EVALUATION OF TRAFFIC CONTROL AT HIGHWAY INTERSECTIONS

by

Clyde E. Lee
Walter C. Vodrazka

Research Report Number 78-1F

Evaluation of Traffic Control
at Highway Intersections
Research Project 3-18-64-78

conducted for

The Texas Highway Department

in cooperation with the
U. S. Department of Transportation
Federal Highway Administration
Bureau of Public Roads

by the

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

March 1970

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads.

PREFACE

This is the first, and final, published report on Research Project 3-18-64-78, "Evaluation of Traffic Control at Highway Intersections." It describes (1) the development of special digital traffic delay recording equipment; (2) field studies conducted in Austin, San Antonio, and Houston, Texas, from 1965 to 1967; (3) data processing and analysis techniques; and (4) an interpretation of the results.

Three unpublished theses based on various phases of the research study have, however, been submitted to The University of Texas at Austin in partial fulfillment of the requirements for the Master of Science degree in Civil Engineering. These are:

"A Technique for Evaluating Vehicular Delay at Intersections," June 1967, by Louis E. Hood,

"Traffic Delay at Stop Sign Controlled Intersections," May 1969, by Frank N. Cunningham, and

"An Analysis of Vehicular Delay at Signalized Intersections," January 1970, by Harold D. Cooner.

Copies of these theses are available for interlibrary loan from the Engineering Library, The University of Texas at Austin, Austin, Texas 78712, or reproductions may be procured from this source for cost of processing.

Digital data tapes and computer programs for the SDS 930 and CDC 6600 computers are on file at the Center for Highway Research.

ABSTRACT

The design and development of a digital delay data recorder (D3 Recorder) and its use for collecting traffic volume and delay data at intersections are detailed in this report.

Extensive field measurements of traffic characteristics observed at 19 locations ranging in complexity from low-volume intersections operating under stop-sign control to high-volume, signalized diamond interchanges were recorded for subsequent analysis.

Computer programs used for data reduction and analysis are described and documented.

Volume versus delay relationships for virtually all types of stop-sign and signal control were formulated. Currently used warrants for selecting various types of traffic control devices and proposed warrants for using traffic-actuated signals were evaluated. A new set of minimum volume warrants for four-way stop-sign installation were formulated.

A number of recommendations regarding potential applications of the recording equipment and the methodology developed for this study are given. The feasibility of using multichannel recording devices for field studies of traffic characteristics has been demonstrated.

KEW WORDS: delay at intersections, stopped-time delay, intersection capacity, traffic volume, traffic flow characteristics, peak-hour traffic, signalized intersection traffic control, stop-sign intersection control, traffic-actuated

signals, traffic signal timing, traffic counters, manual traffic counts, data recorders, data processing, traffic analysis, mathematical models, traffic delay warrants, traffic signal warrants, traffic sign warrants, highway user benefits, traffic planning, traffic research by simulation.

SUMMARY

Traffic delay is one of several criteria that are frequently used by traffic engineers to aid them in selecting the proper type of traffic signs or signals for a specific application. Their objective is to minimize delay while providing safe, orderly traffic flow through street and highway intersections.

Before this research study began, there were no practical means for recording and processing the large amounts of traffic performance data required for evaluating the relative effectiveness of various control devices in minimizing delay. A 12-channel digital delay recorder was developed and used extensively for studying traffic delay characteristics at 19 different intersections controlled by stop signs or signals. The desirability of recording simultaneously input from electromechanical monitoring devices on the traffic signal controllers and from human observers in the field in a format directly suitable for computer processing was demonstrated.

Analysis of the delay data obtained in over 240 hours of field studies led to the development of new minimum traffic volume warrants for installing four-way stop signs. Also, a set of volume warrants for traffic-actuated signals proposed by the Traffic Engineering Section, Maintenance Operations Division of the Texas Highway Department was evaluated in terms of minimizing delay and was recommended for continued use.

Procedures for using the delay-evaluation technique in before-and-after studies and an economic analysis process are described in the report. Suggestions for modernization and further development of the delay recording system are offered, and potential application of the presently available field data for traffic simulation and intersection capacity studies are pointed out.

TABLE OF CONTENTS

ACKNOWLEDGMENTS

In the conduct of this research study, many individuals associated with several public agencies have given generously of their time and talents. The authors of this report express their appreciation for these contributions and for the attitude of cooperation exhibited continuously by each person connected with the project.

Henry R. Mitchell and Georges Vernez developed many of the concepts upon which the delay recorder was based when they were graduate students at The University of Texas at Austin.

Harold H. Dalrymple, James L. Porter, Michael J. Lee, and Peter Y. Lee designed, constructed, and maintained the electromechanical equipment.

Georges Vernez, Louis E. Hood, Ahmed O. Thabit, and Harvey L. Jeane developed a majority of the computer programs used for data reduction.

Dale Marvel, City of Houston; John Lipscomb, Texas Highway Department, Houston; and William R. McCasland, Texas Transportation Institute, cooperated generously in the conduct of the studies in Houston.

Stewart Fischer and personnel from the Traffic Department assisted in the studies in San Antonio.

Walter Klapproth, Harold Beasley, Joe Ternus and other personnel of the Traffic and Transportation Department, City of Austin, cooperated willingly in the many studies performed in Austin.

M. V. Greer and Herman Haenel, D-18, Texas Highway Department, served as contact individuals and contributed freely of their time and resources throughout the study.

Peter Crivello, Frank N. Cunningham, and Edward V. Kristaponis represented the Bureau of Public Roads as contact men and made many helpful observations and suggestions.

Louis E. Hood, Frank N. Cunningham, and Harold D. Cooner each prepared a thesis based on certain phases of this study. Portions of this report are the direct results of their work.

Nasser I. Al-Rashid, Randy B. Machemehl, and Robert F. Inman assisted in data analysis and in the preparation of the report.

Other staff members of the Center for Highway Research too numerous to mention individually contributed in special ways to the study.

Computer facilities of The University of Texas at Austin and the Division of Automation, Texas Highway Department, made this study feasible.

The study was part of the Cooperative Research Program with the Texas Highway Department and the Bureau of Public Roads, Federal Highway Administration, Department of Transportation.

Clyde E. Lee
Walter C. Vodrazka

March 1970

CHAPTER 1. INTRODUCTION

Every intersection presents the motorist with potential sources of hazard and delay in addition to those normally associated with vehicular operation on the open highway. This is due primarily to the need for two or more traffic streams to share a common section of roadway in passing through an intersection. In order to keep hazard and delay experience within reasonable limits, it is necessary to select and install a means of controlling traffic movement through intersections. A variety of traffic control devices ranging from a set of signs and pavement markings to rather complex signal systems have been developed and are in wide use throughout the United States.

The purpose of any intersection control system is to alternate the right-of-way between the several intersection approaches in such a way that traffic may move safely through the intersection with minimum delay. Indeed, the stated criteria of signal timing are to minimize (1) average delay to all vehicles and pedestrians, (2) total delay to any single group of vehicles or pedestrians, and (3) the possibility of accident producing conflicts (Ref 1).

Guidelines in the form of warrants for certain types of traffic control have been established. These are generally expressed in terms of traffic volumes and accident experience, and a certain degree of standardization in the use of traffic control devices has been achieved (Ref 18). Techniques for phasing and timing signal controllers have been proposed and used with some success, and comparatively complicated signal controllers incorporating many adjustable features have been constructed.

Other factors which influence the selection and installation of a traffic control device include the initial equipment cost, installation cost, and the anticipated operating and maintenance costs. Also, in order to make valid economic judgments, it is necessary to acquire information on the relative costs to motorists affected by the traffic control configuration employed. Unnecessary and excessive delays to motorists can result in significant economic losses during the service life of the control equipment. A reduction in these losses, i.e., a reduction in delay, will often justify the installation of more expensive and, correspondingly, more efficient traffic control equipment.

Up to now, however, there has been no suitable method for evaluating the effectiveness of control devices operating with various settings under actual traffic conditions.

PURPOSE AND OBJECTIVES

Traffic performance at controlled intersections may be expressed in terms of the relative delay experienced by vehicles entering the intersections. Consideration must also be given to the maximum delay experienced by any individual vehicle or by a queue of vehicles in the traffic stream. Thus, an accurate, quantitative measure of the delay to each vehicle in the traffic stream is required in order to use this concept for evaluating the effectiveness of existing traffic control schemes and for making economic decisions related to various alternative traffic control systems.

The purpose of this research was to develop a practical means for measuring vehicular delay, a procedure for analyzing the data obtained, and a methodology for evaluating traffic control installations, with particular emphasis on the establishment of suitable warrants for selecting the proper traffic control equipment at individual intersections.

Several basic policy decisions which were made in the early stages of this research directly affected its outcome. One of these concerned the type of delay to be measured and another was the manner in which the data were recorded.

Determination of the total delay which results to a vehicle from slowing below normal running speed, stopping, and then accelerating back to running speed is desirable, but such a determination is impractical because it would require a time-space record of each individual vehicle. However, an observer can sense quite accurately when a vehicle is not moving and the total number of vehicles stopped on a selected approach during any given time interval can be recorded by mechanical means. Stopped time delay can then be calculated as the product of the number of stopped vehicles and the recording time interval in seconds.

Thus, for practical reasons, it was decided to employ stopped time delay in the evaluation of intersection performance in this research study.

In earlier studies of delay involving such methods as time-serial photographs and purely manual data recording, a most serious drawback concerned the very large volume of tedious and time-consuming data reduction required to obtain a relatively small sample of usable delay data. Thus, it was decided that the recording device used in this research must have the capability of producing digital output in a form directly acceptable for computer processing. This would make the analysis of large volumes of data feasible.

The Digital Delay Data Recorder and its operation are described in detail in Chapter 2 this report.

Based on the purpose of the research as given above, the following set of objectives was established:

(1) to design, construct, and test equipment capable of recording vehicular delay data in digital form on punched paper tape for twelve approach lanes,

(2) to prepare computer programs for summarizing delay data and reducing it to a suitable form for analysis,

(3) to develop methods for analyzing traffic delay data,

(4) to evaluate the performance of various control devices in field operation, and

(5) to evaluate the suitability of existing warrants for selecting the type of traffic control required at a number of actual intersections.

An additional objective concerning the optimization of signal settings which was given in the original research proposal could not be met in the performance of this study. To do so would have required a great deal of additional data, collected in the framework of a controlled experiment. This was not anticipated at that time; however, some preliminary findings in this regard are brought out later in the body of this report.

PROCEDURE

This research was carried out in three slightly overlapping phases.

The first phase consisted of the design, construction, and field testing of special equipment for recording delay data in digital form on paper tape. This phase corresponded to the first objective and is described in detail in Chapter 2 of this report.

The second phase involved the formulation of techniques and procedures for processing, collecting, and analyzing the delay data. This corresponded to the second and third objectives and is covered in the latter half of Chapter 2 and in Chapters 3 and 4 of this report.

The third phase was concerned with evaluating the performance of various types of traffic control devices and with establishing suitable warrants for selecting these devices. This corresponded to the fourth and fifth objectives and is covered in the balance of this report, beginning with Chapter 5.

CHAPTER 2. DIGITAL DELAY DATA RECORDER

The Digital Delay Data Recorder (D3 Recorder) developed as part of this research effort is a unique piece of equipment. It provides, for the first time, a reliable means of recording a large amount of accurate data suitable for calculating vehicular delay characteristics at intersections in a form directly accessible to high-speed computer processing. The D3 Recorder is capable of recording the number of stopped vehicles, the cumulative through-traffic volume, and the signal indication for each intersection approach lane (up to 12) virtually simultaneously on a moment-to-moment basis for extended periods of several hours.

The D3 Recorder developed in this research was based to a large extent on a digital data recorder designed and built by Henry R. Mitchell in 1962 at The University of Texas (Ref 22). Many modifications as well as certain electronic and structural improvements were made to the device which increased its durability and enhanced its versatility as a practical research tool.

Although bulky, the equipment is easily transported and can be set up in the field in about 30 minutes. Anywhere from 6 to 18 observers are required to input data which are then recorded automatically on punched paper tape at specific intervals. The number of observers is a function of the number of approach lanes under study.

DEVELOPMENT AND DESIGN

Because the principal function of the D3 Recorder was to collect and record stopped-time delay data at controlled intersections, several factors basic to this application had to be considered in its development. There was no intention to construct a general-purpose data recorder although, with certain modifications, the present model could be used for a number of other applications.

Basic Factors

The basic factors which were considered in designing the recorder may be summarized as follows:

(1) Manual observation was necessary to provide all inputs to the recorder. The measurement of stopped-time vehicle delay presents unique problems because of the nature of traffic accumulation at an intersection. The erratic pattern of stops and starts characteristic of this accumulation does not lend itself readily to automatic detection or measurement. For this reason, manual observation was required.

(2) Punched paper tape was considered to be a feasible method for recording delay data. The Friden tape punch is generally available and its portable nature lends itself to installation in a relatively compact piece of equipment. The paper tape is an intermediate storage location for delay data inasmuch as the data must be transferred to magnetic tape or punched cards in order to make them accessible for computer processing. The use of direct recording on either magnetic tape or punched cards was not feasible due to the bulky, very expensive equipment required in comparison to punched paper tape. Paper tape also provides an inexpensive means of long-term data storage if this is desired.

(3) Multiple data input channels capable of recording all requisite data for a minimum of 12 approach lanes had to be provided. It was felt that this would provide a high degree of versatility, enabling the recorder to simultaneously and continuously collect data for virtually any intersection configuration encountered, ranging from four approach lanes to the six approach legs of a diamond intersection, including the associated separate left turn lanes or even individual lanes of critical approaches.

(4) An appropriate sampling rate for the recording of data had to be selected. Ideally, the sampling rate should be as high as possible; however, for periods of delay typically encountered at intersections, intervals as long as 15 seconds between counts have been used with no appreciable loss of accuracy (Ref 2). The upper limit on the rate is a function of the operating limitations of the equipment and the amount of data which may feasibly be analyzed. The computer makes the analysis of large volumes of data routine, but the punch is limited to about 20 characters per second. When all 12 data channels are in operation, 49 characters must be punched as one data set; this requires a minimum time of about 2.5 seconds. On this basis, the sampling rate was selected such that each input was recorded once every 3.00 seconds. A faster sampling rate (once every 1.44 seconds) was also used, discussed later. The three-second sampling interval approximates the average initial vehicle starting time and thus virtually assures the recording of all delayed vehicles. This time interval is not likely to be a large fraction of a green signal indication, even with traffic-actuated phases. However, the short cycles made

possible with traffic-actuated equipment could result in significant errors if a longer sampling interval were used.

(5) Special provisions had to be made for sensing and recording the indicated number of stopped vehicles on each approach at any given time, the total vehicular flow through each approach up to any given time, and the signal indication facing each approach at any given time.

Design Components

The process of observing traffic behavior and recording the appropriate data, as envisioned in this research, consists of three essential functions:

(1) remote sensing of the data,

(2) translating the data to binary-coded decimal form, and

(3) recording the data on punched paper tape in a specified format by means of a programmer.

Remote Sensors. Three data items were observed for each intersection approach: (1) the number of stopped vehicles, (2) the number of vehicles that had crossed the approach stop line, and (3) the signal indication facing each approach. Both vehicle counts were input to the recorder through manually actuated counter modules operated by individual observers while signal indications were obtained by making connections to the relay contacts in the signal controller.

Two types of manual vehicle counter modules were designed. The first (delay counting module) was used for counting stopped vehicles and the second (incrementing module) was used for counting traffic volume.

Because the number of stopped vehicles could increase when vehicles arrived at the rear of a queue or decrease when the head of a queue was released by the green signal, it was necessary that the delay counting device be capable of both forward and reverse operation. Each delay counter module consists of two push-button switches and a remote count indicator (see Fig 2.1). One switch, when closed by the observer, inputs a signal to the recorder when an arriving vehicle stops and the indicated count of delayed vehicles is increased by one. The other switch is activated when a vehicle just begins to proceed toward the intersection and the indicated count is decreased by one.

The remote count indicator is used to enable the observer (stationed up to 200 feet from the recorder) to visually check the indicated number of stopped vehicles (which will be the number punched on tape) with the actual number of stopped vehicles, thus assuring accurate data. Each delay counter module is connected by multi-conductor electrical cable to one of the twelve programmer delay input channels in the recorder. The cable length of 200 feet allows the observer to select the best observation point.

The incrementing counter, used for volume counting purposes, consists of a single push-button switch which is depressed by an observer at the passage of a vehicle. It is connected to one of the twelve programmer volume-input channels. The incrementing counter is capable of only forward operation because only cumulative volume totals are needed. Volumes for any specific time period may then be obtained easily by subtraction.

The operation of the traffic signal controller is monitored through relay coils connected to the relay contacts in the signal controller which feed power to the red signal faces on each approach. The contacts of the monitor relays are connected to appropriate positions on the function selector stepping switches.

Translators. The purpose of the translators is to convert the sensed data from the ordinary decimal digit mode to the binary coded decimal mode, the form in which they will be punched. The translators consist of the programmer

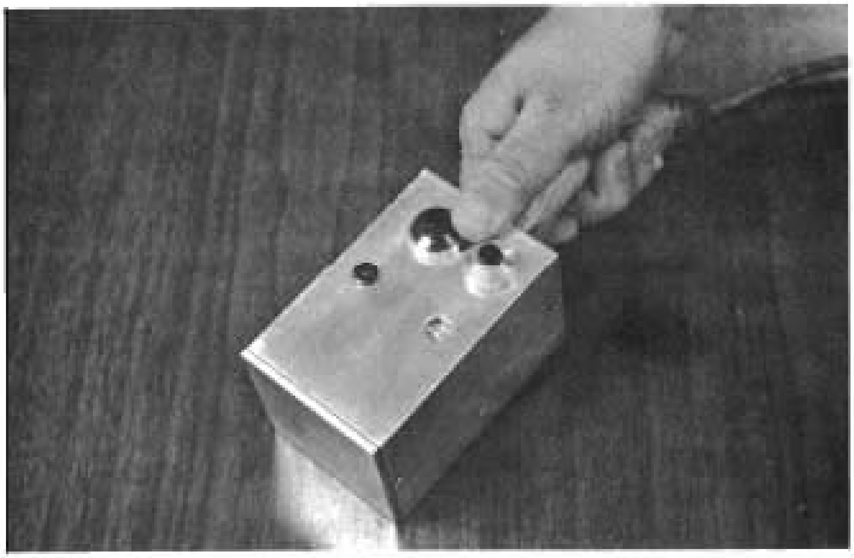

Fig 2.1. Remote count indicator.

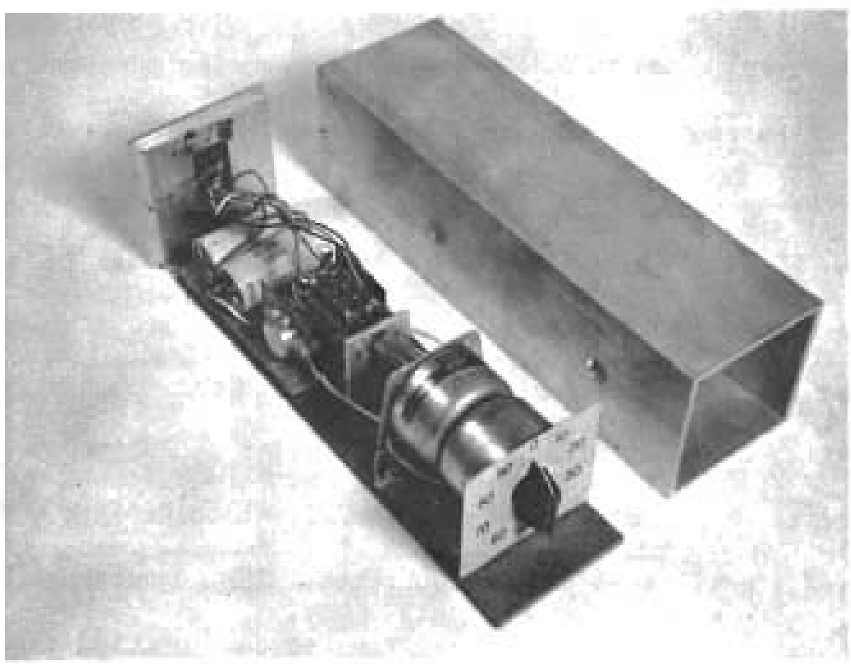

Fig 2.2. Units module.

delay input and the programmer volume input channels mentioned above in the section on remote sensors.

Each programmer input channel is made up of two multi-contact rotary switches. These are four, 10-position switches connected to act as one unit. One switch operates as a units module and the other as a tens module, thus making it possible to keep track of the number of stopped vehicles (up to 99) on an approach at any given time.

A signal from the remote delay counting module drives one of two digimotors (rotary-action solenoids) in the units module. One digimotor rotates the 10-position switch forward through 36 degrees and the other rotates it backwards through 36 degrees, for each actuation, providing a total of 360 degrees for the ten positions. When the units module switch steps from position 9 to position zero, contacts automatically actuate the forward digimotor in the tens module to advance the rotary switch there one position. The reverse is also true. Figure 2.2 shows a units module.

Thus, counting may be done in an uninterrupted manner from zero to 99 either the forward or backward direction. If 100 vehicles are stopped, the counter will register a zero, but counting can still proceed in either the forward or backward direction from this point. The recorded data require additional editing in the computer operations which effect the transfer of the data from punched tape to magnetic tape. Obviously, these counting modules can be used to keep cumulative vehicle totals for other applications.

Each of the units and tens module switches includes a set of four stationary wafers which are wired to count in binary arithmetic. Each succeeding wafer represents an increasing power of two, beginning with zero, so that any ordinary decimal number from zero to 15 can be represented. Binary counting is achieved by wiring the wafer contacts as shown in Fig 2.3, in which 1 indicates "contact wired" and 0 indicates "contact not wired". The wafer

Count Input	Wafer and Code			
	1 (2^0)	2 (2^1)	3 (2^2)	4 (2^3)
0	0	0	0	0
1	1	0	0	0
2	0	1	0	0
3	1	1	0	0
4	0	0	1	0
5	1	0	1	0
6	0	1	1	0
7	1	1	1	0
8	0	0	0	1
9	1	0	0	1

Note: 1 = contact wired.
0 = contact not wired.

Fig 2.3. Wafer contact wired for binary counting.

contact positions are then connected to appropriate contacts on similar wafers on the function selector stepping switch and from there to the punch pins on the paper tape punch.

Thus, when a particular channel is being sampled, only those punch pins which correspond in position to the wired contacts on the module switches for the indicated count input are activated. The electrical circuit needed to actuate a punch pin is completed by connecting the wiper arms of the module switch to the negative terminal of a 48V dc power source and the punch magnet coil to the positive terminal. Punching is done only when the contacts are closed on both the module switch and the function selector stepping switch.

The balance of the programmer input channel consists of several wires connected directly to a selected position row on the function selector stepping switch. Wires which are connected directly to the negative terminal of the power source are also permanently connected to the first three wafers of the function selector stepping switch so that, when that position is sampled, the punches will indicate by the prewired code the approach lane it represents.

The contact of the fourth wafer is connected to the signal monitor relay coil for the approach so that the corresponding punch pin is activated only when the signal indication is red.

This arrangement was used in order to punch both the approach designation and the signal indication on the same row of the paper tape. The number of rows thus conserved allowed for punching all approach volume data along with delay data in one sweep of the function selector stepping switch.

The programmer volume input channel consists of an 11-position stepping switch which functions in virtually the same manner as the units and tens module switches. The 11-position switch causes a minor problem in data conversion because the second zero in the volume count sequence actually indicates 11, the third zero, 22, and so forth. In other words, the volume counts are supplied in terms of base 11 rather than normal base-10 arithmetic. The 11-position stepping switches were used only because they were commercially available and did not require special order as did the 10-position rotary wafer switches used in the units and tens modules.

Recording Programmer. The term "programmer" is used to identify that portion of the D3 Recorder which causes the data from each input unit to be punched sequentially on paper tape at predetermined intervals. The programmer consists of two function selector stepping switches, a synchronous pulse generator, a power supply, and a Friden paper tape punch.

Two function selector stepping switches are used to provide for two sampling rates depending on the number of approach lanes under study. One is a 52-position switch which provides a 3.00-second sampling rate for twelve or fewer approach lanes and the other is a 25-position switch which provides a 1.44-second sampling rate for six or fewer approach lanes. The sampling rate is controlled by the synchronous pulse generator.

The wiper arms which serve to provide a connection with a row of wafer contacts at each position of the stepping switch are connected to the code magnets controlling the punch pins. By advancing the switch at prescribed time intervals, the data from each input are punched on successive rows of the paper tape. A schematic diagram of this operation is shown in Fig 2.4.

Three successive positions of the switch are required for each programmer input channel. After all six (or 12) programmer input channels are scanned and punched, the programmer volume input channels are scanned and punched. The sequence of approach lanes must be exactly the same for each of these two sets of input data.

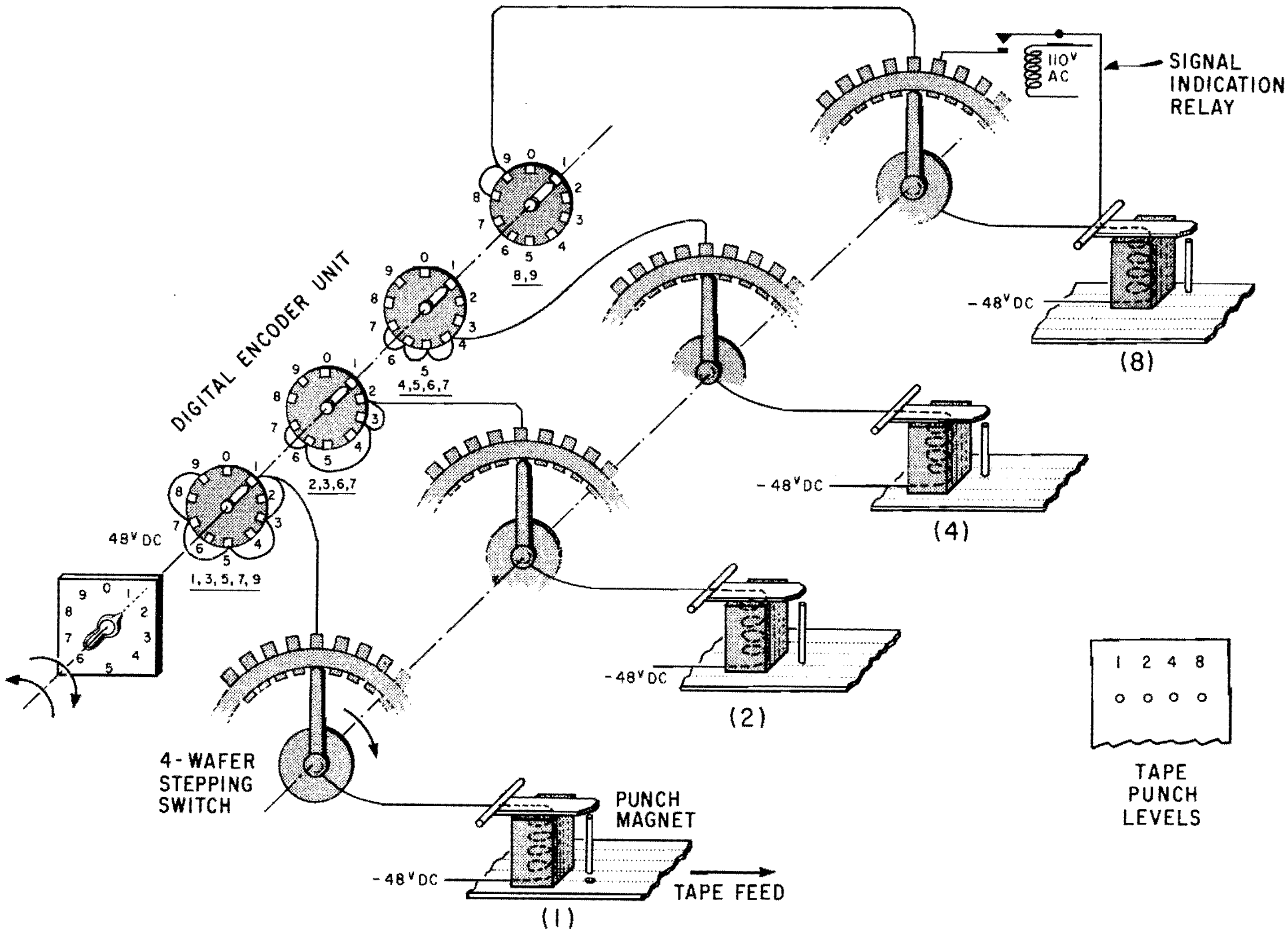

Fig 2.4. Schematic diagram of the programmer operation.

The synchronous pulse generator causes the production of electrical signals of the duration required to properly drive the 52-position stepping switch through one rotation in 3.00 seconds. Thus, a pulse to actuate the switch is required 17-1/3 times per second or approximately every 57.7 milliseconds. If the same pulse rate is used to activate the other stepping switch, the time required to drive it through its 25 positions is computed proportionately as $\left(\frac{25}{52} \times 3.00 \right)$ or about 1.44 seconds.

The components of the pulse generator are a synchronous motor and a set of cams and microswitches. A synchronous motor operates at a uniform speed regardless of the load and, for this reason, was used in order to establish a reliable time base.

Because exactly 17-1/3 pulses per second or 1040 pulses per minute are required, it is necessary to gear down the motor's rated speed of 1800 rpm to 1040 rpm. This was accomplished by means of a set of four gears. A 30-tooth gear is attached to the 1800-rpm shaft of the motor and meshes with a 60-tooth gear on an adjacent shaft, which then operates at 900 rpm. A 52-tooth gear attached to this second shaft meshes with a 45-tooth gear on a third shaft, which then turns at the required 1040 rpm.

Three adjustable cams are then attached to the third shaft. As the cams rotate, each one forces the opening and closing of a microswitch once each revolution. The closure time of each microswitch is adjustable by means of a screw-spring attachment.

One microswitch with a 24V dc power source is used to activate the 52-position stepping switch and another with a 48V dc power source to activate the 25-position stepping switch. The third microswitch, with a 48V dc power source, activates the clutch magnet of the tape punch which initiates the punch cycle.

The significant operations of the stepping switch during the 57.7 millisecond punching cycle are:

(1) The power source of 24V dc for the 52-position switch or 48V dc for the 25-position switch is applied across the coil, which then pulls in the armature and cocks a drive spring. The switch wiper arms are held in position by a rachet. Power must be applied to the coil for 20 to 30 milliseconds through the microswitch of the pulse generator.

(2) When power across the coil is switched off, the drive spring is released and the wiper arms are moved forward to the next position. This movement requires from 8 to 12 milliseconds.

(3) Wiper arms then switch 48V dc power to the appropriate code magnets and thereby position the latches to punch the data set on that position by the corresponding input source.

The significant operations of the punch during the same punching cycle are:

(1) The punch drive motor runs continuously. However, power for rotating the punch mechanism is expended through a friction clutch when not being used.

(2) When 48V dc is applied to the clutch magnet, the armature is pulled in, the clutch released, and actual punching occurs. Power must be applied to the clutch magnet for a minimum of 10 milliseconds.

(3) The punching mechanism turns exactly one revolution and stops.

(4) Each punch pin is controlled by a latch that is positioned by a code magnet. Power for each code magnet is switched by the stepping switch contacts and by the contacts of the input source.

Power must be kept on the code magnets for at least 15 milliseconds after power is first applied to the clutch magnet. This assures the proper data are punched. A short time (2 or 3 milliseconds) after this 15 millisecond period has expired, all power is cut off as the stepper activating microswitch opens and the stepper moves to a new position. The old data are then released and new data signals are transmitted to the code magnets.

A sketch of the time relationships between the operations of the stepping switch and the punch is shown in Fig 2.5. These relationships are established and the final adjustments made on the cam positions and microswitch closure times through the use of an oscilloscope.

The main power source for these operations is the 110V ac line located in each signal controller. The 110V ac power was converted to 48V dc and 24V dc power for the operation of some of the equipment. The voltages in each case are Zener diode controlled to their nominal values. The 24V dc power is used to drive the digimotors in the units and tens modules, the manual counters, and the 52-position stepping switch. The 48V dc power is used to drive the 25-position stepping switch and the tape punch. All other equipment operates on the 110V ac power source.

A Friden tape punch was used because of its general availability and relatively low cost. The characteristics of the tape punch have been included in the context of prior portions of this report. An example of the punched tape output will be given in a later section entitled "Output Data Format."

An overall view of the D3 Recorder which was used in this research is shown in Fig 2.6. The top portion shows the tape punch at the left and the programmer volume input stepping switches at the right. The lower portion of the picture shows the units and tens modules of the 12 programmer input

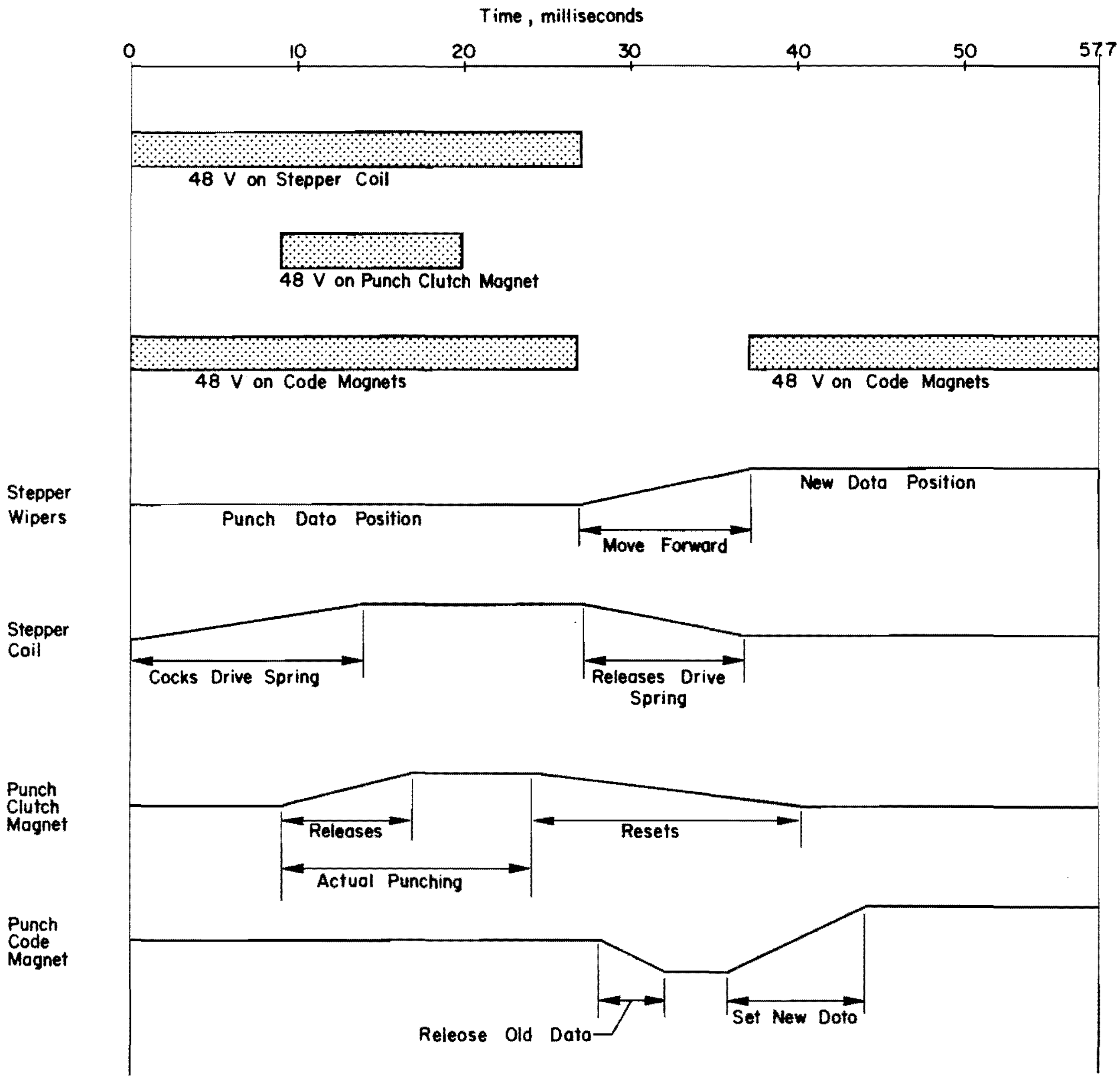

Fig 2.5. Punch sequence timing diagram, 25-position stepping switch.

Fig 2.6. Overall view of D-3 Recorder.

channels at the left. Wiring for the function selector stepping switches as well as other control mechanism switches may be seen at the lower right-hand portion of Fig 2.6.

FIELD OBSERVATION PROCEDURE

Prior to the initial field setup, the D3 Recorder and each of its components was extensively tested in the laboratory. A number of malfunctions were corrected and several improvements were suggested and ultimately incorporated into the design. The same procedure was then followed with several trial runs in the field.

A full-time crew of eight men was assembled and trained in the operations of the D3 Recorder and data collecting operations were begun in the spring of 1966.

The actual setup of the D3 Recorder in the field was quite simple, provided that a certain amount of care was exercised in making all the necessary connections. The initial step was the location of the D3 Recorder near the signal controller to facilitate connection of the signal monitor relays. The counter modules had to be connected and each observer then positioned himself in an advantageous location for observing his assigned approach.

If six or less approaches were being studied, the 25-position function selector stepping switch was turned on, and the 52-position switch was turned on if more than six approaches were under study. After the power systems were turned on and the equipment in operation, the paper tape output was visually checked to be sure that the punching sequence was correct for the approaches under study. When a thorough check of the entire setup had been completed, the actual collection of data began.

A group of observers could be effectively trained for the purposes of data gathering in a period of 10 or 15 minutes. An individual observing the number of stopped vehicles for a particular approach was instructed to depress the "add" button when an arriving vehicle had actually stopped. The "subtract" button was to be depressed only when a vehicle had actually begun to move. The observer was also urged to frequently check the actual number of stopped vehicles with the number indicated on his counter and to quickly make a correction if required. Fortunately, such corrections were seldom required.

Volume counting observers simply had to depress the button on their counters whenever they observed a vehicle cross the stopline of the approach under study and actually clear the intersection.

Thus, the setup of the D3 Recorder and the observation of data were relatively simple tasks and no great problems ever developed along these lines. Occasionally, however, equipment malfunctions did occur which required on-the-spot repair but which rarely resulted in the loss of data or in the need for restudy of the intersection.

OUTPUT DATA FORMAT

The paper tape used for data recording was the standard form in that each row had sufficient space for seven columns of punches with a column of sprocket holes between the third and fourth punch level columns. Only the first four levels were used for data punching in this study but a parity punch was punched in column seven as needed.

The punch format for the various characters used in this study is illustrated in Table 2.1.

Each of the programmer input channels is capable of transmitting three rows of punched output containing four specific items of data. The transmission occurs at prescribed intervals as controlled by one of the function

TABLE 2.1. PUNCH FORMAT OF CHARACTERS USED IN STUDY

Punch Levels	1	2	3	4	7
Codes	1	2	4	8	
Characters					
0					
1	0				0
2		0			0
3	0	0			
4			0		0
5	0		0		
6		0	0		
7	0	0	0		0
8				0	0
9	0			0	
Red Indication				0	*
Green Indication					*
End Data Block	0	0	0	0	0

*The red and green indications will always appear with a lane designation in columns 1 to 3. Thus, the parity punch will vary.

selector stepping switches included in the programmer. Individual rows are transmitted during the dwell interval of the stepping switch contacts and the three rows are transmitted in successive steps of the stepping switch.

Each row is made up of a set of punched holes which constitute a binary number, the magnitude of which indicates the approach being sampled and the signal indication on the first row, a units digit on the second row, and a tens digit on the third row.

Each of the programmer volume input channels transmits one row of data which indicates volume in terms of base-11 arithmetic.

The output data format depends on which function selector stepping switch is in the circuits. When six approach lanes are under study and the 25-position switch is on, 25 rows of punched output constitute one data block. The first 18 rows contain the data from the 6 programmer input channels, the next 6 rows contain the data from the 6 programmer volume input channels, and the 25th row indicates the end of a data block. In each case of transmission from the data input channels, data are always punched in the same sequence, i.e., from lane A through lane F.

When twelve approach lanes are under study and the 52-position switch is on, the first 36 rows contain the data from the 12 programmer input channels, the next 12 rows contain the data from the 12 programmer volume input channels, the next 3 rows are blank, and the 52nd row indicates the end of a data block.

A typical example of stopped vehicles at an intersection is shown in Fig 2.7 and the resulting punch configuration on the paper tape is shown in Fig 2.8. In this example, the 25-position switch was on and data were sampled during an interval shortly after the east-west flow received the green indication. The reader may verify the output illustrated in Fig 2.8 easily by

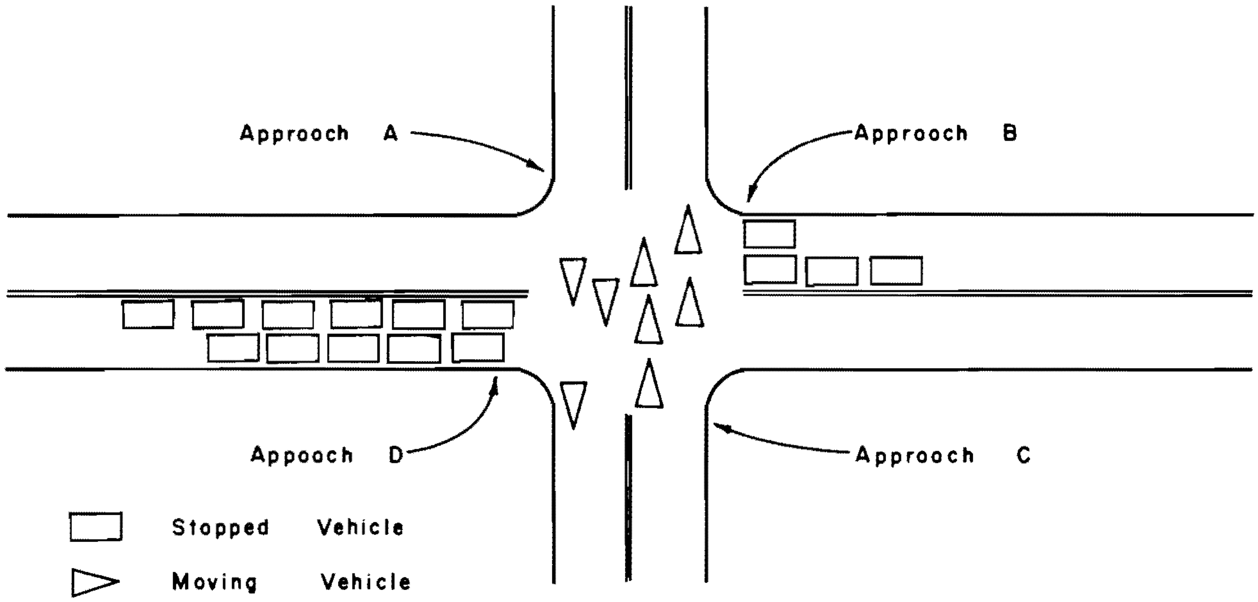

Fig 2.7. Example of stopped vehicles at an intersection.

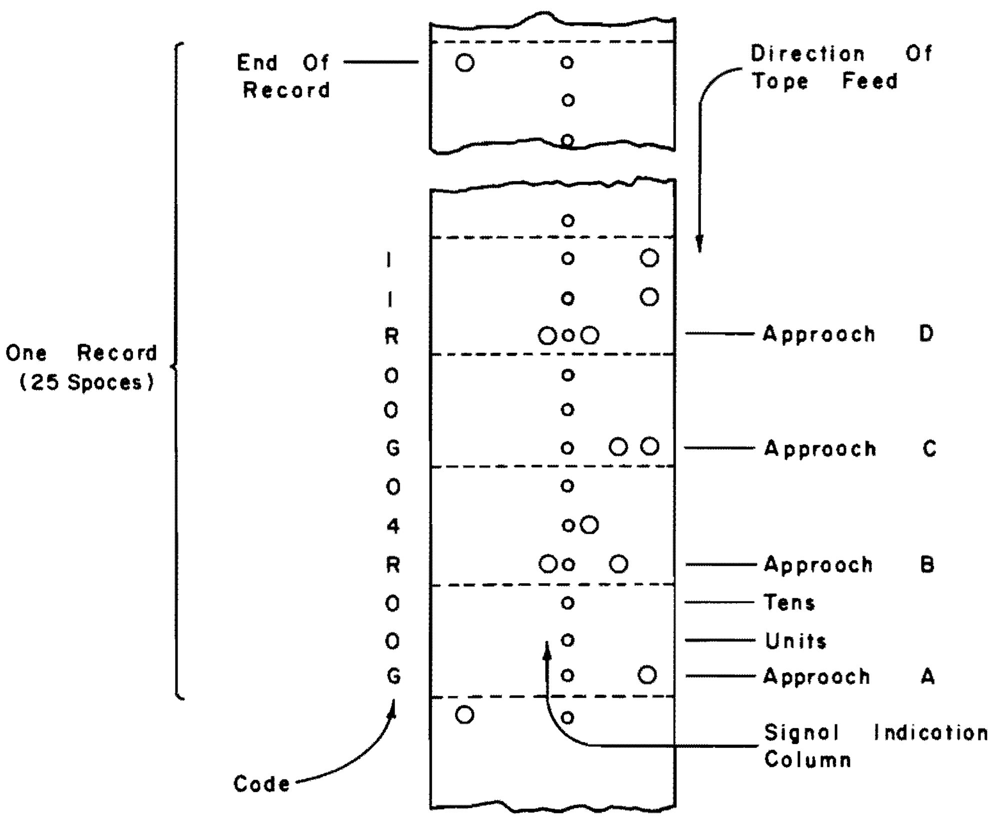

Fig 2.8. Typical data tape representing stopped vehicles in Fig 2.7.

reference to Fig 2.7. It should be noted that lanes E and F are not being observed in this example.

CHARACTERISTICS

Many of the more basic characteristics of the D3 Recorder have been brought out in earlier sections. It remains now to discuss some of these in the context of the advantages as well as the disadvantages and limitations of the use of the D3 Recorder for its intended purpose.

Advantages

The principal advantage of the D3 Recorder is, of course, that if it functions quite well in recording the observations necessary for the calculation of delay statistics at intersections. All data can be accurately observed and subsequently recorded in a form immediately suitable for computer processing and analysis. There is a minimal amount of encoding and card punching required, although some data such as the intersection studied, the time and date, the directional orientation of the approaches, and other system parameters must be so treated.

The paper tape and punch are a readily available, relatively inexpensive, and highly reliable means of recording the data and maintaining them as a somewhat permanent record. Equipment for reading the tape is available at most computer installations.

The equipment is housed in a rugged carrying case. It is fairly reliable with a minimum amount of down time due to malfunctions. Virtually each component is sufficiently compartmentalized so that a replacement unit can be installed in a matter of minutes.

Disadvantages

The principal disadvantage associated with the use of the D3 Recorder is the number of people required to manually observe the data. This is not because of the cost involved as much as it is the gaper's block in the traffic stream which results from curiosity about the reason for the equipment, the people, and the wires which run in every direction. A great deal of this problem was eliminated, however, by placing a sign reading "Traffic Survey" on top of the D3 Recorder.

Another disadvantage was concerned with the general lack of directly applicable component equipment for the construction of the D3 Recorder. For instance, the 10-position stepping switches used in the units and tens modules had to be especially manufactured, for two reasons. Stepping switches are available with 11 positions but not with 10 positions. Also, switches capable of both forward and backward operation are unavailable, and a special design was required.

The D3 Recorder, as has been stated earlier, is bulky and requires two men to handle it. It is anticipated, however, that a more sophisticated design incorporating greater use of electronic components and solid-state circuitry will give a more compact unit, one easily carried by one man.

SUMMARY

The design and construction of the Digital Delay Data Recorder, which was one of the major objectives of this research, has been described in considerable detail. The D3 Recorder makes available, for the first time, a practical research tool for the comprehensive analysis and evaluation of intersection phenomena, particularly those having to do with vehicular delay experience.

Virtually any intersection configuration covering the entire range of traffic control systems can be studied. When one considers the very large number of intersections which require some degree of control, it is readily apparent that significant user cost due to excessive or unnecessary vehicular delay can develop. The D3 Recorder provides the traffic engineer with a definitive means for analyzing intersection delay experience, thus enabling him to make sound judgments regarding the installation and efficient adjustment of the most suitable traffic control device for the geometric and traffic conditions prevailing at individual intersections.

