)	Time in seconds						
	2.0	3.0	4.0	5.0	10.0	20.0	30.0
110	9.1	10.9	16.4	16.4	32.7	49.1	58.2
92	4.3	13.0	19.6	21.7	26.1	45.6	63.0
216	12.0	18.5	23.1	27.8	47.2	64.8	84.3
224	8.0	19.6	25.9	32.1	47.3	73.2	83.0
190	4.2	12.6	17.9	21.0	43.2	66.3	76.8
204	9.9	18.8	23.8	25.7	48.5	70.3	85.1
284	9.2	23.2	33.8	39.4	59.2	75.4	90.1
294	9.5	20.4	29.9	35.4	63.9	83.0	89.8
276	17.4	28.3	37.7	43.5	62.3	80.4	87.0
314	12.1	29.9	40.8	45.9	66.2	81.5	87.4
368	13.6	23.9	33.7	42.4	68.0	88.0	94.6
368	13.0	27.7	35.9	41.3	64.7	87.5	95.6
360	11.7	22.8	34.4	40.0	64.4	86.7	93.3
430	19.5	41.4	53.5	60.5	74.4	87.0	94.4

TABLE 15 (continued)
Observed Headway Distributions for Opposing Volume of 300 vph
Percent Headways Less than Time Shown

Observed Volume (vph)	Time in seconds						
	2.0	3.0	4.0	5.0	10.0	20.0	30.0
478	19.2	37.2	44.8	52.7	75.3	91.2	96.2
486	13.6	33.7	46.1	56.0	78.6	92.2	97.1
484	27.7	36.8	45.9	51.7	74.4	91.7	97.1
462	16.0	34.6	43.3	50.6	74.9	91.3	97.0

TABLE 16
Observed Headway Distributions for Opposing Volume of 400 vph
Percent Headways Less than Time Shown

Observed Volume (vph)	Time in seconds						
	2.0	3.0	4.0	5.0	10.0	20.0	30.0
126	17.5	22.2	23.8	27.0	36.5	55.5	68.3
74	13.5	18.9	27.0	27.0	32.4	40.5	43.2
68	11.8	11.8	14.7	20.6	38.2	41.2	52.9
218	7.3	15.6	28.4	31.2	51.4	72.5	84.4
226	13.3	24.8	30.1	35.4	52.2	73.4	83.2
186	6.4	18.3	22.6	23.6	41.9	64.5	72.0
308	11.7	24.0	29.9	35.7	55.8	81.8	90.3
292	13.0	24.0	31.5	38.4	61.6	78.1	89.7
322	13.0	34.2	41.0	46.0	64.0	82.6	91.3
294	18.4	32.6	42.2	44.2	60.5	85.0	92.5
352	10.8	26.7	34.7	42.0	65.3	83.0	93.2
372	9.2	26.5	41.6	50.3	68.1	86.5	92.4
396	11.1	25.8	37.4	43.9	67.2	88.9	97.0
388	10.4	25.9	38.3	46.1	71.5	88.6	92.7

TABLE 16 (continued)
Observed Headway Distributions for Opposing Volume of 400 vph
Percent Headways Less than Time Shown

Observed Volume (vph)	Time in seconds						
	2.0	3.0	4.0	5.0	10.0	20.0	30.0
514	22.2	47.9	59.5	64.6	81.3	91.8	95.3
458	9.2	23.6	34.1	44.5	72.9	92.1	98.7
532	18.4	39.8	51.5	61.3	78.9	92.5	99.2

TABLE 17
Observed Headway Distributions for Opposing Volume of 500 vph
Percent Headways Less than Time Shown

Observed Volume (vph)	Time in seconds						
	2.0	3.0	4.0	5.0	10.0	20.0	30.0
112	17.8	23.2	30.3	33.9	50.0	58.9	60.7
116	20.7	24.1	29.3	34.5	46.6	55.2	58.6
162	19.8	23.5	30.9	34.6	45.7	65.4	72.8
172	10.5	18.6	27.9	34.9	46.5	62.8	74.4
230	7.0	12.2	18.3	22.6	46.1	69.6	84.3
242	5.8	20.8	28.3	34.2	48.3	74.2	84.2
270	8.9	17.8	23.7	30.4	51.1	77.8	88.9
342	12.9	24.0	32.2	39.8	63.7	86.5	94.2
258	16.3	25.6	34.1	39.5	60.5	76.0	84.5
268	15.7	25.4	31.3	38.8	50.7	79.1	90.3
354	16.4	27.7	37.3	46.3	62.1	87.0	93.2
374	11.2	23.0	31.6	39.0	68.4	89.8	95.7
378	16.4	34.4	40.7	46.0	68.3	83.6	95.2