CHAPTER 3. DATA PROCESSING

The reduction of the delay data into a form suitable for analysis consisted of a certain amount of intermediate data processing prior to the actual calculation of the delay relationships to be studied (see Fig 3.1).

INTERMEDIATE DATA PROCESSING

The first step in reducing the data was to transfer the raw field data from the punch paper tape to a 200 BPI magnetic tape, noting any incomplete data blocks and flagging them on the newly built tape. Upon completion of this task, which was performed on the SDS 930 computer, the second step in data reduction was performed. It consisted of producing a new 556 BPI magnetic tape on the CDC 6600 computer. This tape excluded all incomplete data blocks which had previously been flagged on the 200 BPI tape and also contained an added time base on each record for identification purposes. The computer program which produced the 556 BPI tape also produced a listing of that tape containing the data in tabular form and its added time base. An example of this listing is shown in Table 3.1. This also illustrates the data which were collected for a typical intersection. The first column indicates the time base, that is, the time at the start of each 1.44-second sampling interval, in hours, minutes, and seconds.

The data printed in columns 2 through 7 under the headings "Lane A" to "Lane F" are those recorded by the programmer input channels. The letters "R" and "G" represent the red and green signal indications which face each

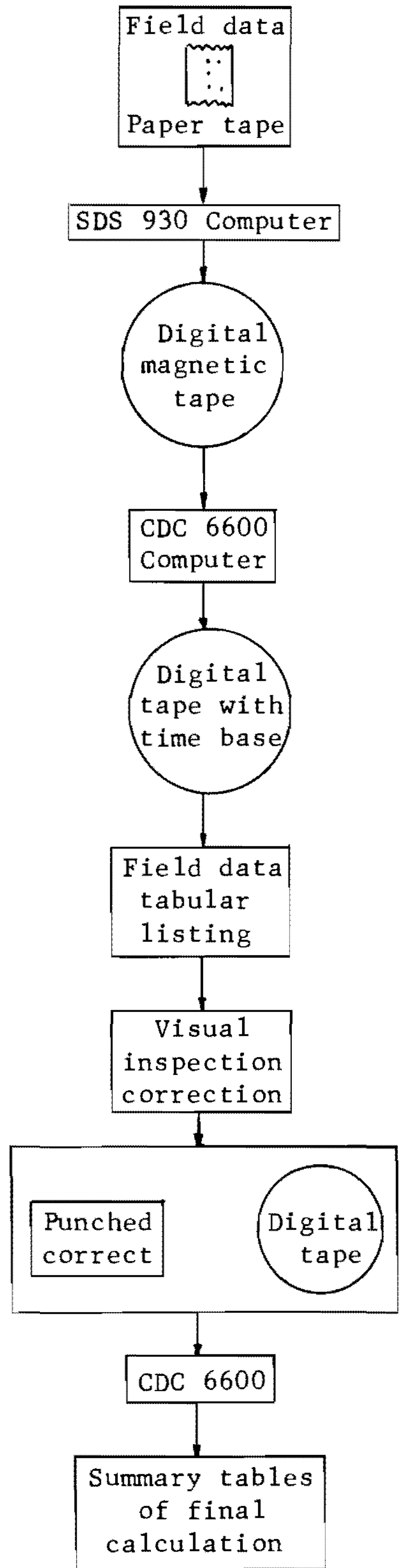

Fig 3.1. Flow chart for data processing procedure.

TABLE 3.1. TABULAR DATA LISTING

WOODROW AND KOENIG - JULY 25TH 1967 - 0715 TO 0915
FILE NO. 4 STORED IN MAG. TAPE - 858

HR	MIN	SEC	LANE A	LANE B	LANE C	LANE D	LANE E	LANE F	VOLU A	VOLU B	VOLU C	VOLU D	VOLU E	VOLU F
7	41	24.00	G 0	R 2	G 2	R 5	G 0	G 0	1	2	1	7	0	0
7	41	25.44	G 0	R 2	G 2	R 5	G 0	G 0	3	2	1	7	0	0
7	41	26.88	G 0	R 2	G 2	R 5	G 0	G 0	4	2	1	7	0	0
7	41	28.32	G 1	R 2	G 3	R 6	G 0	G 0	4	2	1	7	0	0
7	41	29.76	G 1	R 2	G 3	R 6	G 0	G 0	4	2	1	7	0	0
7	41	31.20	G 2	R 2	G 2	R 7	G 0	G 0	4	2	2	7	0	0
7	41	32.64	G 4	R 2	G 2	R 7	G 0	G 0	4	2	2	7	0	0
7	41	34.08	G 4	R 2	G 2	R 7	G 0	G 0	5	2	3	7	0	0
7	41	35.52	G 5	R 2	G 2	R 7	G 0	G 0	5	2	3	7	0	0
7	41	36.96	G 4	R 2	R 2	G 7	G 0	G 0	5	2	3	7	0	0
7	41	38.40	R 4	G 2	R 2	G 7	G 0	G 0	5	2	3	7	0	0
7	41	39.84	R 4	G 2	R 2	G 8	G 0	G 0	6	2	3	8	0	0
7	41	41.28	R 4	G 2	R 2	G 6	G 0	G 0	6	2	3	9	0	0
7	41	42.72	R 4	G 1	R 2	G 5	G 0	G 0	6	2	3	9	0	0
7	41	44.16	R 5	G 0	R 2	G 5	G 0	G 0	6	3	3	10	0	0
7	41	45.60	R 8	G 0	R 2	G 3	G 0	G 0	6	3	3	1	0	0
7	41	47.04	R 11	G 0	R 2	G 2	G 0	G 0	6	4	3	2	0	0
7	41	48.48	R 11	G 0	R 2	G 1	G 0	G 0	6	4	3	3	0	0
7	41	49.92	R 12	G 0	R 2	G 0	G 0	G 0	6	4	3	4	0	0
7	41	51.36	R 12	G 0	R 2	G 0	G 0	G 0	6	4	3	4	0	0
7	41	52.80	R 12	G 0	R 2	G 0	G 0	G 0	6	4	3	5	0	0
7	41	54.24	R 12	G 0	R 3	G 0	G 0	G 0	6	4	3	5	0	0
7	41	55.68	R 12	G 0	R 3	G 0	G 0	G 0	6	4	3	7	0	0
7	41	57.12	R 12	G 0	R 3	G 0	G 0	G 0	6	4	3	7	0	0
7	41	58.56	R 13	G 0	R 3	G 0	G 0	G 0	6	4	3	8	0	0
7	41	60.00	R 14	G 0	R 3	G 0	G 0	G 0	6	4	3	8	0	0
7	42	1.44	R 14	G 0	R 3	G 0	G 0	G 0	6	4	3	8	0	0
7	42	2.88	R 15	G 0	R 3	G 0	G 0	G 0	6	4	3	9	0	0
7	42	4.32	R 15	G 0	R 3	G 0	G 0	G 0	6	4	3	10	0	0
7	42	5.76	R 15	G 0	R 3	G 0	G 0	G 0	6	4	3	0	0	0
7	42	7.20	R 16	G 0	R 3	G 0	G 0	G 0	6	4	3	2	0	0
7	42	8.64	R 17	G 0	R 3	G 0	G 0	G 0	6	4	3	2	0	0
7	42	10.08	R 18	G 0	R 3	G 0	G 0	G 0	6	4	3	3	0	0
7	42	11.52	R 18	G 0	R 3	G 0	G 0	G 0	6	4	3	3	0	0
7	42	12.96	R 18	G 0	R 3	G 0	G 0	G 0	6	4	3	5	0	0
7	42	14.40	R 18	G 0	R 3	G 0	G 0	G 0	6	4	3	5	0	0
7	42	15.84	R 20	G 0	R 3	G 0	G 0	G 0	6	4	3	6	0	0
7	42	17.28	R 20	G 0	R 3	G 0	G 0	G 0	6	5	3	7	0	0
7	42	18.72	R 20	G 0	R 3	G 0	G 0	G 0	6	5	3	8	0	0
7	42	20.16	R 21	G 0	R 3	G 0	G 0	G 0	6	6	3	9	0	0
7	42	21.60	R 22	G 0	R 3	G 0	G 0	G 0	6	7	3	10	0	0
7	42	23.04	R 22	G 0	R 3	G 0	G 0	G 0	6	7	3	10	0	0
7	42	24.48	R 23	G 0	R 3	G 0	G 0	G 0	6	7	3	10	0	0
7	42	25.92	R 23	G 0	R 3	G 0	G 0	G 0	6	7	3	10	0	0
7	42	27.36	G 24	R 0	G 4	R 0	G 0	G 0	6	7	3	10	0	0
7	42	28.80	G 24	R 0	G 5	R 1	G 0	G 0	7	7	4	10	0	0
7	42	30.24	G 23	R 0	G 5	R 1	G 0	G 0	7	7	4	10	0	0
7	42	31.68	G 23	R 1	G 3	R 1	G 0	G 0	7	7	5	10	0	0
7	42	33.12	G 22	R 1	G 2	R 1	G 0	G 0	8	7	5	10	0	0
7	42	34.56	G 20	R 2	G 1	R 1	G 0	G 0	9	7	6	10	0	0
7	42	36.00	G 20	R 2	G 0	R 2	G 0	G 0	10	7	6	10	0	0
7	42	37.44	G 21	R 2	G 0	R 2	G 0	G 0	10	7	6	10	0	0
7	42	38.88	G 20	R 3	G 0	R 3	G 0	G 0	1	7	7	10	0	0
7	42	40.32	G 18	R 4	G 0	R 3	G 0	G 0	2	7	7	10	0	0
7	42	41.76	G 17	R 4	G 0	R 3	G 0	G 0	3	7	7	10	0	0

approach, and the tabulated number represents the number of stopped vehicles on each approach at the instant the data were recorded.

The data printed in columns 8 through 13 under the headings "Volu A" to "Volu F" are those recorded by the programmer volume input channels. These values represent the individual approach volumes in terms of base 11 arithmetic.

This tabular presentation thus gives an essentially instantaneous and continuous record of the traffic volume, number of stopped vehicles, and the signal indication for each approach.

This listing was then checked visually for inconsistent and obviously inaccurate data. For example, the output for an approach that had been showing zero stopped vehicles would suddenly show five or six consecutive values in the eighties and nineties.

The location and nature of all necessary corrections were noted, punched on cards, and input in the calculation program.

The exact duration of individual green times or signal cycle lengths could not be determined precisely from this data because of the 1.44-second sampling interval. However, the mean length of such time periods may be determined with reasonable accuracy by averaging over a longer time period of 10 or 15 minutes.

At this point, the data on the second magnetic tape were ready for the next step: the calculation of the delay relationships to be studied. It can be noted that data at a specific intersection were collected for three 2-hour periods each day: an AM peak, PM peak, and an off-peak afternoon period. Thus, 6 hours of data containing almost 300,000 individual data items were generally available for calculation purposes within one or two days.

CALCULATIONS

The calculation of all delay relationships was performed on the CDC 6600 computing facility of The University of Texas at Austin. The program was written in FORTRAN and is on file and fully documented at the Center for Highway Research. The program was designed with a certain degree of flexibility so that delay relationships could be studied in several different ways. Delay relationships for individual approaches and for the intersection as a whole could be calculated for any desired time period.

The values that were calculated for each approach over a given time period were

(1) traffic volume,

(2) total vehicle-seconds of delay,

(3) total number of vehicles stopped,

(4) average delay per vehicle,

(5) average delay per vehicle stopped,

(6) percent of vehicles stopped,

(7) total green time,

(8) number of complete cycles,

(9) average green time per cycle, and

(10) average cycle length.

The first six items were calculated for the sum of all approaches as well. Items seven and nine were characteristic for a given direction while items eight and ten were characteristic of the intersection control. Attempts were made to calculate other relationships such as the vehicle-seconds of delay due to left turns, the total number of stops, and the average delay to the first vehicle. However, difficulties arose in the calculation of these values which limited their usefulness in the analysis of intersection delay characteristics.

An example of the calculations made for a 15-minute period at a typical intersection are shown in Table 3.2. All of the values listed above are illustrated along with one additional value, "Total X Time." This refers to the total time in the time period during which data were missing or otherwise unusable.

The traffic volume was determined as the difference between the recorded volumes at the beginning and end of the time period under study.

Vehicle-seconds of delay were computed as the product of the sum of the indicated number of stopped vehicles for each recording interval in the time period and the length of the interval, which was either 1.44 or 3.00 seconds. If the indicated number of stopped vehicles is plotted as the ordinate versus the mid-point of each recording interval on a continuous time scale, the area under the curve is equivalent to this calculation.

The total number of vehicles stopped was determined for each approach by counting the increases in the indicated number of stopped vehicles during each red signal and in the first few seconds of green signal time. Here, the assumption was that an arriving vehicle was forced to stop at the rear of the queue. When an increase in the indicated number of stopped vehicles occurred during the green signal indication, it was observed in the field that it was most often due to a previously stopped vehicle waiting to make a left turn.

The addition of the latter and the former number of increases yields a quantity called the total number of stops. If an increase in the indicated number of stopped vehicles occurred during a green signal, the number of stopped vehicles was accumulated for each interval until a decrease was observed. The vehicle-seconds of delay due to left turns were then calculated by multiplying this accumulated number by the recording interval length.

TABLE 3.2. TYPICAL CALCULATIONS FOR A 15-MINUTE PERIOD

WOODROW AND KOENIG JULY 25, 1967 0715 TO 0915 FULL ACTUATED

TIME PERIOD 800 - 815

COMPUTED INFORMATION	APPROACH A	APPROACH B	APPROACH C	APPROACH D
TRAFFIC VOLUME	97.00	77.00	28.00	84.00
TOTAL VEH-SECS OF DELAY	578.88	544.32	220.32	612.00
VEH-SECS OF DELAY DUE TO LEFT TURNS	4.32	7.20	18.72	8.64
TOTAL NO OF VEHS STOPPED	40.00	49.00	14.00	44.00
TOTAL NO OF STOPS	43.00	51.00	18.00	47.00
AVERAGE DELAY PER VEHICLE STOPPED	14.47	11.11	15.74	13.91
AVERAGE DELAY PER VEHICLE	5.97	7.07	7.87	7.29
AVERAGE DELAY TO THE FIRST VEHICLE	15.16	14.47	14.94	14.23
PERCENTAGE OF VEHICLES STOPPED	41.24	63.64	50.00	52.38
TOTAL GREEN TIME	406.08	485.28	410.40	485.28
AVERAGE GREEN TIME PER CYCLE	17.10	19.56	17.16	19.56
NUMBER OF CYCLES	24.00	24.00	24.00	24.00
AVERAGE LENGTH OF CYCLE	36.96	36.84	36.90	36.84
TOTAL X TIME	0.	0.	0.	0.

TOTAL TRAFFIC VOLUME	286.00
TOTAL VEH-SEC OF DELAY ALL APP..	1955.52
TOTAL NUMBER OF VEHS STOPPED ALL APP.	147.00
TOTAL NUMBER OF STOPS ALL APP.	159.00
AVERAGE DELAY PER VEH STOPPED ALL APP.	13.30
AVERAGE DELAY PER VEHICLE ALL APP.	6.84
PERCENTAGE OF VEHICLES STOPPED ALL APP.	51.40
AVERAGE DELAY TO FIRST VEH. ALL APP.	14.65

APPROACH A IS SOUTHBOUND
APPROACH B IS WESTBOUND
APPROACH C IS NORTHBOUND
APPROACH D IS EASTBOUND

Of course, this method of determining the number of vehicles stopped and the left-turn delay is not foolproof. A vehicle could arrive at the rear of a queue just as a vehicle departs from the front and the indicated number of stopped vehicles would remain unchanged. The reader may easily visualize other circumstances which, if they occurred, would produce erroneous results. However, the actual observation of the phenomena in the field led to the conclusion that these methods would provide results in close agreement with actual conditions. It may be possible to combine the stopped vehicle data with the concurrently observed volume data to eliminate some of these problems and provide not only more but more accurate information.

This could not be done for the bulk of the data collected in this study because the method of recording volumes on paper tape was a relatively recent innovation. Prior to this, volumes were manually counted and recorded in the field at five-minute intervals.

The average delay per vehicle and per vehicle stopped was calculated by dividing the total vehicle-seconds of delay by the volume and the total number of vehicles stopped, respectively.

Total green time was measured by counting the number of intervals in which the green signal was displayed and then multiplying by the interval length. The determination of the number of complete signal cycles was slightly more complicated. The interval at which the red signal indication first changed to green was noted. The next time red changed to green marked the end of the first cycle. Thus, the total number of times that red changed to green during the time period under study was one more than the number of complete cycles.

The average green time per complete cycle and the average length of a complete cycle were then easily computed.

The average delay to the first vehicle was calculated as the total delay to all first vehicles observed in the time period divided by the number of first vehicles observed. A first vehicle was considered observed at the first recording interval which indicated at least one stopped vehicle after the preceding recording interval had indicated no stopped vehicles, subject to the limitation that only those events taking place during a red signal indication would be counted. For each first vehicle observed, the number of recording intervals was counted, up to but not including the interval when the indicated number of stopped vehicles decreased. The total of these intervals multiplied by the interval length yielded the total delay to first vehicles. A precaution was taken so that once a first vehicle was observed, the associated decrease had to occur in the same time period. Otherwise, the observation was counted for the very next time period.

Those values which were applicable to the intersection as a whole were obtained by appropriate summation and subsequent manipulation.

TIME PERIOD FOR ANALYSIS

The hour is generally the time unit employed in the planning, design, and operation of highway facilities. Such terms as the Design Hour Volume and the peak hour are in wide use. The variation of hourly volumes throughout the day and the variation of daily volumes throughout the year are well known and fully documented.

Just as important a variation, however, occurs within each hour. Short-term rates of flow based on 1, 5, or 15-minute intervals within an hour are often quite variable. The 15-minute flow on a given intersection approach may range from slightly more than the uniform rate of 25 percent to more than 50 percent of the total hourly flow.

Therefore, the time base used in a delay study should be some period less than an hour in order to take into account the fluctuations in traffic volumes within the hour. This is because the expectation is that vehicle-seconds of delay have some direct functional dependence on traffic volume. If there were no vehicles, there would be no delays, and if each approach had a continuous vehicle backup, the delays would be quite large and an unacceptable situation would exist.

The question remains, then, of selecting an appropriate time period for analysis. Obviously, one minute is much too short because at certain intersections some approaches may not even receive a green signal indication during this time. Some consideration must also be given to the fact that certain of the parameters calculated are average values, such as the average vehicle-seconds of delay per vehicle. If the time period ends at the point when an approach is just given the green signal, the great proportion of the delay associated with the then stopped vehicles will have been accumulated, but none of the vehicles will be included in the volume count. Consequently, the average delay per vehicle could be seriously overestimated. This particular effect will decrease, however, as the time period gets longer.

In order to arrive at a reasonable time period, it was decided to plot certain of the relationships for 5, 10, 15, and 30-minute periods and base the selection on which time period appeared to provide reasonably smooth curves. Figure 3.2 illustrates the volume versus time curves for 5 and 15-minute time periods. The obvious influence of the 15-minute time period in providing smoother, more regular curves is readily apparent. Additional plots of delay versus volume as well as other relationships also showed the decided efficacy of a 15-minute time period over all others considered.

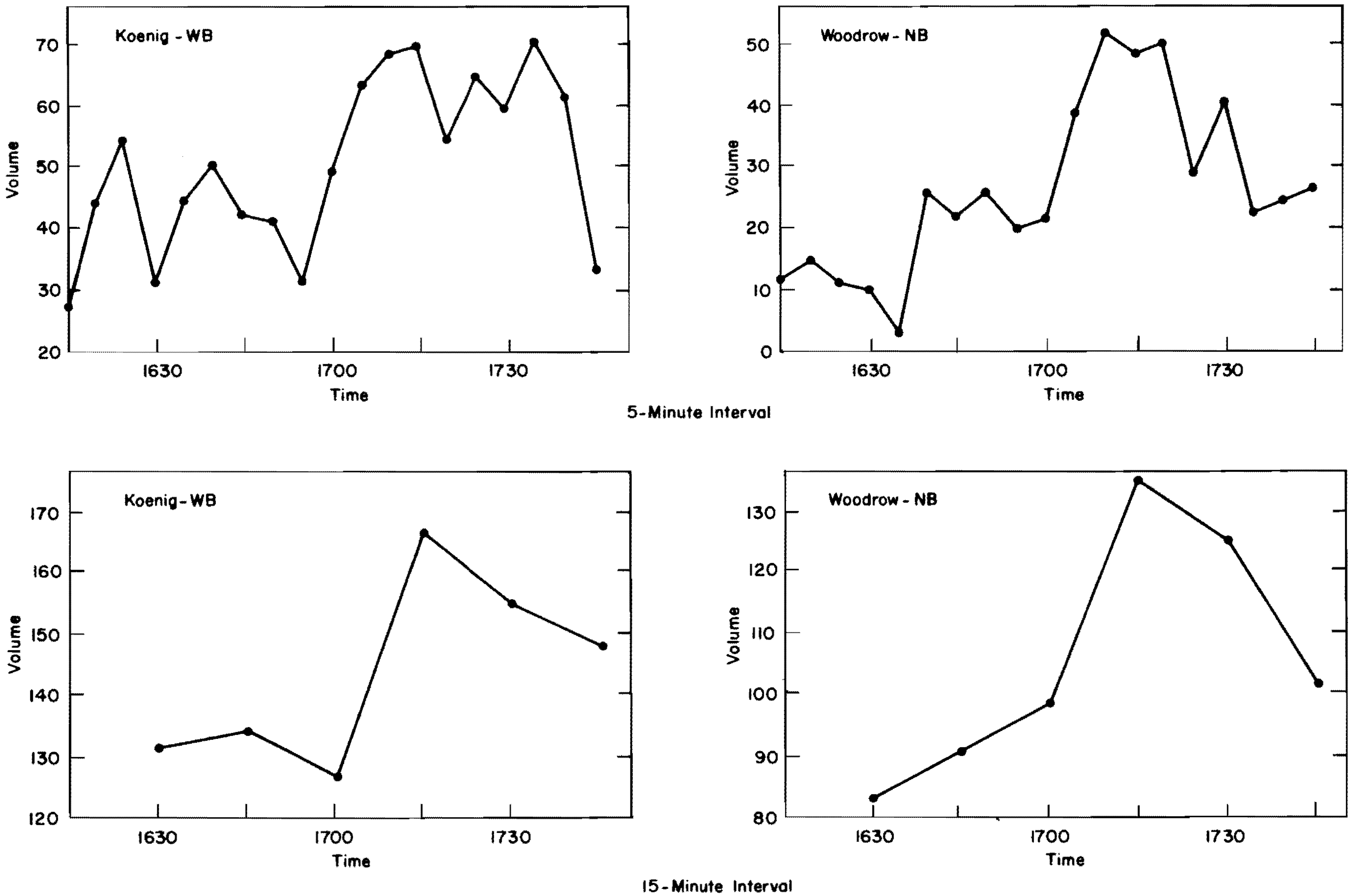

Fig 3.2. Traffic volume plotted on a 5- or 15-minute interval. Full-actuated control, Woodrow and Koenig intersection.

For these and other practical reasons, the delay relationships are presented and discussed in terms of 15-minute time periods unless specifically noted otherwise.

DISCUSSION

A great deal of effort was expended in the course of this research in gathering extensive quantities of data and in the writing and debugging of appropriate computer programs designed to reduce the data for analysis. Many intersections representing the several control methods were studied. It should be appreciated that the quantities calculated and the programs required varied with the type of control.

Obviously, for two-way and four-way stop-sign controlled intersections there were no signal characteristics to consider. On four-way stops, 100 percent of the vehicles were required to stop, and on two-way stops, only one direction of flow had to stop.

With pretimed control at an intersection, the cycle length and the distribution of green time could not vary unless, of course, the settings were changed for studies made on different days.

If this line of thinking is carried one step further, the analysis of the delay relationships as well as their interpretation should also vary, depending on the type of control. For example, the approach delay at a two-way stop controlled intersection should be more a function of the cross-street volume and the approach volume, while at a four-way stop controlled intersection, the approach delay should be more a function of the approach volume and the total volume. At a full-actuated signal the approach delay should be more a function of the cross-street volume while, at a pretimed signal the approach delay should be more a function of the approach volume.

In subsequent chapter of this report there will be much more detailed discussions and exhibits designed to point out the delay characteristics of intersections subject to various methods of control.

CHAPTER 4. STUDY SITES

The majority of the sites selected for this study were located in Austin, Texas. However, one intersection in San Antonio was included and a special before-and-after study of a diamond interchange on the Gulf Freeway in Houston was also performed.

A total of 19 intersections were selected at which 124 individual studies, consisting of approximately 240 hours of observed data, were run.

INTERSECTIONS SELECTED

The intersections included in this study are listed in Table 4.1. The normal type of control and subsequent modifications are also shown in Table 4.1 along with other information concerning the date and hours of the individual data runs and some traffic characteristics at each intersection.

Line drawings representing the physical layout of each intersection are contained in Appendix A.

DISCUSSION OF INTERSECTIONS

Except for the diamond interchange, all of the intersections have four approaches and are essentially right-angle crossings. The sites are generally situated in suburban areas which may be classified as either outlying business districts or residential fringe areas. Parking was prohibited on all approaches in virtually all instances. Sight distances were generally adequate. The volume of pedestrian and truck traffic at each intersection location was negligible. Exceptions to these characteristics are noted in this report as part of the discussion of individual intersections and their delay characteristics.

TABLE 4.1. INTERSECTION STUDIES

Date	Time	Hours	Traffic Split, % of Total Major	Traffic Split, % of Total Minor	Left Turns, % of Total Major	Left Turns, % of Total Minor	Modifications to Control Device
Two-Way Stop Control							
19th and Chicon (4x2)							
Aug. 10, 1967	0715-0915	2	65	35	3.9	6.1	
Aug. 10, 1967	1300-1500	2	72	28	8.9	7.0	
Aug. 11, 1967	1300-1500	2	77	23	4.3	5.9	
Aug. 10, 1967	1600-1800	2	78	22	5.4	6.2	
29th and Jefferson (2x2)							
July 11, 1967	0700-0900	2	78	22	17.2	2.8	
July 10, 1967	1315-1515	2	80	20	4.9	3.7	
July 10, 1967	1600-1800	2	77	23	3.2	3.3	
38th and Speedway (2x2)							
July 6, 1967	0700-0900	2	55	45	6.2	4.2	
July 11, 1967	1300-1500	2	83	17	3.1	6.4	
July 10, 1967	1600-1800	2	79	21	4.8	4.4	
Four-Way Stop Control							
19th and Chicon (4x2)							
June 24, 1967	1345-1545	2	74	26	10.9	7.3	
June 24, 1967	1615-1815	2	56	44	6.1	5.7	
June 27, 1967	0710-0910	2	71	29	4.7	7.2	
June 27, 1967	1300-1500	2	73	27	9.1	6.9	
June 27, 1967	1600-1800	2	75	25	6.4	6.1	
June 28, 1967	0700-0900	2	73	27	5.1	7.6	

(Continued)

TABLE 4.1 (CONTINUED)

Date	Time	Hours	Traffic Split, % of Total Major	Traffic Split, % of Total Minor	Left Turns, % of Total Major	Left Turns, % of Total Minor	Modifications to Control Device
North Loop and Woodrow (4×4)							
June 7, 1966	0700-0900	2	52	48	4.5	9.3	
June 7, 1966	1645-1815	2	54	46	7.8	3.9	
June 8, 1966	0700-0900	2	52	48	4.7	11.1	
June 8, 1966	1330-1530	2	65	35	5.4	4.9	
June 8, 1966	1630-1830	2	67	33	7.4	3.5	
Justin and Woodrow (4×4)							
June 9, 1966	1340-1510	1.5	59	41	3.3	11.7	
June 16, 1966	0700-0900	2	55	45	2.4	11.7	
June 16, 1966	1630-1830	2	50	50	3.3	8.8	
15th and Congress (4 × 4) Median on 15th							
June 14, 1966	0715-0845	1.5	74	26	7.4	4.7	
June 14, 1966	1330-1530	2	70	30	3.9	3.9	
June 14, 1966	1600-1800	2	70	30	3.8	2.6	
Hancock and Balcones (2×2×2×4)							
June 15, 1966	0715-0915	2	72	28	22.7	1.9	
June 15, 1966	1330-1530	2	65	35	26.1	3.4	
June 15, 1966	1600-1800	2	65	35	15.5	3.7	
Full-actuated Control							
Koenig and Woodrow (4×4)							
July 15, 1966	1330-1530	2	78	22	3.0	5.2	
Aug. 31, 1966	0730-0900	1.5	73	27	3.0	11.0	

(Continued)

TABLE 4.1. (CONTINUED)

Date	Time	Hours	Traffic Split, % of Total Major	Traffic Split, % of Total Minor	Left Turns, % of Total Major	Left Turns, % of Total Minor	Modifications to Control Device
Aug. 31, 1966	1630-1830	2	67	33	3.1	6.0	
July 6, 1966	1340-1530	2	77	23	2.5	7.0	Pretimed
July 6, 1966	1650-1850	2	67	33	2.1	6.7	Pretimed
July 15, 1966	0715-0915	2	66	34	1.7	10.4	Pretimed
July 18, 1966	1330-1530	2	72	28	4.0	5.7	Semiactuated
July 18, 1966	1630-1830	2	62	38	3.1	5.8	Semiactuated
July 22, 1966	0715-0915	2	63	37	4.7	10.2	Semiactuated
July 19, 1967	0730-0930	2	57	43	2.0	10.0	
July 21, 1967	0715-0915	2	75	25	4.2	6.0	
July 21, 1967	1300-1500	2	71	29	3.1	6.4	
July 21, 1967	1600-1800	2	62	38	2.2	10.1	
July 25, 1967	0700-0900	2	63	37	2.0	10.0	Short vehicle interval
July 25, 1967	1300-1500	2	75	25	4.0	6.0	Short vehicle interval
July 25, 1967	1600-1730	1.5	68	32	3.0	6.0	Short vehicle interval
Aug. 3, 1967	0715-0915	2	56	44	2.0	10.0	Short maximum interval
Aug. 3, 1967	1330-1530	2	78	22	4.0	6.0	Short maximum interval
Aug. 9, 1967	0715-0915	2	57	43	2.0	10.0	Short initial interval
Aug. 9, 1967	1310-1510	2	73	27	4.0	6.0	Short initial interval
Aug. 9, 1967	1610-1810	2	72	28	3.0	6.0	Short initial interval
South First and Oltorf (4x4)							
July 14, 1966	1330-1530	2	56	44	9.2	6.5	
July 14, 1966	1630-1800	1.5	55	45	6.6	7.7	
July 29, 1966	0700-0900	2	51	49	6.1	8.6	
July 19, 1966	0700-0900	2	54	46	5.9	7.7	Pretimed
July 19, 1966	1330-1530	2	52	48	6.8	8.7	Pretimed
July 19, 1966	1630-1830	2	51	49	6.0	4.7	Pretimed

(Continued)

TABLE 4.1. (CONTINUED)

Date	Time	Hours	Traffic Split, % of Total Major	Traffic Split, % of Total Minor	Left Turns, % of Total Major	Left Turns, % of Total Minor	Modifications to Control Device
July 20, 1966	0715-0915	2	54	46	6.6	8.1	Semiactuated
July 20, 1966	1330-1530	2	57	43	9.5	7.2	Semiactuated
July 20, 1966	1630-1830	2	51	49	6.7	8.7	Semiactuated
July 26, 1967	0700-0900	2	55	45	6.1	7.6	
July 26, 1967	1600-1800	2	52	48	7.0	8.1	
Aug. 1, 1967	0700-0900	2	55	45	7.6	5.3	Short vehicle interval
Aug. 1, 1967	1315-1515	2	52	48	7.1	7.6	Short vehicle interval
Aug. 1, 1967	1600-1800	2	53	47	7.1	6.7	Short vehicle interval
Aug. 8, 1967	0700-0900	2	55	45	7.6	5.3	Short maximum interval
Aug. 8, 1967	1300-1500	2	52	48	7.1	7.6	Short maximum interval
Aug. 8, 1967	1600-1800	2	53	47	7.1	6.7	Short maximum interval
Ben White and Manchaca (4x4) Median on Ben White							
July 27, 1966	1330-1530	2	57	43	10.3	7.4	
July 27, 1966	1630-1830	2	58	42	14.6	6.2	
July 28, 1966	0700-0900	2	51	49	6.7	7.3	Short vehicle interval
July 28, 1966	1330-1530	2	62	38	9.2	4.7	Short vehicle interval
July 28, 1966	1630-1800	1.5	61	39	11.3	5.8	Short vehicle interval
Aug. 1, 1966	1400-1600	2	64	36	12.0	4.3	Short initial interval
Aug. 1, 1966	1630-1830	2	59	41	15.0	5.7	Short initial interval
Exposition and Windsor (4 × 4)							
July 27, 1966	0715-0845	1.5	68	32	7.1	6.1	
July 21, 1966	1330-1530	2	59	41	7.9	10.5	
July 21, 1966	1630-1830	2	62	38	5.8	6.7	
Aug. 4, 1966	0700-0900	2	69	31	8.4	7.5	Pretimed
July 25, 1966	1330-1530	2	57	43	9.9	10.6	Pretimed

(Continued)

TABLE 4.1. (CONTINUED)

Date	Time	Hours	Traffic Split, % of Total Major	Minor	Left Turns, % of Total Major	Minor	Modifications to Control Device
July 25, 1966	1630-1830	2	62	38	6.8	10.6	Pretimed
July 26, 1966	0715-0915	2	68	32	7.5	6.0	Semiactuated
July 26, 1966	1330-1530	2	59	41	9.2	10.8	Semiactuated
July 26, 1966	1630-1830	2	60	40	4.9	10.3	Semiactuated
Windsor and Hartford (4x2)							
June 28, 1967	0700-0900	2	61	39	2.6	11.2	
June 28, 1967	1300-1500	2	71	29	4.2	7.0	
June 28, 1967	1615-1815	2	33	67	4.1	7.8	
July 12, 1967	0730-0900	1.5	63	37	3.1	10.6	Short vehicle interval
July 12, 1967	1330-1530	2	70	30	4.1	6.4	Short vehicle interval
July 12, 1967	1410-1610	2	32	68	3.0	8.1	Short vehicle interval
Hildebrand and Blanco, San Antonio (4 × 4)							
Aug. 23, 1966	0800-0930	1.5	60	40	6.7	5.5	
Aug. 23, 1966	1330-1530	2	63	47	6.5	5.7	
Aug. 23, 1966	1600-1800	2	57	43	5.1	4.4	
Aug. 24, 1966	0700-0900	2	55	45	8.1	5.0	Pretimed
Aug. 24, 1966	1330-1530	2	61	39	6.6	4.8	Pretimed
Aug. 24, 1966	1600-1700	1	56	44	4.8	3.7	Pretimed
Aug. 24, 1966	1745-1830	0.75	61	39	6.3	5.5	Pretimed
Pretimed Control							
19th and Interregional (Diamond Interchange)							
July 18, 1967	0700-0900	2	60	40	7.3	7.3	
July 18, 1967	1300-1500	2	66	34	14.6	7.1	
Aug. 2, 1967	0700-0900	2	60	40	7.3	7.3	

(Continued)

TABLE 4.1. (CONTINUED)

Date	Time	Hours	Traffic Split, % of Total Major	Traffic Split, % of Total Minor	Left Turns, % of Total Major	Left Turns, % of Total Minor	Modifications to Control Device
Aug. 2, 1967	1300-1500	2	66	34	14.6	7.1	
Aug. 2, 1967	1600-1800	2	64	36	13.3	5.6	
Volume Density Control							
Ben White and South First (4 × 4) Median on Ben White							
Aug. 2, 1966	0715-0915	2	72	28	12.8	6.1	
Aug. 2, 1966	1330-1530	2	73	27	8.4	4.3	
Aug. 2, 1966	1630-1830	2	72	28	8.4	4.3	
Aug. 3, 1966	1330-1530	2	79	21	8.6	5.4	Settings varied
Aug. 3, 1966	1630-1830	2	72	28	9.6	4.7	Settings varied
Aug. 5, 1966	0715-0915	2	75	25	15.0	4.8	Settings varied
Lamar and 38th (4×4)							
Aug. 9, 1966	0700-0900	2	64	36	3.7	5.7	
Aug. 9, 1966	1335-1535	2	68	32	5.6	5.3	
Aug. 9, 1966	1630-1830	2	69	31	3.1	9.1	
Aug. 10, 1966	0700-0900	2	64	36	3.3	6.4	Settings varied
Aug. 10, 1966	1330-1530	2	67	33	4.9	8.6	Settings varied
Aug. 10, 1966	1630-1815	1.75	68	32	3.5	10.0	Settings varied
Lamar and 24th (4×4)							
Aug. 11, 1966	0700-0900	2	59	41	4.3	2.3	
Aug. 17, 1966	1330-1530	2	68	32	8.3	4.3	
Aug. 17, 1966	1630-1730	1	69	31	11.3	3.8	
38-1/2 and Interregional (Diamond Interchange)							
Aug. 19, 1966	0715-0915	2	29	71	12.4	14.1	
Aug. 18, 1966	1425-1555	1.5	52	48	8.0	17.4	
Aug. 18, 1966	1630-1830	2	56	44	10.9	16.5	

Every effort was made to select intersections which had similar geometric proportions and which included several in each control type category. It was also desired that each intersection be isolated so that the delay characteristics of the type of control were measured without being greatly influenced by nearby similarly controlled intersections. This was virtually impossible to do, however.

The number of two-way and four-way stop-sign controlled intersections in the vicinity of Austin that had appreciable traffic volumes was severely limited. Thus, the stop-controlled intersections included in this study cover a wide range of geometrical proportions and could not be classified by a simple set of characteristics.

The number of intersections with pretimed and semiactuated signal control as their normal control mode was also limited. In fact, only one pretimed and no semiactuated controlled intersections which were deemed suitable for inclusion in this research effort were found in Austin.

However, several very similar full-actuated intersections were studied. These intersections were first studied in their "as-is" condition and then the controller was modified and the various dial settings changed. The initial, maximum, and vehicle intervals were varied for separate data runs. The controller was then made to operate under pretimed and semiactuated control and delay data under these conditions were recorded. Reference to Table 4.1 will show the extent to which these modifications were made at specific intersections.

This made it possible to investigate the delay characteristics at an intersection operating under several different control modes and provided much valuable data.

The volume-density controlled intersections also were operated under several different settings of the controller.

Even though the study was somewhat hampered by a dearth of specific intersection types, the amount and quality of the collected data enabled the formulation of many worthwhile conclusions and recommendations concerning the operation of intersections under the several control modes. This will be demonstrated in the balance of this report.

CHAPTER 5. STOP-SIGN CONTROLLED INTERSECTIONS

Seven different stop-sign controlled intersections located in Austin, Texas, were studied in the course of this research: two intersections under two-way stop control, four under four-way stop control, and one under both two and four-way stop control. The studies performed at each location are detailed individually in Table 4.1.

DATA COLLECTION

The D3 Recorder was set up at each field location in accordance with the field observation procedure described in Chapter 2 of this report. There were, however, several differences in procedure when working with stop-sign controlled intersections as opposed to signal-controlled intersections.

The most significant difference in procedure was concerned with the measurement of stopped-time delay. In both cases, the delay interval was considered to begin when a vehicle actually stopped. The delay interval for a particular vehicle at a signalized intersection ended when the vehicle resumed its motion, although a subsequent delay interval could be experienced if the vehicle did not clear the intersection on the green signal or if delayed by a left-turning vehicle. The delay interval for stop-sign controlled vehicles ended when the vehicle crossed the stop line on its way through the intersection.

This procedure included some in-motion time as delay time, but it was carried out in this manner because all vehicles are required to stop at the stop line prior to entering the intersection, regardless of their initial location in the queue of vehicles.

Obviously, there were no signal indications to be recorded at stop-sign controlled intersections. Therefore, the only data recorded during a given sweep of the data channels were the number of vehicles in the queue on each approach and the cumulative volume on each approach.

Computed information consisted of the vehicular volume, vehicle-seconds of delay, and the average delay per vehicle for each approach and for the intersection as a whole. Each value was calculated for 5, 15, and 60-minute periods.

TWO-WAY STOP CONTROL

Three intersections in Austin (29th and Jefferson, 19th and Chicon, and 38th and Speedway) were studied under two-way stop control at various times during the day, including the morning and evening peak periods as well as a midday period. Preliminary work showed no discernible evidence that delay characteristics were affected by the time of day for the data recorded in this study. Thus, no further mention of time of day is made in this chapter.

An idealized relationship between vehicle-seconds of delay and vehicular volumes on individual stop-sign controlled approaches is shown in Fig 5.1. Delay time increases at an increasing rate as the approach volume increases; the through-traffic volume ranges from about 65 to 85 percent of the total intersection traffic volume for the data represented by this relationship.

A more meaningful presentation is given in Fig 5.2, in which the sum of vehicle delay on the two stop-sign controlled approaches is plotted as a function of the total volume on all four approaches for 15-minute intervals. It may be observed in Fig 5.2 that delay increases rather gradually to a volume of about 200 to 250 vehicles per 15-minute interval. At this volume, a break in the curve occurs and delay increases quite sharply with further volume increases.

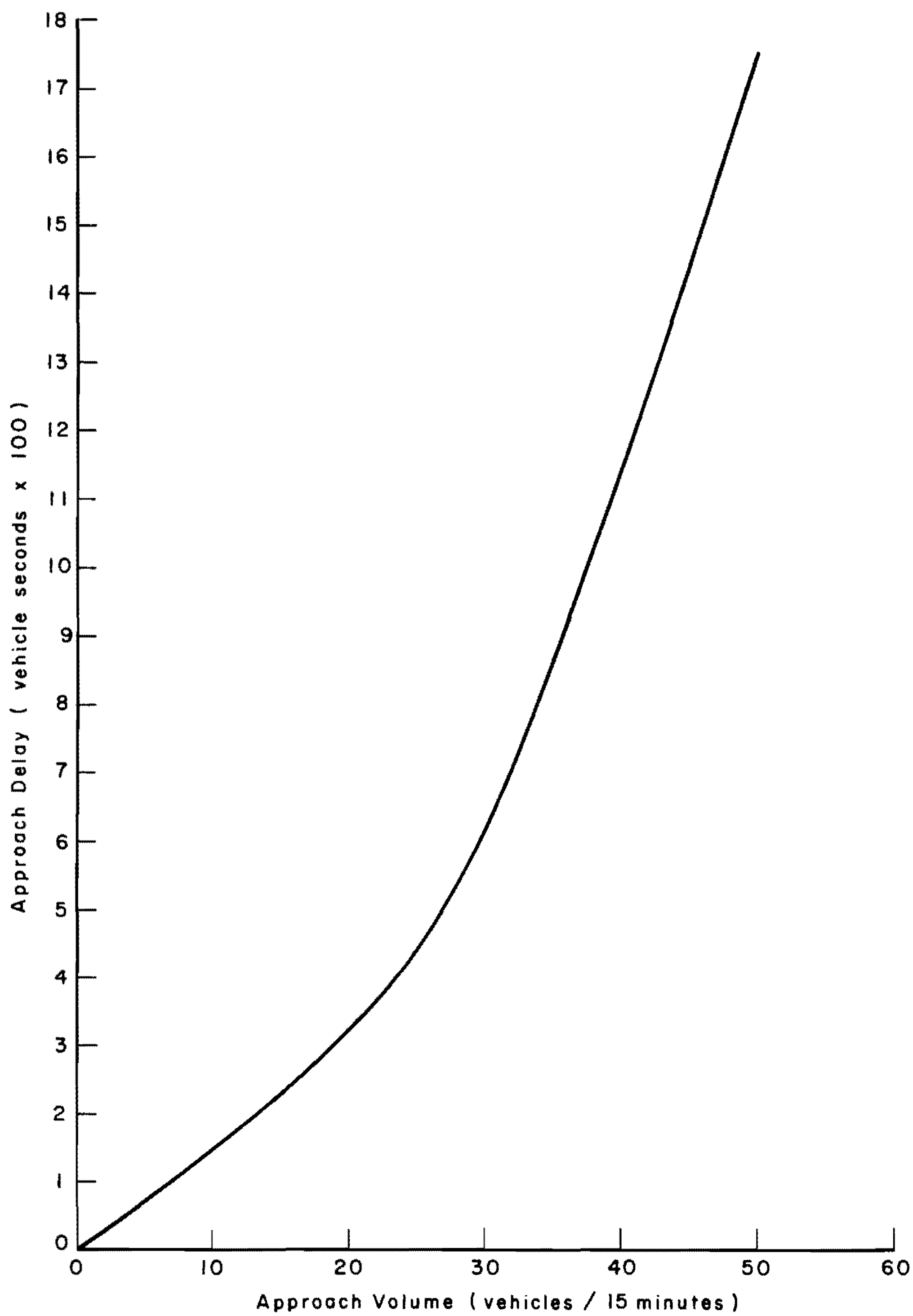

Fig 5.1. Idealized relationship of delay versus volume by approach. Two-way stop control, 15-minute intervals.

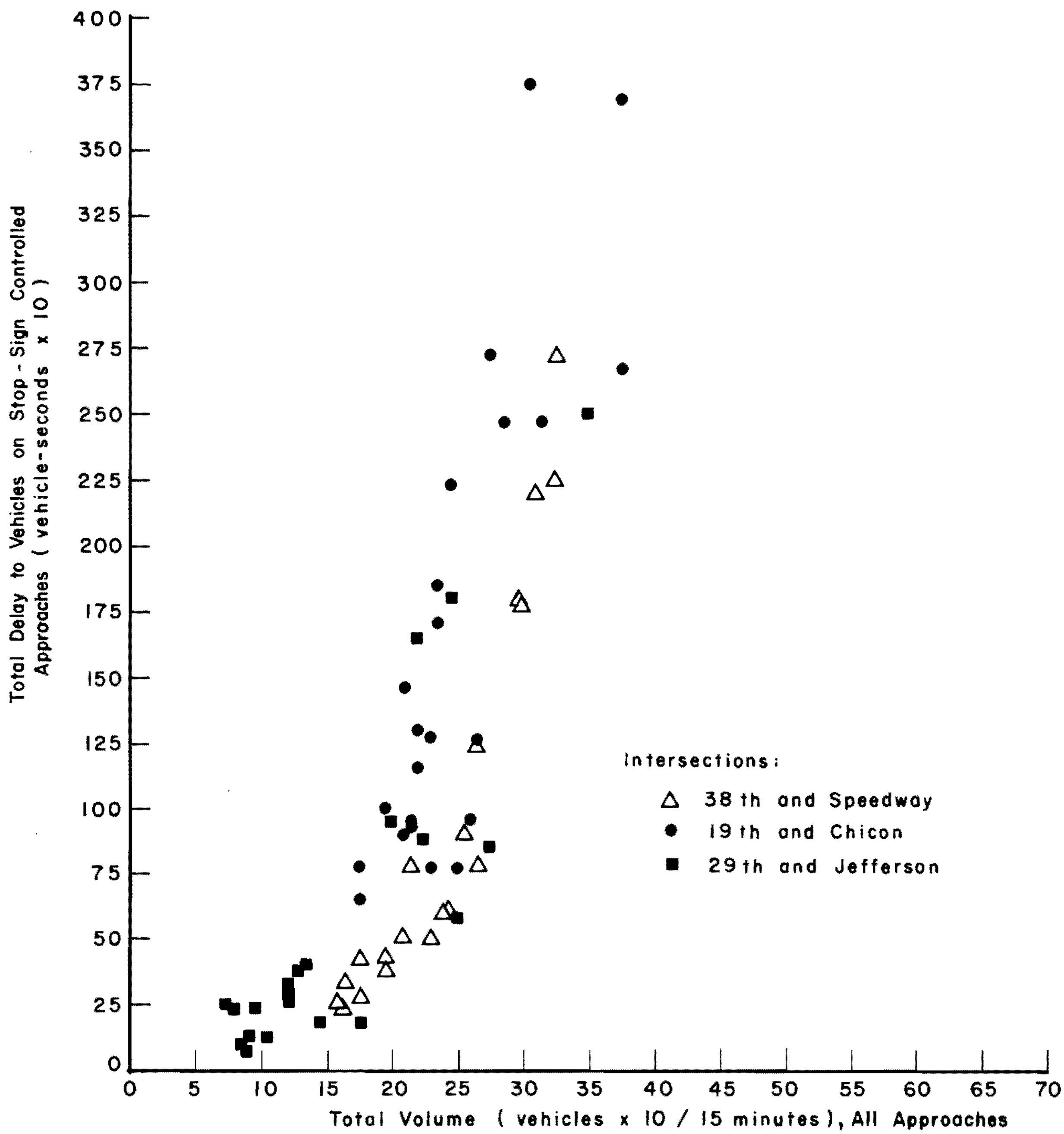

Fig 5.2. Total delay to vehicles on stop-sign controlled approaches versus total volume. Two-way stop control, 15-minute intervals.

The BMD-2R statistical computer program for stepwise multiple regression (Ref 5) was used in an effort to explain some of the variability in the data collected in this and subsequent phases of the research reported here. The data illustrated in Fig 5.2 were subjected to this analysis.

The following model was developed:

$$y = 456 - 6.89\, x_1 + 0.08464\, x_2 - .0712\, x_3$$

where

y = total vehicle-seconds of delay on the stop-sign controlled approaches for 15-minute intervals,

x_1 = the total volume (on all four approaches),

x_2 = the square of the total volume,

x_3 = the square of the through volume.

This model had an R^2 of 0.834 and a root mean square of 375. This was based on a total of 64 data sets covering the three intersections studied in research. A graph of this model is shown in Fig 5.3. This model is used later in this chapter to develop a set of volume warrants for four-way stop installations.

Another approach used in explaining some of the variability is illustrated in Fig 5.4. In this case, the data of Fig 5.2 were aggregated into one-hour rather than 15-minute intervals. The striking linearity exhibited by the data for 38th and Speedway and 19th and Chicon are readily apparent. The explanation for the translation of one line relative to the other is not quite so apparent. It does seem reasonable, however, to attribute this translation

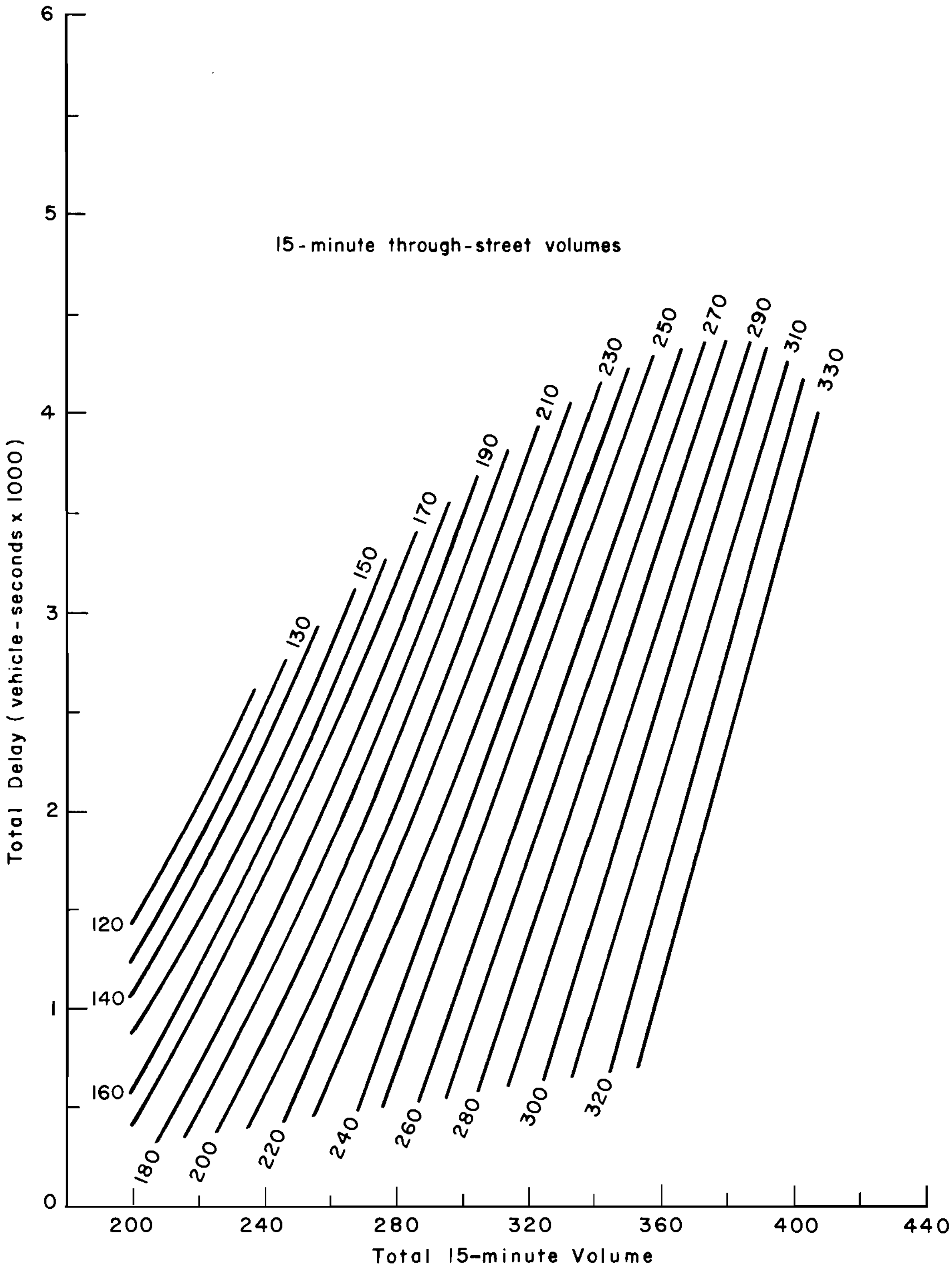

Fig 5.3. Volume versus delay relationships for two-way stop controlled intersections.

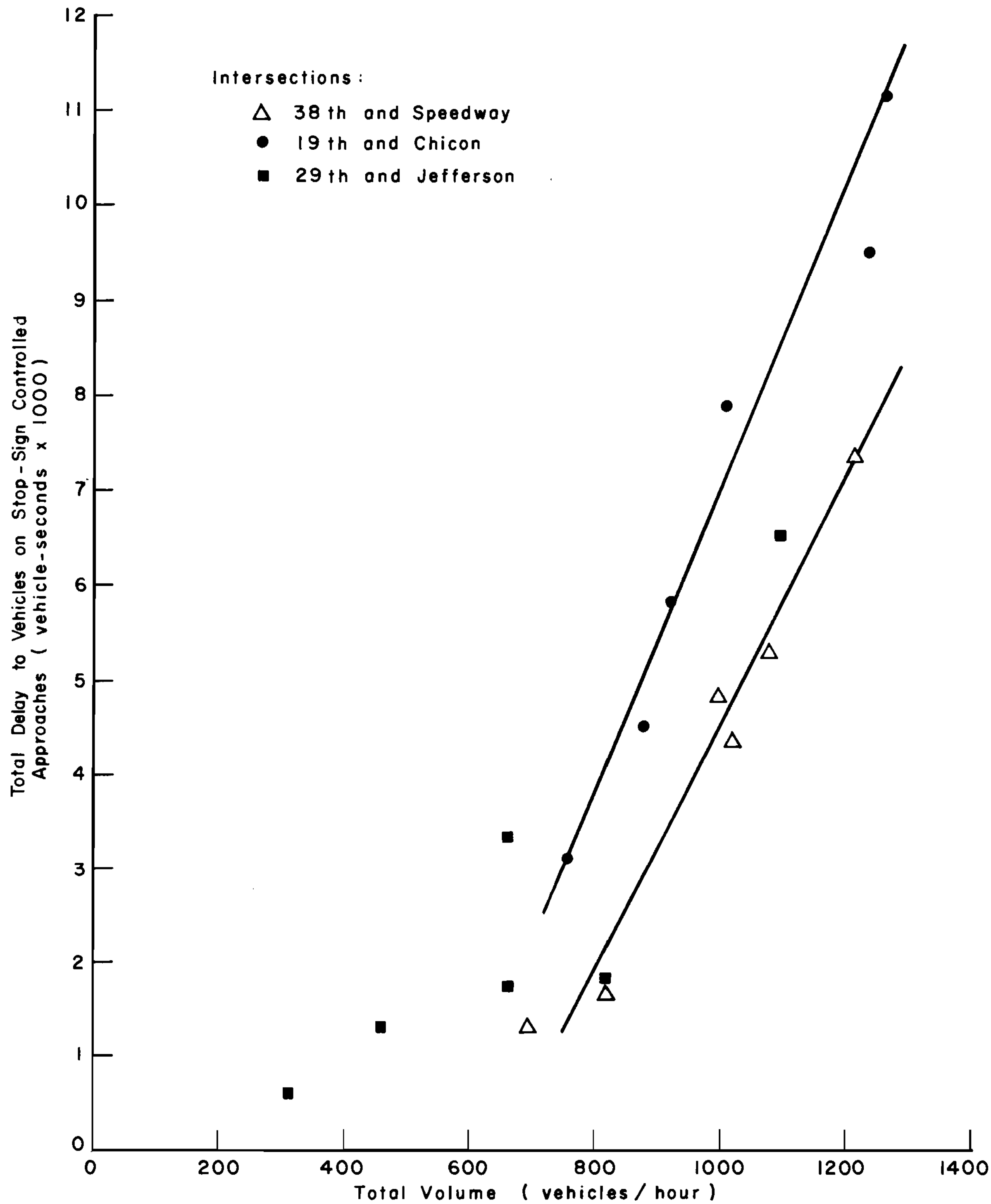

Fig 5.4. Total delay to vehicles on stop-sign controlled approaches versus total volume. Two-way stop control, one-hour intervals.

to geometric differences at the two intersections. The sight-distance restrictions at 19th and Chicon were severe in comparison to those at 38th and Speedway. This fact could explain the higher delay experience at 19th and Chicon.

Some idea of average delays may be obtained by reference to Figs 5.5 and 5.6. The average delay per stop-sign controlled vehicle is shown in Fig 5.5 as a function of the 15-minute total intersection volume. The relationship appears to be linear for 38th and Speedway, but there seems to be a break in the curve at about 250 vehicles per 15 minutes at 19th and Chicon. This may again be attributed to the aforementioned sight-distance restrictions.

The average delay of both stop-sign controlled vehicles and all vehicles as a function of total hourly intersection volume is shown in Fig 5.6.

Perhaps a more significant reason for the higher delays observed at 19th and Chicon was the fact that the intersection normally operated under four-way stop control rather than the sight distance restrictions *per se*. The intersection was converted to two-way control and the data collected after a one-week period of driver adjustment. This may not have been enough time for the everyday drivers to adjust completely to the change.

FOUR-WAY STOP CONTROL

Five intersections in Austin (Woodrow and Justin, North Loop and Woodrow, 19th and Chicon, 15th and Congress, and Balcones and Hancock) were studied under four-way stop-sign control, each at various times during the day.

An idealized relationship between vehicle-seconds of delay and vehicular volumes on individual approaches is shown in Fig 5.7. These curves are based on data collected at 19th and Chicon for both two and four-way stop operation. For the same approach volume, the total delay and the average delay were greatly

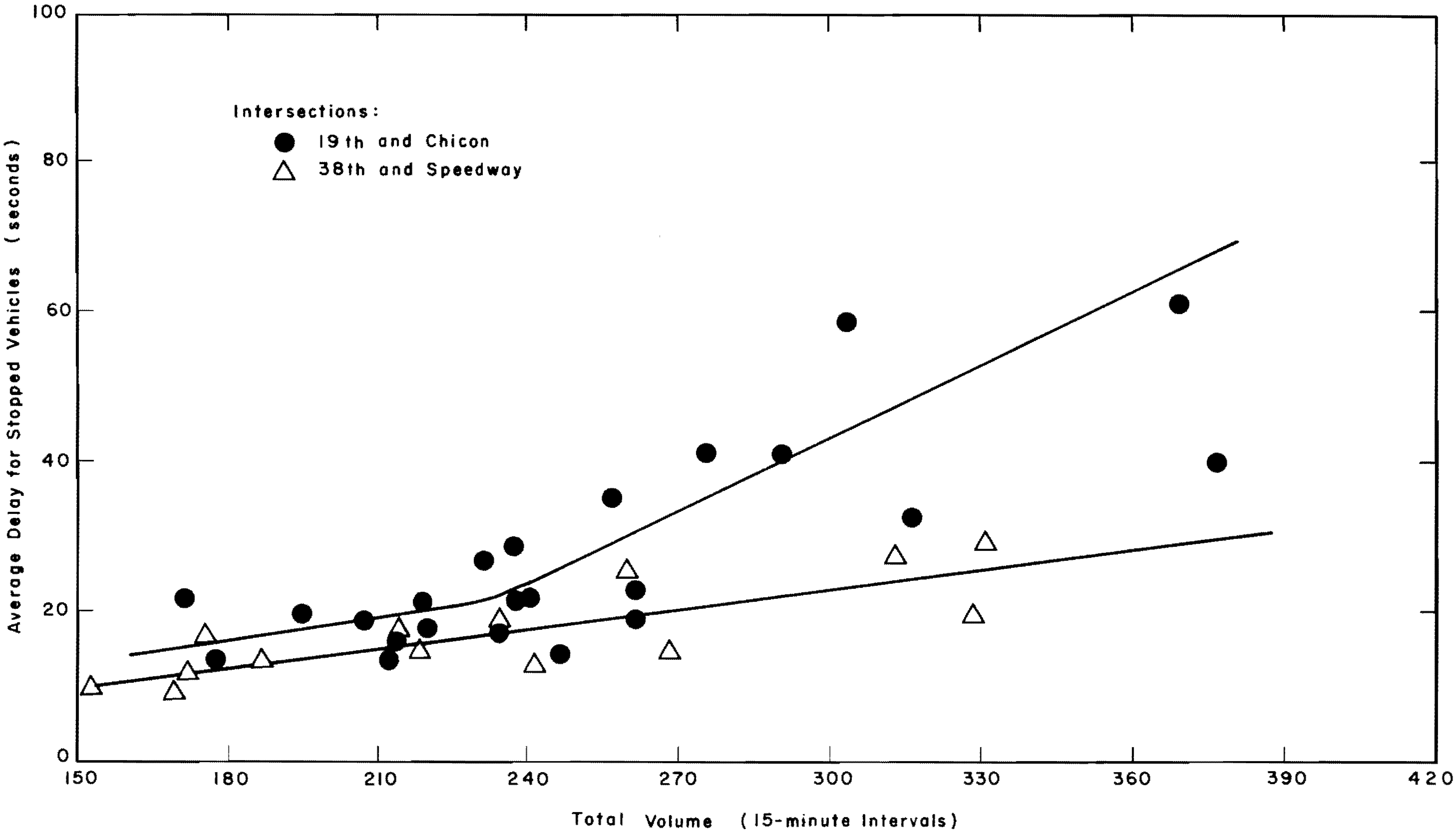

Fig 5.5. Average delay versus total volume. Two-way stop, 15-minute intervals.

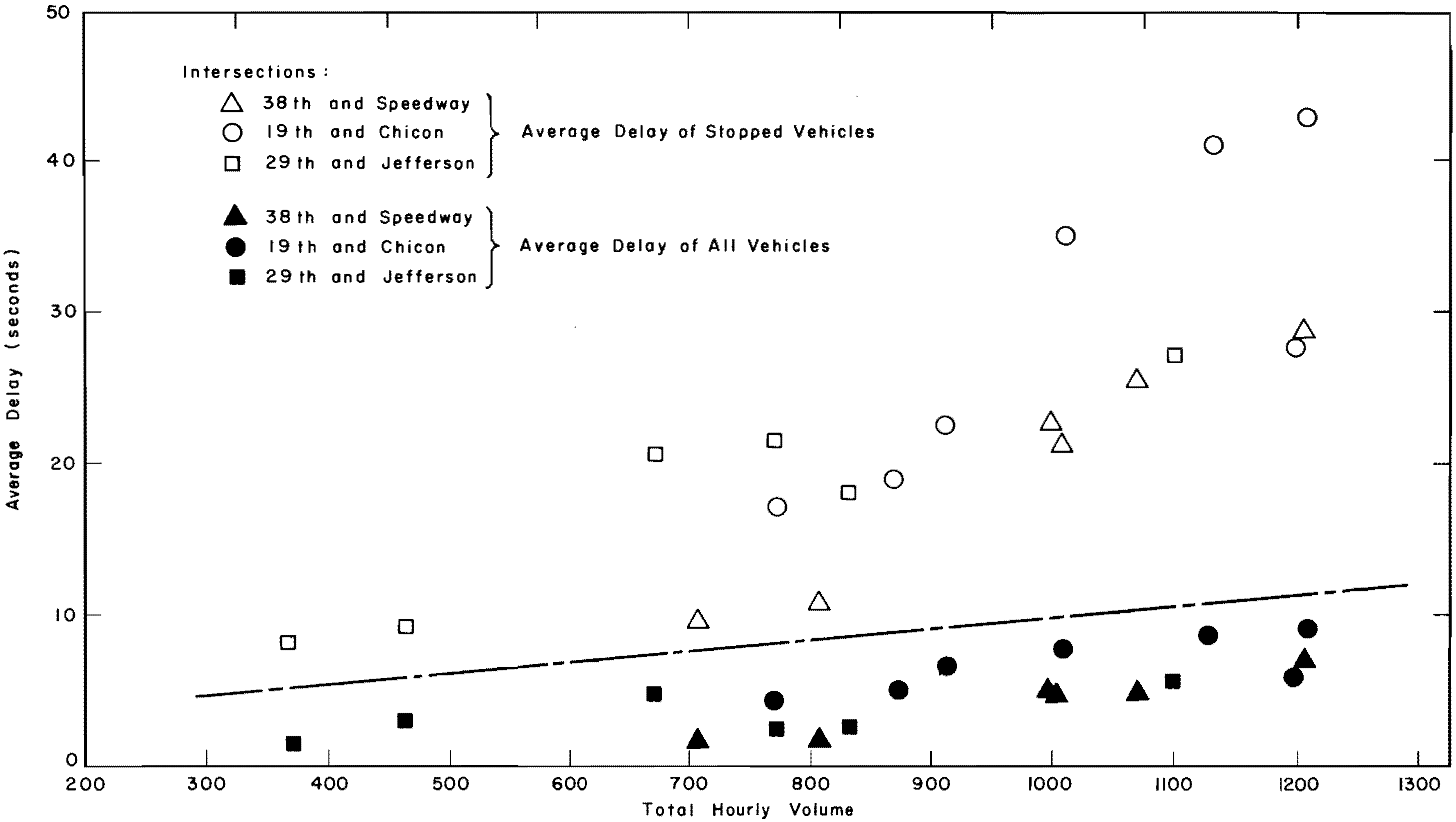

Fig 5.6. Average delay versus total volume. Two-way stop, hourly intervals.

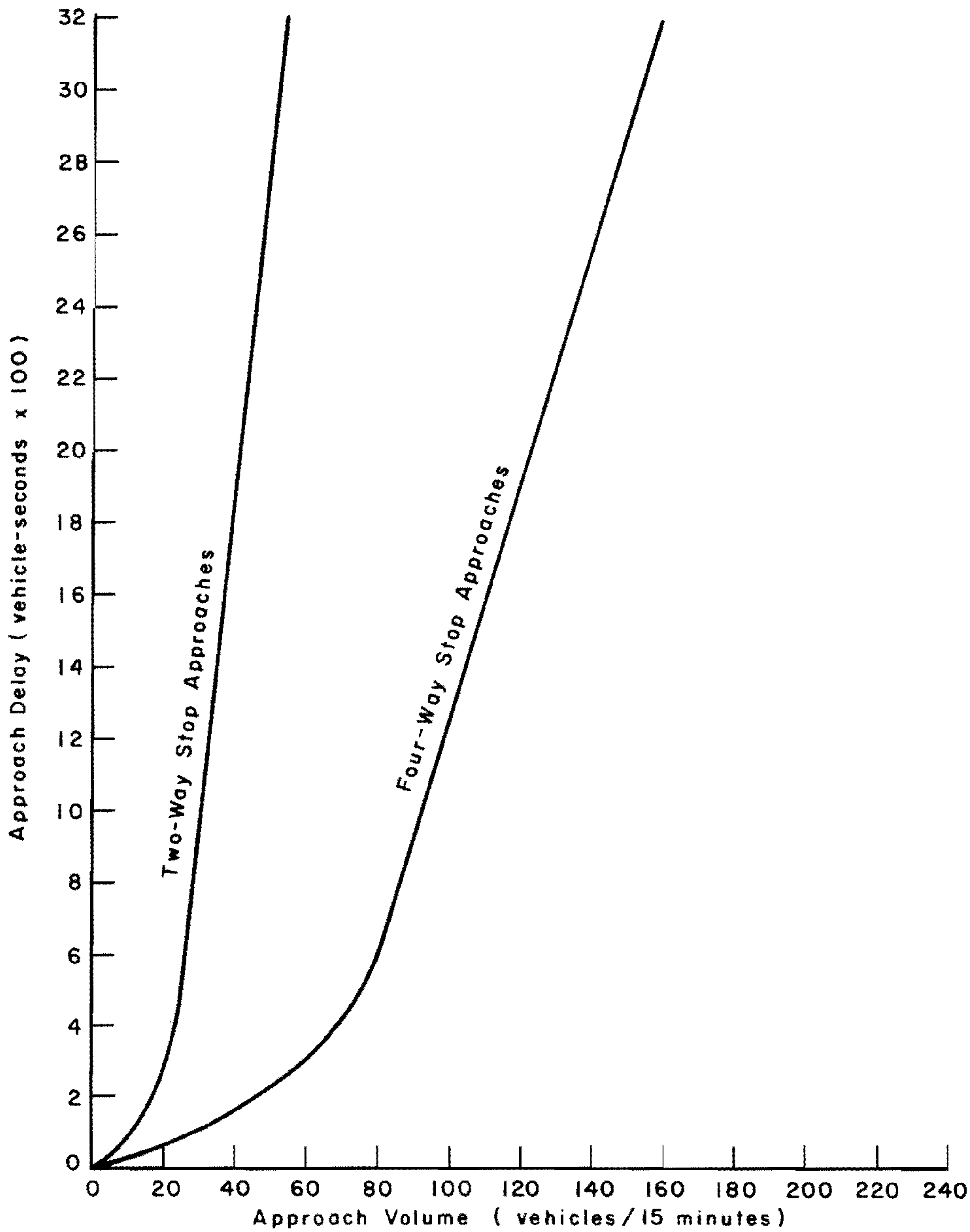

Fig 5.7. Idealized delay versus volume by approach. Two and four-way control at 19th and Chicon, 15-minute intervals.

reduced for a stop-sign controlled approach when intersection control was changed from two-way to four-way stop control.

However, the total delay experienced on all intersection approaches is greater for four-way than for two-way stop control for equal volumes. This is due, of course, to the fact that all traffic must stop and suffer delay under four-way stop control but only minor-street traffic must stop under two-way stop control.

The relationship between total delay and total volume for four-way stop control is shown in Fig 5.8. A direct comparison of Figs 5.2 and 5.8 illustrates the larger total delay experienced at four-way stop controlled intersections. Thus, a reduction in average delay experience (for the stopped vehicles) must be traded off with an increase in total delay when converting from two-way to four-way stop control.

It is significant to note that in Fig 5.8 the plotted data were observed at five different intersections. The consistency of these data is rather marked and indicates that a strong relationship exists. A regression yielded the following model:

$$y = -420 + .05147\ x^2$$

where

y = the total vehicle-seconds of delay per 15-minute interval,

x = the total vehicular volume per 15-minute interval.

This particular relationship had an R^2 of 0.897.

It is of interest to note that if a square-root transformation is made on the delay variable, a regression yields a relationship having an R^2 of 0.984 with the following functional form:

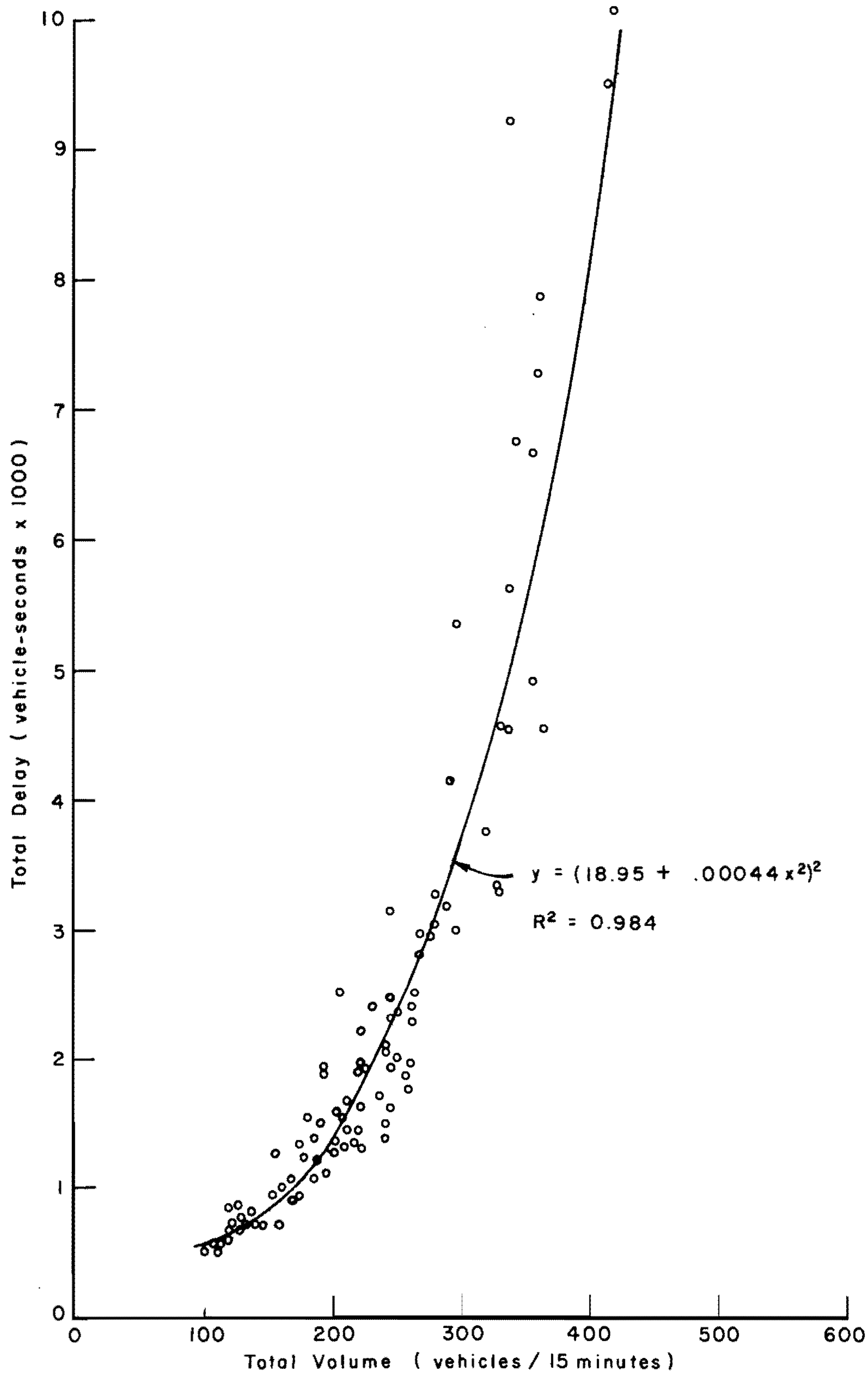

Fig 5.8. Total delay versus total volume. All intersections, four-way stop control, 15-minute intervals.

$$y = (18.95 + .00044\ x^2)^2$$

where the variables are as defined immediately above. This relationship is plotted in Fig 5.8. A square-root transformation is often of value in working with data which are Poisson-distributed. The hypothesis of Poisson-distributed data could not easily be tested, however.

Some effort was expended in working with the delay characteristics of individual approaches. While some very good models (by virtue of a high R^2) were formulated through regressions, a usable model characteristic of approaches in general was not formulated. In almost all instances, approach delay appeared to be a function of the approach volume raised to both the first and second powers and the total volume squared.

An example of the delay characteristics by approach is shown in Fig 5.9. Hourly totals of delay and volume for each approach at the five intersections studied have been plotted. It is observed that the total delay begins to increase much more rapidly at approach volumes above 300 vehicles per hour than it does at volumes below 300 vehicles per hour. The lines on Fig 5.9 may be thought of as the lower limit of delay at given approach volumes.

Hourly totals of delay and volume for all intersection approaches are shown in Fig 5.10. In this case, delays appear to begin increasing very rapidly at an intersection volume of about 900 vehicles per hour.

Average delays are illustrated in Fig 5.11. The actual data points are not plotted in Fig 5.11, to avoid cluttering the figure. However, least-square lines which were fitted to the data are plotted (Ref 8). The volume break point, above which the average delay increases rapidly, ranges from about 270 vehicles per 15-minute interval to about 330 vehicles per 15-minute interval.

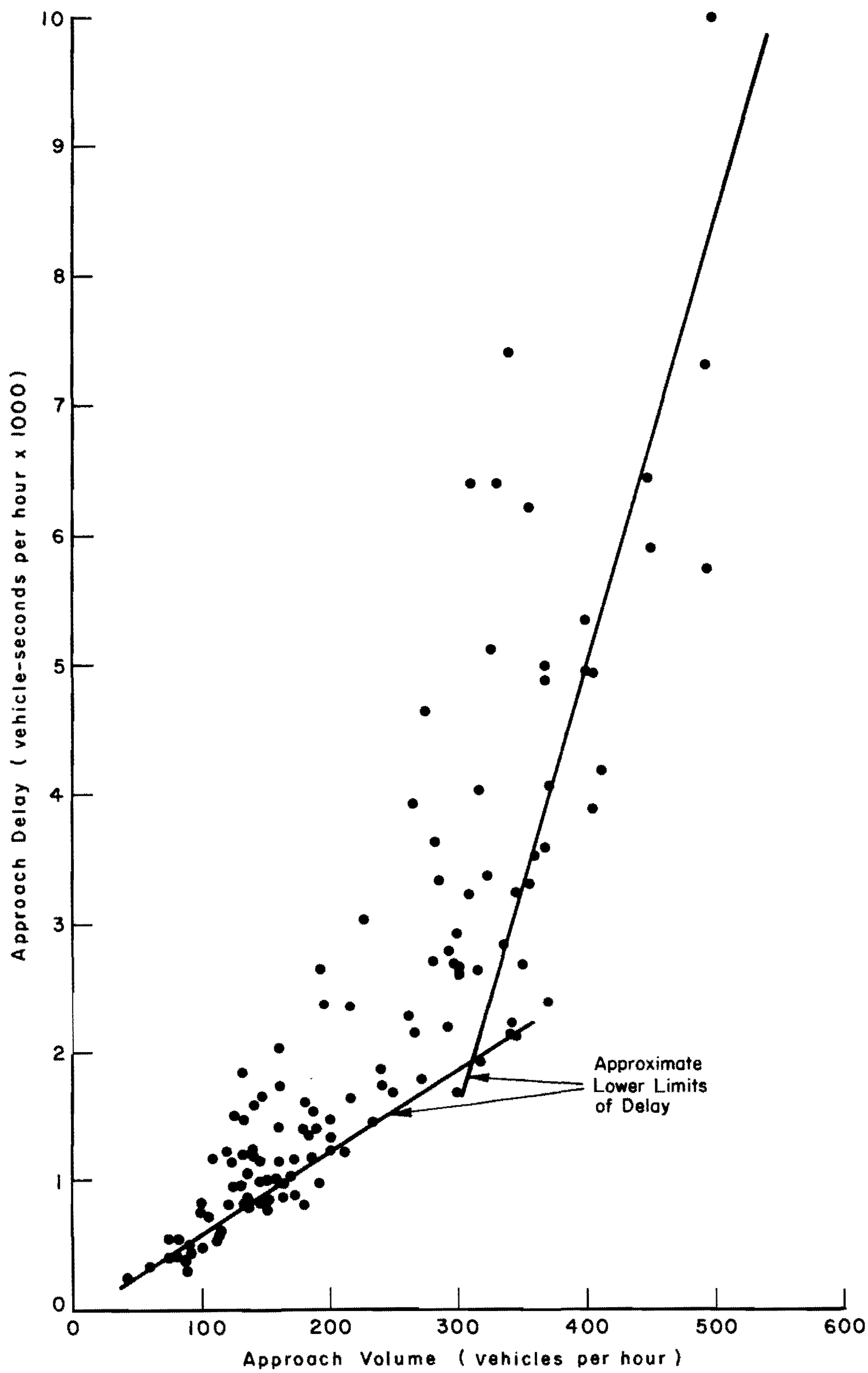

Fig 5.9. Approach delay versus approach volume. One-hour totals, four-way stops.

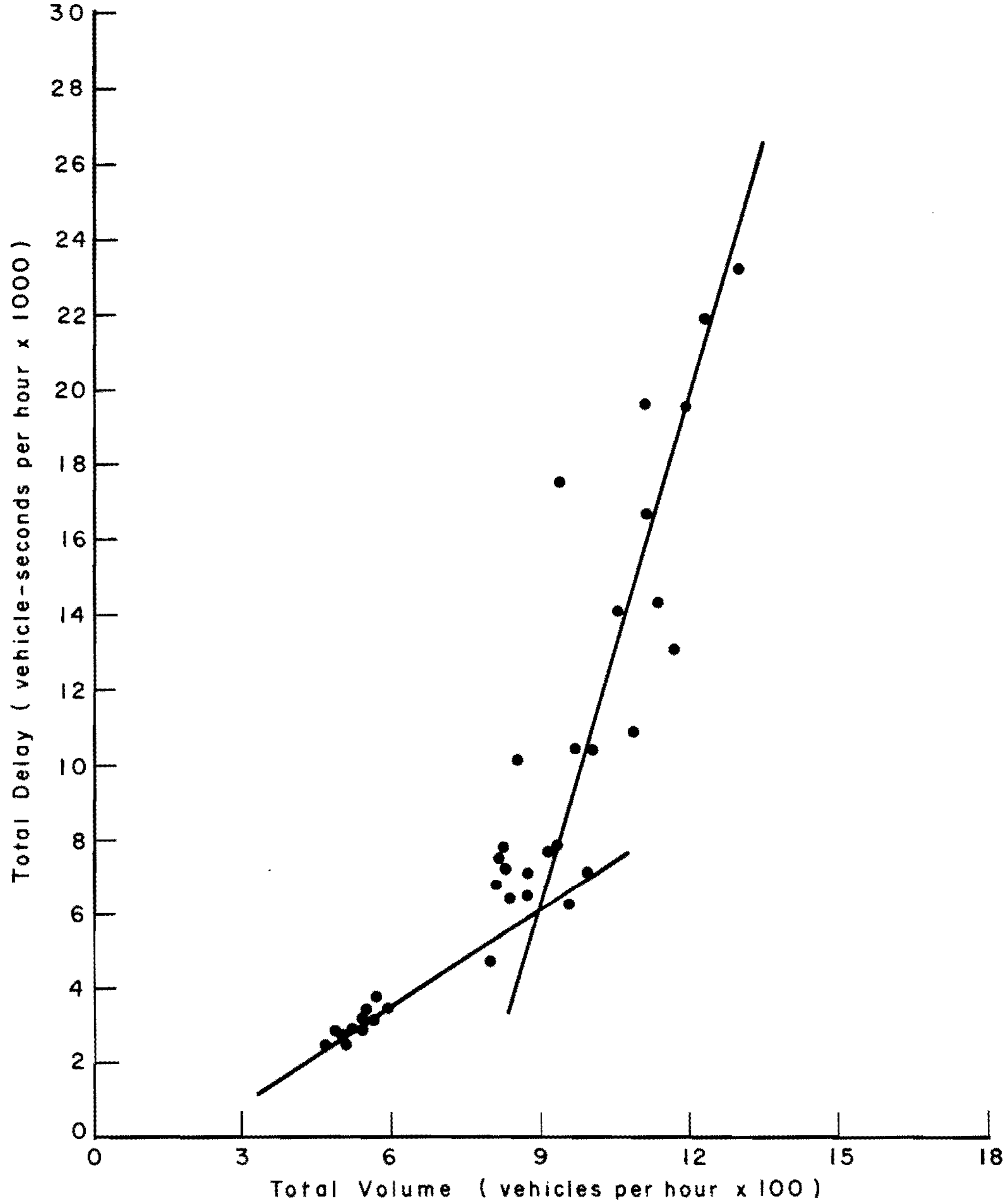

Fig 5.10. Total delay versus total volume. One-hour totals, four-way stops.

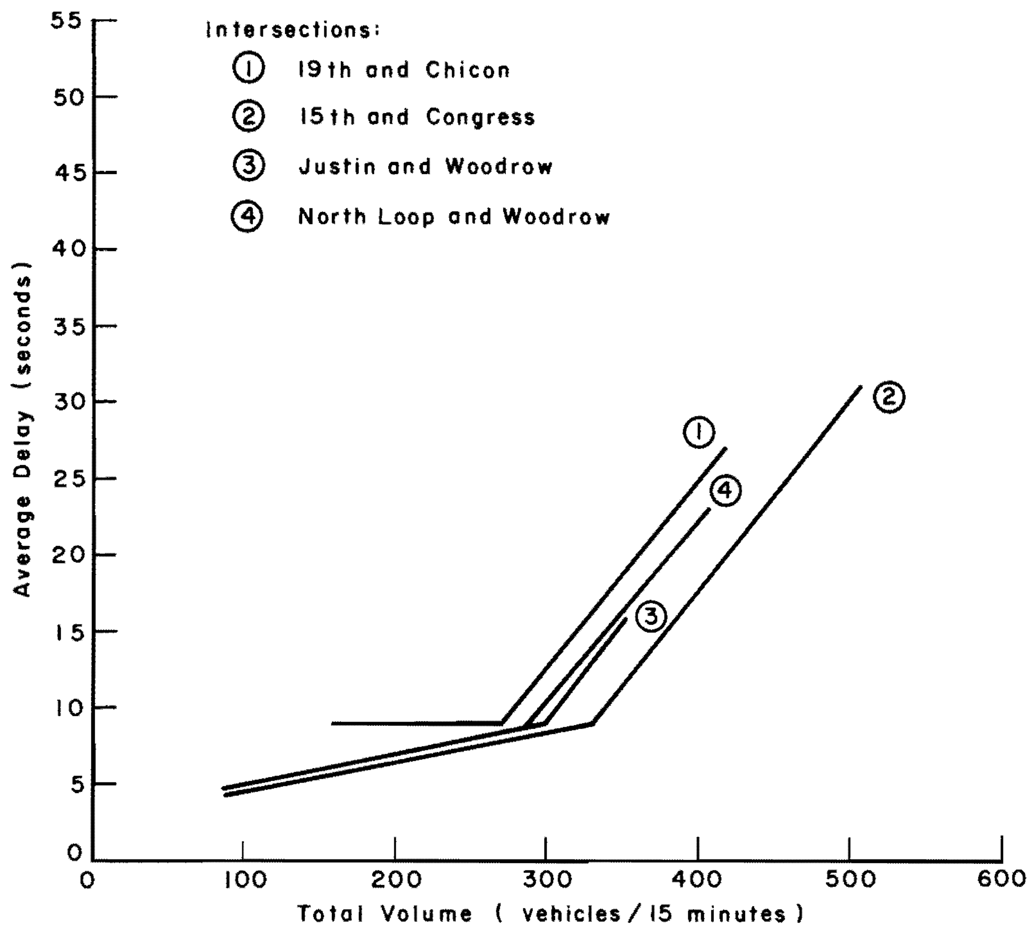

Fig 5.11. Average delay versus total volume. Fifteen-minute intervals, four-way stops.

The slope of the line for volumes greater than the break point is virtually a constant for each of the intersections listed in Fig 5.11. The slope is about one vehicle-second per eight vehicles per 15-minute interval. Stated in another way, at total volumes above approximately 300 vehicles per 15-minute interval, the average delay to all vehicles increases by one second for each increase in total volume of eight vehicles.

In comparing two-way and four-way stop operation, Figs 5.2 and 5.4, for two-way stops, are directly comparable to Figs 5.8 and 5.10 for four-way stops, respectively. Reference to Figs 5.2 and 5.8 shows that total delay began to increases very rapidly at total volumes of from 200 to 250 vehicles per 15-minute interval for two-way stops and from 250 to 300 vehicles per 15-minute interval for four-way stops. These 15-minute volumes of 250 and 300 vehicles may be termed the critical volumes for two-way and four-way stops, respectively.

The corresponding critical hourly volumes are 750 and 900 as determined from Figs 5.4 and 5.10, respectively. At volumes greater than critical, Fig 5.4 shows an increase in total delay of about 13 to 15 vehicle-seconds for each unit increase in total volume at a two-way stop controlled intersection. On the average, one in every four vehicles must stop if the major-minor street traffic split is about 75/25, which was approximately the average split observed in this study at two-way stops. Thus, the equivalent of 50 to 60 vehicle-seconds of additional delay was observed for each stopped vehicle.

Fig 5.10 shows an increase in delay of about 50 seconds for each additional vehicle over the critical volume at four-way stop intersections. Because all vehicles must stop at four-way stops, the increase in delay per stopped vehicle approximates 50 seconds at volumes just beyond the critical volumes for both two-way and four-way stop control. The value of 50 seconds increases as the volume increases. These values of additional delay can be shown to be approximately the minimums required to satisfy the average delay data illustrated in Figs 5.5 and 5.11.

There was some indication that the critical volume range of 250 to 300 vehicles per 15-minute interval was dependent upon the traffic split. It appeared that the upper value of 300 was associated with a 50/50 split and that the critical volume decreased as the deviation from a 50/50 split increased. However, the data were insufficient for this indication to be considered.

Additionally, the data collected at 19th and Chicon for both two-way and four-way stop controlled operations are directly comparable. Hourly averages of total delay and volume as well as average vehicular delays were calculated for each of the eight studies performed at the intersection of 19th and Chicon. The results are listed in Table 5.1.

The combined data represented in Table 5.1 indicate what occurred when control was changed from two-way to four-way stop. Chicon experienced a 15 percent increase in traffic volume and a 62 percent reduction in total delay when the intersection was converted from two-way to four-way stop operation. It is not known whether or not the volume increase was due to the change in operation, but the reduction in delay certainly was attributable to the change in control.

The traffic volume on 19th Street was virtually unchanged, but delay was increased considerably. As a consequence, the intersection had a 4 percent increase in traffic volume and an 86 percent increase in total delay when the conversion was effected. However, the average delay for stopped vehicles was 27.7 seconds under two-way stop control and 11.7 seconds under four-way stop control, a reduction of 58 percent.

These same types of comparisons can be made for the morning and evening peak hours, and they might be more impressive. However, excessive average

TABLE 5.1. COMPARISON OF TWO-WAY AND FOUR-WAY DELAY CHARACTERISTICS AT 19TH AND CHICON (HOURLY AVERAGES)

		19th Street		Chicon		Total			Average Delay, sec		
									Stopped Vehicles		
Time Period	Stop Control	Volume, vph	Delay, veh-sec	Volume, vph	Delay, veh-sec	Volume, vph	Delay, veh-sec	Major-Minor Split, %	19th	Chicon	All Vehicles
Morning	Two-way	764	--	220	5,148	984	5,148	78/22	0.0	23.4	5.2
	Four-way	838	13,915	297	2,600	1,135	16,516	74/26	16.6	8.8	14.6
Afternoon	Two-way	646	--	246	5,147	892	5,147	72/28	0.0	20.9	5.8
	Four-way	701	4,659	279	1,970	980	6,629	72/28	6.6	7.1	6.8
Evening	Two-way	886	--	244	9,707	1,130	9,707	78/22	0.0	39.8	8.6
	Four-way	835	13,228	291	3,384	1,126	16,613	74/26	15.8	11.6	14.8
Combined	Two-way*	760	--	236	6,535	996	6,535	76/24	0.0	27.7	6.6
	Four-way**	766	9,687	271	2,476	1,037	12,162	74/26	12.6	9.1	11.7

* Represents 5.75 hours of data.

** Represents 8.25 hours of data.

delays to stopped vehicles under two-way stop control can be ameliorated effectively by conversion to four-way stop control.

Some measure of what constitutes excessive average delay is needed but it should be remembered that total delay increases drastically when the control mode is changed to a four-way stop.

DISCUSSION OF RESULTS

In studying the characteristics of intersections, many variables deserve consideration; including directional volumes, turning movements, approach speeds, width and number of lanes, truck and pedestrian traffic, intersection geometry, and distance to adjacent intersections, among others. In almost all cases in this study, such factors as directional volumes, lane widths, intersection geometry, and the location of adjacent intersections were measured or could be determined. Truck and pedestrian traffic was very minor and was considered to have negligible effects in most instances.

Little data on turning movements and approach speeds were available. Some manual counts of left-turn movements were kept, but these did not appear to have much influence on the delay characteristics of the intersections studied. In general, for almost all variables other than delay and volume, the range of the recorded varaible was so limited that its significance, if any, was masked.

Occasionally such information as the sight-distance restrictions at 19th and Chicon helped to explain the greater delays experienced at that intersection than at others, especially for two-way stop control. However, few of these variables appeared to have much influence on the delay characteristics as four-way stop controlled intersections. This was pointed out in Fig 5.8, in which the data from five different intersections were plotted without distinction. Actually, there was no observed differentation among the data when plotted.

The geometric layout of each intersection studied under stop-sign control is included in Appendix A. Reference to these drawings will show marked differences in geometry, but these seemingly did not influence delay characteristics.

It is of particular importance to recognize that no conclusion is drawn regarding the irrelevance of these variables to delay characteristics other than in the limited range to which the variables were included in the studies reported. Additional studies designed especially to measure the influence of these variables must be carried out if the variable are to be understood thoroughly.

The Highway Capacity Manual (Ref 13) lists some basic (ideal) capacities in terms of total vehicles per hour for various intersection types. These capacities are 1900 for a 2 x 2, 2800 for a 2 X 4, and 3600 for a 4 x 4 intersection type. Corresponding practical capacities are given as 1200, 1800, and 2200 vehicles per hour. The largest one-hour volume observed at a four-way stop intersection in this study was about 1300. The fact that the observed volumes at intersections of various types were much smaller than the suggested capacities helps to explain why little or no influence due to intersection geometry was detected in this study. Apparently, the traffic volumes were not large enough with respect to capacity to allow geometry to appear as a significant influence on the delay characteristics of different intersections.

An analysis was made in an effort to gain some information on the departure distribution at four-way stop intersections. A set of data was observed in which the intervals between successive departures were counted. The range in the number of intervals between successive departures was from zero to six. On interval in this study was equivalent to 1.44 seconds.

The frequency distribution is as listed below:

No. of Interval	Frequency (percent)
0	14.5
1	36.9
2	25.2
3	19.2
4	2.2
5	1.3
6	0.6

These intervals were recorded only when at least two approaches were continuously occupied by stopped vehicles. A total of 317 observations were made. This sample had a mean of 1.64 intervals and a variance of 1.32. The mean of 1.64 intervals in equivalent to 2.36 seconds. A simple statistical test will show that this mean is not significantly different from the 2.4 seconds that represents a departure rate of 1500 vph through the four approaches of a stop-sign controlled intersection where a supply of stopped vehicles is always available for departure.

This is significant because the Highway Capacity Manual (Ref 13) states that a line of vehicles stopped by an interruption will only rarely move away from the interruption at a rate greater than 1500 passenger cars per lane per hour. The data collected in this study suggest that this also applies to four-way stop intersections. However, capacities may be greater than the figure of 1500 vehicles per hour for several reasons. A very important one is the lane configuration. With two lanes on an approach and a continuous supply of vehicles, it should be possible to have twice as many departures per time interval as with one approach lane. The important point however, is that the additional efficiency afforded by multilane approaches appears to be effected only at relatively high volumes.

A similar analysis was made when there was a vehicle on only one approach. In order to eliminate the effects of interference, a vehicle's delay time was observed only if no one had departed within two intervals of arrival and if no other vehicles arrived within four intervals of departure. This would provide a measure of delay experience with virtually no restriction on movement except the stop sign and the driver's capability. In this case, the mean value for initial vehicle delay was about 3.5 seconds.

WARRANTS

The generally accepted warrants pertaining to the installation of stop-signs, yield signs, and the various types of signals are published in the Manual on Uniform Traffic Control Devices (Ref 18). The purpose of these signs and signals is to assign right-of-way to traffic on the approaches of an intersection where conditions of hazard exist such that an uncontrolled intersection is not feasible and the normal rule, " the vehicle on the right has the right-of-way," cannot be applied safely or efficiently.

The normal hierarchy of control devices, with respect to both cost and effectiveness, is probably the following: yield sign; two-way stop sign; four-way stop sign; and the several signal configurations, including pretimed, semi-actuated, full-actuated, and volume-density devices.