TABLE 17 (continued)
Observed Headway Distributions for Opposing Volume of 500 vph
Percent Headways Less than Time Shown

Observed Volume (vph)	Time in seconds						
	2.0	3.0	4.0	5.0	10.0	20.0	30.0
538	26.8	47.2	56.9	63.6	80.7	93.3	96.7
476	17.6	40.8	49.6	58.4	75.6	90.8	97.1

TABLE 18
Calculated Values of c for the Exponential Curve
Fitted at Various Levels of Opposing Volume

Curve fitted to fraction of headways less than time shown (seconds)	Opposing volume (vph)				
	100	200	300	400	500
2.0	-0.000907	-0.000588	-0.000439	-0.000395	-0.000508
3.0	-0.001459	-0.001158	-0.000941	-0.000968	-0.001033
4.0	-0.001854	-0.001541	-0.001333	-0.001392	-0.001398
5.0	-0.002201	-0.001838	-0.001643	-0.001716	-0.001750
10.0	-0.003401	-0.003174	-0.003079	-0.003065	-0.003038
20.0	-0.005062	-0.005097	-0.005350	-0.005236	-0.005440
30.0	-0.006670	-0.006993	-0.007492	-0.007893	-0.007605

TABLE 19

Observed Headway Distributions on Two-Lane, Two-Way, Urban Arterial Streets
Percent Headways Less than Time Shown

Observed Volume (vph)	Time (seconds)									
	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
94	0.0	2.1	2.1	10.6	12.8	19.1	19.1	23.4	23.4	25.5
120	0.0	1.7	3.3	8.3	15.0	20.0	23.3	25.0	30.0	31.7
110	0.0	1.8	9.1	10.9	16.4	16.4	18.2	21.8	23.6	29.1
68	5.9	8.8	11.8	11.8	14.7	20.6	32.4	35.3	35.3	35.3
74	2.7	10.8	13.5	18.9	27.0	27.0	27.0	27.0	29.7	32.4
104	0.0	3.8	3.8	5.8	9.7	17.3	23.1	23.1	26.9	32.7
224	0.0	1.8	8.0	19.6	25.9	32.1	36.6	39.3	42.0	44.6
162	0.0	12.3	19.8	23.5	30.9	34.6	37.0	38.3	40.7	42.0
186	1.1	5.4	5.4	9.7	16.1	21.5	25.8	29.0	33.3	38.7
230	0.0	2.6	7.0	12.2	18.3	22.6	31.3	33.9	37.4	41.7
206	1.9	7.8	16.5	29.1	38.8	41.7	44.7	46.6	46.6	47.6
218	0.9	3.7	9.2	14.7	19.2	24.8	32.1	36.7	41.3	46.8
294	2.0	6.8	9.5	20.4	29.9	35.4	40.8	46.3	53.1	57.8
292	0.0	4.1	13.0	24.0	31.5	38.4	46.6	49.3	54.1	57.5
276	1.4	8.0	10.1	16.7	24.7	33.3	39.8	47.1	50.7	52.9

TABLE 19 (continued)
Observed Headway Distribution on Two-Lane, Two-Way, Urban Arterial Streets
Percent Headways Less than Time Shown

Observed Volume (vph)	Time (seconds)								
	10.0	12.0	14.0	16.0	18.0	20.0	25.0	30.0	40.0
94	25.5	29.8	34.0	36.2	40.4	44.7	44.7	48.9	61.7
120	33.3	38.3	41.7	43.3	46.7	48.3	55.0	63.3	70.0
110	32.7	36.4	45.5	49.1	49.1	49.1	49.1	58.2	70.9
68	38.2	38.2	41.2	41.2	41.2	41.2	52.9	52.9	55.9
74	32.4	35.1	37.8	37.8	40.5	40.5	43.2	43.2	45.9
104	34.6	38.5	38.5	38.5	44.2	46.2	55.8	61.5	73.1
224	47.3	56.2	62.5	66.1	70.5	73.2	78.6	83.0	87.5
162	45.7	48.1	51.8	56.8	58.0	65.4	69.1	72.8	79.0
186	40.9	44.1	49.5	59.1	62.4	65.6	72.0	76.3	87.1
230	46.1	53.0	58.3	60.0	63.5	69.6	78.3	84.3	93.0
206	53.4	59.2	64.1	64.1	65.0	67.0	71.8	78.6	84.5
218	47.7	52.3	56.0	59.6	62.4	69.7	77.9	85.3	89.9
294	63.9	68.7	72.8	77.6	80.3	83.0	86.4	89.8	95.2
292	61.6	68.5	72.6	76.0	77.4	78.1	87.7	89.7	93.8
276	56.5	60.9	66.7	68.1	71.7	76.1	83.3	91.3	96.4