In general, a yield sign is employed for special intersection configurations such as channelized right-turn lanes, intersections with a divided highway, or ramp entrances with inadequate or no acceleration lanes. Yield signs should also be considered applicable at intersections where stop signs are warranted but visibility and speed conditions are such that a full stop is not necessary for safety.

Stop signs may be warranted at almost any intersection of a minor road with a main road or an intersection of two main roads, at an unsignalized intersection in a signalized area, and at railroad crossings. However, stop signs are warranted at any intersection where hazard or accident history indicates a need for stop-sign control. Generally, the two opposing minor-stream flows are stopped while the larger, major-stream flows are not stopped. Under certain conditions, all four approach flows must stop, necessitating four-way stop control, for which the Manual (Ref 18) lists more specific warrants, as opposed to the general policy outlined for yield and two-way stop control.

A four-way stop may be used as temporary measure at an intersections to be signalized and at an intersection with turning and right-angle accidents accumulating to at least five within a 12-month period. In addition, certain minimum traffic volumes are established:

(1) The total, all-approach vehicular volume must average at least 500 vehicles per hour for any eight hours of an average day.

(2) The combined vehicular and pedestrian volume from the minor approaches must average at least 200 units per hour for the same eight hours with an average delay of 30 seconds per vehicle or more for the minor-street traffic during the maximum hour.

(3) The volume warrants are reduced to 70 percent of those given above when the 85-percentile approach speed of major-street traffic exceeds 40 miles per hour.

The Manual (Ref 18) suggest, among several qualifications regarding the installation of stop signs, that a four-way stop not be used where the traffic volumes on the intersecting streets are very unequal. If the volumes are heavy enought to warrant additional controls in this instance, a signal installation might be preferrable. Chapter 7 of the present report includes a discussion of traffic-signal warrants.

RECOMMENDATIONS

It does not appear practical to specify other than general policy statements, such as those given on pages 27 and 30 of the Manual (Ref 18), as warrants for two-way stop and yield signs, respectively. The installation of these types of devices is often discretionary on the part of the traffic engineer.

Citizens often demand an increase in intersection control in their residential neighborhoods. The choice of control, if any, at these low-volume locations is between yield signs and two-way stop signs. Yield signs should be considered where a full stop is not necessary and where sight distances are adequate. They should not be used as substitutes for stop signs if stop signs are warranted. It should be recognized that demands from citizens in residential neighborhoods often stem from a desire to limit speeds rather than to control intersections *per se*.

However, the primary concern at this stage is the warrants for the installation of four-way stop signs.

Traffic Split as a Warrant

The warrants as presented on page 28 of the Manual on Uniform Traffic Control Devices begin with the statement that four-way stop signs should not be used where the intersecting flows are very unequal. The results of this study show that the total delay experienced at four-way stop intersections is virtually unaffected by traffic splits ranging from 50/50 to about 80/20 (Fig 5.8).

Table 5.1, however, shows that the higher-volume approaches tend to have higher average delays, but this is probably the result of the relatively high volume rather than of the unequal intersecting flows. Furthermore, the data give no indication of any influence on delay due to the traffic split when plotted on an approach basis (Fig 5.9).

It appears reasonable to conclude that total vehicle-seconds of delay at four-way stop intersections are not seriously influenced by unequal traffic splits as great as 80/20 where the total intersection volume is limited to between 1300 and 1400 vehicles per hour for 4 x 4 intersections and between 1000 and 1100 vehicles per hour for 2 x 2 intersections. These were the greatest hourly volumes observed at these intersection types in this study. Of course, this does not imply that the delay experience at these volumes is satisfactory.

Therefore, it is recommended that when the installation of a four-way stop sign is under consideration, the traffic split not be a factor in making the decision. At larger volumes at which the traffic split might be a factor, a signal installation, rather than a four-way stop installation, should be given consideration.

Average Delay as a Warrant

The minimum-volume warrant suggests an all-approach total averaging at least 500 vehicles per hour for any eight hours of an average day, at least 200 vehicles and pedestrians entering from the minor street, and an average delay of at least 30 seconds per vehicle during the maxiumum hour.

The critical all-approach hourly volume for two-way stop intersections has been established as approximately 750 vehicles per hour (Fig 5.4). At greater volumes, the delay begins to increase very rapidly. Thus, the suggested value of 500 vehicles per hour appears to be conservative on a vehicular delay basis. However, the average of eight hours of an average day may represent the 1000th to 1200th highest hour of the year.

The warrant also suggests an average delay of 30 seconds per vehicle during the maximum hour. Fig 5.12 shows average delay per stopped vehicle at a two-

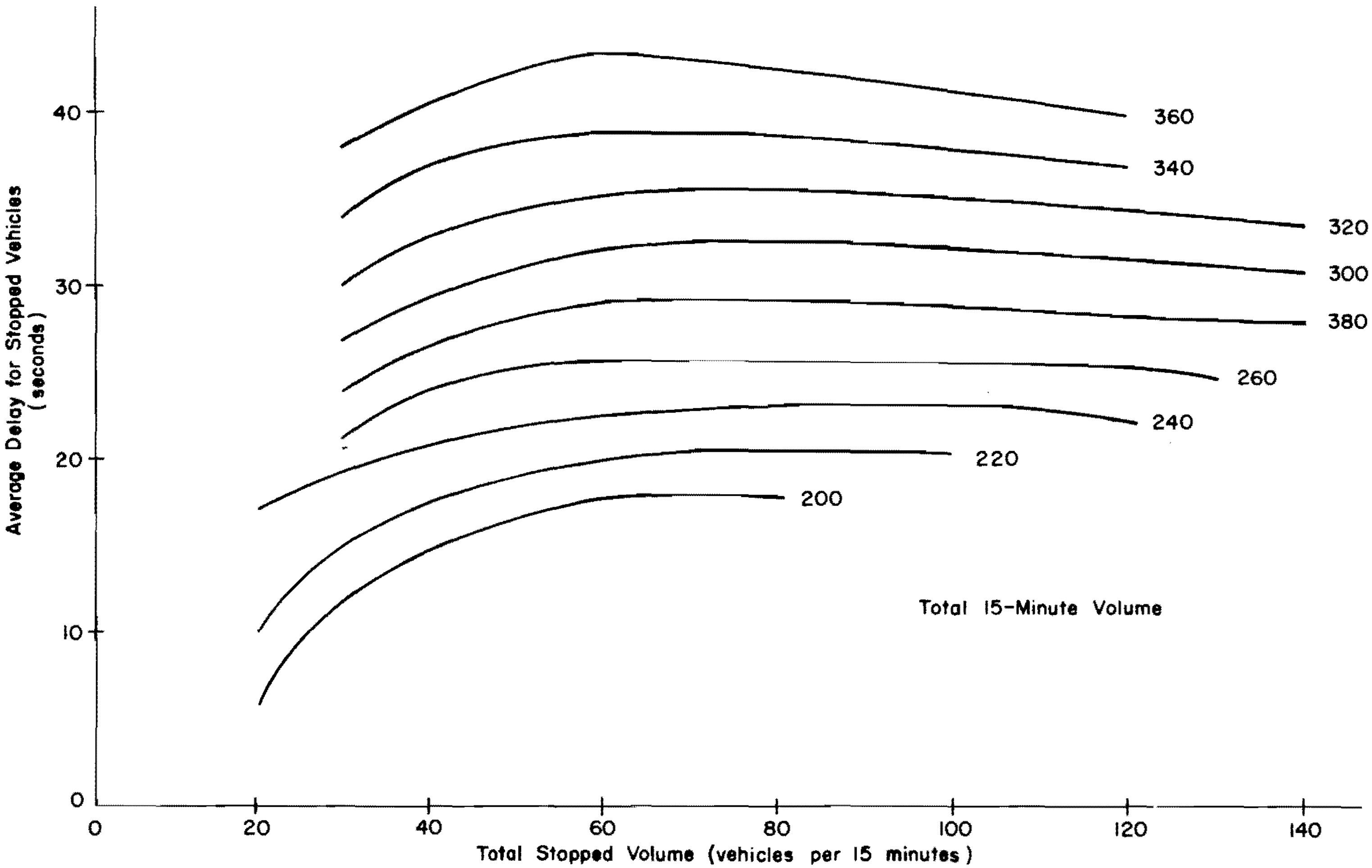

Fig 5.12. Volume-average delay relationships for two-way stop controlled intersections.

way stop controlled intersection as a function of the total volume on the stop-sign controlled streets and the total intersection volume for 15-minute intervals. The curves were obtained by replotting Fig 5.3. The average delay was relatively constant over a wide range of traffic volumes on the controlled approaches for a given total volume, i.e., a wide range of traffic splits. However, the maximum average delay occurs at a traffic split of approximately 85/15 for a 15-minute total volume of 360 vehicles and close to a 50/50 split for a 15-minute total volume of about 200 vehicles. The worst average delay to the stopped vehicles thus occurs at the higher volumes and very unequal traffic splits, the very situation for which the Manual (Ref 18) suggests that a four-way stop installation not be used.

There is an average delay of approximately 30 seconds for a volume of about 300 vehicles per 15 minutes (Fig 5.12). The peaking factor may be expected to vary from about 0.75 in the peak-volume hours to about 0.90 in the low-volume hours. Thus, a 15-minute volume of 300 may represent a volume of from 900 to 1100 vehicles per hour. The warrant appears to stipulate that, for an average day, the average volume for any eight hours must be at least 500 vehicles per hour and the maximum volume for hour must be in excess of approximately 1000 vehicles per hour. This would seem to be a difficult warrant to meet. The added requirement of a minimum stopped streeet vehicular and pedestrian volume of 200 makes the warrant even more difficult to meet. Average delay at even relatively low stopped volumes is only slightly lower than the average delay at higher stopped volumes for equivalent total volumes (Figs 5.12).

The results of this study show that the total delay is greater at four-way stops than at two-way stops for a given total volume throughout the range of total volumes observed. Thus, a warrant for four-way stops should be meant to limit the average delay experience rather than the total delay experience.

The warrants, as given on page 28 of the Manual (Ref 18), set two main conditions: first, to impose a minimum average volume over an eight-hour period and, second, to impose a minimum deviation from the maximum-hour volume such that an average delay to stopped vehicles of at least 30 seconds is experienced during the maximum hour. This means that at least four, and possibly five or six, of the eight hours will have volumes under 500 vehicles per hour, but the highest hour must have between 900 and 1000 vehicles per hour.

It would be more realistic, perhaps, to set a limit on average delay and work backwards to establish a set of volume warrants. The numbers of hours to use in computing an average volume must be selected first. As stated above, four to six of the eight hours would have volumes under 500 vehicles per hour, which is below the critical volume of 750 vehicles per hour as established previously (Fig 5.4) for two-way stops. In establishing a new warrant, it was decided to use four hours, probably both of the two-hour periods centered around each of the morning and afternoon peak periods.

The following procedure was used in establishing the warrants:

(1) An average delay was selected. Average delays of 20, 30, and 35 seconds per vehicle were used and the corresponding 15-minute volumes of 220, 285, and 320 were read from Fig 5.12. It may be observed that, for the stated volumes, the average delays hold over a wide range of stopped-vehicle volumes. In fact, these average delays are characteristic of through to stopped vehicle ratios of about 80/20 to 60/40. Average delays are lower for ratios outside this range.

(2) A peak-hour factor was selected. A peak-hour factor was necessary to convert the 15-minute volume of step 1 to a maximum-hour volume. Three ranges of peak-hour factors were used: 0.75 to 0.80, 0.80 to 0.85, and 0.85 to 0.90

(3) A peak-period factor was selected. This factor was used to convert the maximum-hour volume of step 2 to the average hourly volume observed during the two-hour peak period. The peak-period factor is similar to the well-known peak-hour factor and is calculated in the following manner:

$$PPF = \frac{\text{Sum of Volumes for Four Peak Hours}}{4 \times \text{Maximum-Hour Volume}}$$

or

$$PPF = \frac{\text{Average Hourly Volume}}{\text{Maximum-Hour Volume}}$$

Thus, the average hourly volume for the four-hour period is the product of the maximum-hour volume and the peak-period factor. Four peak-period factors which were representative of the observed data from this study were used in this analysis: 0.60, 0.70, 0.80, and 0.90.

The application of this procedure resulted in the establishment of the minimum volume warrants for four-way stop signs (Table 5.2). It is the province of the engineer in charge to decide on the average delay and peaking factors to be used in each specific case. However, it is recommended that

(1) the peaking factors be based on field observations (or local experience),

(2) an average delay of 30 seconds per stopped vehicle be used, and

(3) the maximum average intersection volume permitted for two-way stop operation be set within the range of 750 to 800 vehicles per hour.

It is also recommended that when the 85-percentile speed on the major street exceeds 40 miles per hour, the warrants be reduced to 70 percent of the values in Table 5.2.

TABLE 5.2. VOLUME WARRANTS FOR FOUR-WAY STOP-SIGN INSTALLATION

Peak-Period Factor	Minimum Four-Hour Average Intersection Volumes for Average Delays of		
	20 sec	30 sec	35 sec
		Peak-Hour Factor = 0.75-0.80	
.60	400	525	600
.70	475	625	700
.80	550	700	800
.90	625	800	900
		Peak-Hour Factor = 0.80-0.85	
.60	425	550	625
.70	500	650	750
.80	575	750	850
.90	650	850	950
		Peak-Hour Factor = 0.85-0.90	
.60	450	600	675
.70	550	700	800
.80	625	800	900
.90	700	900	1000

Notes:
(1) An average delay of 30 seconds per stopped vehicle is recommended for general use.
(2) Intersection volumes are all-approach totals.
(3) Major-minor flow ratios from 80/20 to 60/40 are included.
(4) Maximum hourly volume for two-way operation is 800 vehicles per hour (four-hour average).
(5) Peak-period factor equals the average hourly volume for four hours divided by the maximum-hour volume.

CHAPTER 6. EVALUATION OF SIGNAL CONTROL

GENERAL

The purpose of this chapter is to present an analysis of the delay studies conducted at the signalized intersections described in Chapter 4. Eight four-leg intersections which were at or near right angle crossings, with light pedestrian traffic, were studied. All were relatively isolated from the effects of other signalized intersections; geometric features and amount of pedestrian traffic were almost identical. The percentage of traffic approaching the intersections on the minor streets was similar, generally varying between 30 and 40 percent. Accumulated delay data also reflected essentially the same characteristics; therefore, only two intersections have been selected for separate and thorough analysis and for presentation in this chapter. These two four-leg intersections are: (1) Woodrow and Koenig and (2) South First and Oltorf.

ANALYSIS PROCEDURE

Vehicular delay at intersections is significant from two standpoints to the traffic engineer. First, he attempts to minimize total stop-time delay, thereby minimizing overall user costs. Second, since this minimum total delay condition may, in some cases, subject a few vehicles to unreasonable amounts of delay, he must also evaluate the effects of the control system on individual vehicles on each intersection approach. For example, the first vehicle in a queue can be considered as representative in the latter case.

A delay study should include a number of relationships besides the total vehicle-seconds of delay. These relationships, even though of varying degrees of importance, give a clear insight and understanding of the problems which exist at street and highway intersections. The average delay per vehicle is thought to be an important traffic statistic, since it is a measure of the length of time a representative vehicle is delayed. The average delay per vehicle stopped takes into consideration only the vehicles actually stopped before proceeding through the intersection.

Delay to the first vehicle in a queue gives, in most cases, an indication of the maximum delay which will be experienced by a particular vehicle. Quantitative measures of this factor may be averaged over a period of time, thereby giving a measure of the delay to which a representative vehicle stopped at the head of a queue might be subjected.

Values such as the number of vehicles stopped, number of stops (a vehicle may be forced to stop more than once before proceeding through the intersection), and the percentage of vehicles stopped give indications of how and where delay is being accumulated.

Finally, although an hour is frequently used as a time base in design and operation of certain traffic facilities, this is too long a period for studying delay at signalized intersections where traffic volumes fluctuate moment by moment. In this study, the delay relationships were calculated and summed for 5, 15, and 60-minute intervals. Figures 6.1 and 6.2 show the average delay per vehicle plotted with the traffic volumes for 5 and 15-minute intervals, respectively, for Woodrow and Koenig. As might be expected, there is considerably more scatter in the delay data summed over 5-minute periods than that accumulated for 15-minute intervals. A study of the delay versus volume relationships for all the intersections observed revealed the same tendency. A 15-minute interval was therefore selected as the shortest practical time period for analysis of delay data, and all examples are based on this period.

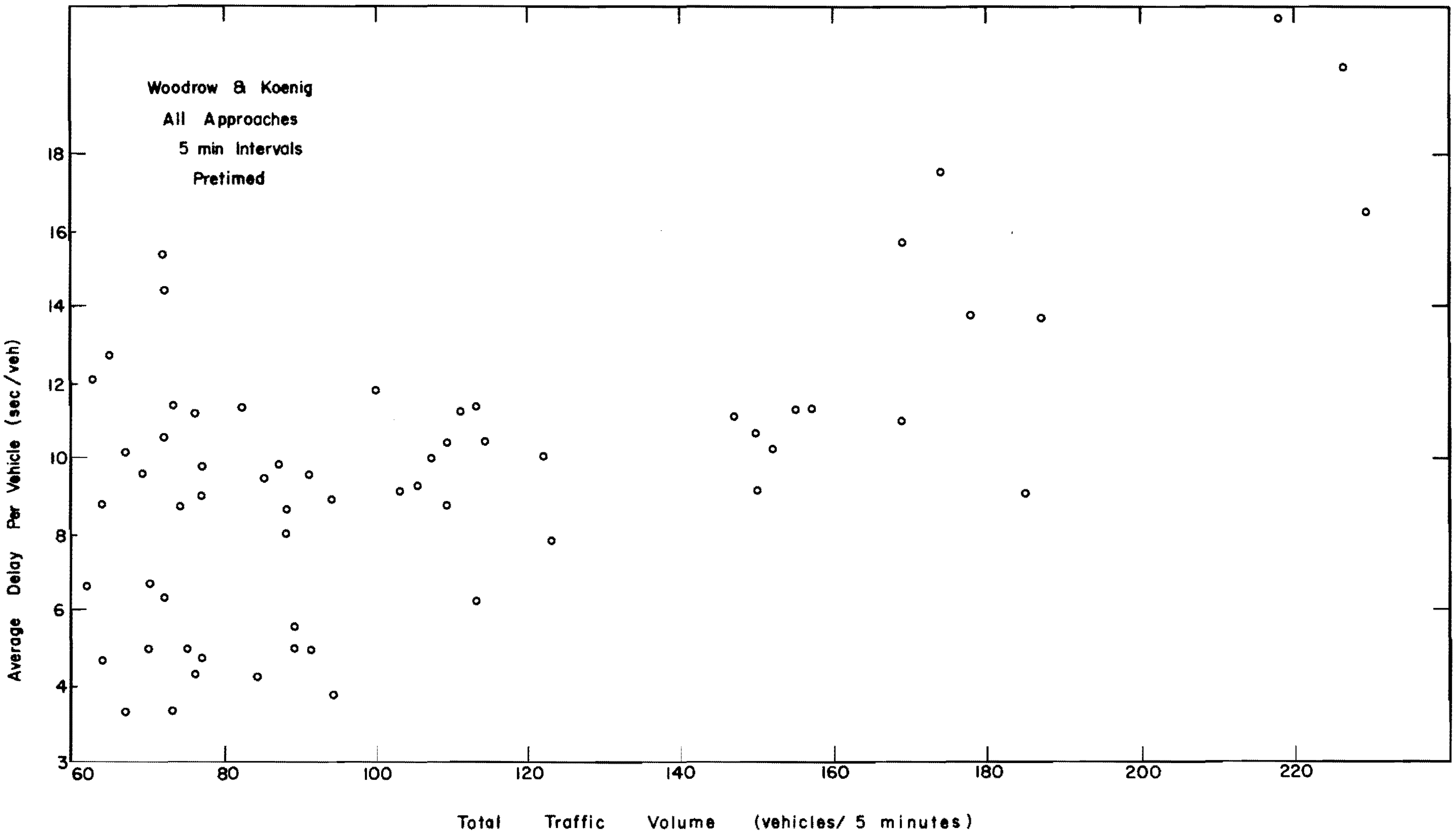

Fig 6.1. Average delay per vehicle, all approaches, pretimed, 5-minute intervals, Woodrow and Koenig.

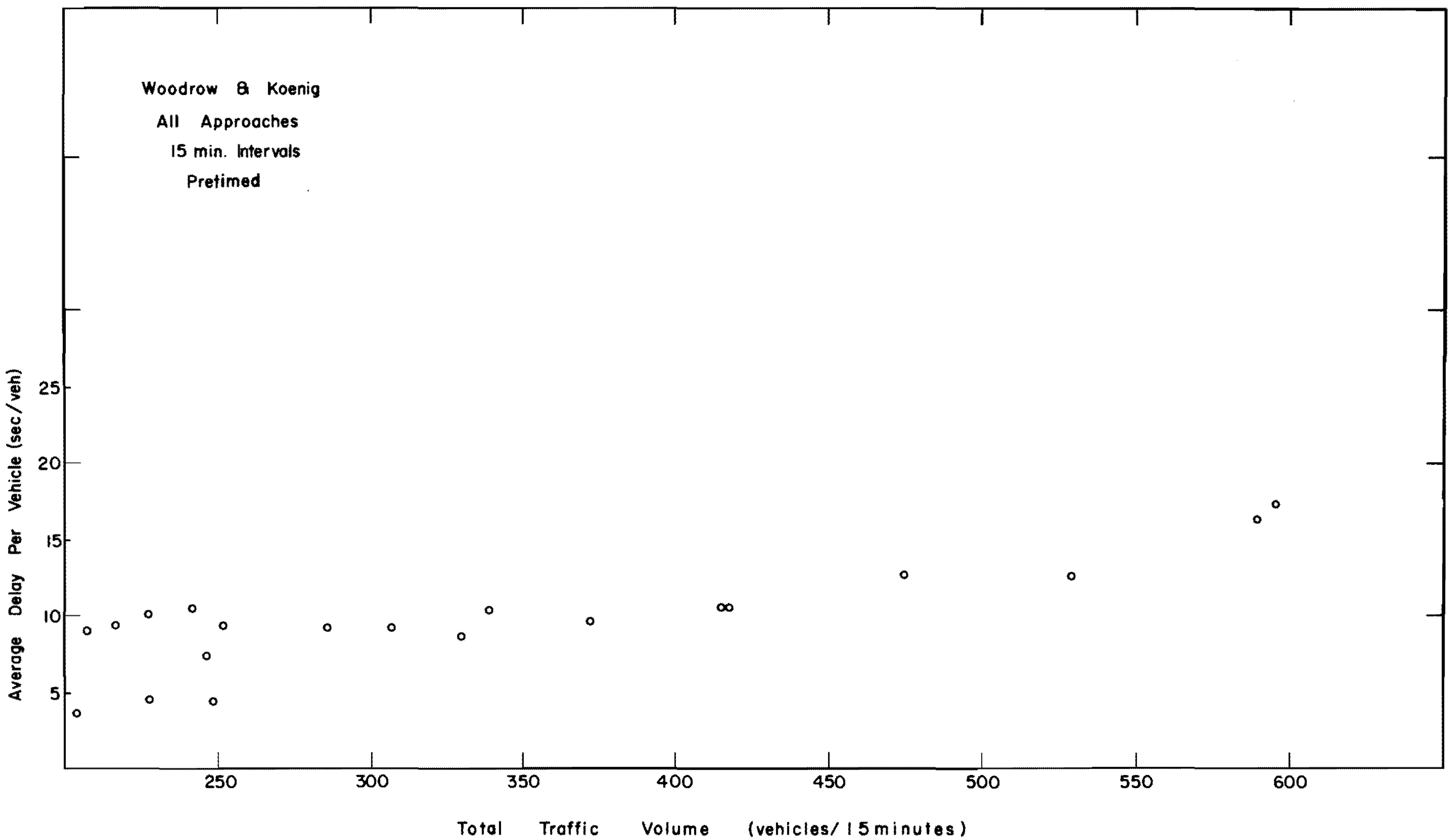

Fig 6.2. Average delay per vehicle, all approaches, pretimed, 15-minute intervals, Woodrow and Koenig.

In the two studies presented, delay relationships will be discussed, first in terms of all approaches combined, and then in relation to major and minor streets. In the discussion of major and minor streets, consideration will also be given to delay of the first vehicle in the queue which forms at the red signal indication. Intersection layouts, signal settings, and traffic volumes for each intersection are shown in Appendix A.

ANALYSIS OF WOODROW AND KOENIG INTERSECTION

In this section, the delay characeristics which include volume versus delay relationships for Woodrow and Koenig in Austin, Texas are discussed.

Total Delay

The relationship between the total traffic volume and the total vehicle-seconds of delay for Woodrow and Koenig is shown in Fig 6.3. Total delay increased as the total volume increased for each of the three types of controllers studied. The actuated equipment generally caused less total delay than the pretimed controller over the range of volumes observed.

At total 15-minute volumes greater than approximately 450, total delay increased at a greater rate than for lower volumes. This tendency was noted for all types of controllers (Fig 6.4).

The actuated controllers produced average cycle lengths ranging from 42 seconds to 84 seconds during the studies at Woodrow and Koenig (Fig 6.5) while the pretimed controller was set for a 60-second cycle. It is interesting to note in Fig 6.3 that, even with the longer cycle lengths provided by the actuated equipment, the total delay increased at approximately the same rate as for pretimed control when the total volume exceeded 450 vehicles per 15 minutes. The flexibility in cycle length available with actuated control, however, resulted in less total delay than for pretimed control throughout the range of traffic volumes.

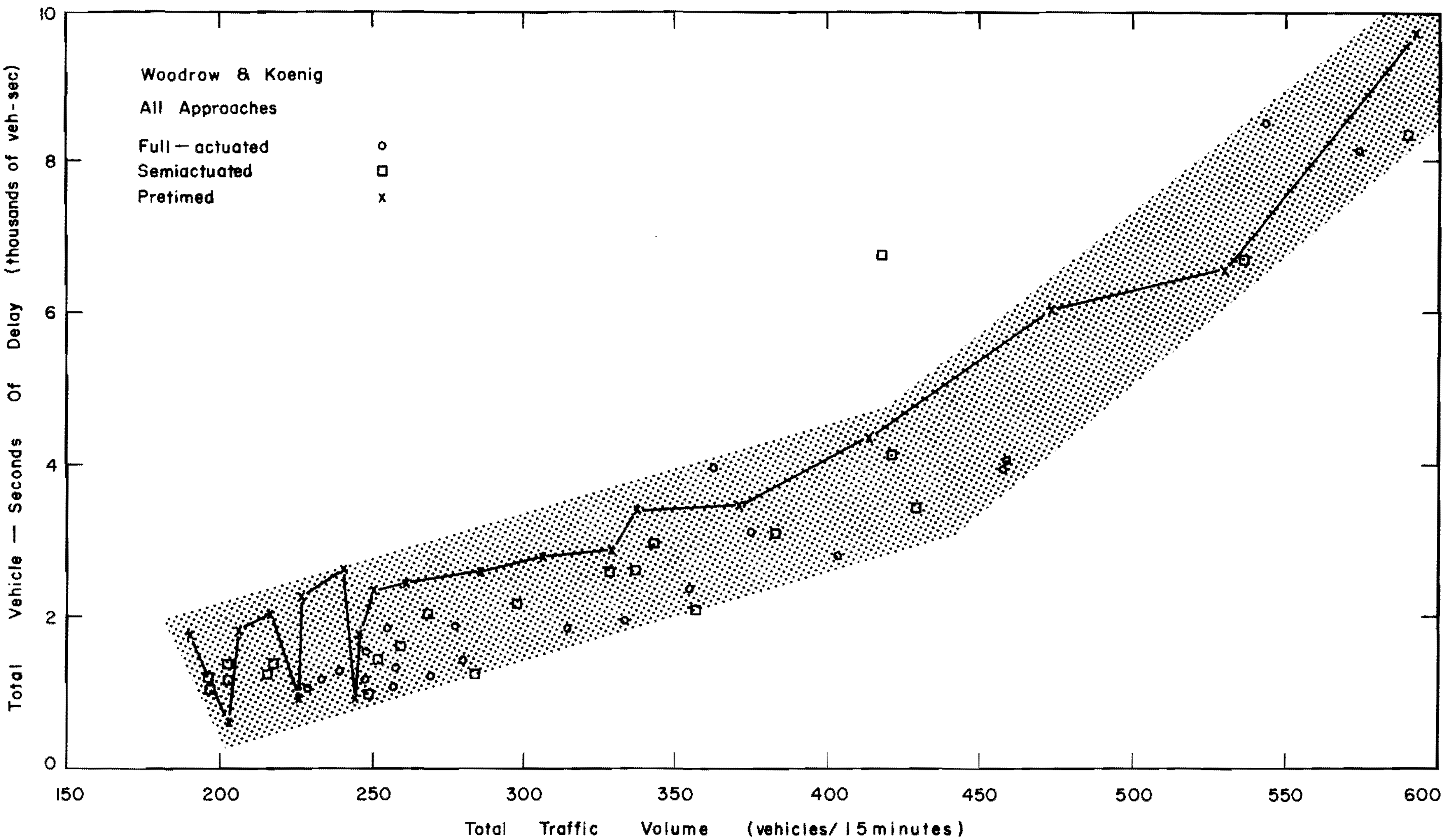

Fig 6.3 Total vehicle-seconds of delay, all approaches, Woodrow and Koenig.

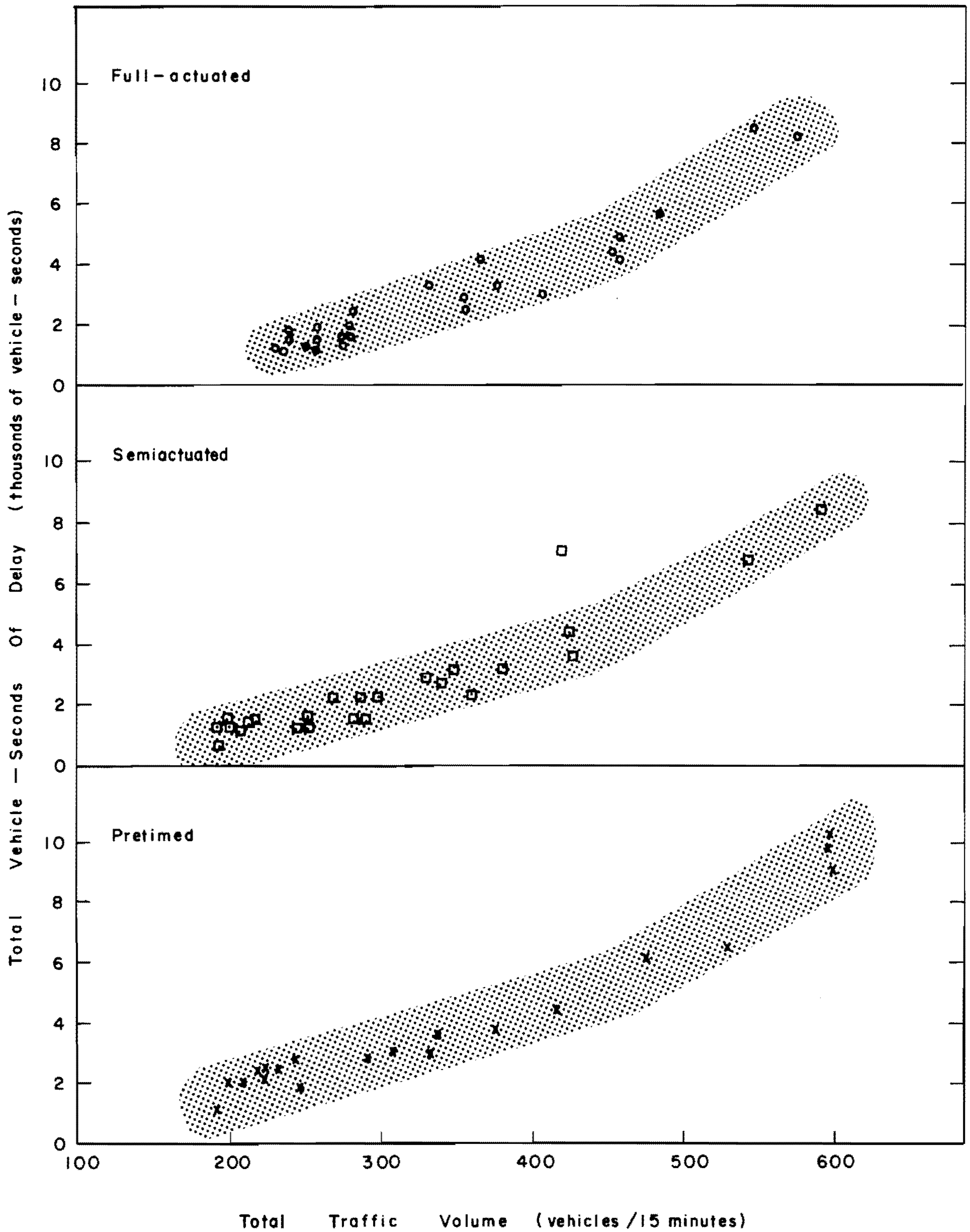

Fig 6.4. Total vehicle-seconds of delay, all approaches, Woodrow and Koenig.

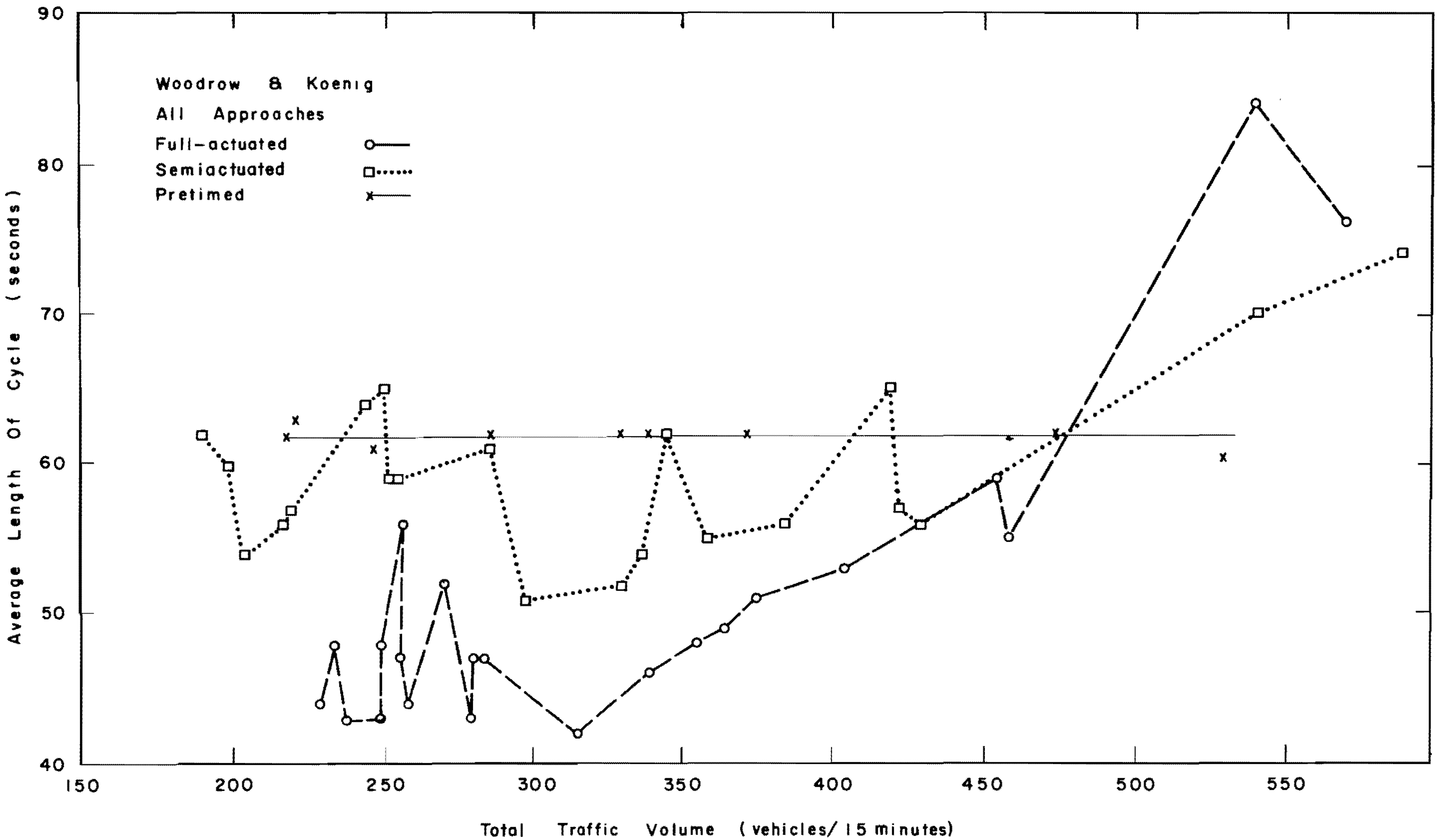

Fig 6.5. Average Length of cycle, all approaches, Woodrow and Koenig.

The relationships between major or minor-street traffic volume and the total vehicle-seconds of delay for the major or minor-street traffic (Figs 6.6 and 6.7) followed the same general trends shown by the total traffic volume versus delay relationships for all approaches (Fig 6.4). Delay increased uniformly as the volume increased until the major street volume reached approximately 300 vehicles per 15 minutes, or the minor-street volume reached about 170 vehicles per 15 minutes. As the volume continued to increase beyond these values, delay increased at a faster rate, except for full-actuated control on the major street (Koenig) and semiactuated control on the minor street (Woodrow).

Settings of the controllers help to explain these exceptions. Throughout the studies at Woodrow and Koenig, total traffic volume was split approximately 35 percent on Woodrow and 65 percent on Koenig during the morning and evening study periods and 25 percent on Woodrow and 75 percent on Koenig in the off-peak afternoon studies. Under full-actuated control, the maximum interval was set at 60 seconds on both major (Koenig) and minor (Woodrow) streets with similar initial and vehicle intervals (6 or 8 seconds) allowed on each street. At the higher volumes, traffic on the major street extended the green time to near the maximum and thereby caused no increase in the ratio of delay to volume on the major street. Traffic on the minor street, however, experienced more delay at the higher minor-street volumes.

The studies of semiactuated control were conducted at Woodrow and Koenig with conventional intervals set on Woodrow but a long vehicle interval (30 seconds) and a relatively short maximum interval (29 seconds) set on Koenig. This maximum was chosen to be the same as the major-street green phase under pretimed control, and the resulting delay to major-street traffic was in fact similar in both cases (Fig 6.6). Green time on the major street was extended to the maximum on virtually every cycle during the morning and evening periods,

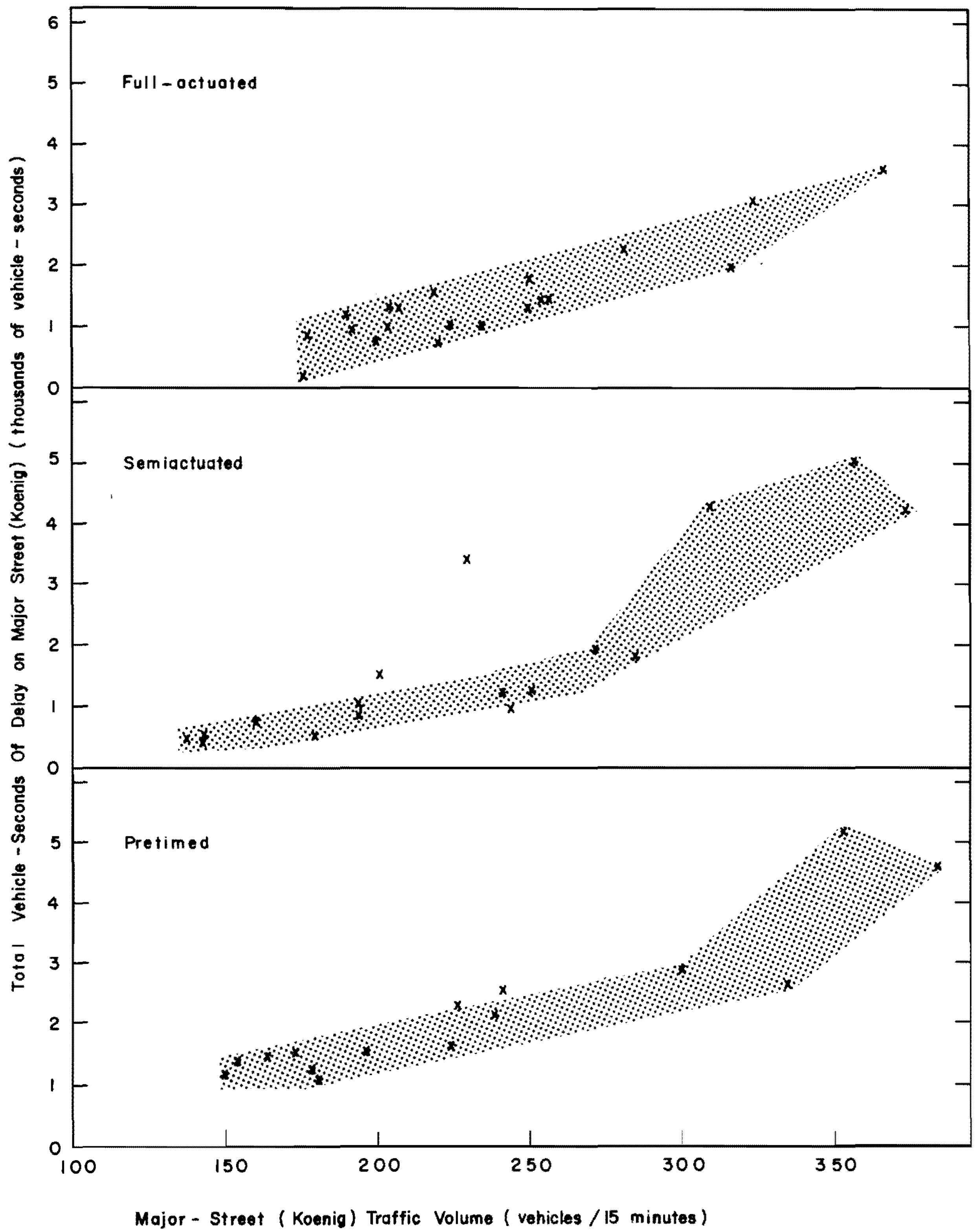

Fig 6.6. Total vehicle-seconds of delay to major-street (Koenig) traffic, Woodrow and Koenig.

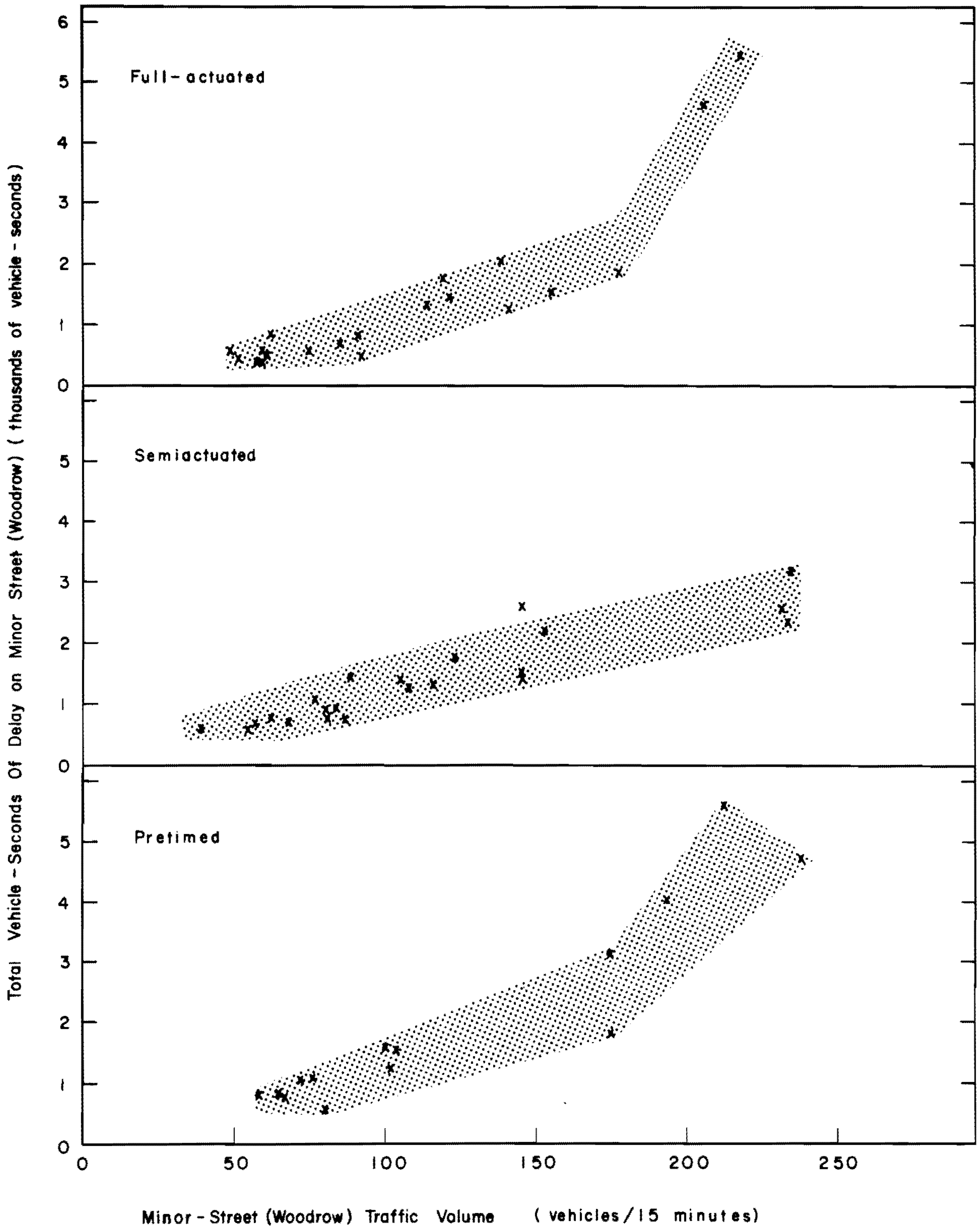

Fig 6.7. Total vehicle-seconds of delay to minor-street (Woodrow) traffic, Woodrow and Koenig.

while the average green time per cycle averaged only about 20 seconds per cycle on the minor street.

For this 4 x 4 intersection carrying traffic which was split approximately 35 percent on the minor street and 65 percent on the major street, total delay increased more beyond about 450 vehicles per 15 minutes, regardless of the type of signal controller used.

Average Delay

Figure 6.8 shows the relationship between the average delay per vehicle and the traffic volume for Woodrow and Koenig. As shown in this figure, the full-actuated controller produced the lowest average delay per vehicle at low volumes, about 5 seconds at 200 vehicles per 15 minutes. The semiactuated yielded about 6 seconds average delay at the same volume, while the pretimed controller produced a 8-second average delay. As the traffic volume increased, the difference between controllers was reduced. At a volume of 600 vehicles per 15 minutes, all the controllers produced about a 15-second average delay per vehicle.

While the average delay values for the remaining intersections are somewhat different, the trends remain the same at all the location except Exposition and Windsor. At this location, the semiactuated controller yielded the lowest average delay up to a total volume of 250 vehicles per 15 minutes.

On the basis of these data, it appears that the full-actuated controller yielded the lowest average delay for volumes up to approximately 450 vehicles per 15 minutes. At higher volumes all three types of control produced approximately the same average delay.

Figure 6.9 shows the major-street traffic volume plotted against the major-street average delay per vehicle. The full-actuated and semiactuated controllers yielded a lower average delay than the pretimed controller at major-street volumes less than 250 vehicles per 15 minutes. At higher volumes, there was very little difference between controllers.

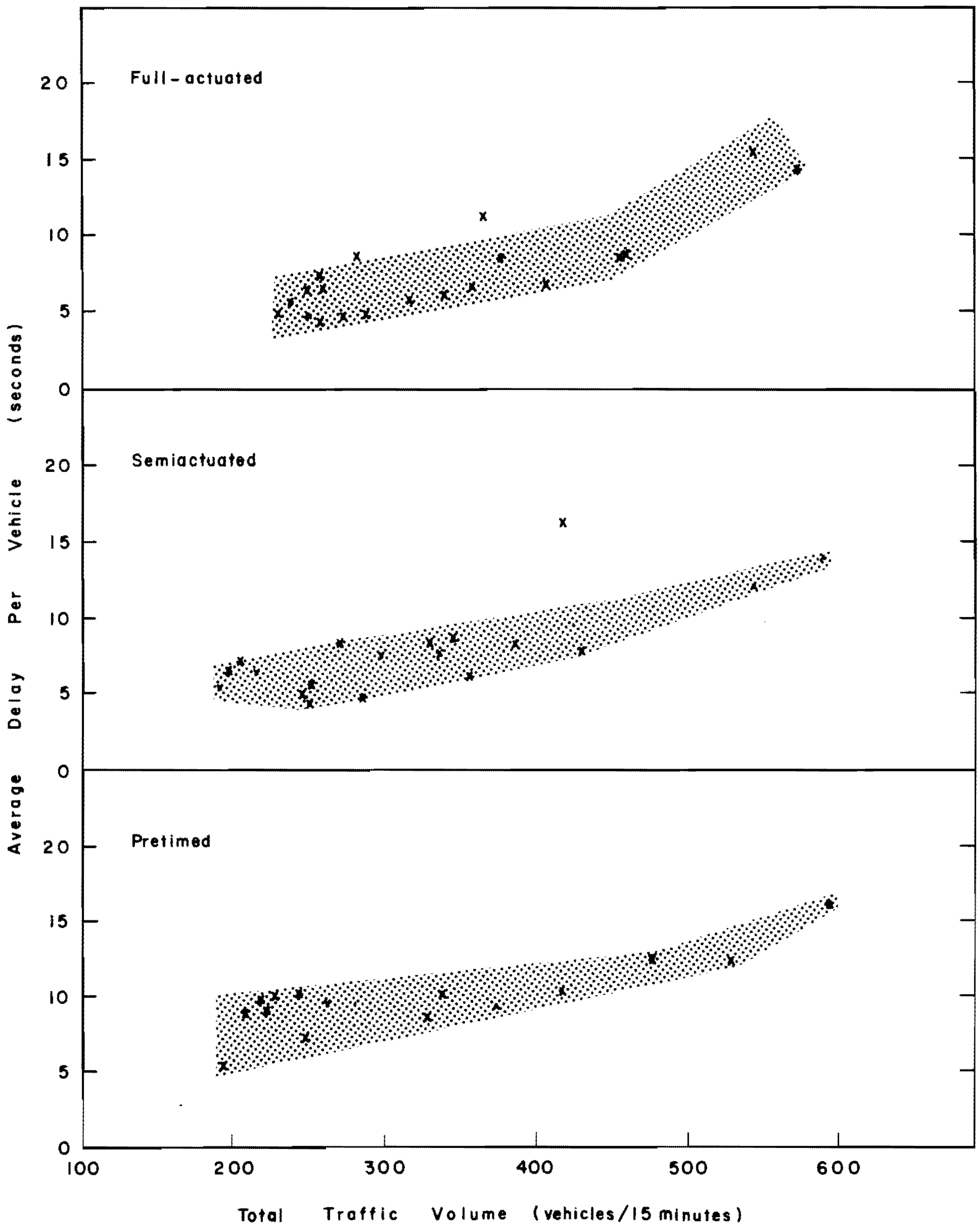

Fig 6.8. Average delay per vehicle, all approaches, Woodrow and Koenig.

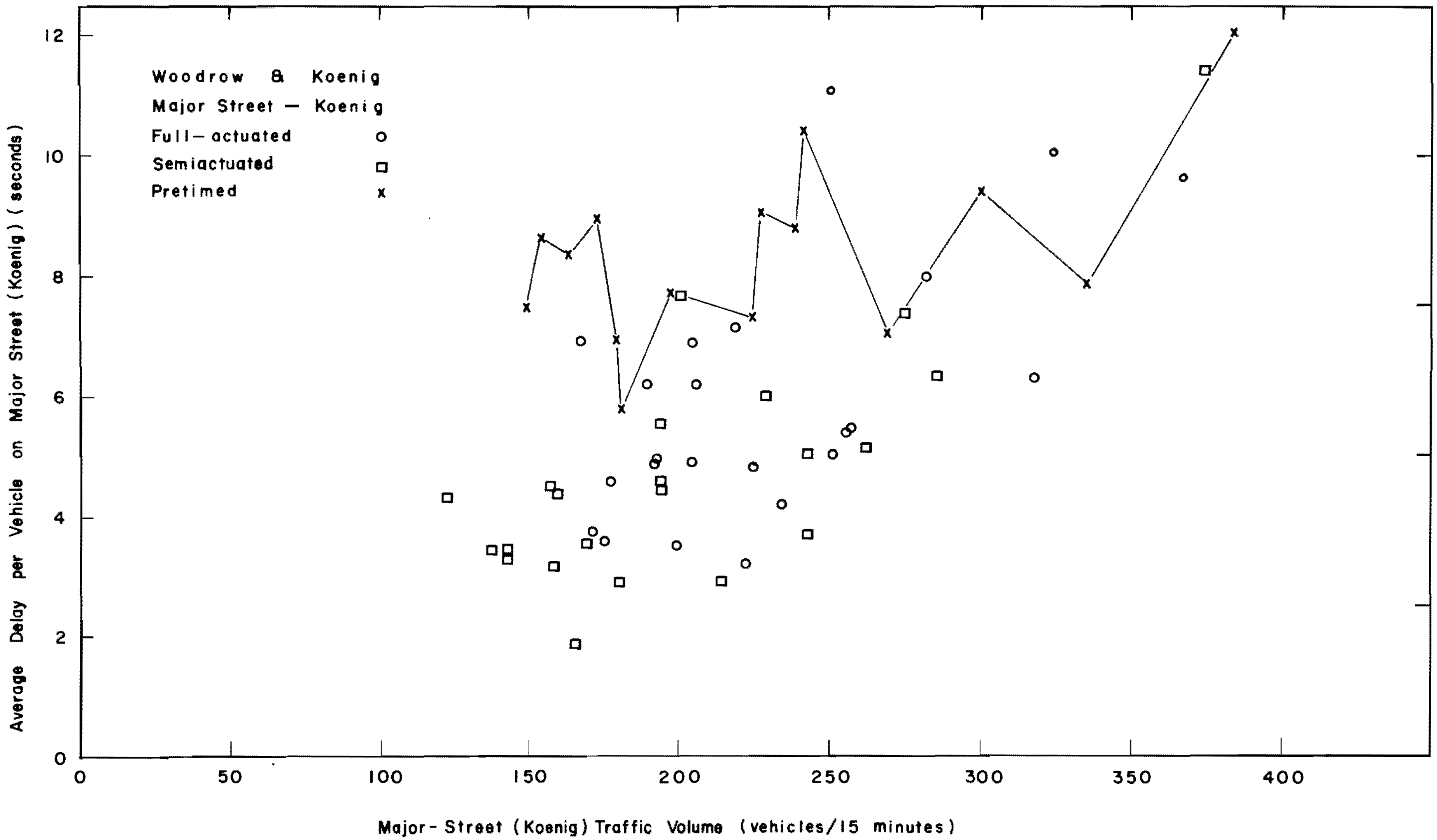

Fig 6.9. Average delay per vehicle on major street (Koenig), Woodrow and Koenig.

There was a wider variation of average delay per vehicle on the minor street than on the major. Figure 6.10 shows that there was not a consistent relationship between average delay and volume on the minor street for the range of volumes observed. It appears, however, that the full-actuated controller generally gave lower average delays to minor-street traffic than the semiactuated or pretimed controller for the settings used in these studies.

A comparison of Figs 6.9 and 6.10 shows that the traffic on the minor street experienced higher average delays (5 to 16 seconds) than the major-street street traffic (2 to 12 seconds). This clearly shows the preference given to the major flow.

Delay per Vehicle Stopped

As stated before, the average delay per vehicle stopped includes only the vehicles forced to stop before proceeding through the intersection.

This relationship for data observed at Woodrow and Koenig is shown in Fig 6.11, which shows that the average delay per vehicle stopped increased slightly as the total volume increased from 200 to 400 vehicles per 15 minutes. The actuated controllers tended to produce lower average values than the pretimed controller. The distinction between semiactuated and pretimed control values was recognizable, with the semiactuated being somewhat lower.

Perhaps the most significant factor shown in this figure is that for the pretimed control the average delay per vehicle increased only slightly as the total volume increased to over 500 vehicles per 15 minutes. The average delay increased from 17 seconds at low volumes to about 22 seconds at higher volumes. While the average delay per vehicle stopped for the actuated equipment was lower at total volumes less than about 450 vehicles per 15 minutes, it was approximately the same for all controllers at total volumes of 600 vehicles per 15 minutes.

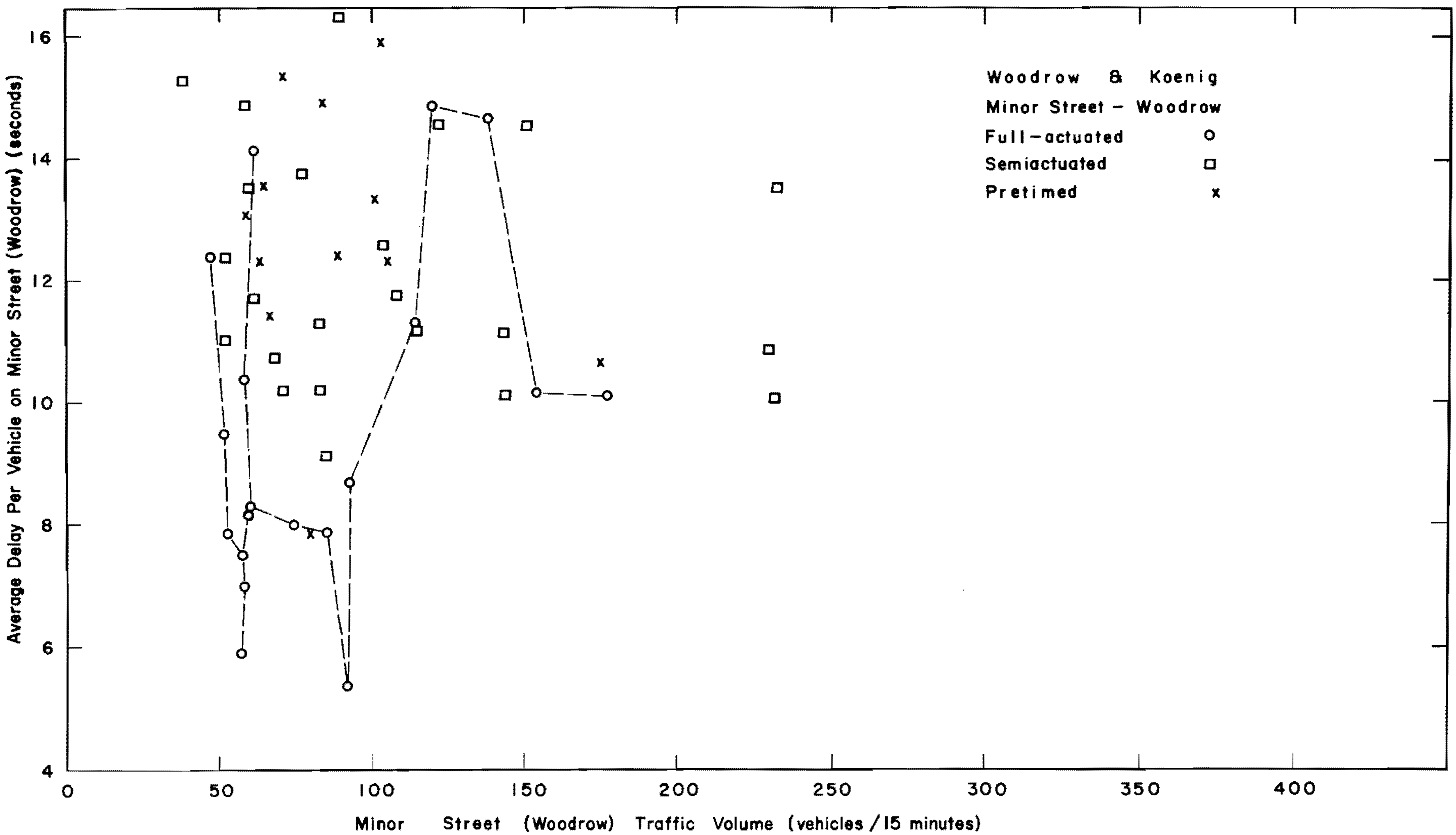

Fig 6.10. Average delay per vehicle on minor street (Woodrow), Woodrow and Koenig.

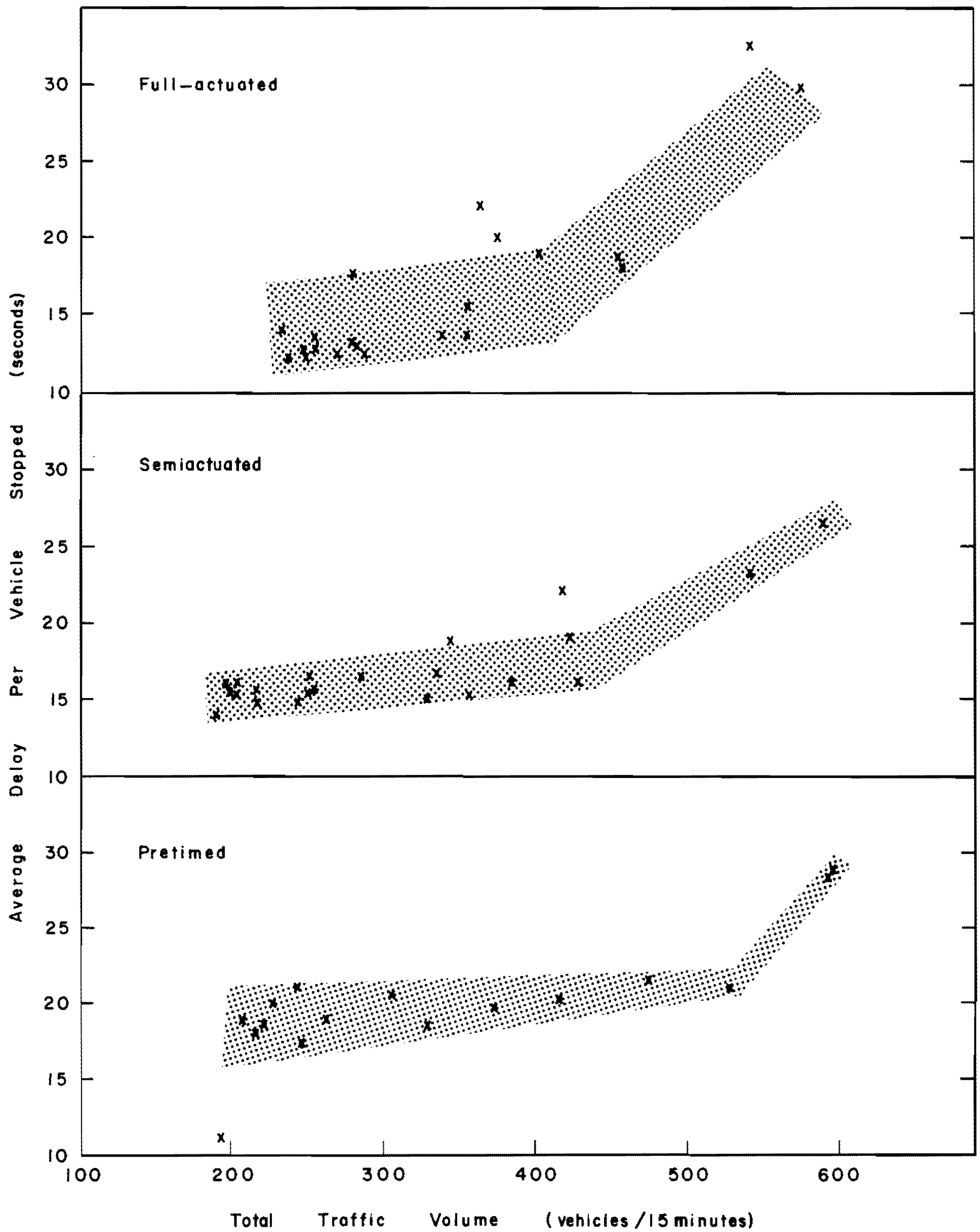

Fig 6.11. Average delay per vehicle stopped, all approaches, Woodrow and Koenig.

Figure 6.12 shows that the average delay per vehicle stopped on the major street varied from 10 to 15 seconds for both the full-actuated and semiactuated controllers and from 15 to 20 seconds for the pretimed controller. The values remained within these limits until the volume reached approximately 300 vehicles per 15 minutes on the major street.

There was a wider range of values for the average delay per vehicle stopped on the minor street than on the major street (Fig 6.13). At volumes less than 100 vehicles per 15 minutes on the minor street, the full-actuated controller yielded the lowest average delay per vehicle stopped; while the pretimed control generally gave the highest delay throughout the range of volumes studied. Perhaps this observed tendency for pretimed control to produce larger average delays when the minor-street total volume exceeded 100 vehicles per 15 minutes (or 200 vehicles per hour on the high-volume minor-street approach) lends credence to the minor-street volume warrants for pretimed control suggested in the Manual on Uniform Traffic Control Devices (Ref 18).

A comparison of Figs 6.12 and 6.13 indicates that vehicles stopped on the minor street experienced longer average delays than those stopped on the major street. Average delay per stopped vehicle ranged from about 8 to 33 seconds at Woodrow and Koenig.

Vehicles Stopped

Figure 6.14 shows the realtionship between the total number of vehicles stopped and total traffic volume per 15-minute interval. The number of vehicles stopped increased with an increase in total volume. The percentage of vehicles stopped, on the other hand, remained relatively constant, as shown in Fig 6.15, and was almost equal for all controllers. This percentage ranged between 30 and 60 percent, regardless of the type of control.

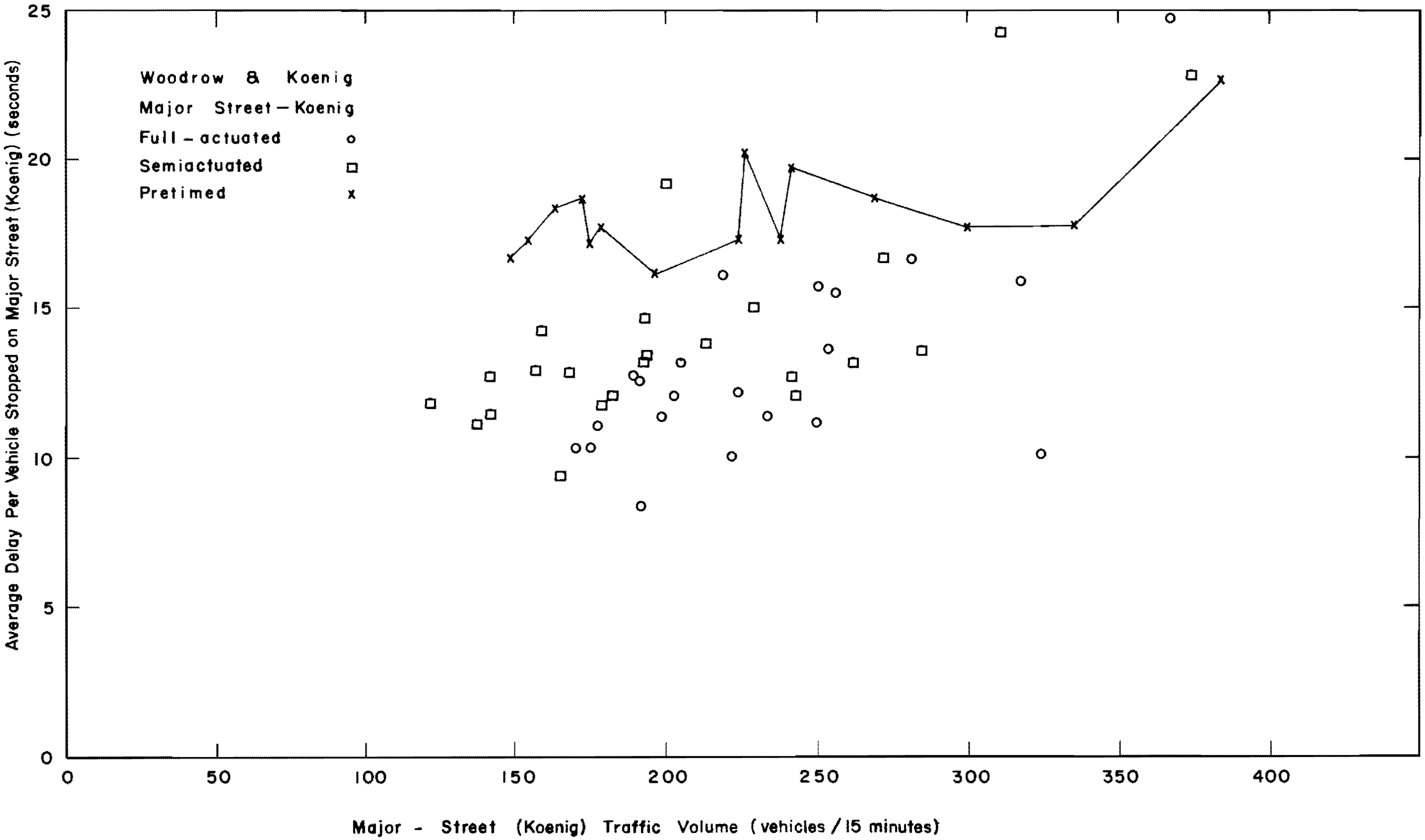

Fig 6.12. Average delay per vehicle stopped on major street (Koenig), Woodrow and Koenig.

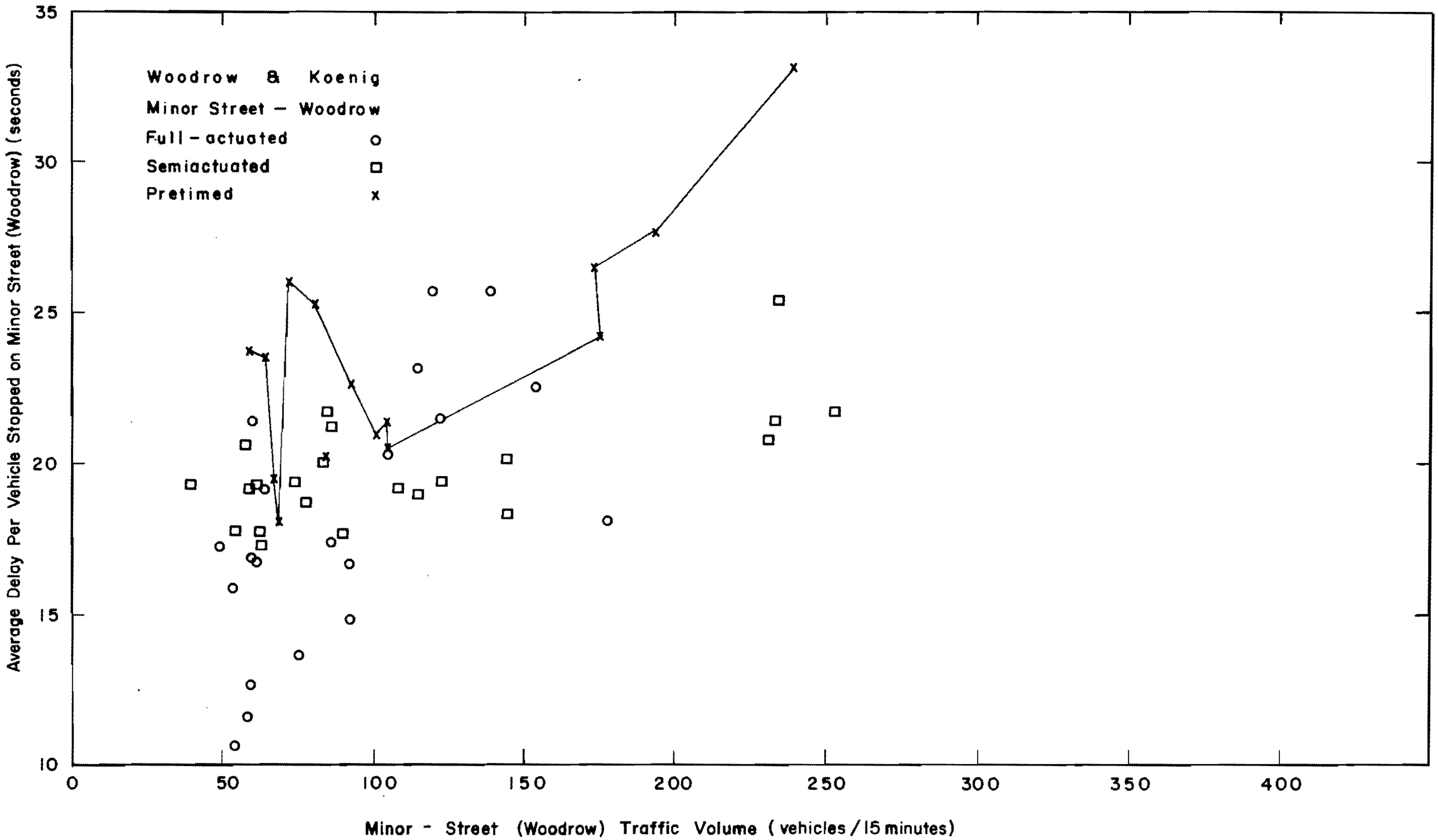

Fig 6.13. Average delay per vehicle stopped on minor street (Woodrow), Woodrow and Koenig.

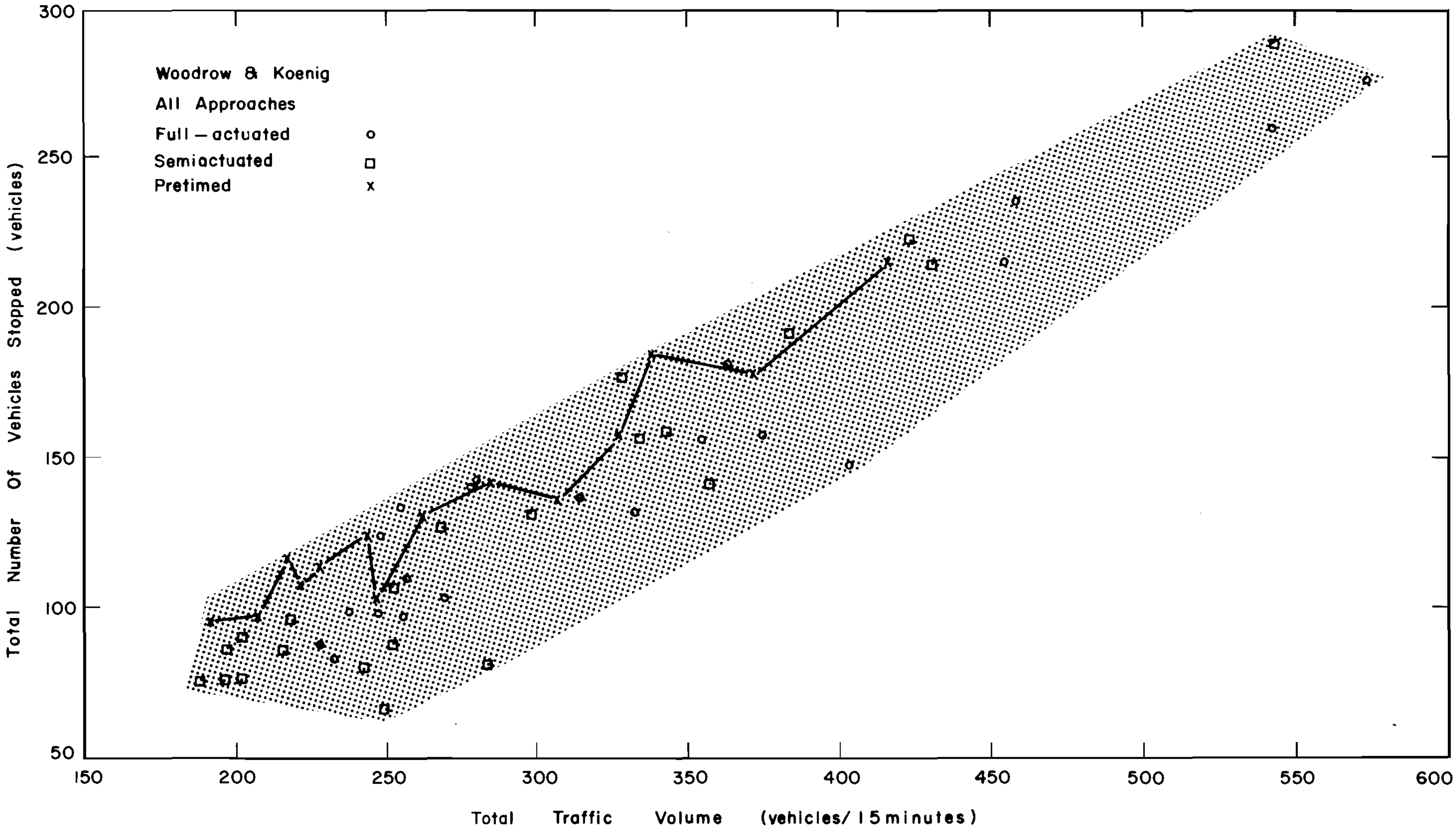

Fig 6.14. Total number of vehicles stopped, all approaches, Woodrow and Koenig.

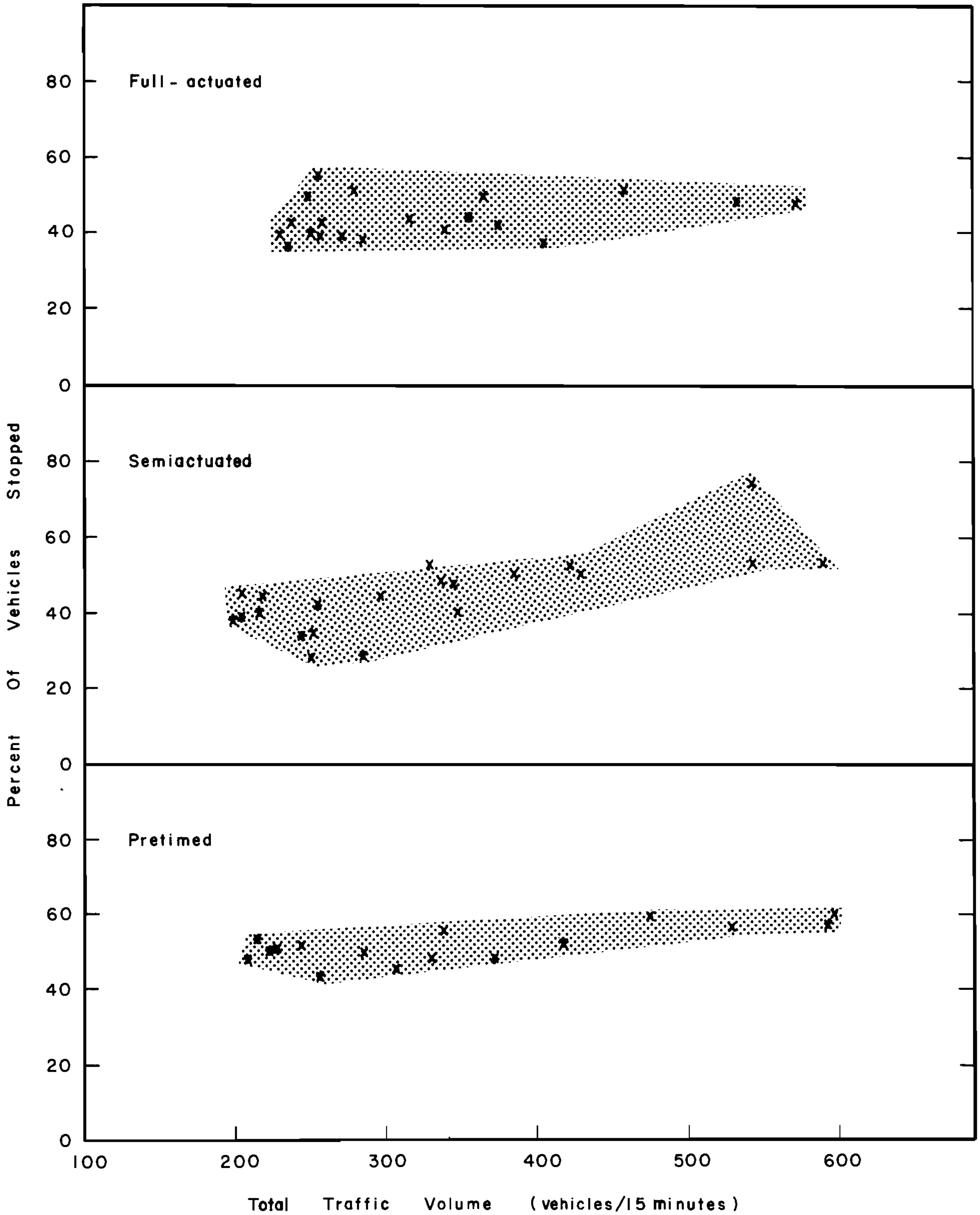

Fig 6.15. Percentage of vehicles stopped, all approaches, Woodrow and Koenig.

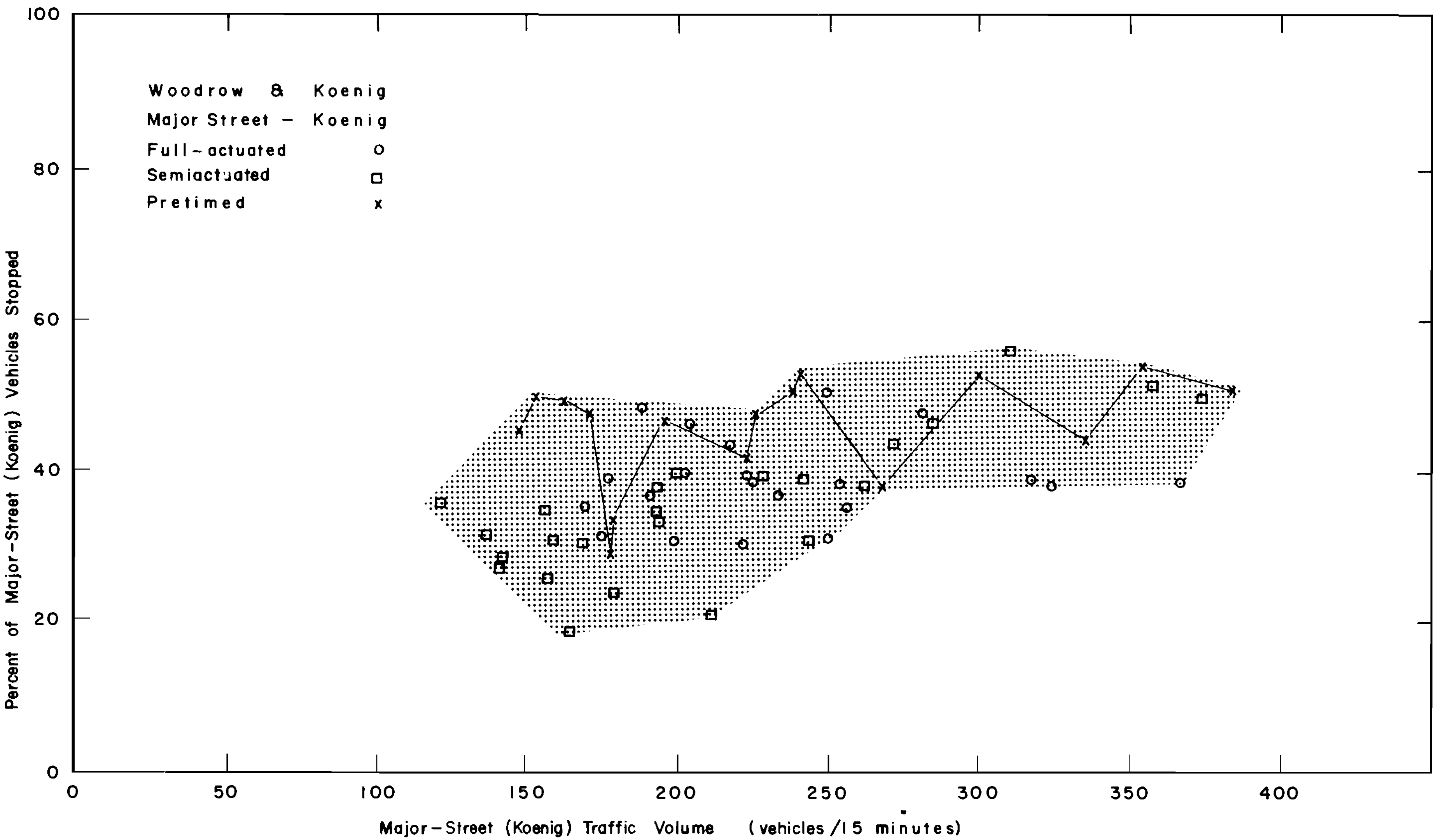

Fig 6.16. Percentage of major-street (Koenig) vehicles stopped, Woodrow and Koenig.

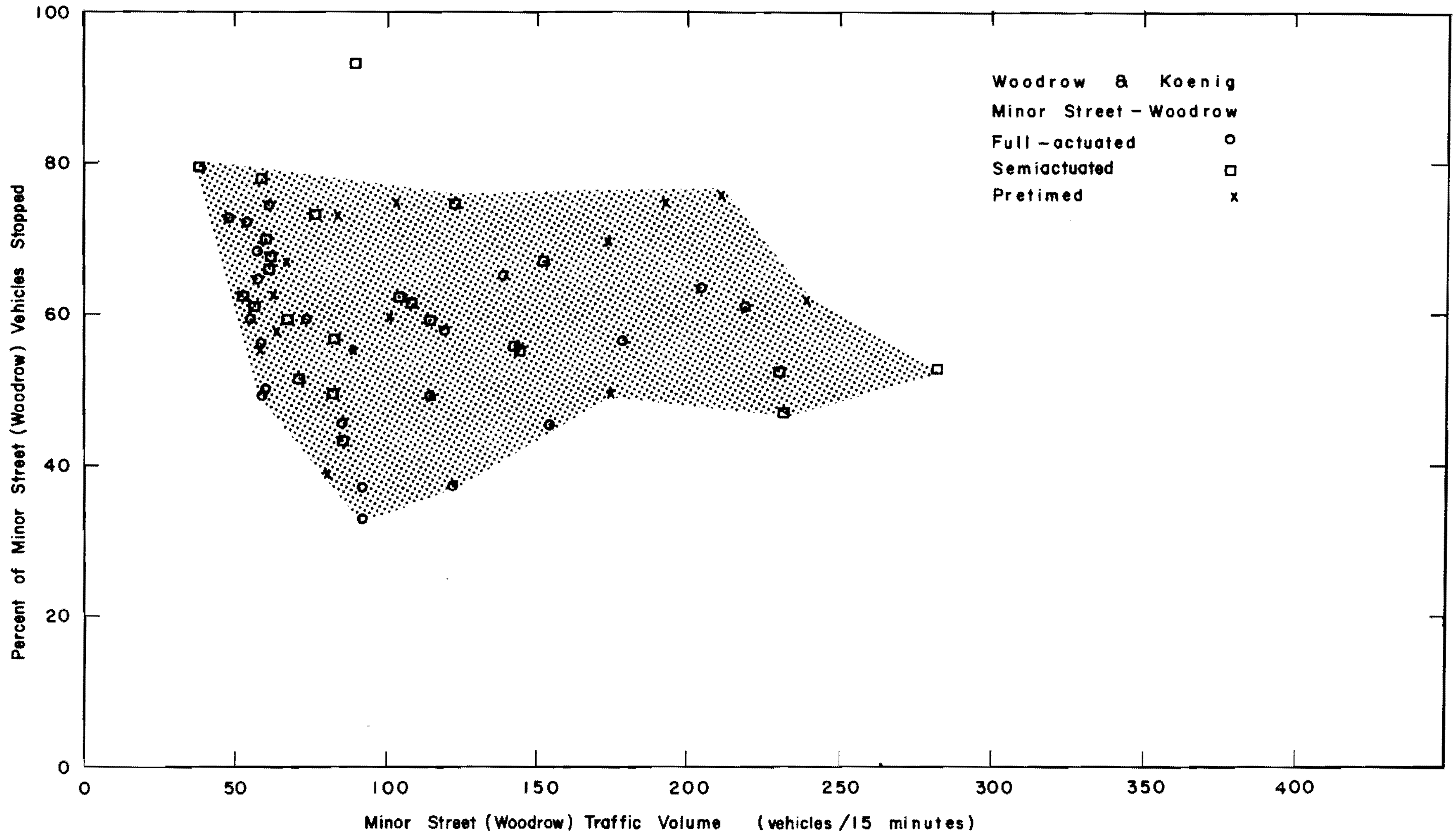

Fig 6.17. Percentage of minor-street (Woodrow) vehicles stopped, Woodrow and Koenig.

The percentage of vehicles stopped for the major street is shown in Fig 6.16. It appears that the full-actuated and semiactuated controllers produced a lower percentage of vehicles stopped than pretimed control. There was a slight tendency for the percentage to increase as the volume increased. The values of the pretimed controller generally varied from about 40 to 55 percent, regardless of the volume. The full-actuated and semiactuated controllers gave values ranging from about 30 percent at a volume of 150 vehicles per 15 minutes to about 55 percent at a major-street volume of 350 vehicles per 15 minutes.

The values for the minor street are scattered in Fig 6.17 and do not appear to depend on the volume or type of controller. A larger percentage of the vehicles (up to 75 percent) on the minor street were forced to stop than on the major street.

Delays to the First Vehicle in a Queue

Figure 6.18 shows the average delay experienced by vehicles at the head of a queue on the higher-volume approach of the major street plotted against the total traffic volume. This was the higher-volume approach on Koenig. It can be seen that the pretimed system produced higher average delay to the first vehicle than the actuated system. This was expected, since the delay to the first vehicles on the major street depends on the length of time that the green signal faces the minor street, and the actuated equipment can return the green to the major street as soon as the minor demand is satisfied.

Figure 6.19 shows the average delay experienced by first vehicles in a queue on a minor-street approach lane (southbound on Woodrow). This figure shows that the full-actuated equipment gave a lower average delay until the total traffic volume reached about 400 vehicles per 15 minutes. At total volumes greater than 400, there was little difference in the control systems.

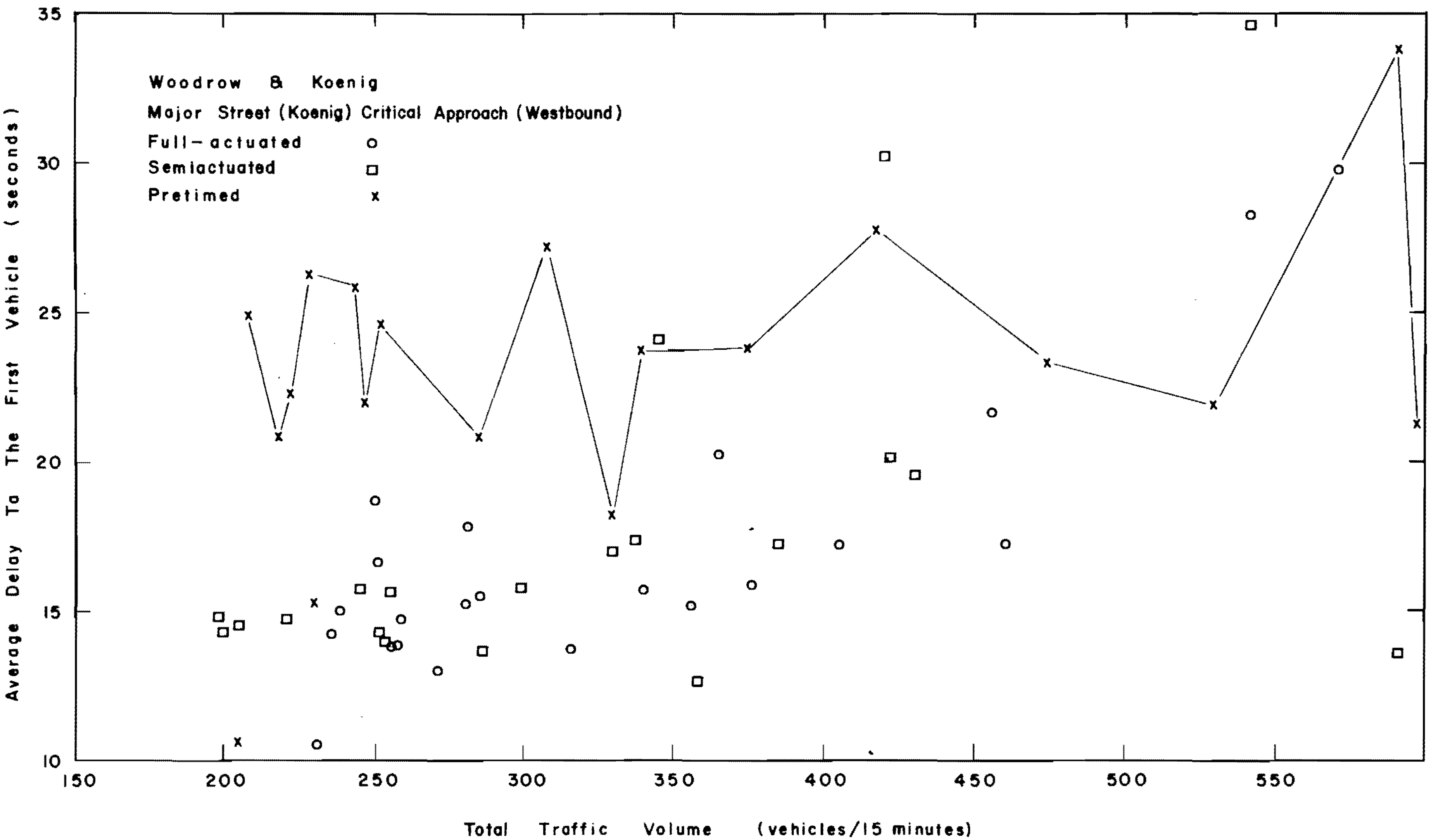

Fig 6.18. Average delay to first vehicle in queue, major-street (Koenig) critical approach, Woodrow and Koenig.

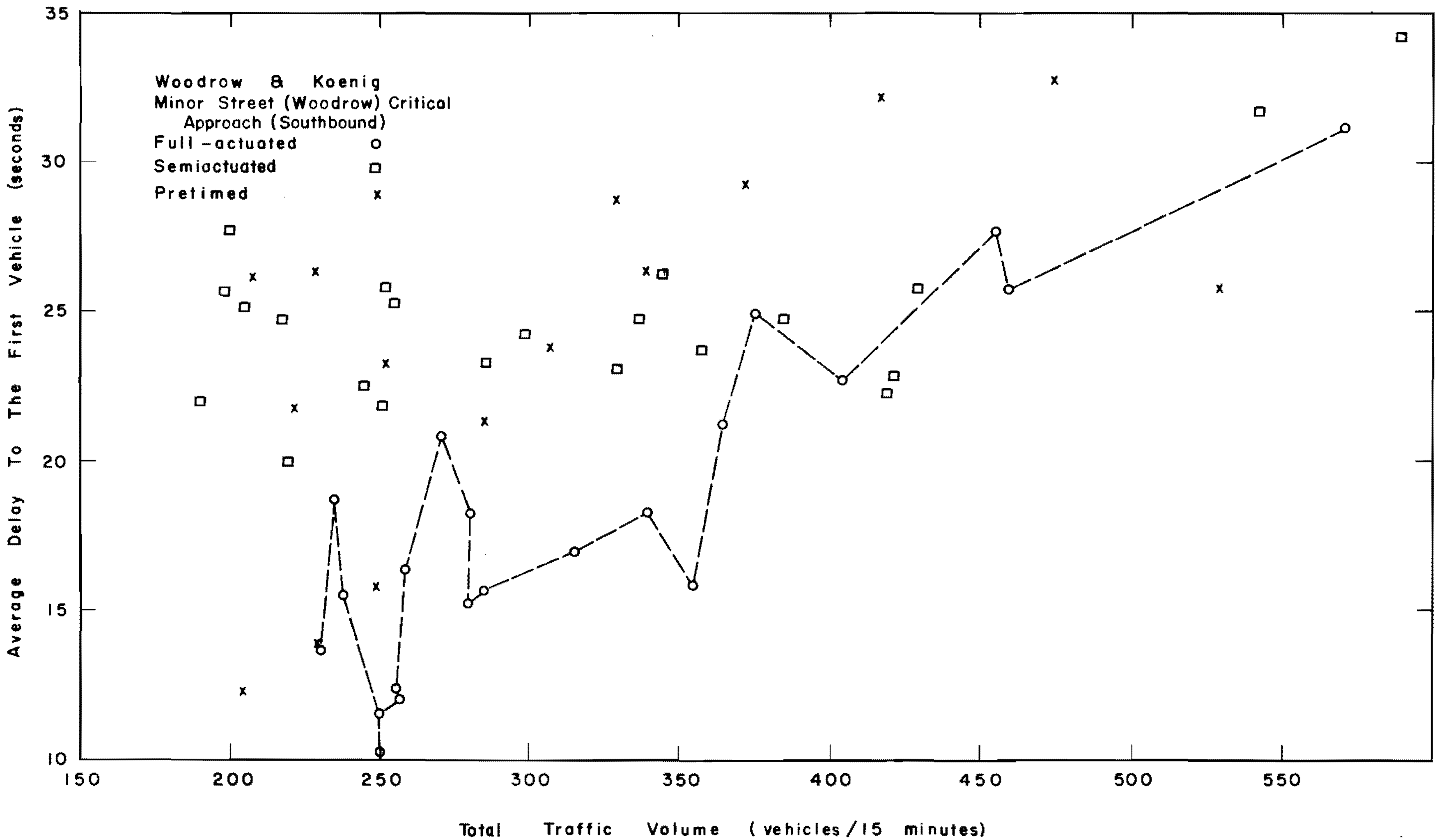

Fig 6.19. Average delay to first vehicle in queue on minor-street (Woodrow) critical approach (southbound), Woodrow and Koenig.