TABLE 19 (continued)

Observed Headway Distributions on Two-Lane, Two-Way, Urban Arterial Streets
Percent Headways Less than Time Shown

Observed Volume (vph)	Time (seconds)									
	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
310	2.6	14.8	32.3	46.5	55.5	59.4	61.9	63.9	66.5	70.3
268	0.0	4.5	15.7	25.4	31.3	38.8	41.8	43.3	46.3	50.0
322	0.6	6.8	13.0	34.2	41.0	46.0	49.1	54.6	59.0	62.1
372	0.0	2.7	9.2	26.5	41.6	50.3	57.3	60.5	63.8	65.4
386	1.0	10.4	19.7	31.6	37.8	44.6	50.8	55.4	62.2	66.8
352	0.6	4.5	10.8	26.7	34.7	42.0	49.4	56.3	59.1	60.8
388	0.5	4.1	10.4	25.9	38.3	46.1	53.4	61.6	64.2	67.4
360	0.0	3.3	11.7	22.8	34.4	40.0	47.2	51.7	57.2	61.1
396	0.0	3.0	11.1	25.8	37.4	43.9	50.0	55.1	63.1	65.7
514	1.6	9.7	22.2	47.9	59.5	64.6	68.9	71.2	75.5	78.2
476	1.7	19.7	37.0	50.0	57.6	63.0	66.8	71.8	73.9	74.8
452	4.4	21.2	34.1	48.2	54.9	64.1	66.8	70.8	71.7	74.3
462	0.4	6.5	16.0	34.6	43.3	50.6	57.6	64.5	71.0	73.2
476	0.8	8.0	17.6	40.8	49.6	58.4	63.4	66.8	69.7	72.3
532	0.8	7.1	18.4	39.8	51.5	61.3	65.4	70.3	72.9	76.7

TABLE 19 (continued)
Observed Headway Distribution on Two-Lane, Two-Way, Urban Arterial Streets
Percent Headways Less than Time Shown

Observed Volume (vph)	Time (seconds)								
	10.0	12.0	14.0	16.0	18.0	20.0	25.0	30.0	40.0
310	71.0	73.5	74.8	76.1	78.7	81.3	84.5	86.5	89.7
268	50.7	59.0	63.4	68.6	74.6	79.1	85.1	90.3	93.3
322	64.0	68.9	73.9	77.0	80.7	82.6	88.2	91.3	94.4
372	68.1	74.0	77.8	82.2	84.9	86.5	90.8	92.4	96.2
386	68.9	75.1	78.8	83.4	86.0	88.6	90.2	94.8	96.9
352	65.3	71.6	76.7	80.1	81.8	83.0	91.2	92.7	96.9
388	71.5	76.2	79.8	83.4	85.5	88.6	88.6	93.2	96.6
360	64.4	72.8	76.7	82.8	84.4	86.7	91.7	93.3	95.6
396	67.2	72.7	76.3	79.8	84.3	88.9	93.9	97.0	98.0
514	81.3	85.2	86.8	90.3	91.1	91.8	93.4	95.3	97.7
476	76.5	81.5	84.4	84.9	86.9	88.6	92.8	93.7	96.2
452	75.7	80.5	83.2	83.6	87.2	88.0	91.6	93.8	96.0
462	74.9	80.1	85.3	87.4	90.5	91.3	94.8	97.0	99.1
476	75.6	79.0	84.0	87.0	88.7	90.8	94.9	97.1	99.2
532	78.9	82.3	86.1	90.2	91.7	92.5	96.2	99.2	99.6

TABLE 20

Calculated Values of c in Exponential Curve Fitted to Observed Distribution of Headways Less than Time Shown

Time (seconds)	c
1.0	-0.000028
1.5	-0.000238
2.0	-0.000517
3.0	-0.001063
4.0	-0.001463
5.0	-0.001808
6.0	-0.002102
7.0	-0.002380
8.0	-0.002646
9.0	-0.002874
10.0	-0.003099
12.0	-0.003583
14.0	-0.004027
16.0	-0.004462
18.0	-0.004854
20.0	-0.005233
25.0	-0.006254
30.0	-0.007427
40.0	-0.009147
50.0	-0.011679
60.0	-0.014015