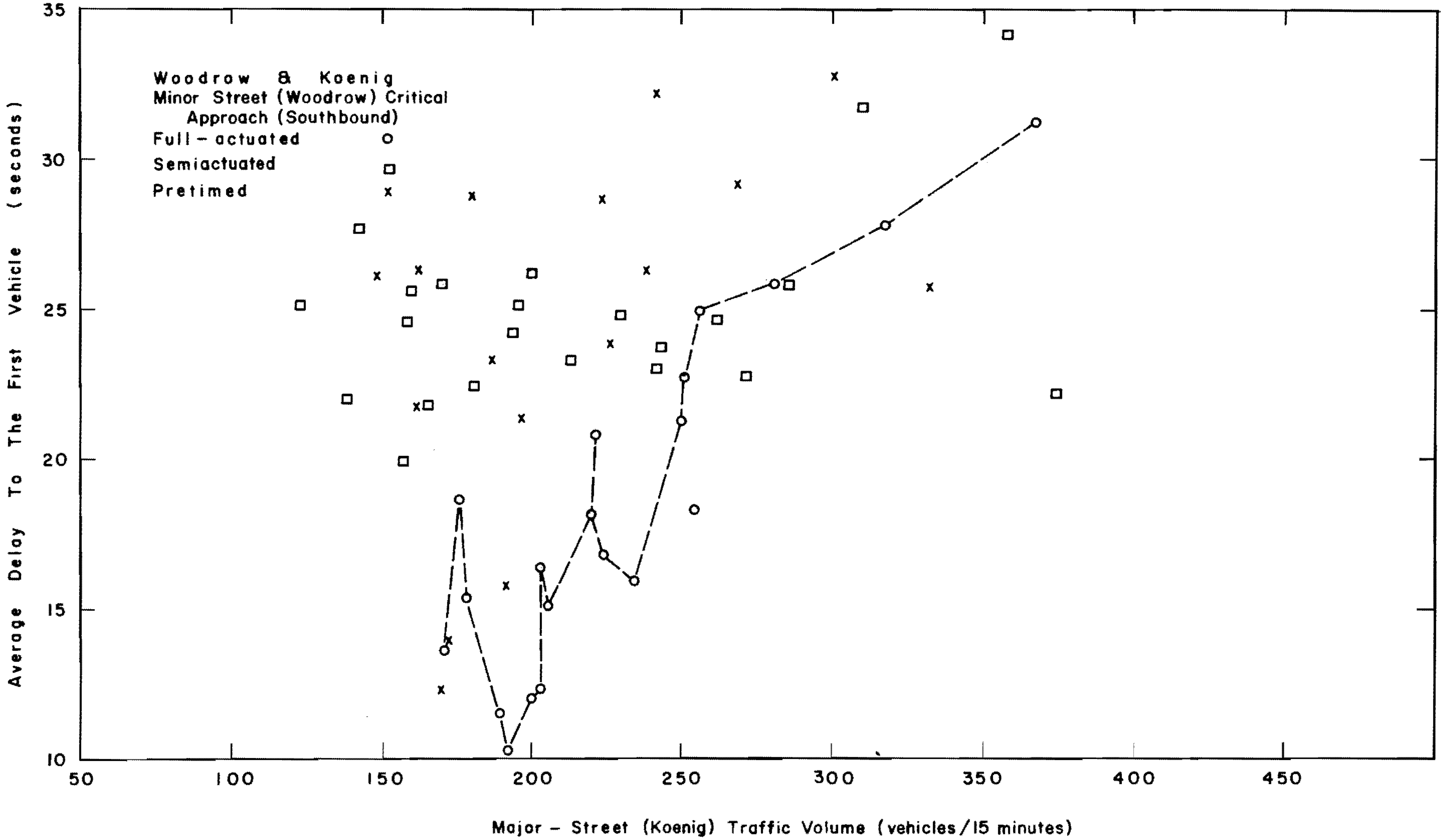

Fig 6.20. Average delay to first vehicle in queue on minor street (Woodrow) critical approach (southbound), Woodrow and Koenig.

The relationship between the average delay to the first vehicle on a minor approach lane and the major-street volume is interesting. Figure 6.20 shows that the values for the full-actuated controller increase steadily with an increase in volume, while the semiactuated and pretimed values remain fairly constant as the volume increases.

The relationships between the minor-street volume and the average delay to the first vehicles on the major-street, higher-volume approach is shown in Fig 6.21. This figure shows that the actuated equipment yielded lower averages which increased with the volume until the volume reached about 200 vehicles per 15 minutes. The pretimed values remained almost constant for all volumes.

ANALYSIS OF SOUTH FIRST AND OLTORF INTERSECTION

In this section, the delay characteristics which include volume versus delay relationships for South First and Oltorf intersection in Austin are discussed. This intersection was generally similar in geometric characteristics and traffic to Woodrow and Koenig described previously; therefore, similar delay studies were made. Total traffic volumes at South First and Oltorf were not quite as high as at Woodrow and Koenig; thus, a 50- second cycle was used for pretimed control rather than 60 seconds as at Woodrow and Koenig. Other differences in controller settings are detailed in Appendix A, but these were relatively minor.

Two pertinent differences in traffic should be noted in comparing these two intersections:

(1) Total traffic was split virtually 50/50 on South First and Oltorf during all the morning and evening studies while Woodrow (minor street) carried only 30 to 40 percent of the total traffic at Woodrow and Koenig.

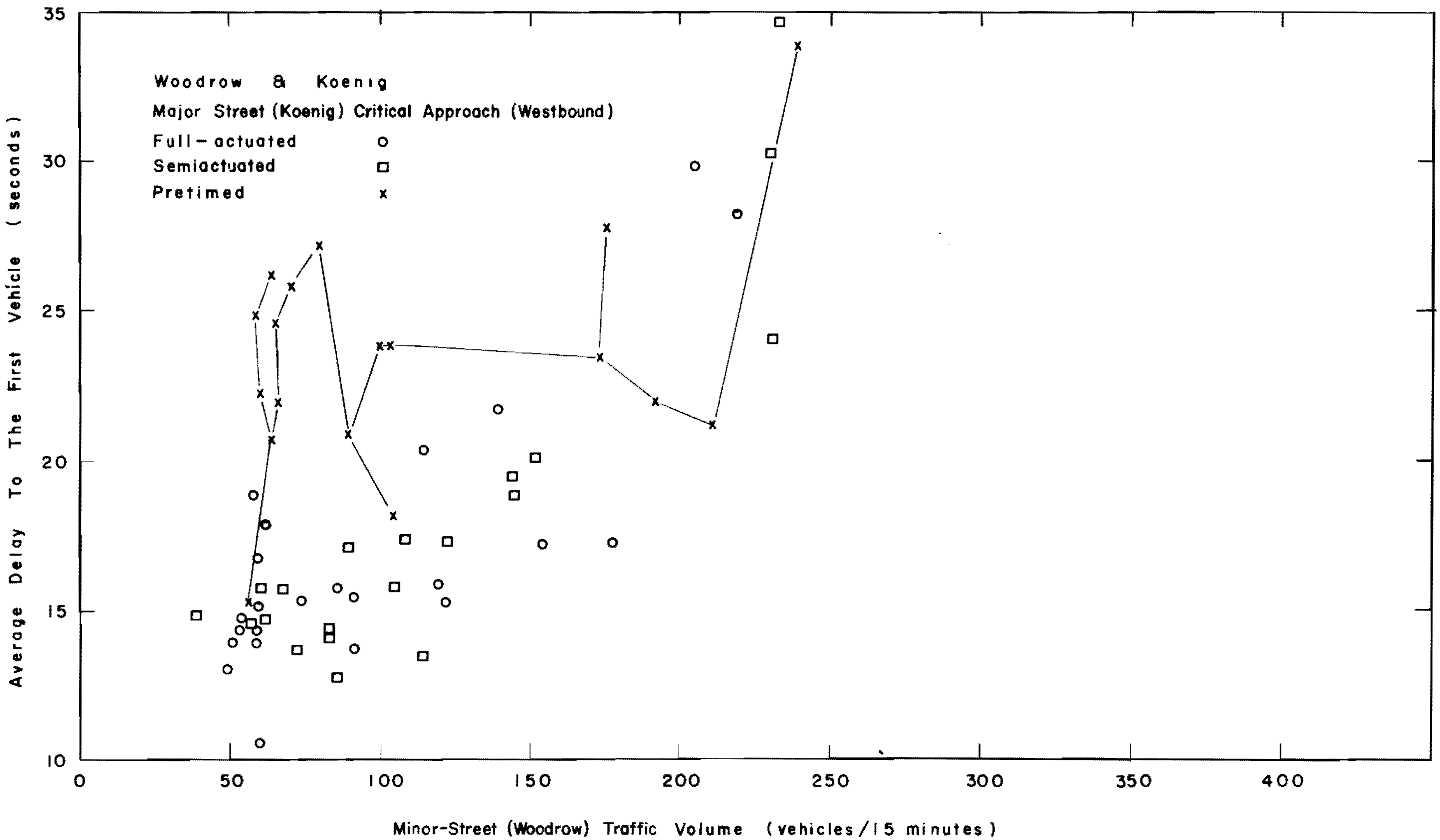

Fig 6.21. Average delay to first vehicle in queue on major street (Koenig) critical approach (westbound), Woodrow and Koenig.

(2) About 30 percent of the traffic on both South First and Oltorf was turning traffic, whereas at Woodrow and Koenig, only about 8 to 15 percent of the traffic on the major street (Koenig) turned and about 30 percent of the minor street (Woodrow) turned.

Total Delay

The relation between the total traffic volume for 15-minute periods and the associated total vehicle-seconds of delay for the three types of control studied at South First and Oltorf is shown in Fig 6.22. This figure shows that there was a tendency for delay to increase as the total volume increased regardless of the type of signal control. The volume versus delay relationship is strikingly similar to that shown in Fig 6.3 for Woodrow and Koenig. Even though the total volume observed at South First and Oltorf exceeded 450 vehicles per 15 minutes for only a few periods, the same pattern of increasing delay beyond this volume as seen for Woodrow and Koenig is evident here also. Full-actuated control caused the least total delay throughout the range of volumes observed.

Figure 6.23 shows the relationship between total vehicle-seconds of delay on the major street (South First) and traffic volume on the major street. Even though the traffic was split approximately equally, South First carried more traffic than Oltorf during certain periods (see Appendix A) and was therefore designated the major street for this analysis. At major-street volumes of approximately 100 vehicles per 15 minutes, actuated control caused less total delay to major-street traffic than pretimed control. Full-actuated control resulted in less delay than the other types for all major-street volumes observed at this intersection, especially at major-street volumes over 200 vehicles per 15 minutes. This was also true at Woodrow and Koenig (see Fig 6.6).

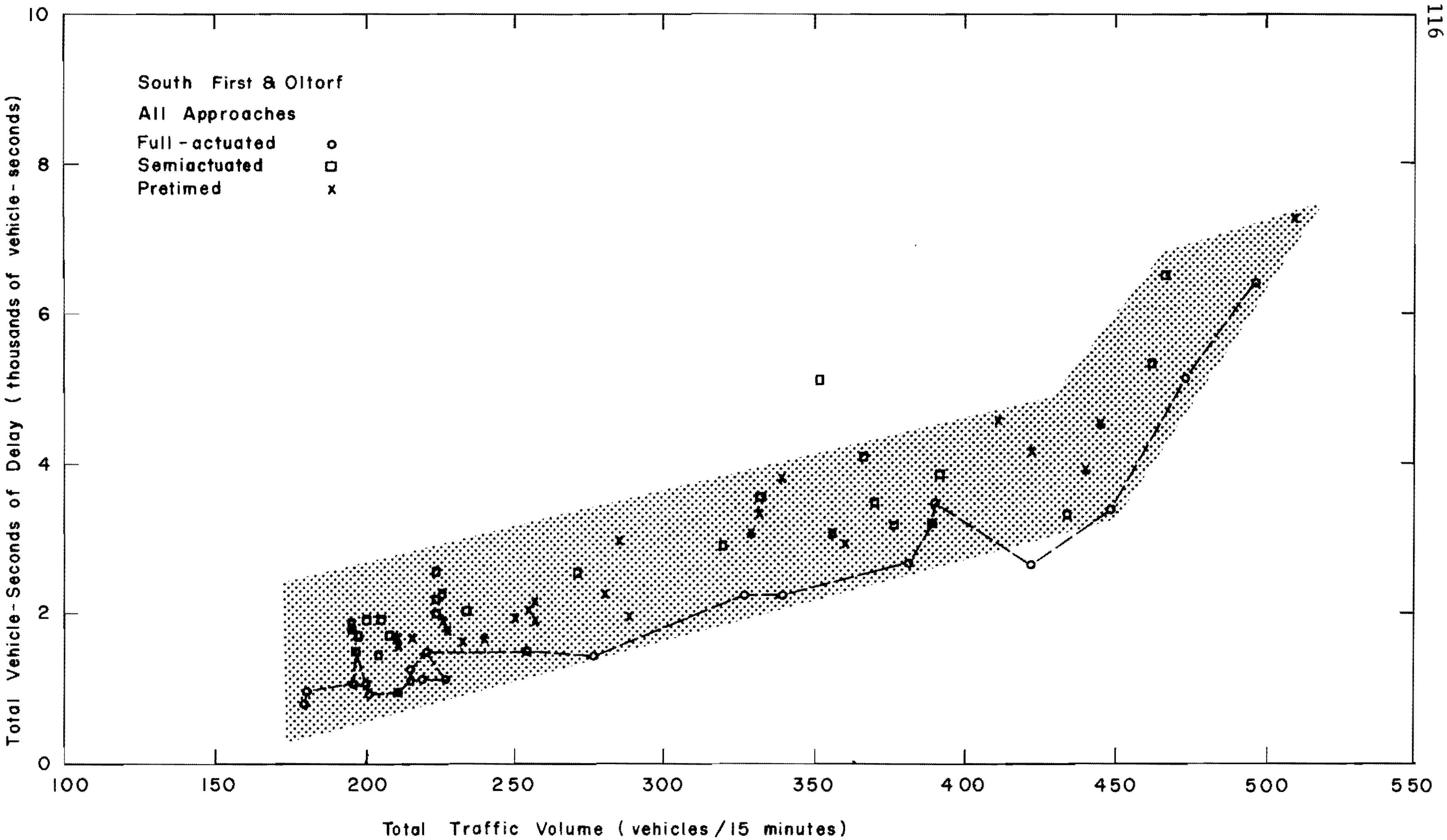

Fig 6.22. Total vehicle-seconds of delay, all approaches, South First and Oltorf.

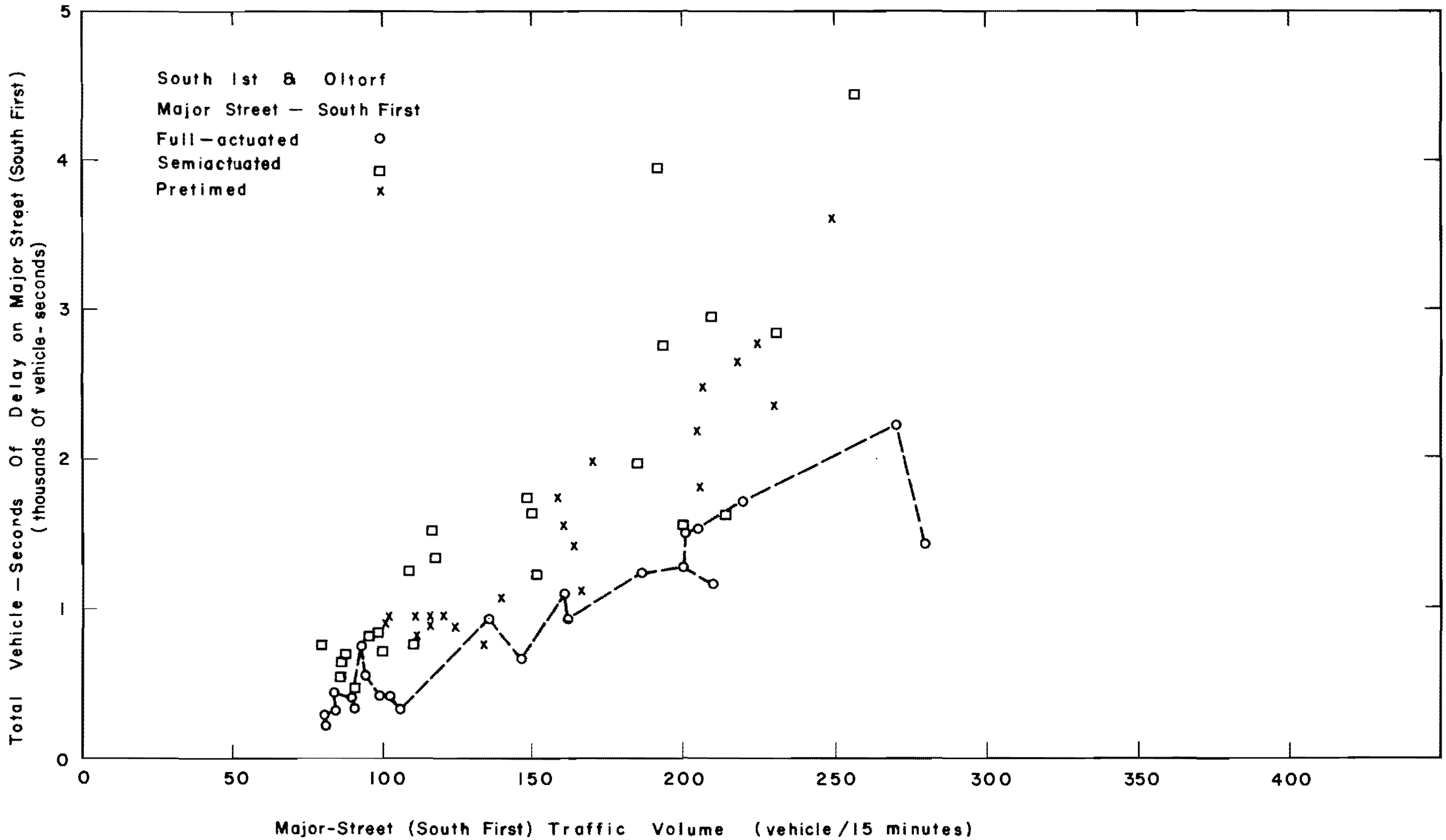

Fig 6.23. Total vehicle-seconds of delay, major street (South First), South First and Oltorf.

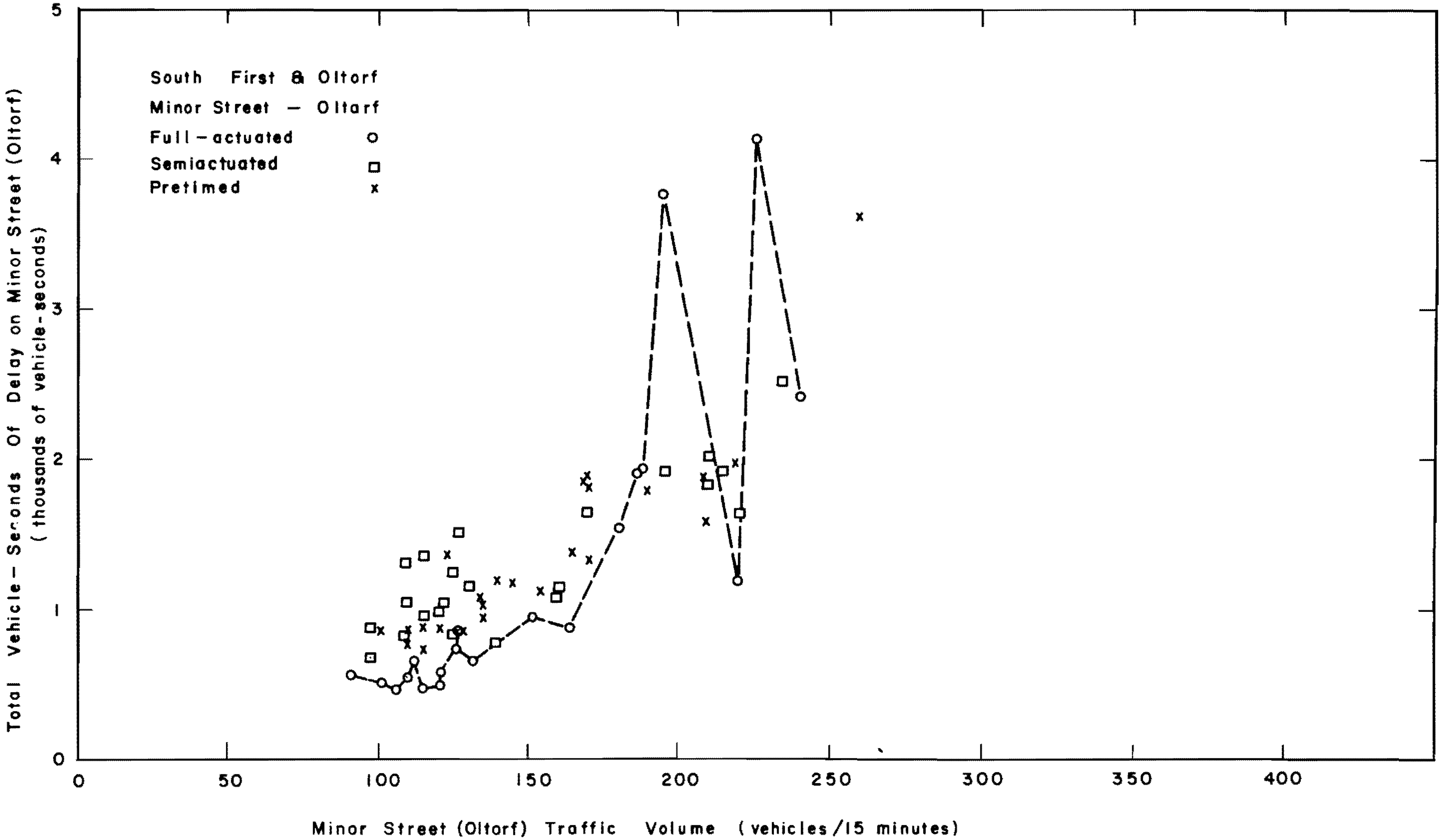

Fig 6.24. Total vehicle-seconds of delay, minor street (Oltorf), South First and Oltorf.

The total vehicle seconds of delay on the minor street (Oltorf) and minor street traffic volume relationship is shown in Fig 6.24. At minor-street volumes less than 150 vehicles per 15 minutes, the full-actuated controller caused less delay than either pretimed or semiactuated control, but at higher volumes, the largest delays to minor-street traffic resulted from full-actuated control. For comparison with Woodrow and Koenig, see Fig 6.7. It is interesting that, even though the traffic split and the full-actuated controller settings were virtually the same on both streets, at South First and Oltorf, traffic on Oltorf experienced larger delays for volumes over about 150 vehicles per 15 minutes than did the traffic on South First for this type of control.

Average Delay

As stated previously in this chapter, average delay per vehicle is an important statistic that gives a single-valued measure of how long a representative vehicle was delayed under specified conditions. Figure 6.25 shows the relation between the average delay per vehicle and total traffic volume per 15 minutes at South First and Oltorf.

Full-actuated control resulted in the least average delay per vehicle (3 to 12 seconds) at South First and Oltorf, while semiactuated and pretimed control caused approximately the same average delay (6 to 14 seconds). The average delay per vehicle at South First and Oltorf (Fig 6.25) was quite similar in magnitude to that at Woodrow and Koenig (Fig 6.8) for pretimed and full-actuated control, but was considerably greater for semiactuated control.

This larger average delay is a reflection of the limited response of the semiactuated controller to the traffic demands at South First and Oltorf where traffic was split approximately 50/50 on the two streets. While the settings used in the study were perhaps not optimum, they were selected so that comparisons could be made among the three types of control. As at Woodrow and Koenig,

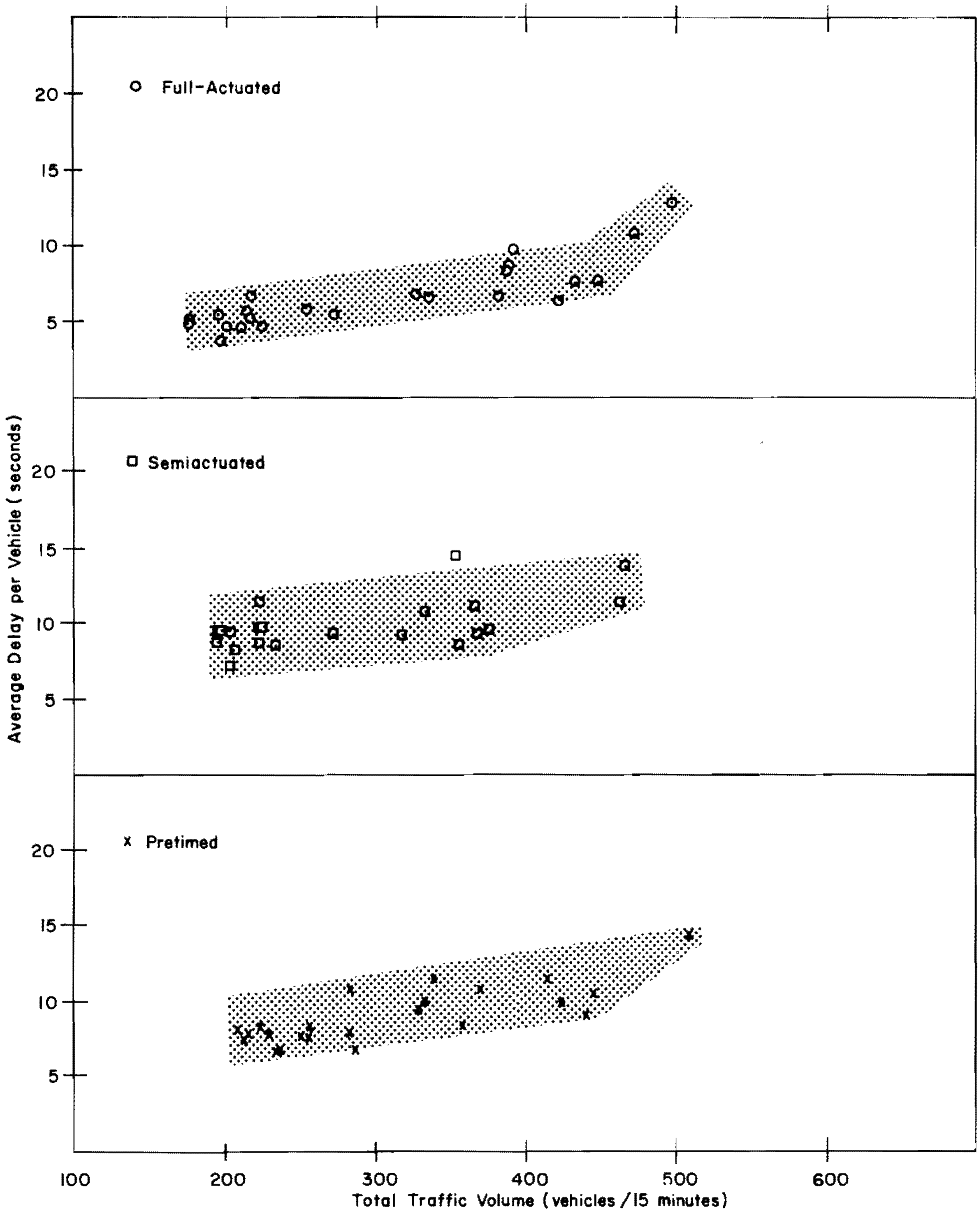

Fig 6.25. Average delay per vehicle, all approaches, South First and Oltorf.

for the semiactuated studies the maximum "go" interval on the major street was set at 50 percent of the cycle length used in the pretimed studies, and the maximum interval on the minor street was set at 60 seconds.

At South First and Oltorf, the semiactuated controller assigned 25 seconds of "go" time to the major street (South First), and traffic actuations on Oltorf accounted for the comparatively large average delay observed at this intersection under semiactuated control. By contrast, at Woodrow and Koenig, where the minor street carried only about 30 percent of the total traffic, the "go" time was extended by traffic actuations to an average value of only 20 seconds, and average delays were about 50 to 80 percent less than those observed at South First and Oltorf for any given traffic volume. These studies of average delay per vehicle substantiate the basic concept that semiactuated control functions most effectively at isolated intersections where traffic on the minor-volume street is consistently less than about 30 to 40 percent of the total volume.

Delay per Vehicle Stopped

Relationships showing average delay per vehicle stopped and traffic volumes reveal the amount of delay that a typical stopped vehicle would normally experience. Figure 6.26 shows this relationship when all approaches at South First and Oltorf are considered. Pretimed and semiactuated control exhibit similar relationships; average delay increased from approximately 16 to 18 seconds per stopped vehicle as total volume increased from 200 to 450 vehicles per 15 minutes. Full-actuated control caused considerably less delay per vehicle stopped than the other types at total volumes less than 450 vehicles per 15 minutes. The pattern here was similar to that at Woodrow and Koenig (see Fig 6.11).

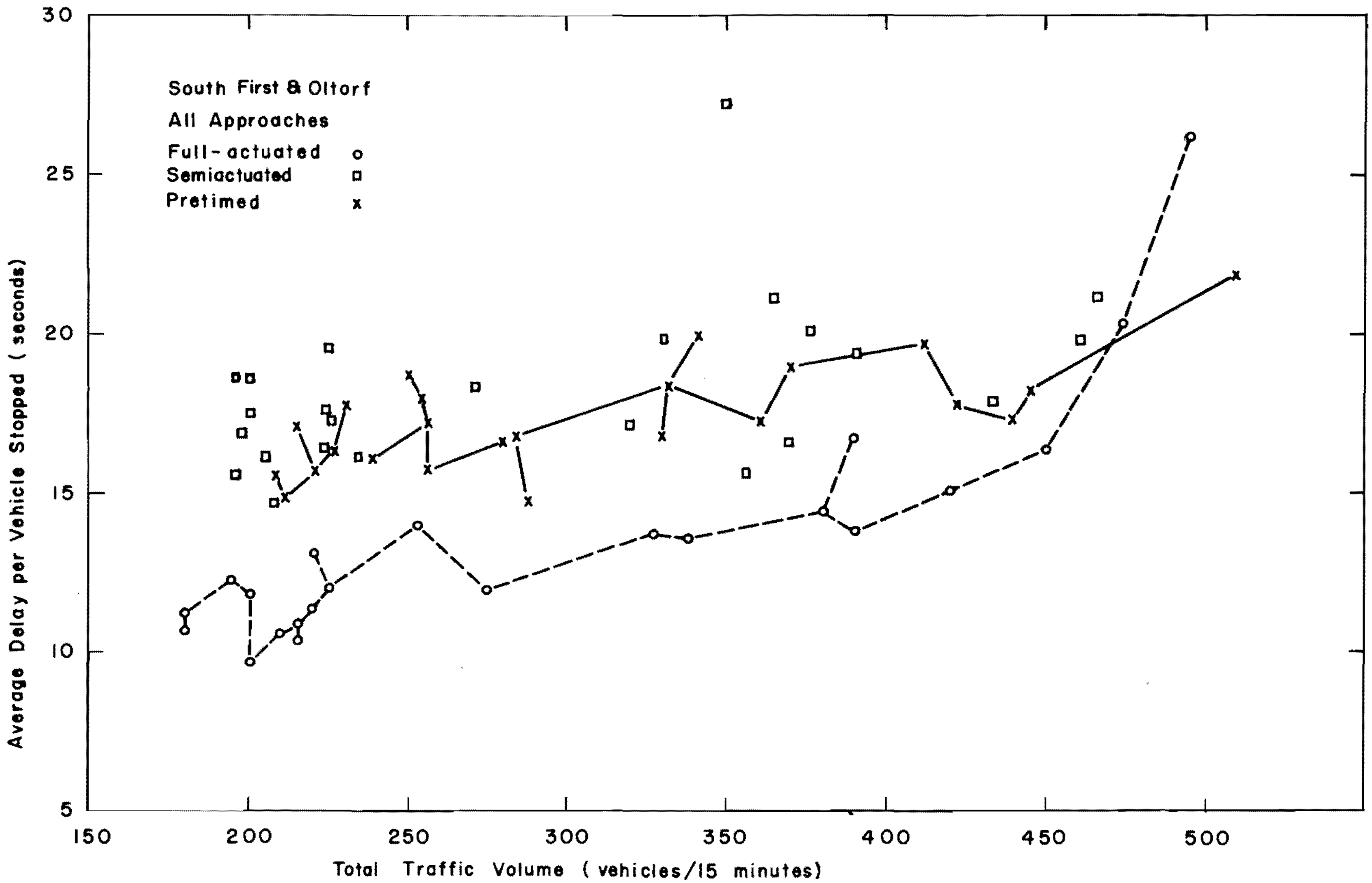

Fig 6.26. Average delay per vehicle stopped, all approaches, South First and Oltorf.

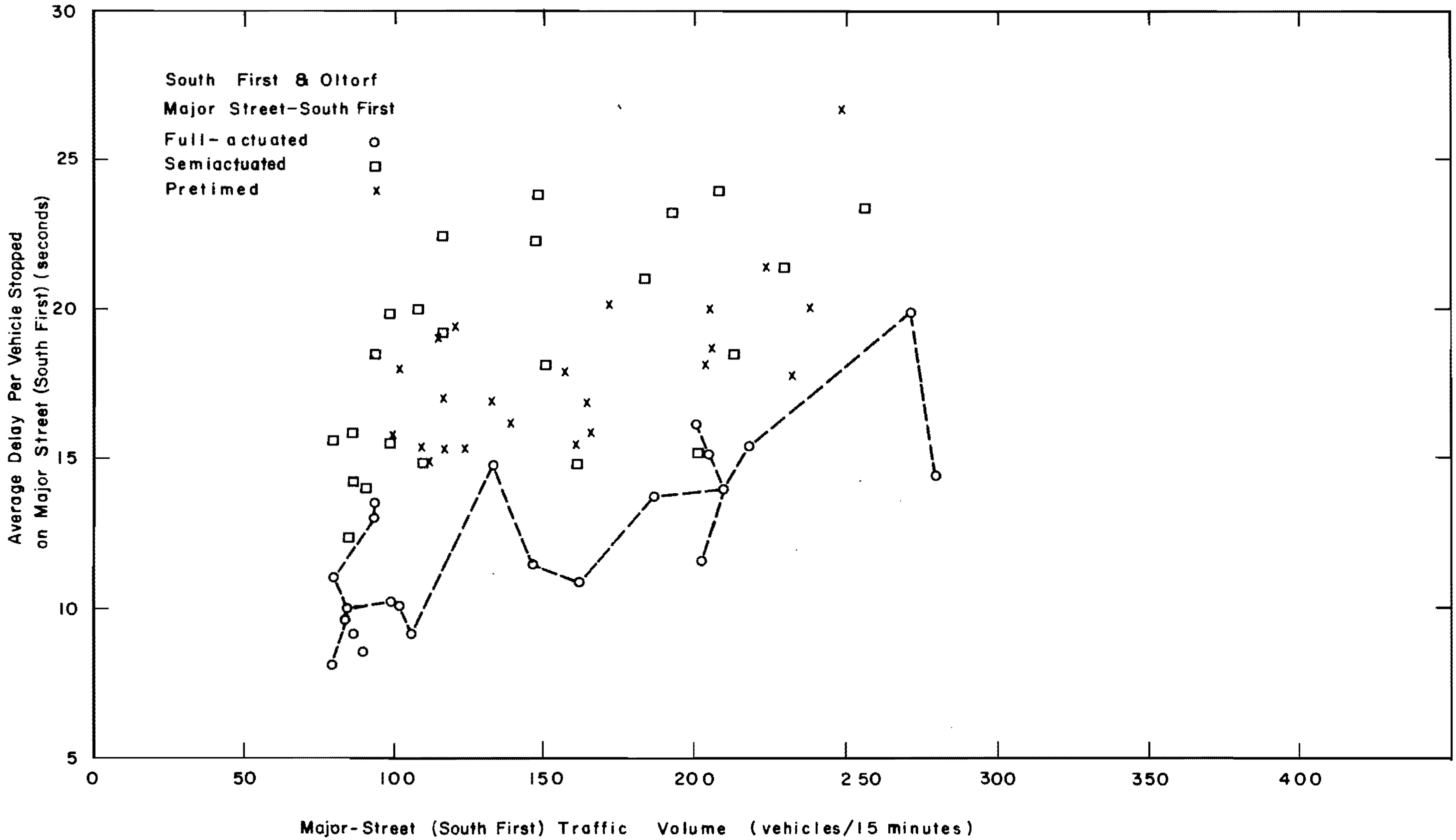

Fig 6.27. Average delay per vehicle stopped, major street (South First), South First and Oltorf.

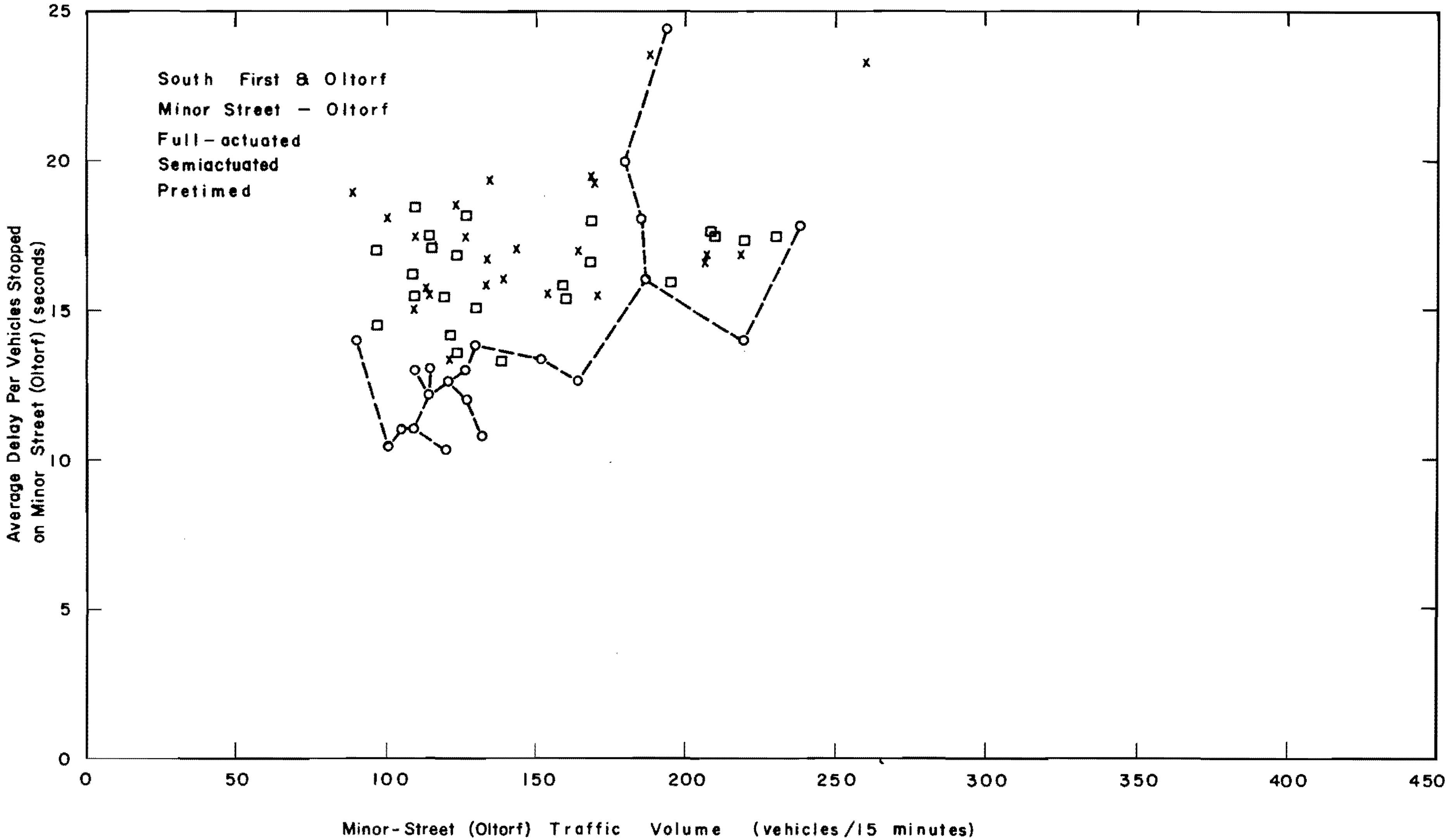

Fig 6.28. Average delay per vehicle stopped, minor street (Oltorf), South First and Oltorf.

The relations between average delay per vehicle stopped and traffic volume per 15 minutes are shown in Fig 6.27 for the major street (South First) and in Fig 6.28 for the minor street. Average delay per vehicle stopped is of the same magnitude as that observed at Woodrow and Koenig. Full-actuated control caused lower average delay to the vehicles that were stopped than the other types of control in virtually all the cases shown.

<u>Vehicles Stopped</u>

The relation of the total number of vehicles stopped on all approaches to the total traffic volume at South First and Oltorf is shown in Fig 6.29. Full-actuated control generally stopped fewer vehicles than either pretimed or semiactuated control at this intersection (50/50 volume split), whereas semi-actuated control frequently resulted in the smallest number of stopped vehicles at Woodrow and Koenig (30/70 volume split, see Fig 6.14). The number of vehicles stopped was proportional to total volume and similar in magnitude at both intersections.

The number of vehicles stopped may be expressed as a percentage of the total traffic and presented as shown in Fig 6.30. The percentage of vehicles stopped ranged from 40 to 65 percent for all control types. At this intersection where the traffic volume was split approximately 50/50, semiactuated control consistently stopped a higher percentage of the total traffic than either pre-timed or full-actuated control at total volumes less than about 300 vehicles per 15 minutes. Pretimed control stopped 45 to 55 percent of the vehicles throughout the range of volumes from 200 to 450 vehicles per 15 minutes.

Figures 6.31 and 6.32 show the relationships of traffic volume and percentage of vehicles stopped on the major and minor street respectively. There was no pronounced difference in the percentages as at Woodrow and Koenig (see Figs

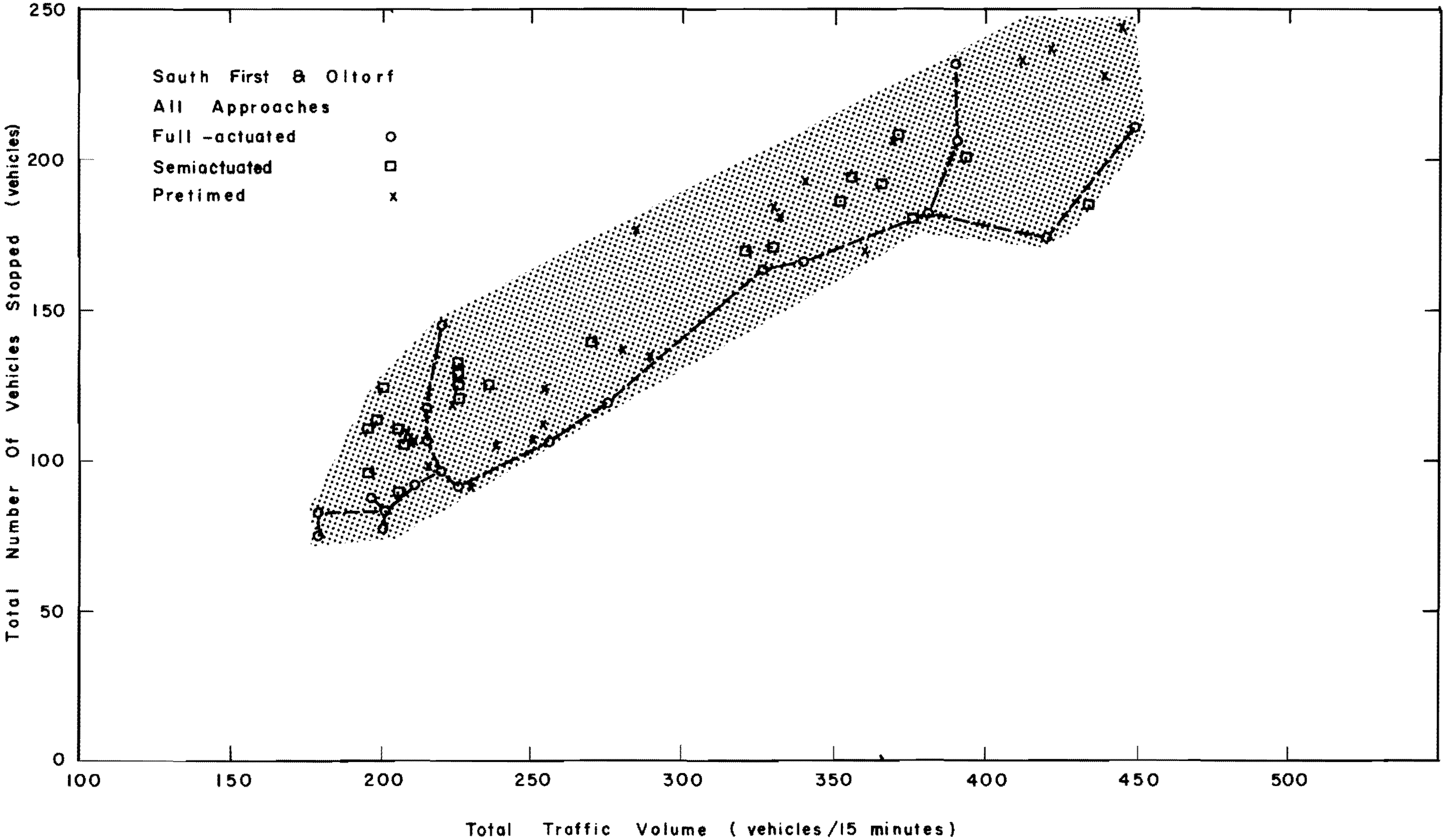

Fig 6.29. Total number of vehicles stopped, all approaches, South First and Oltorf.

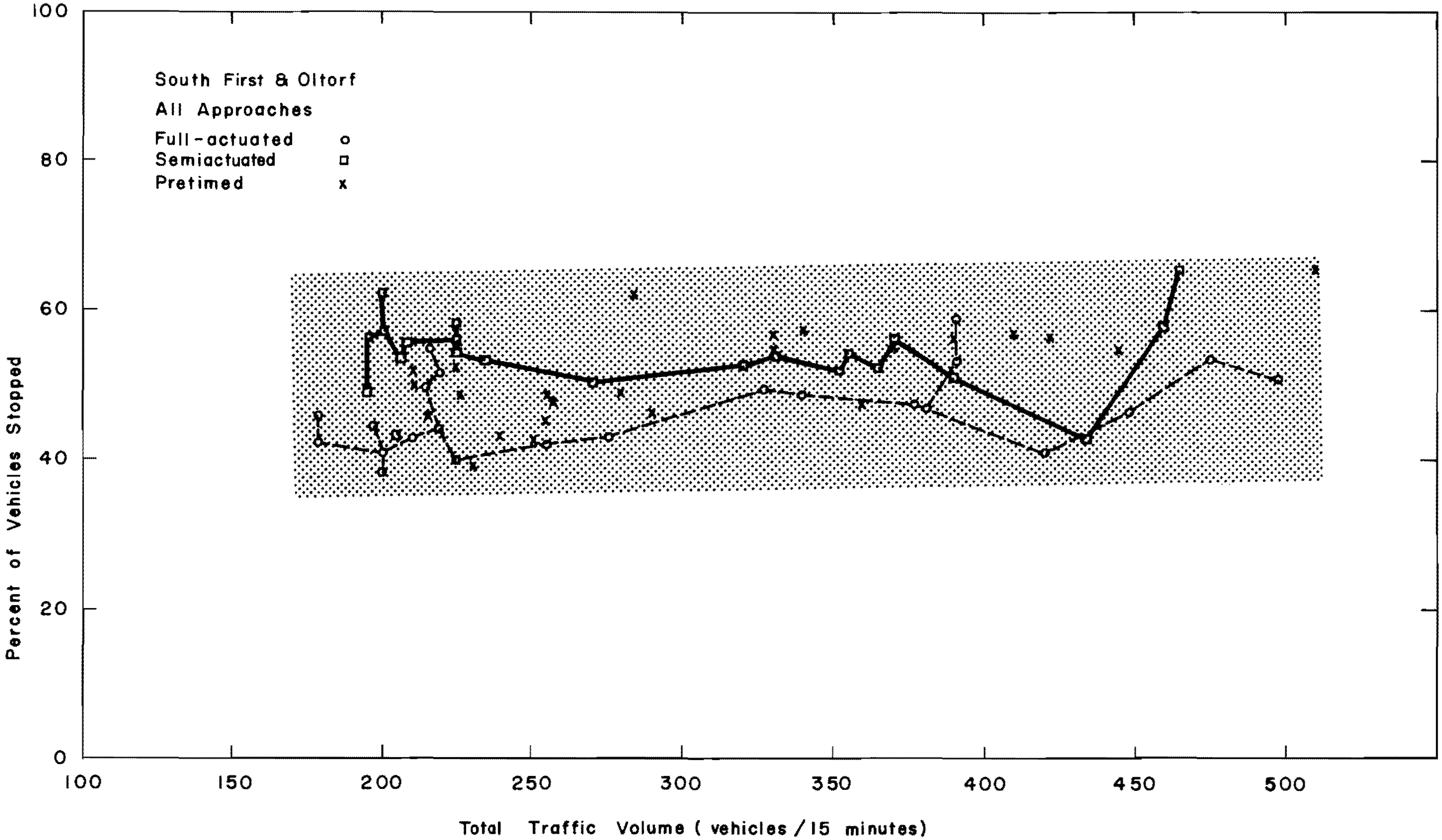

Fig 6.30. Percentage of vehicles stopped, all approaches, South First and Oltorf.

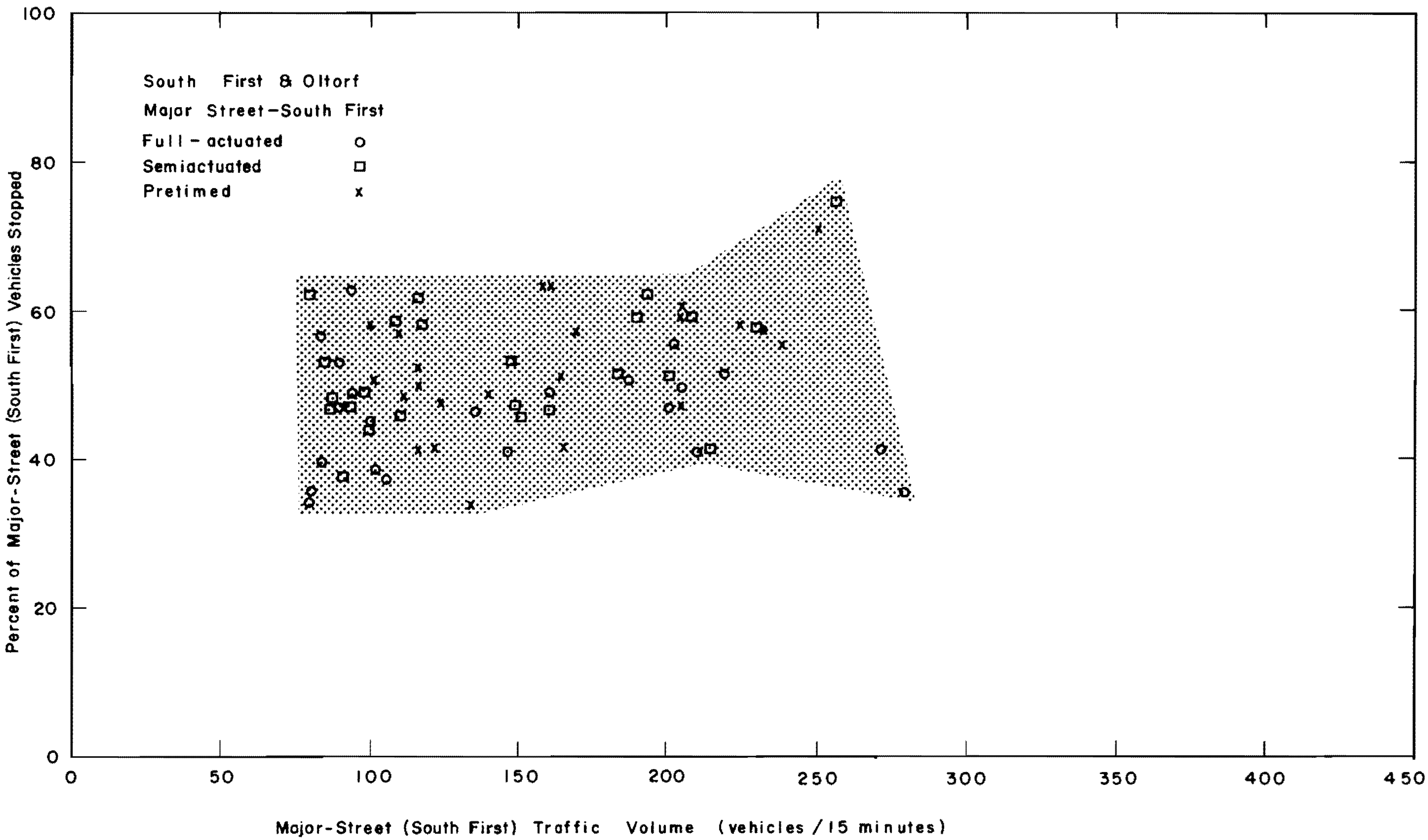

Fig 6.31. Percentage of vehicles stopped, major street (South First), South First and Oltorf..

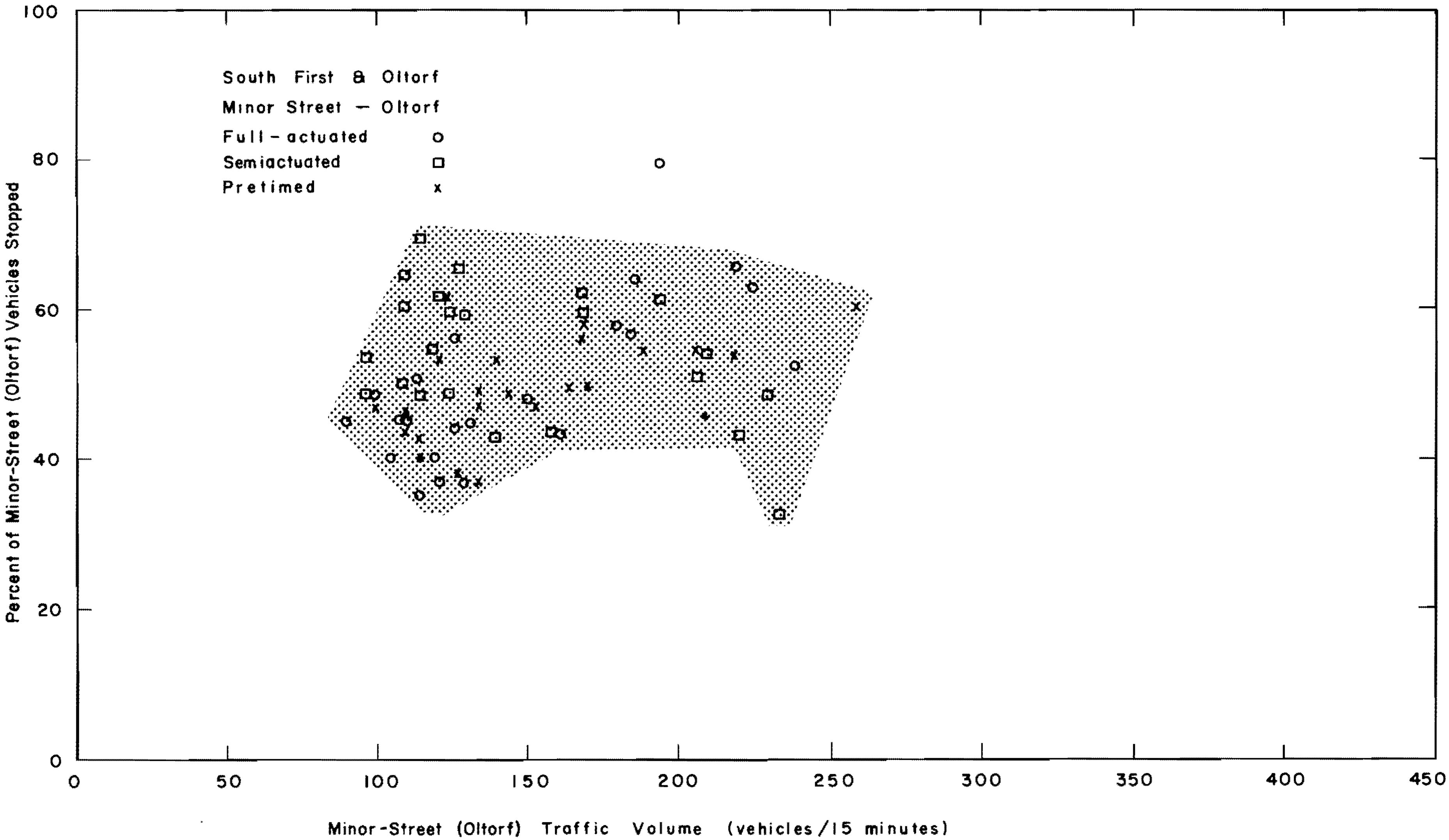

Fig 6.32. Percentage of vehicles stopped, minor street (Oltorf), South First and Oltorf.

6.16 and 6.17), where the traffic on the minor street had a larger percentage stopped than the major street which carried about 60 to 70 percent of the total traffic.

OTHER SIGNALIZED INTERSECTIONS

Although most of the signalized intersections studied had similar characteristics, it is desirable to point out several volume versus delay relationships which lend credence to the analyses presented previously in this chapter.

Exposition and Windsor

Exposition and Windsor is a four-leg, relatively isolated intersection in a primarily residential area in west Austin. Exposition Boulevard, the major street, is 40 feet in width with traffic moving on two approach lanes per leg (see Fig A.10 in Appendix A). Windsor Road, the minor street, is also 40 feet wide, but there are no lane stripes, other than the centerline, to guide traffic flow. Observers noted that vehicles approaching Windsor Road were in single file, but near the intersection two lanes of traffic usually formed under heavy volume conditions. Approximately 200 feet west of the intersection, Windsor Road is narrowed to a width of approximately 30 feet. Here vehicles are forced to merge into a single stream and some turbulence in vehicular flow was noticeable. Stopped-time delay was not visibly affected by this geometric restriction, but this factor should be considered when relating stopped-time delay to travel time.

Almost 18 hours of delay studies were conducted at Exposition and Windsor in 1966. The signal controller settings during the studies and the approach volumes and turning movements for each 15-minute period of the studies are presented in Appendix A. Minor-street (Windsor) traffic ranged from a low of 31 percent to a high of 43 percent of the total volume, and left turns were less than 10 percent of the total volume during all the studies.

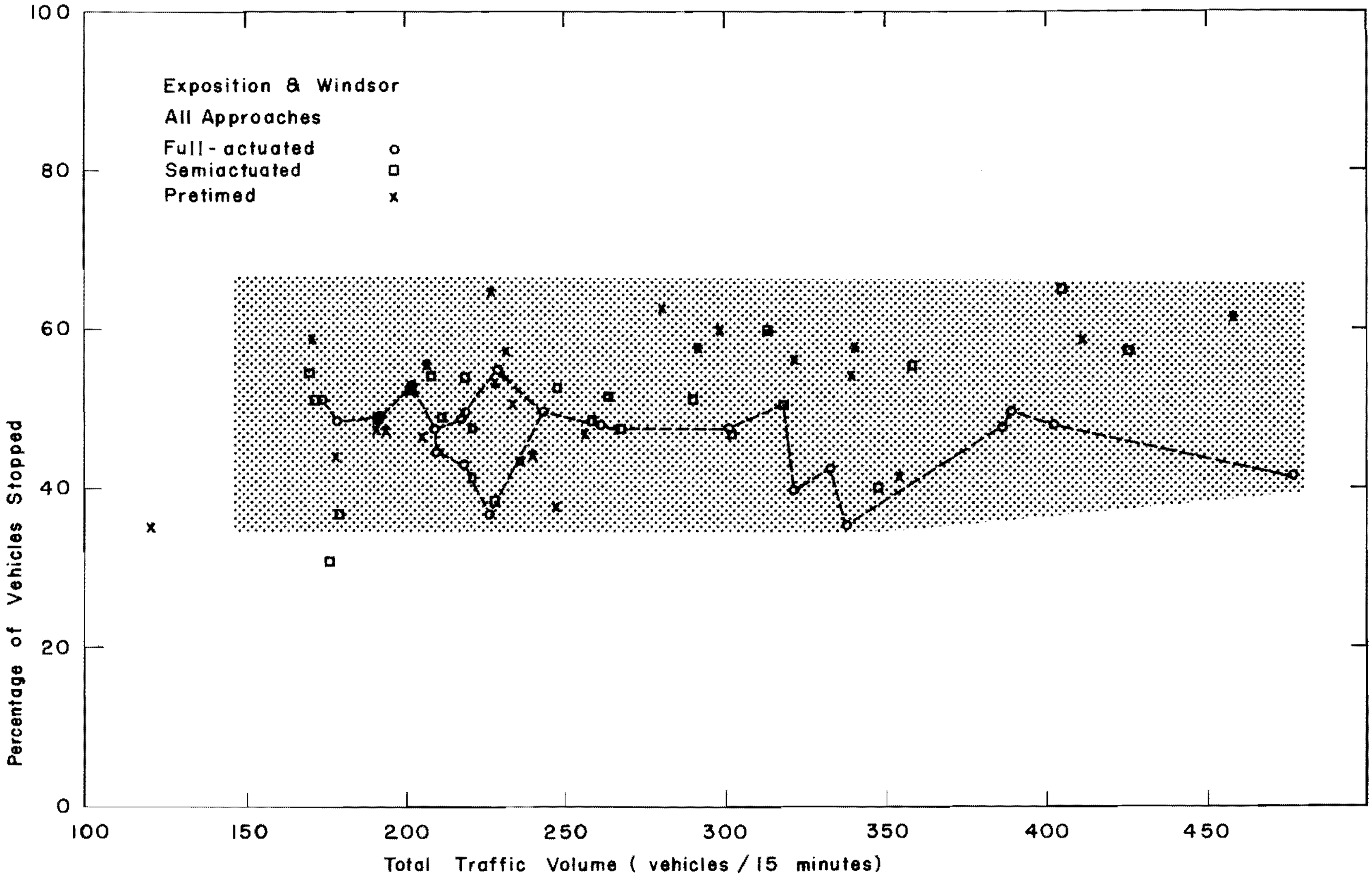

Fig 6.33. Percentage of vehicles stopped, all approaches, Exposition and Windsor.

Figure 6.33 shows the percentage of vehicles stopped at this intersection. The percentages range from approximately 35 to 65 percent and average about 50 percent. Percentages increased slightly with volume when semiactuated or pretimed traffic control was used, but the percentage of stops when using full-actuated control was relatively consistent with respect to volume and slightly less than the percentages resulting from pretimed or semiactuated traffic control.

Figures 6.34 and 6.35 show the average delay per major-street vehicle and average delay per minor-street vehicle and vehicular volume relationships. The data in Fig 6.34 indicate that average delay per major-street vehicle was generally greater when pretimed control was used than when using vehicle-actuated control. But Fig 6.35 shows that all three types of control resulted in similar average delays to minor-street traffic. It should be pointed out that for the studies at Exposition and Windsor green intervals were set at 30 seconds on both the major and the minor streets under pretimed control, even though the traffic volume split (30 to 40 percent on minor street) would ordinarily indicate unequal green intervals for optimum performance. The relatively large average delays to major-street vehicles for pretimed control (Fig 6.34) reflect the improper proportioning of the green time for the 30/70 volume split. Similar effects were observed at Woodrow and Koenig (Fig 6.9) where the split was approximately the same.

Figure 6.36 shows the average delay per vehicle and total traffic volume relationships at Exposition and Windsor. Average delay per vehicle increased slightly with total volume for each control type; moreover, the rate of increase for each control type was about equal. There was no clearly defined advantage to using any particular type of control that was studied at this intersection as far as average delay per vehicle was concerned; however, if total delay is

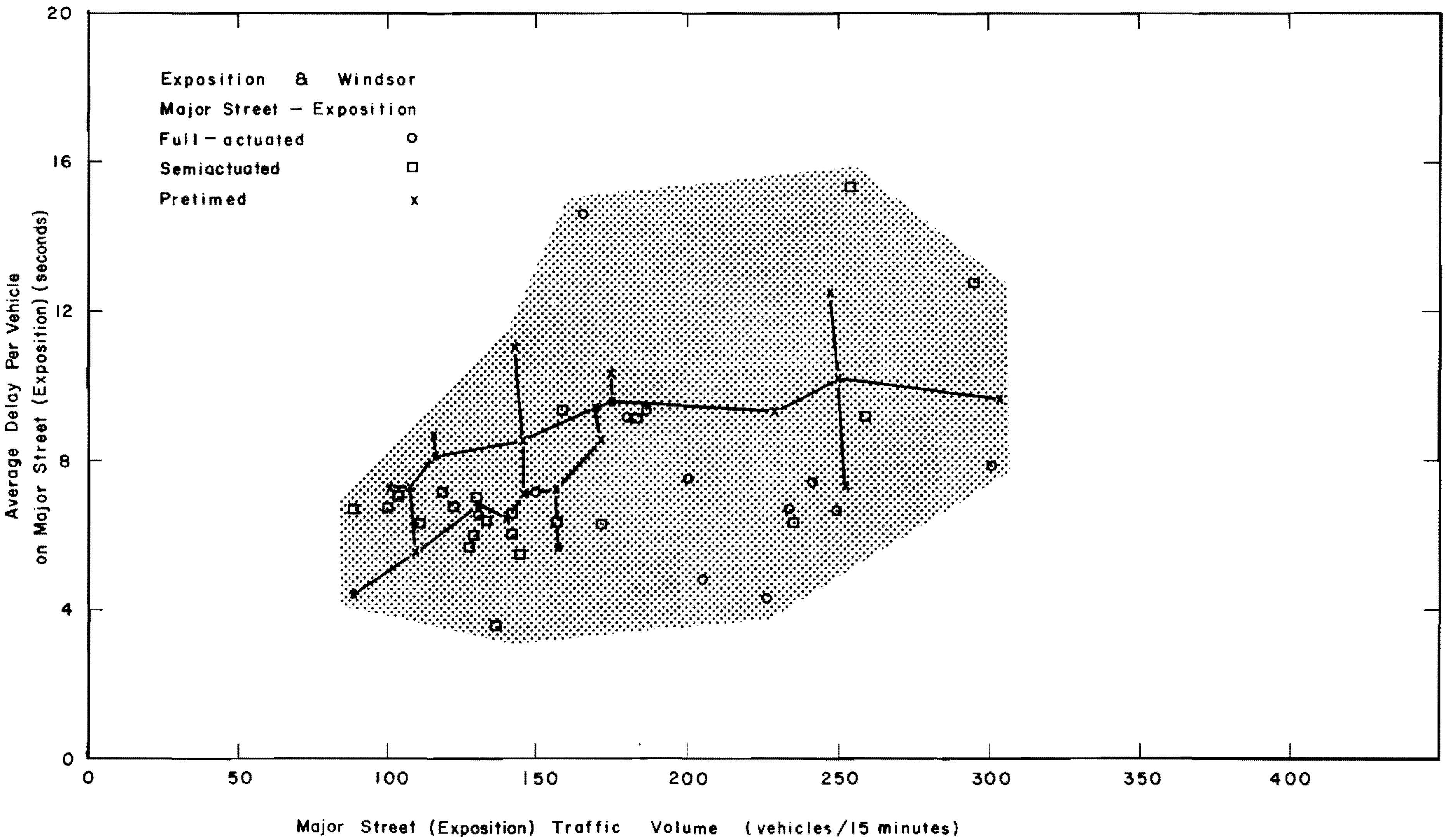

Fig 6.34. Average delay per vehicle, major street (Exposition), Exposition and Windsor.

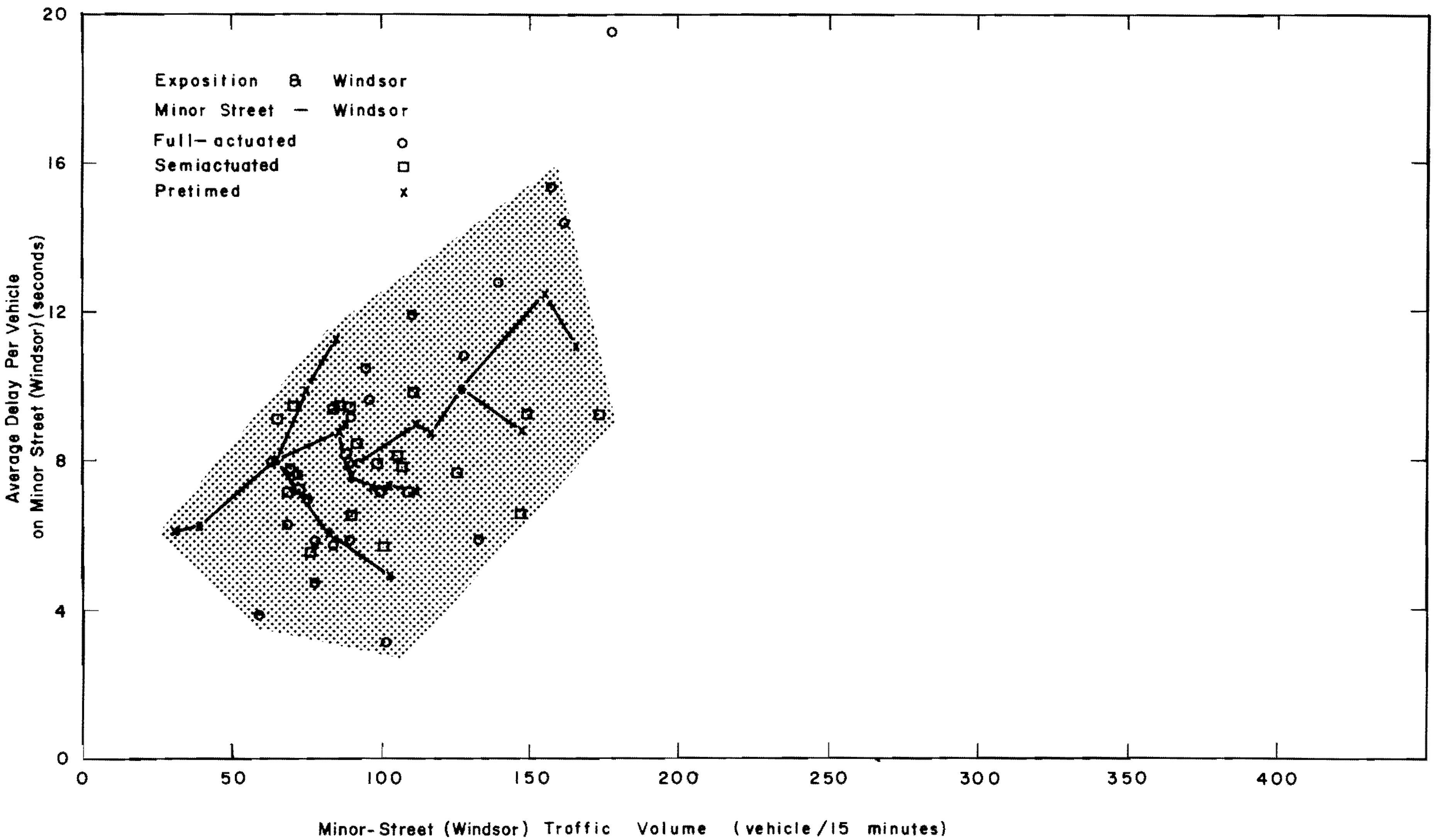

Fig 6.35. Average delay per vehicle, minor street (Windsor), Exposition and Windsor.

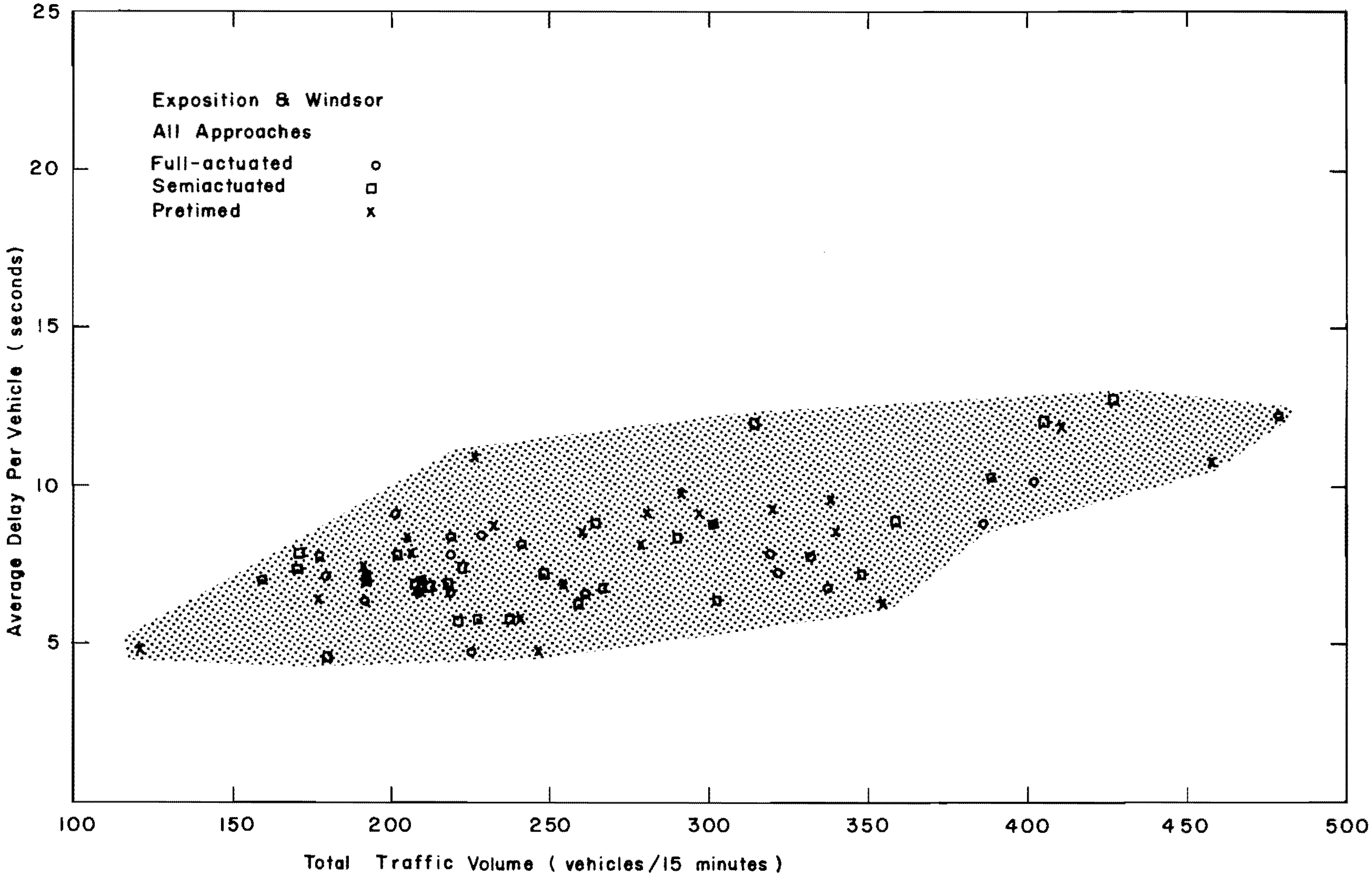

Fig 6.36. Average delay per vehicle, all approaches, Exposition and Windsor.

considered, semiactuated equipment exhibited an advantage over pretimed and full-actuated controllers at this intersection (see Table 7.7).

Hildebrand and Blanco

Hildebrand and Blanco is a four-leg, relatively isolated intersection in San Antonio. Approaches are 43 feet wide, with two approach lanes on each leg.

Delay studies were conducted at Hildebrand and Blanco in August 1966 for full-actuated and pretimed traffic control. Each intersection leg had two approach lanes for vehicular flow. Dial settings and 15-minute volumes are summarized in Appendix A.

Figure 6.37 shows the percentage of vehicles stopped versus 15-minute volumes. For most of the 15-minute volumes shown, the percentage ranges between 40 and 60 percent, as at the other intersections studied.

Figures 6.38 and 6.39 show the average delay per major-street vehicle and average delay per minor-street vehicle and vehicular volume relationships for Hildebrand and Blanco. Both of these figures indicate that full-actuated control is a slightly more efficient method of controlling traffic for most of the street volumes shown at this intersection.

Figure 6.40 shows the average delay per vehicle on all approaches versus 15-minute vehicular volumes. At total volumes ranging from 350 to 500 vehicles per 15 minutes, full-actuated control is slightly more efficient than pretimed control. But as volumes exceed 500 vehicles per 15 minutes, there is more scatter in the plotted data, and full-actuated control appears to have lost its operational advantages over pretimed control.

Finally, Fig 6.41 shows the relationship between total vehicle-seconds of delay and 15-minute volumes for Hildebrand and Blanco and Woodrow and Koenig. These intersections are similar both in operating characteristics and in geometric features but are located in two different cities. As shown in

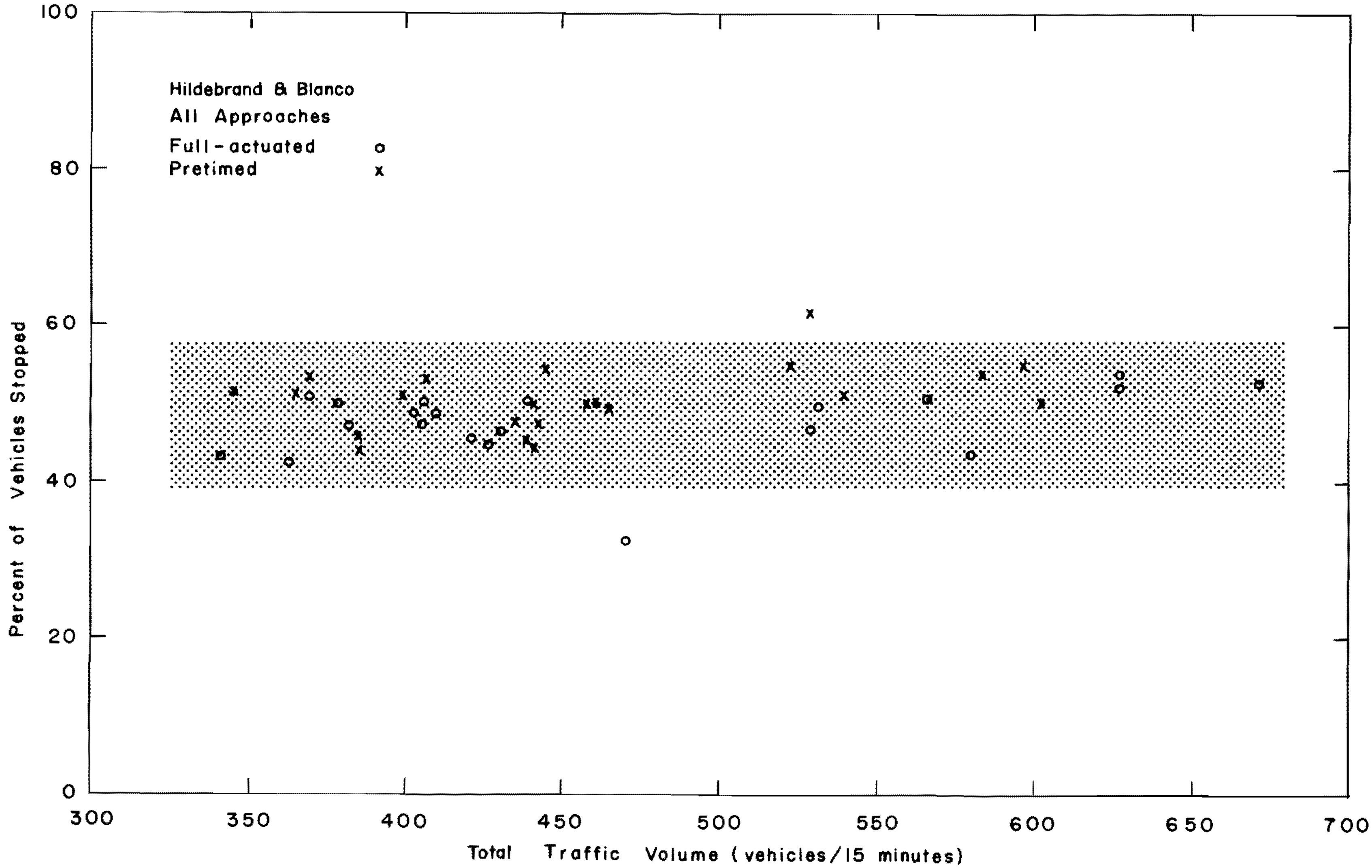

Fig 6.37. Percentage of vehicles stopped, all approaches, Hildebrand and Blanco.

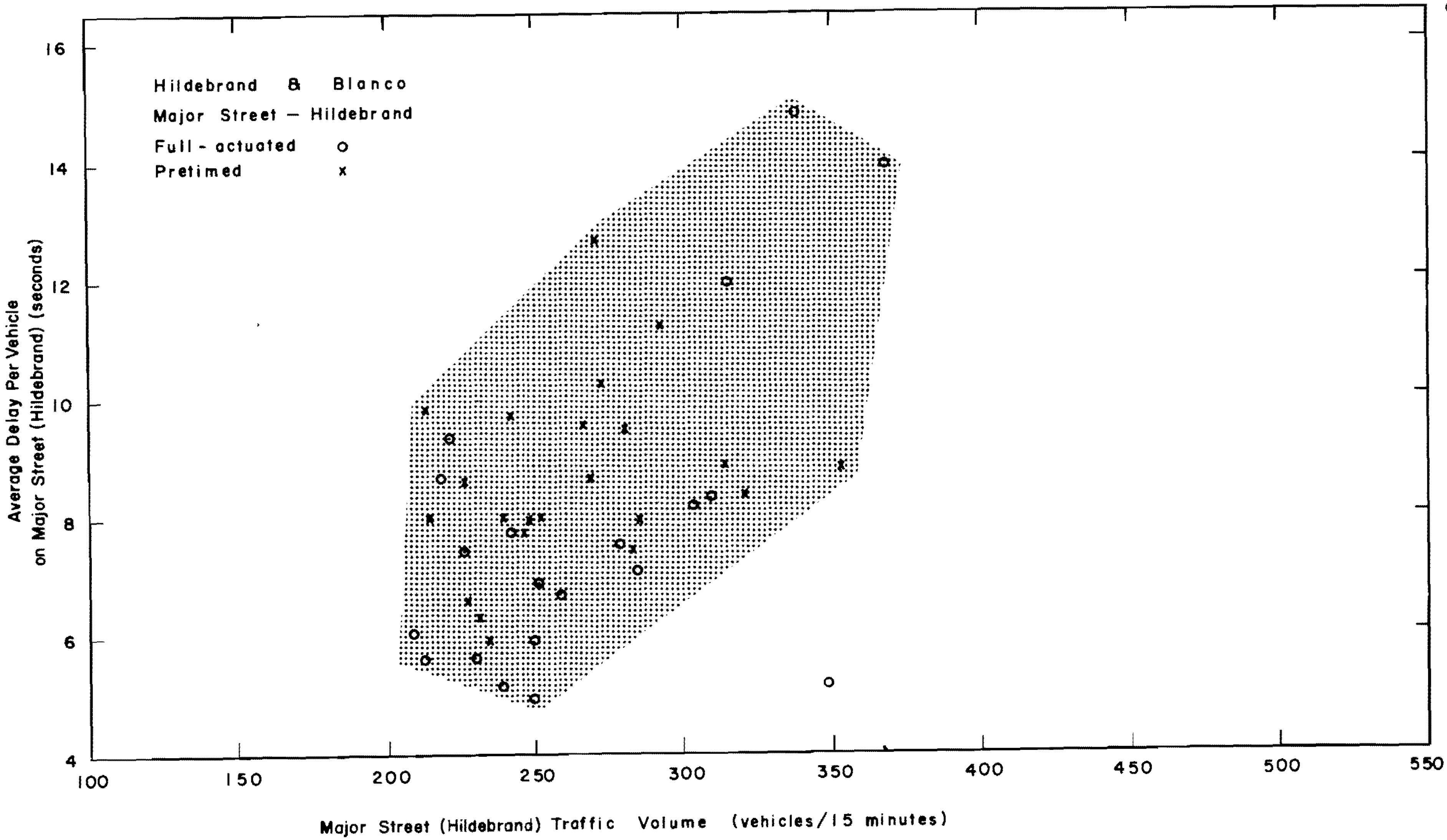

Fig 6.38. Average delay per vehicle, major street (Hildebrand), Hildebrand and Blanco.

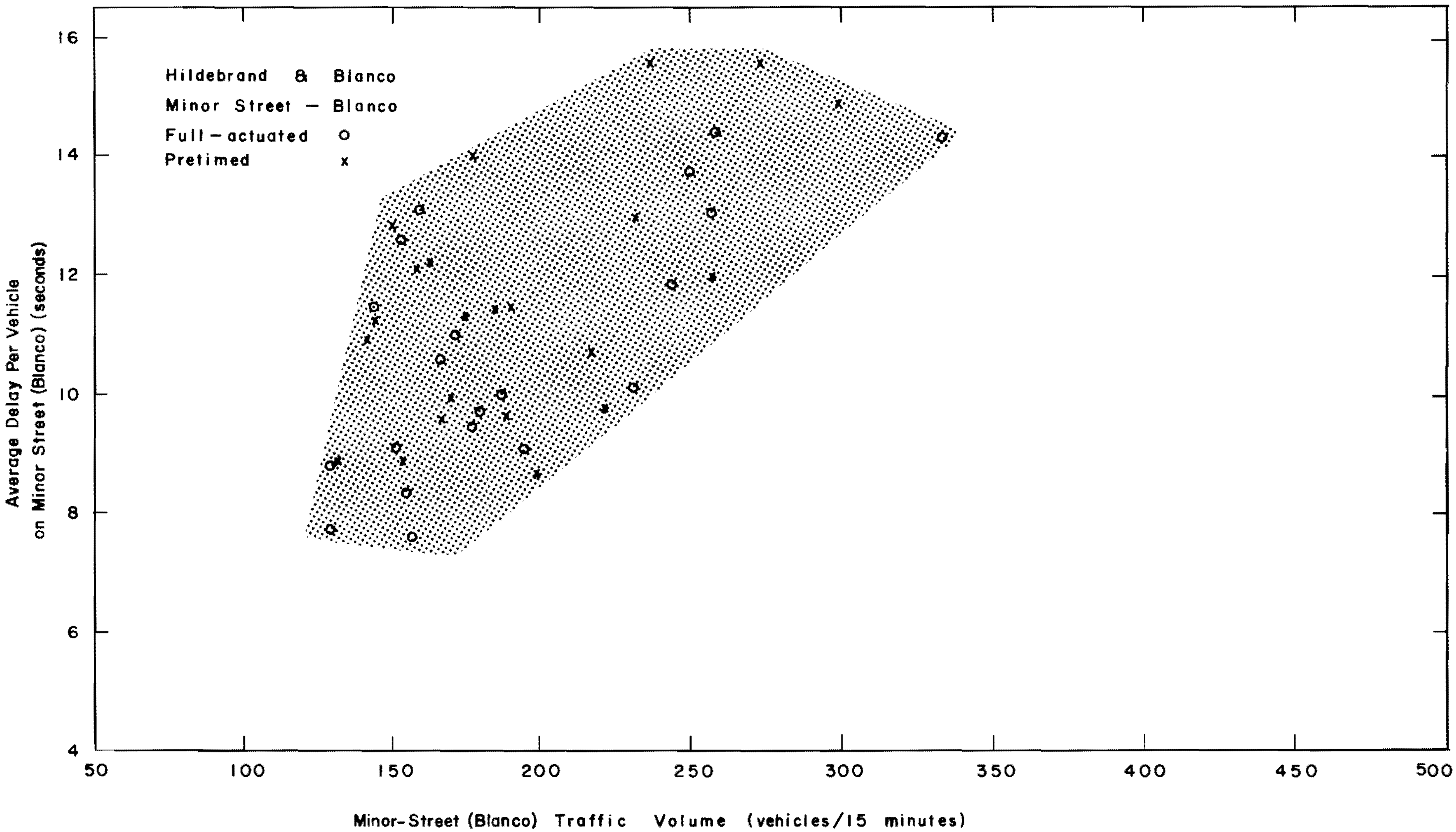

Fig 6.39. Average delay per vehicle, minor street (Blanco), Hildebrand and Blanco.

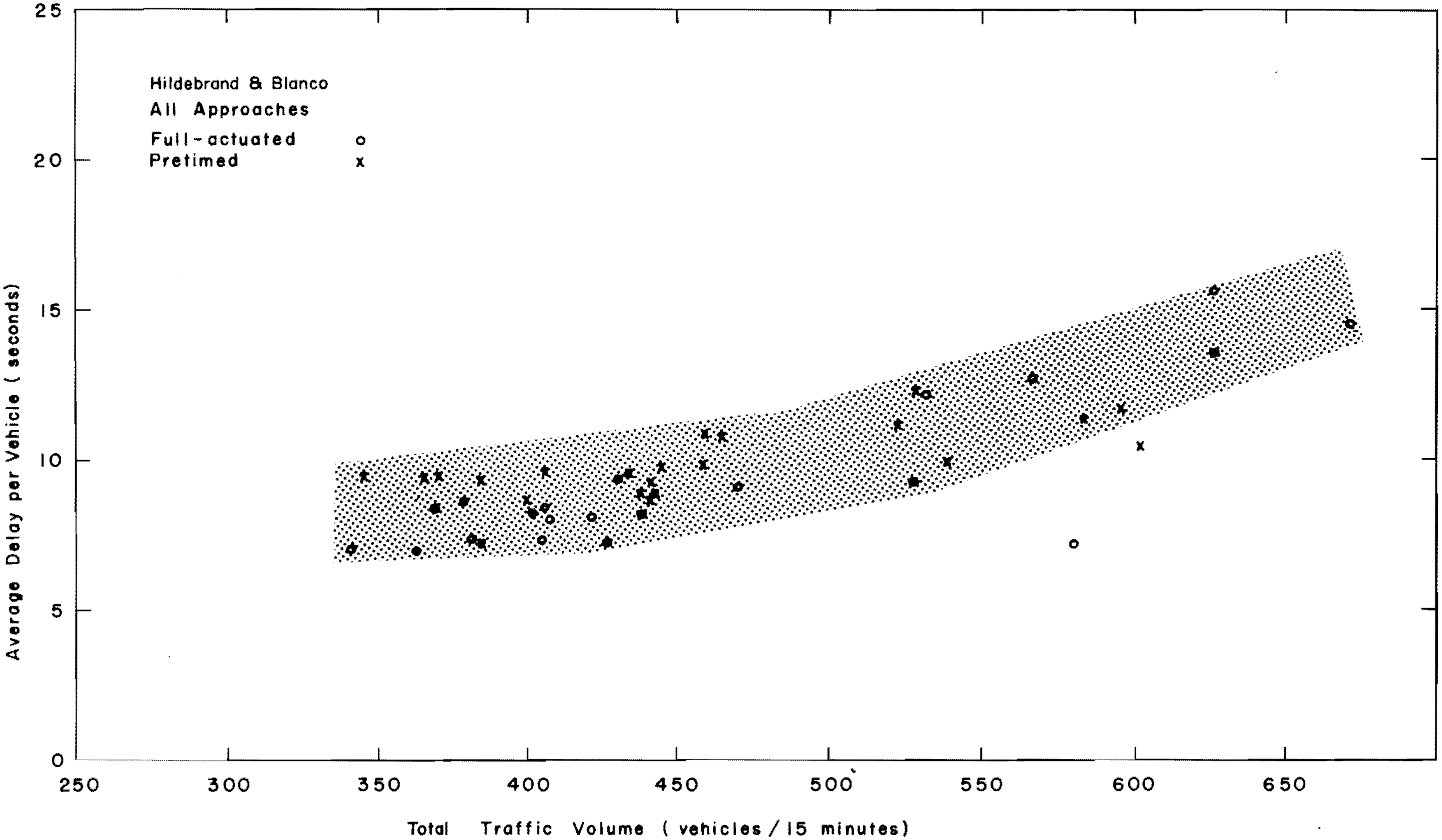

Fig 6.40. Average delay per vehicle, all approaches, Hildebrand and Blanco.

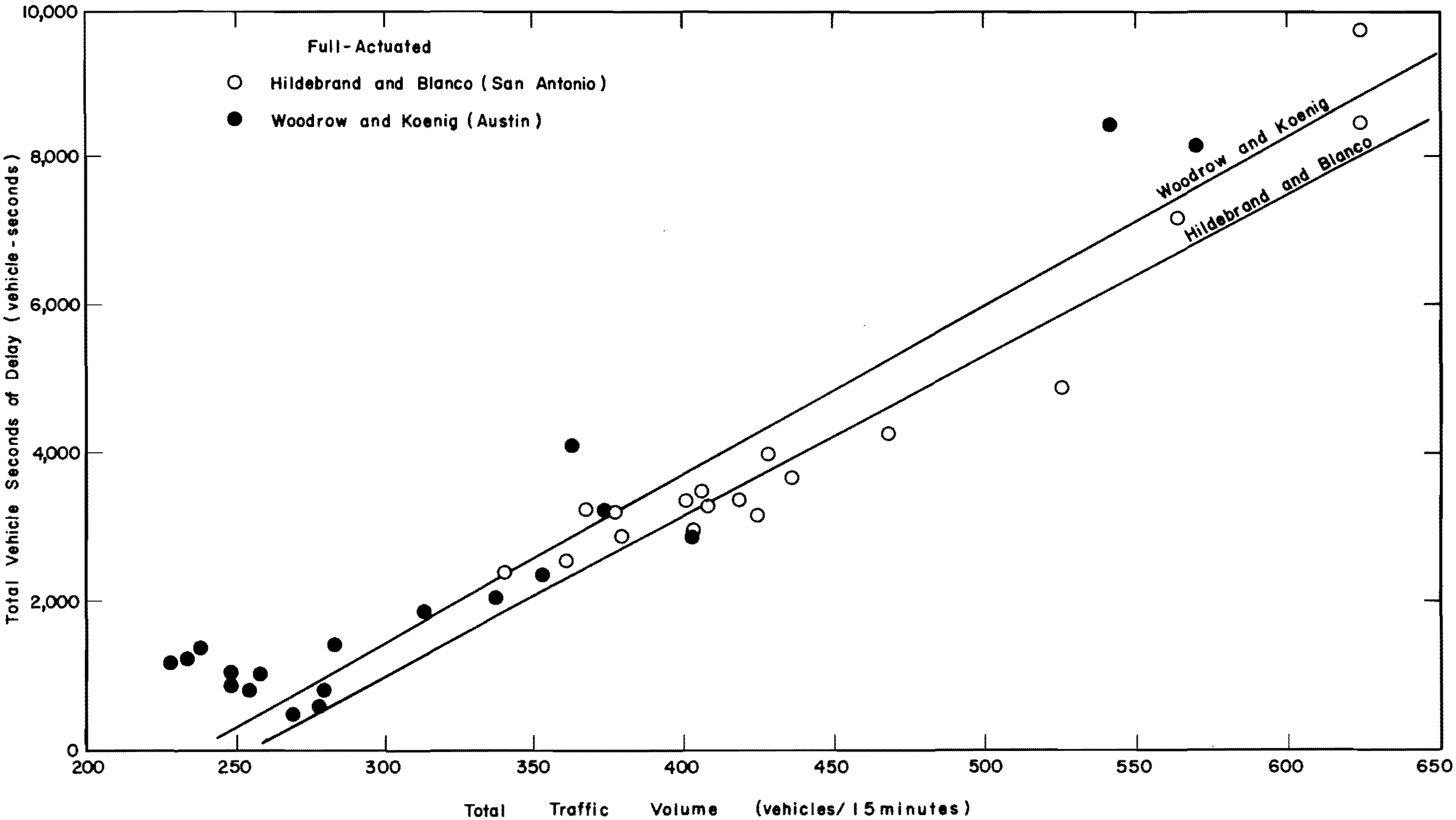

Fig 6.41. Vehicular delay versus 15-minute volume for similar intersection in two Texas cities (San Antonio and Austin), full-actuated.

this figure, the delay versus volume relationship follows the same trend for both intersections and there appears to be no significant effect of location. Drivers responded in the same manner to similar situations.

COST ANALYSIS

In determining which type of control system could be installed at a particular intersection, consideration must be given to the effect of the system on the intersection users as well as to the cost of buying and maintaining the hardware. The following oversimplified example is presented to illustrate the importance of using the proper control system to minimize delay. While the cost of a signal installation will vary from one location to the next, depending on a number of factors, the cost data used in this example can be considered representative for a typical installation.

Approximate costs of three types of signal installations are shown in Table 6.1. The installation cost includes items such as mast arms and poles, signal heads, wire, pull boxes, controller with cabinet, detectors where applicable, and labor costs. The total annual cost shown in Table 6.1 includes maintenance costs and the annual amount that would have to be deposited at 5 percent interest in a sinking fund to accumulate the original installation cost at the end of a ten-year design life.

Using the data presented in Figs 6.11 and 6.15, Table 6.2a was developed to show the major portion of total average hourly cost incurred by stopped vehicles when the intersection was operating at a volume of 1,400 vehicles per hour. This table shows that the cost of moving 1,400 vehicles per hour through the intersection operating under full-actuated control was about $3.17 less per hour than when pretimed control was used.

Assuming that the intersection accommodated at least 1,400 vehicles per hour during 15 hours each week (a conservative estimate; see Appendix A), the

TABLE 6.1. TYPICAL COST OF SIGNAL INSTALLATIONS

Type of Control	Representative Installation Cost*	Annual Maintenance Cost*	Total Annual Cost Assuming 5 Percent Interest Compounded Annually, Ten-Year Design Life, Zero Salvage Value
Full-Actuated	$ 8,000.00	$ 175.00	$ 811.00
Semiactuated	$ 6,500.00	$ 150.00	$ 666.75
Pretimed	$ 4,000.00	$ 100.00	$ 418.00

* Ref 7, p 11.

TABLE 6.2a. VEHICLE OPERATING COST

Type of Control	Cost Estimates* Stopping Cost, $/stop	Idling Cost, $/veh-hr	Value of Time Lost in Idling, $/veh-hr	Percentages of Vehicles Stopped, 1,400 veh/hr	Number of Vehicles Stopped Per Hour, 1,400 veh/hr	Average Delay Per Stopped Vehicle, 1,400 veh/hr, sec	Average Cost Per Stopped Vehicle	Total Average Hourly Cost Incurred By Stopped Vehicles
Full-Actuated	0.00710	0.140	1.70	42	588	17.0	$ 0.0158	$ 9.29
Semiactuated	0.00710	0.140	1.70	45	630	18.5	$ 0.0166	$ 10.45
Pretimed	0.00710	0.140	1.70	50	700	21	$ 0.0178	$ 12.46

* Ref 38, p 662.

TABLE 6.2b. ANNUAL COSTS

Type of Control	Annual Cost = Total Annual Cost (From Table 6.1) Plus Average Hourly Cost (Table 6.2a) × 15 hrs/wk × 52 wks/yr
Full-Actuated	$ 8,057.00
Semiactuated	$ 8,818.00
Pretimed	$ 10,137.00

annual cost of operating the intersection under full-actuated control was nearly $2,100 less than for pretimed control as shown in Table 6.2b. This analysis does not include the savings accrued when the traffic volume through the intersection was less than 1,400 vehicles per hour.

Another way of interpreting the costs incurred from the various types of control can be in terms of the time required to offset the difference in initial costs. For the conditions stated above, the excess initial cost of full-actuated control over pretimed control would be compensated in less than two years of operation.

These computations indicate that, from economic considerations alone, it appears that the most sophisticated control equipment can be easily justified, even though equipment and maintenance costs may be higher than for less efficient control.

CHAPTER 7. WARRANTS FOR TRAFFIC SIGNALS

Information recorded by the digital delay data recorder has been used to study existing as well as proposed warrants for the installation of traffic signals. Due to the fact that the D3 Recorder was used to measure only vehicular traffic volume and delay characteristics, warrant factors such as pedestrian volume and accident experience cannot be evaluated from available data. Volume and resulting delay characteristics concerning warrants can be evaluated, however, as will be demonstrated in the following paragraphs.

INTERSECTIONS STUDIED

Three intersections in Austin, Texas (Woodrow at Koenig, South First at Oltorf, and Exposition at Windsor) were used for the studies conducted on June 20, August 4, and June 23, 1967, respectively. All three intersections were equipped with full-actuated traffic controllers which were adjusted to function as either semiactuated or pretimed controllers during certain phases of the study. In addition to the six hours of vehicular delay studies per day normally conducted in this research study, 12-hour traffic volume surveys were conducted from 0700 to 1900 hours on the day of the traffic study at each intersection. The data for the eight highest hours of traffic volume from each intersection, along with other pertinent information, may be seen in Tables 7.1, 7.2, and 7.3. From this information, which shows peak-hour volumes approaching 2,000 vph (probably too great for stop-sign control), and from the fact that these intersections have been signalized for over ten years, it should be clear that the question to be resolved was not the one

TABLE 7.1. TRAFFIC VOLUME DATA FOR EIGHT HIGHEST HOURS AT WOODROW AND KOENIG, JUNE 20, 1967, 0700-1900 HOURS

Rank	Total Volume, vph	Time of Occurrence	Major Street, vph	Higher Minor Approach, vph
1st	1,860	1645-1745	1,188	451
2nd	1,767	0715-0815	951	662
3rd	1,351	1145-1245	960	230
4th	1,220	1745-1845	814	240
5th	1,087	1245-1345	954	183
6th	1,081	1545-1645	802	143
7th	896	1045-1145	680	119
8th	894	1345-1445	644	156

TABLE 7.2. TRAFFIC VOLUME DATA FOR EIGHT HIGHEST HOURS AT SOUTH FIRST AND OLTORF, AUGUST 4, 1967, 0700-1900 HOURS

Rank	Total Volume, vph	Time of Occurrence	Major Street, vph	Higher Minor Approach, vph
1st	1,918	1645-1745	1,379	336
2nd	1,424	1745-1845	759	402
3rd	1,298	0715-0815	586	563
4th	1,215	1145-1245	595	319
5th	1,157	1545-1645	582	292
6th	1,040	1345-1445	573	235
7th	976	1245-1345	487	270
8th	974	1045-1145	514	240

TALBE 7.3. TRAFFIC VOLUME DATA FOR EIGHT HIGHEST HOURS AT EXPOSITION AND WINDSOR, JUNE 23, 1967, 0700-1900 HOURS

Rank	Total Volume, vph	Time of Occurrence	Major Street, vph	Higher Minor Approach, vph
1st	1,460	1645-1745	1,033	209
2nd	1,005	0730-0830	1,023	223
3rd	960	1200-1300	673	312
4th	90	1745-1845	631	321
5th	815	1545-1645	572	290
6th	742	1100-1200	520	271
7th	726	1300-1400	531	243
8th	635	0930-1030	491	203

of signals versus other control equipment, but what type of signalization was warranted and which type functioned best from a delay standpoint.

WARRANTS FOR PRETIMED SIGNALS FROM THE MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES

The data from the intersections mentioned above facilitates the investigation of warrants 1, 2, and 6 for the installation of pretimed signals, as found on pages 185 and 190 of the Manual on Uniform Traffic Control Devices (Ref 18). Warrants 1 and 2 state that for their terms to be satisfied, the traffic volume at a given intersection for each of eight hours of an average day must be equal to or greater than the values specified in Tables 7.4 and 7.5, respectively. Warrant 6 states that, even if no individual warrant is satisfied, signals may still be justified if 80 percent of the values specified in any two warrants is provided.

The data shown in Tables 7.1, 7.2, and 7.3 were checked against these warrants. The results of this test are shown in Table 7.6. Because none of the intersections satisfied any of these warrants, it could be said that, according to these pretimed signal warrants based on vehicular volume alone, a pretimed signal was not warranted at any of the intersections.

WARRANTS FOR TRAFFIC-ACTUATED SIGNALS

No warrants for traffic-actuated signals are presented as such in the Manual on Uniform Traffic Control Devices, and, for purposes of this study, portions of a set of warrants proposed by the Texas Highway Department were evaluated. (The entire system of warrants and the accompanying commentary is included in Appendix B.) These warrants were developed by combining practical experience with intersection capacity information appearing

TABLE 7.4. MINIMUM VEHICULAR VOLUMES FOR WARRANT 1 (MINIMUM VEHICULAR VOLUME WARRANT)*

Number of Lanes Per Approach		Volumes for Each of Any Eight High Hours**	
Major Street	Minor Street	Vehicles Per Hour on Major Street (Total of Both Approaches)	Vehicles Per Hour on Higher-Volume Minor Street Approach
1	1	500	150
2 or more	1	600	150
2 or more	2 or more	600	200
1	2 or more	500	200

*Manual on Uniform Traffic Control Devices for Streets and Highways, U. S. Bureau of Public Roads, Washington, D. C., June 1961, p 185.

**Same eight hours for both major-street and minor-street volume.

TABLE 7.5. MINIMUM VEHICULAR VOLUMES FOR WARRANT 2 (INTERRUPTION OF CONTINUOUS TRAFFIC WARRANT)*

Number of Lanes Per Approach		Volumes for Each of Any Eight High Hours**	
Major Street	Minor Street	Vehicles per Hour on Major Street (Total of Both Approaches)	Vehicles Per Hour on Higher-Volume Minor Street Approach
1	1	750	75
2 or more	1	900	75
2 or more	2 or more	900	100
1	2 or more	750	100

*Manual on Uniform Traffic Control Devices for Streets and Highways, U. S. Bureau of Public Roads, Washington, D. C., June 1961, p 186.

**Same eight hours for both major-street and minor-street volumes.

TABLE 7.6. NUMBER OF HOURS PASSING EACH WARRANT FOR EACH INTERSECTION (EIGHT REQUIRED TO PASS)

Warrant Number	Intersection: Woodrow and Koenig	South First and Oltorf	Exposition and Windsor
1 (Minimum volume)	4	2	4
2 (Interruption of traffic)	4	1	2
6 (Combination)	5	2	2

TABLE 7.7. TOTAL DELAY AND RESPECTIVE VOLUMES FOR SIX HOURS OF DELAY STUDY (VEHICLE-SECONDS OF DELAY)

Intersection	Controller: Pretimed	Semiactuated	Full-actuated
Woodrow and Koenig	57,099 (4,497 vehicles)	42,075 (4,502 vehicles)	40,856 (4,449 vehicles)
South First and Oltorf	68,250 (6,810 vehicles)	65,458 (6,815 vehicles)	45,768 (6,452 vehicles)
Exposition and Windsor	50,132 (5,601 vehicles)	44,117 (5,500 vehicles)	49,569 (5,833 vehicles)

in Ref 9. The portion of this system of warrants which was investigated consists of four warrants which were developed for use in urban areas. The system also contains a set of four warrants for use in isolated communities having a population of less than 10,000 (latest Federal census), or in areas where the 85-percentile speed of major-street traffic exceeds 40 miles per hour.

The first of the urban warrants deals with peak-hour volumes and is considered to be satisfied when for one hour (any four consecutive 15-minute periods) of an average day the plotted point representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) falls above the curve in Fig 7.1 for the particular existing combination of approach lanes. The major and minor-street volumes must be for the same time period.

The second warrant deals with the traffic volume for the two highest hours, which are to consist of the four consecutive 15-minute periods having the highest volume and the four consecutive 15-minute periods having the second highest volume. This warrant is considered to be satisfied when for each of the two hours the plotted points representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) both fall above the curve in Fig 7.2 for the particular existing combination of approach lanes. The major and minor-street volumes are for the same hour and the volume on the minor street is not necessarily on the same approach for both hours.

The third and fourth warrants deal with the four highest hour volumes and eight highest hour volumes, respectively. In both cases, the plotted points representing major street and higher-volume minor-street approach are plotted

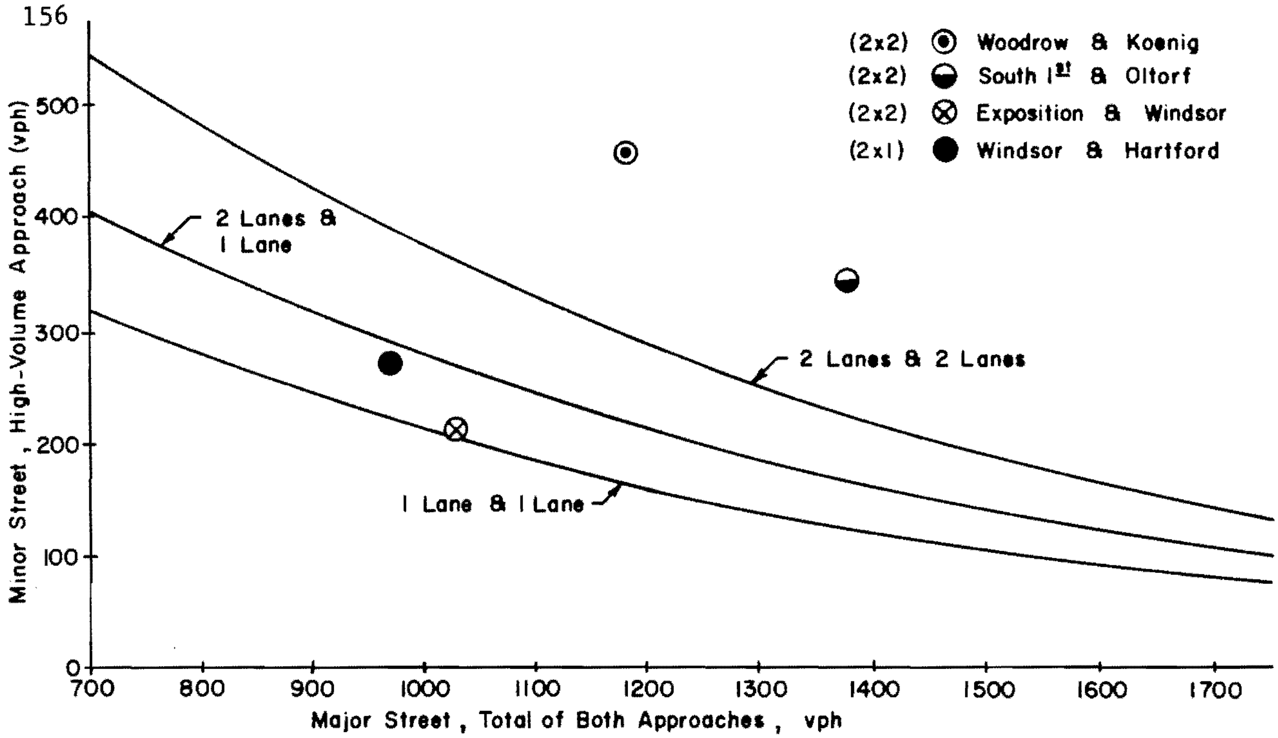

Fig 7.1. Peak-hour volume warrant factor for traffic-actuated signals, highest hour, urban area.

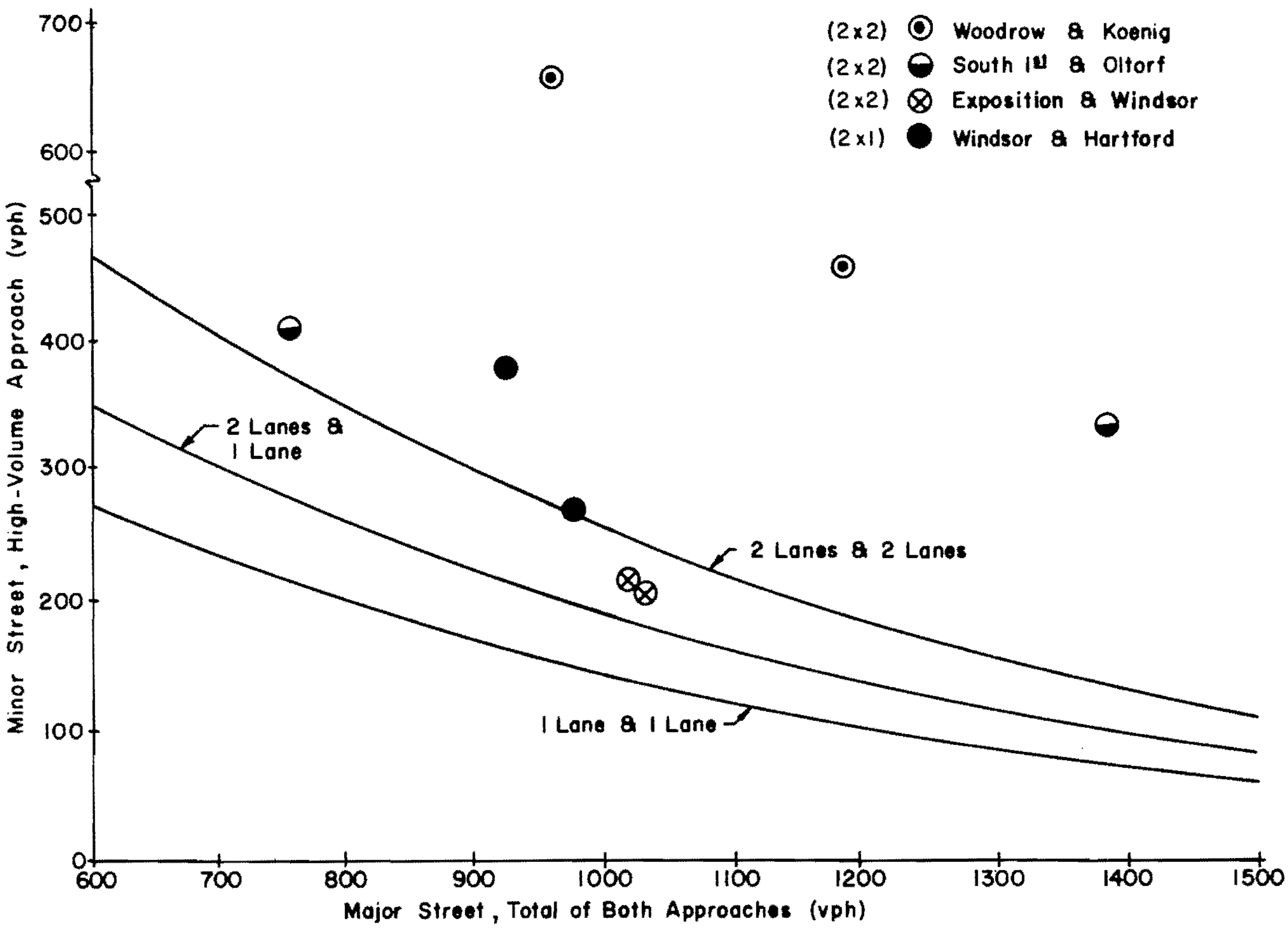

Fig 7.2. Peak-hour volume warrant factor for traffic-actuated signals, two highest hours, urban area.

on Figs 7.3 and 7.4. The same specifications that were applied in the second warrant to the calculation and plotting of the points must also be applied in the third and fourth warrants.

The results of plotting the data from the three previously mentioned intersections, together with that from one additional intersection (Windsor Road at Hartford Road), show that all four intersections satisfy at least one of the warrants and one, South First at Oltorf, satisfies all four (Figs 7.1, 7.2, 7.3, and 7.4).

According to all the previously mentioned warrants, traffic-actuated signals were warranted at these intersections, while pretimed control was not. It was interesting to compare the operation of these two basic types of controllers in terms of vehicular delay at the three intersections selected for evaluation.

The results of delay studies for pretimed, semiactuated, and full-actuated controllers at Woodrow and Koenig, South First and Oltorf, and Exposition and Windsor are illustrated in Figs 7.5, 7.6, and 7.7, respectively. The tabular results of the entire six hours of delay study (Table 7.7) reveal that all intersections had a noticeable decrease in delay when controlled by actuated equiment. Traffic on Sourth First and Oltorf, showed the most pronounced trend, a 33 percent reduction in delay when the control was converted from pretimed to full-actuated at similar volumes (6,810 vehicles under pretimed control versus 6,452 vehicles for the same amount of time under full-actuated control). Woodrow and Koenig showed a 29 percent reduction in delay for the full-actuated condition when compared to pretimed, with a total of 4,497 vehicles during the full-actuated study and 4,449 vehicles during the pretimed condition. Exposition and Windsor, however, showed a 12 percent reduction for the semiactuated controller compared to pretimed, with 5,601 vehicles compared to 5,500 vehicles, respectively.

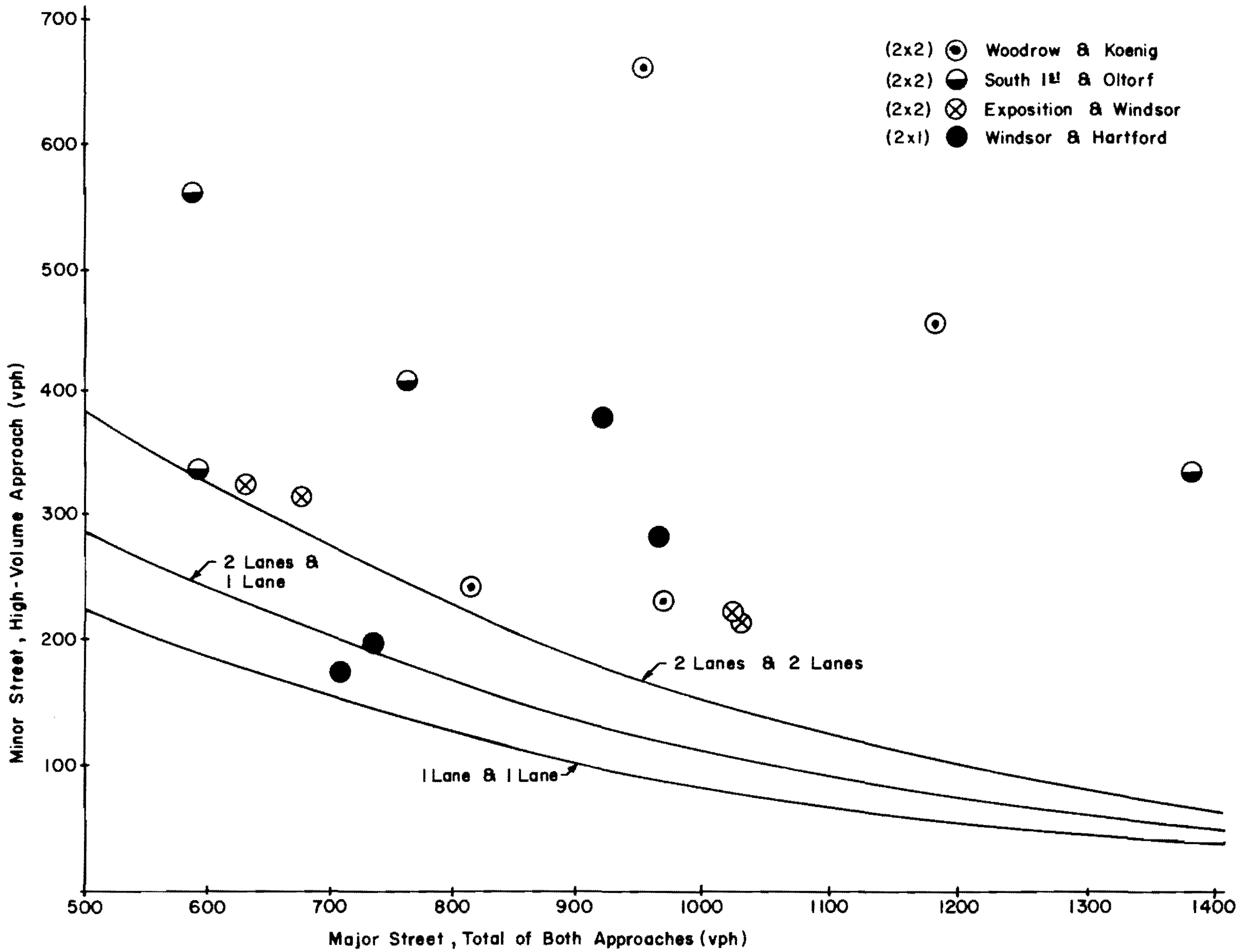

Fig 7.3. Peak-hour volume warrant for traffic-actuated signals, four highest hours, urban area.

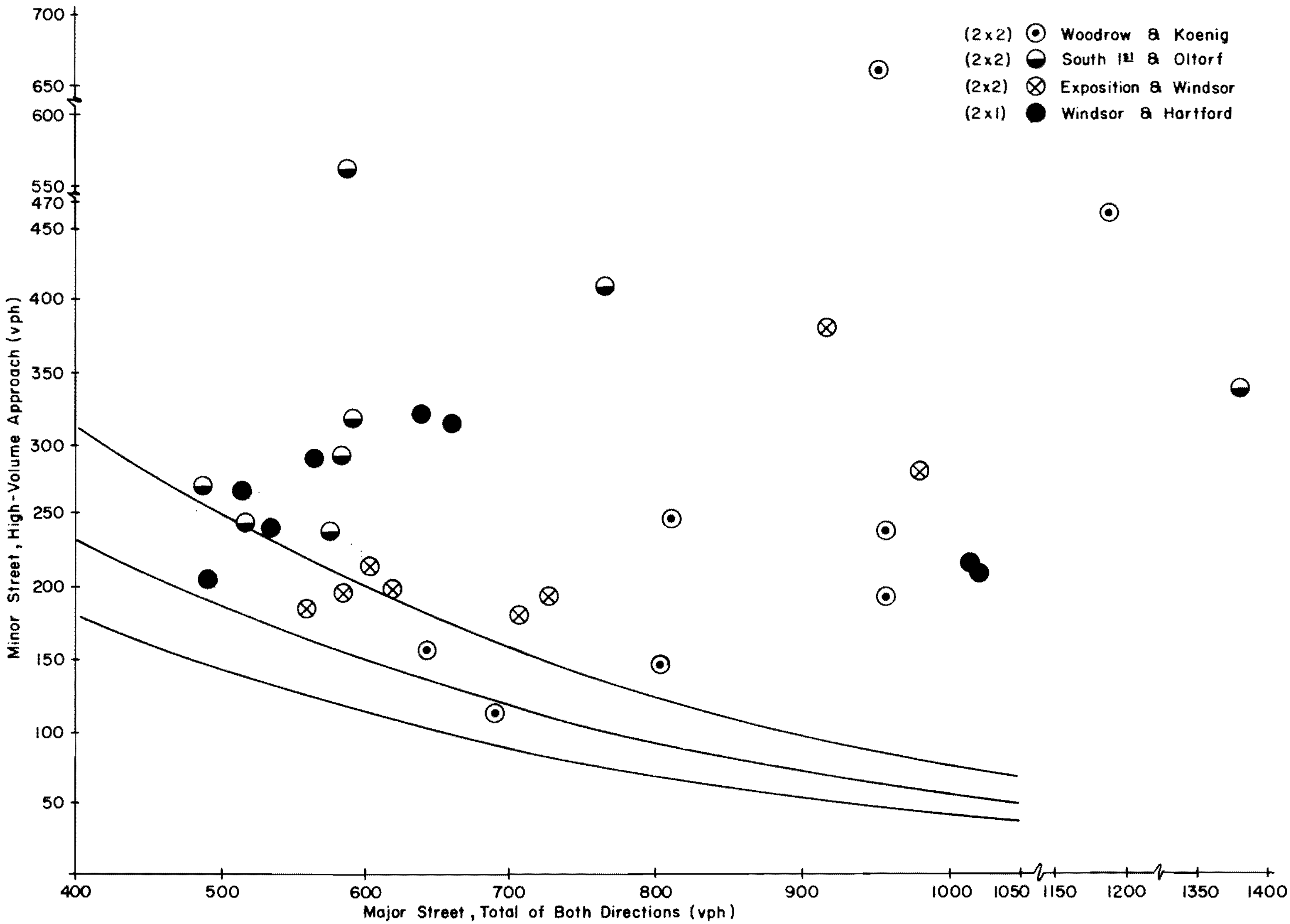

Fig 7.4. Peak-hour volume warrant for traffic-actuated signals, eight highest hours, urban area.

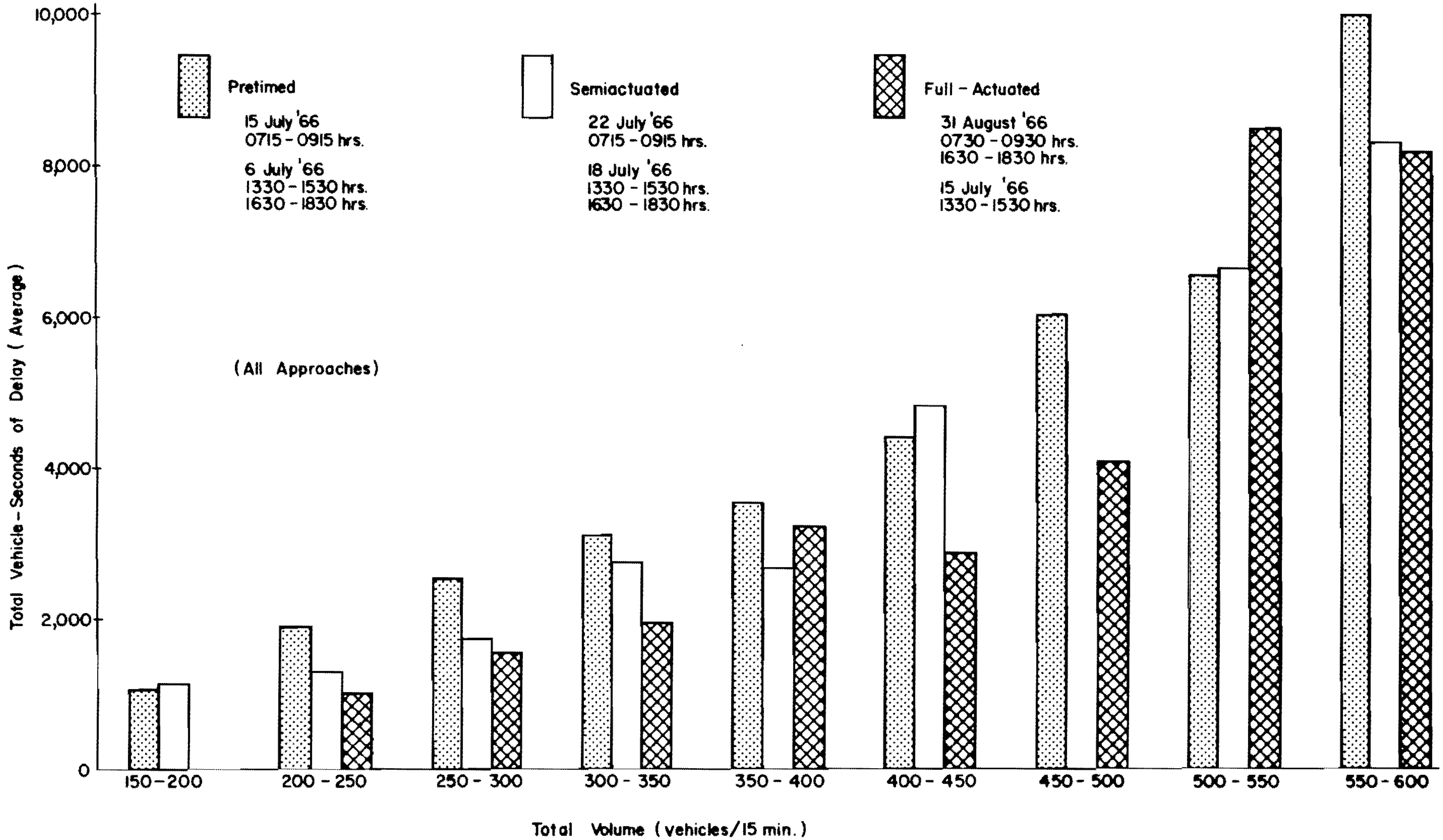

Fig 7.5. Comparison of pretimed, semiactuated, and full-actuated controllers at Woodrow and Koenig.

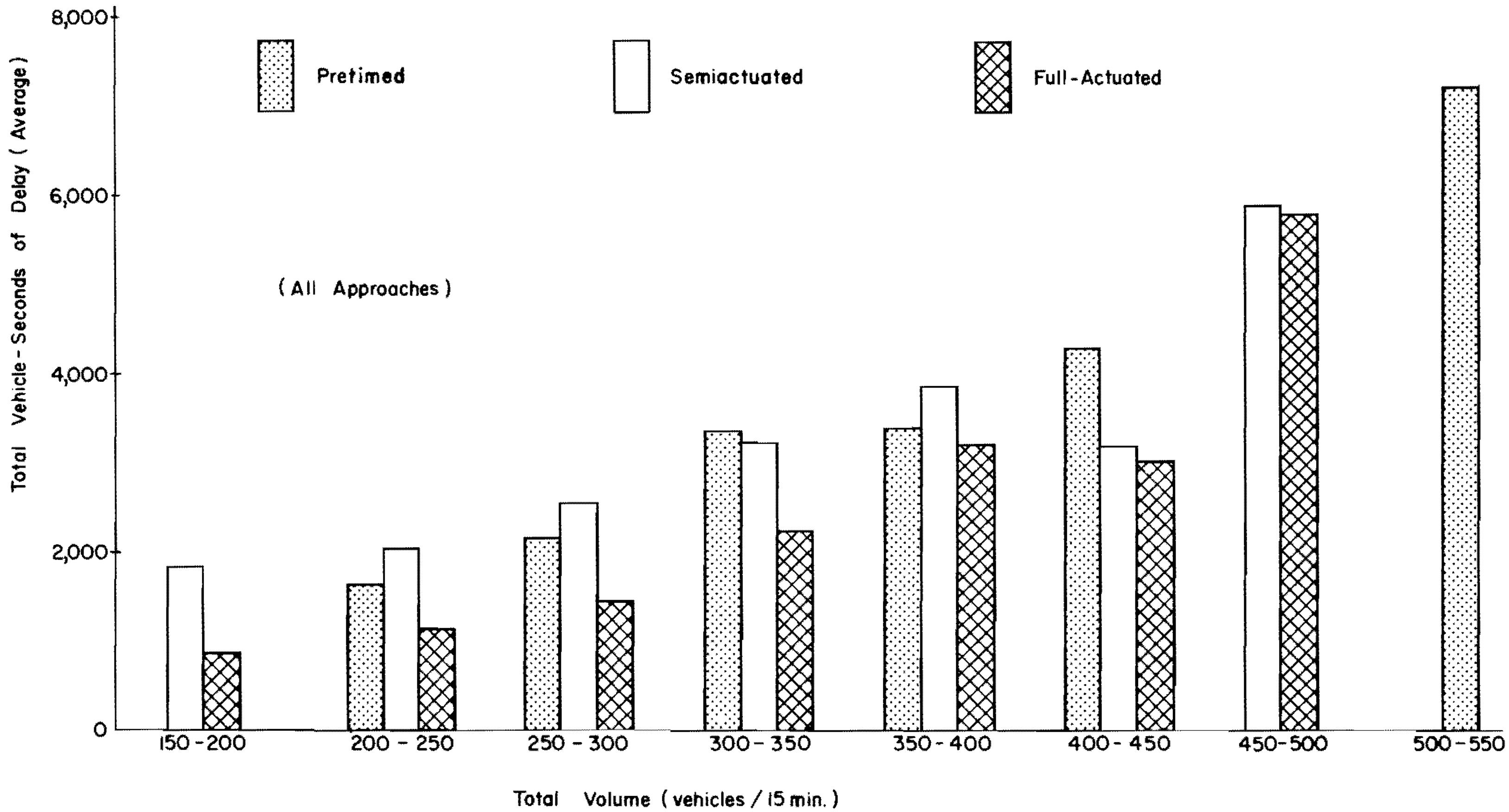

Fig 7.6. Comparison of pretimed, semiactuated, and full-actuated controllers at South First and Oltorf.

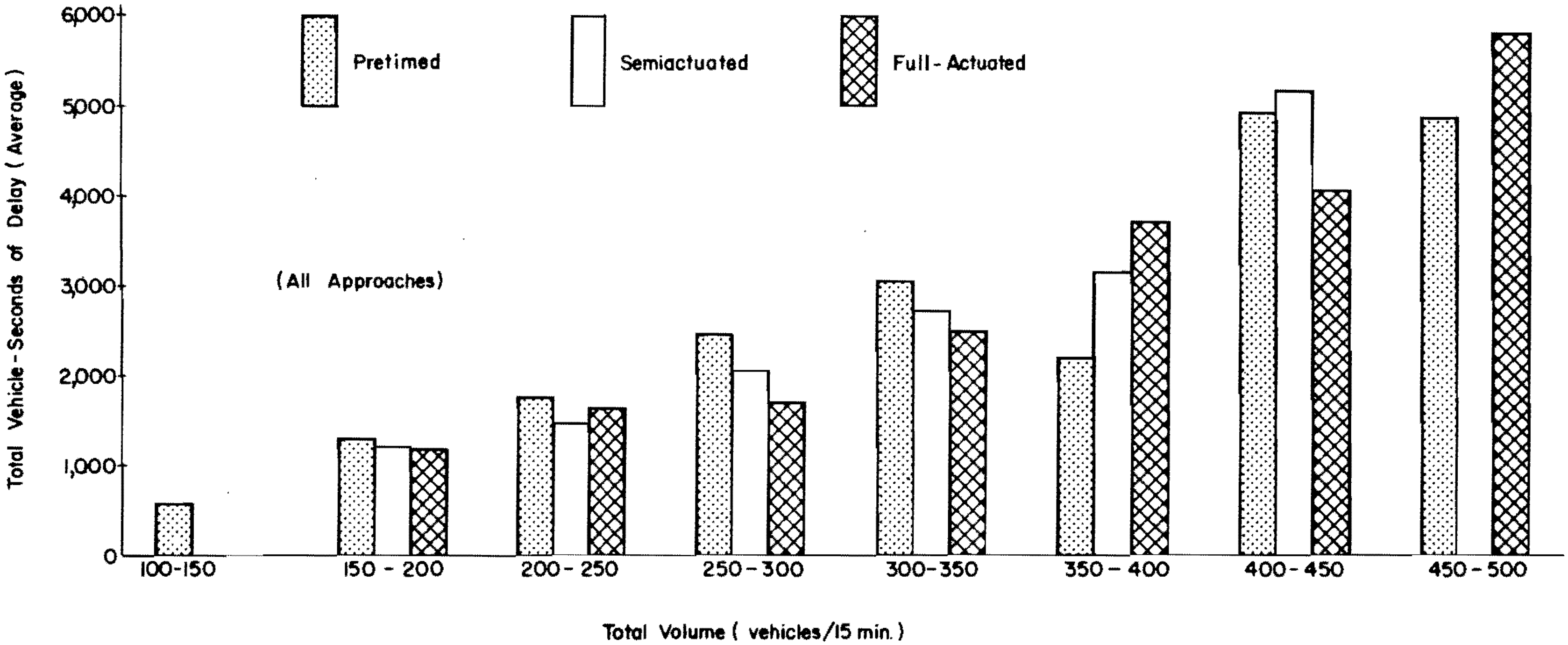

Fig 7.7. Comparison of pretimed, semiactuated, and full-actuated controllers at Exposition and Windsor.

It appears, then, that in terms of vehicular delay, actuated controllers would be the proper choice for those intersections. This substantiates the validity of the proposed warrants and suggests that at least the urban portion for traffic-actuated controllers is sound.

SUMMARY

From the foregoing dicussion of portions of the presently accepted warrants for pretimed signals and the proposed warrants for traffic-actuated signals, two conclusions may be drawn:

(1) When considering only volume-related warrant factors, both the warrants for pretimed and for traffic-actuated signals that have been evaluated herein provide helpful guides in selecting the proper type of traffic signal controller for specific uses.

(2) The D3 Recorder is a valuable tool for evaluating volume-related portions of warrants for the installation of traffic signals.

DELAY WARRANTS

It is probably desirable to have a method whereby, given a set of traffic conditions at an intersection, it would be possible to predict the delay experience likely to result from using each particular type of traffic-signal control device. Then, if a set of delay-based warrants were available, these could be used, in conjunction with other warrants, to determine the most practicable traffic control device to use.

A procedure which is available for developing this method is model building by means of regression analysis. In general, the procedure involves gathering appropriate field data and then finding a mathematical relationship which can predict reasonably well the dependent variable, delay, as a function of a few easily measured independent variables. The data gathered in this research is currently being used in an effort to develop this methodology.

As an example of the model-building technique, consider a fully actuated signal for which 15-minute data summaries are available, such as the Woodrow and Koenig intersection. An equation of the following form could be developed:

$$y = 35.7 - x$$

where

y = the average delay for stopped vehicle, in seconds;

x = the number of complete signal cycles during the 15-minute interval.

However, the number of cycles would not be known until after installation of the signal. Thus, a relationship explaining average delay in terms of another variable was needed. A new model was developed:

$$y = 3.2 + .03125x_1$$

where

y = average delay as defined above,

x_1 = intersection volume per 15-minute interval.

Then, if a model of this type were available for each type of control, a better decision could be made regarding the proper signal installation. This model-building technique can be continued until a suitable model is developed.

Models using up to five or six variables and their interactions to predict delay have been tested against actual observed delays as part of this research study, but no consistently adequate model has yet evolved. Work in this direction is continuing.

A valid model, once available, can be used to investigate a wide variety of intersection conditions and aid in the development of delay-based warrants for various types of signal control.

CHAPTER 8. COMPARATIVE DELAY STUDIES

In addition to its usefulness as a research instrument, the D3 digital delay data recorder has many potential applications for assessing the relative effectiveness of modifications to highway intersections. Quantitative measures of delay can be used as the basis for comparing two or more alternative modifications or improvements to an intersection. Although evaluations of only two types of intersection changes (one a change in geometry and the other a change in traffic control) are described in this chapter, the equipment and the data summary techniques which have been employed have numerous practical applications in before-and-after studies.

GEOMETRIC DESIGN MODIFICATIONS: GULF FREEWAY (IH 45) AND WAYSIDE DRIVE IN HOUSTON

To evaluate the influence of a change in geometric design on intersection efficiency and to test the usefulness of the delay recording equipment for such purposes, two field studies were carried out at a diamond-type interchange located on the Gulf Freeway in Houston, Texas. Data were collected at the Wayside interchange on October 5, 1965, when the two adjacent intersections were operating in the usual diamond-interchange manner. Then, on December 7, 1965, another study was made after new U-turn lanes on the frontage roads had been opened to traffic for about five weeks.

Geometric Features

The geometric features of this interchange during the first study are shown in Fig 8.1. Possibly this should not be considered a typical diamond

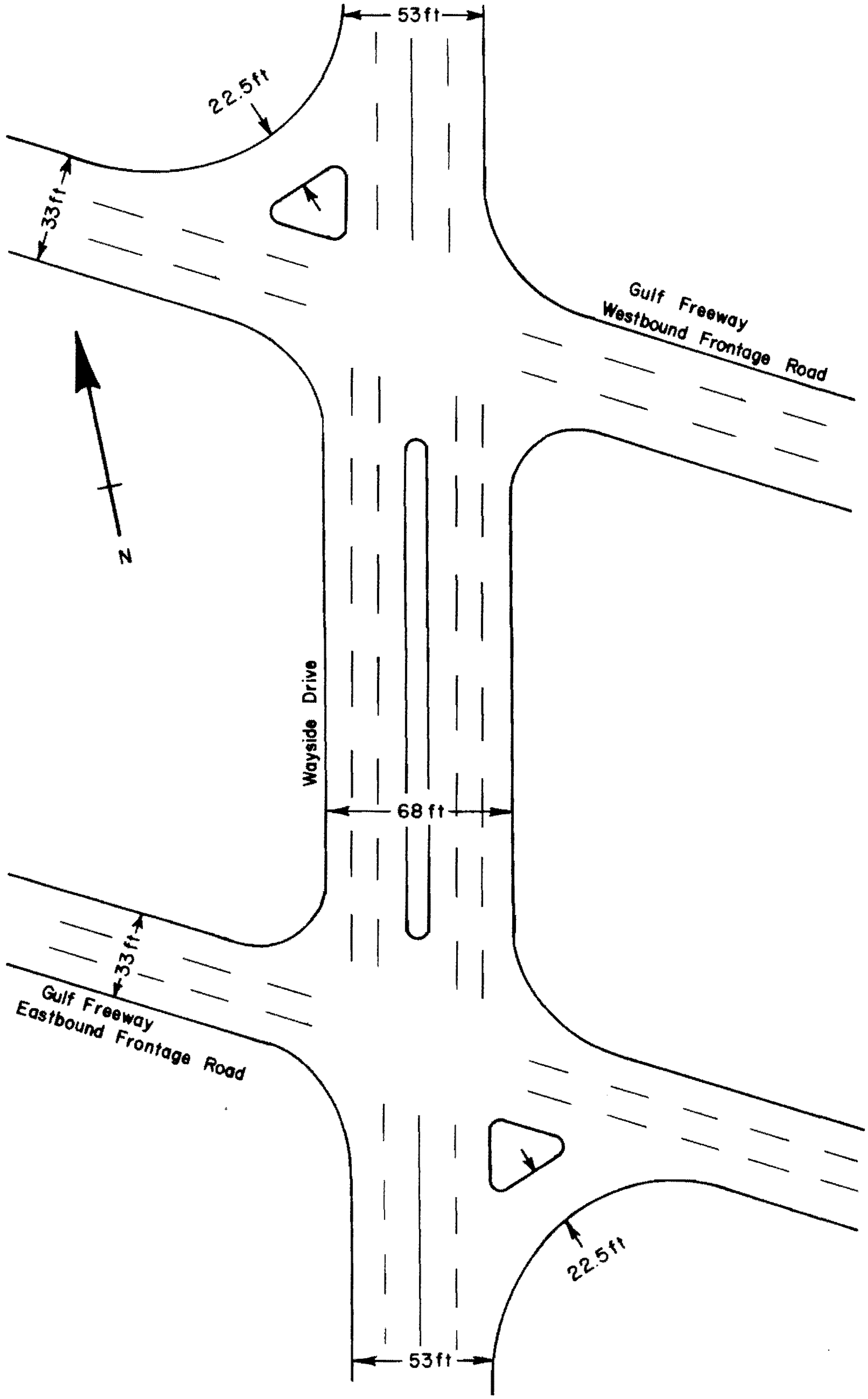

Fig 8.1. Layout of Gulf Freeway frontage roads and Wayside Drive intersection (before changes).

interchange because of the arrangement of the ramps, but the signalization of the two adjacent intersections is conventional. The frontage roads (westbound and eastbound) are three-lane, one-way approaches. Wayside Drive is a two-way street with two-lane approaches.

During the peak traffic flows, Wayside interchange had been loaded beyond capacity. A report by the Texas Transportation Institute published in September 1964 (Ref 20) recommended geometric design modifications for increasing the capacity of this interchange. These modifications were made by the Texas Highway Department and are shown in Fig 8.2. The primary changes involved the addition of U-turn lanes on the frontage roads and provision of a left-turn lane on the westbound frontage road.

Signal Phasing

The signal phasing as recorded by the D3 equipment for the evening peak is shown in Fig 8.3. The cycle length is 70 seconds and the phasing is four phase with two 5-second overlaps. The same signal timing was in operation for both studies.

During the analysis of the data, it was noted that more green time was allowed on the eastbound approach (15 seconds) than on the westbound (12 seconds), despite heavier traffic on the westbound approach. This imbalance is believed to be one cause of the excessive delay experienced on the westbound approach and is discussed later in this chapter.

Traffic Volumes

Although data were recorded continuously during the evening peak period between 1600 and 1800 hours for both studies, a 30-minute period, 1655 to 1725 hours, was selected for comparative analysis. The choice of this period was based on the equivalence of the total traffic volume passing through the intersection on both days. There were 2,008 vehicles on October 5, as compared with

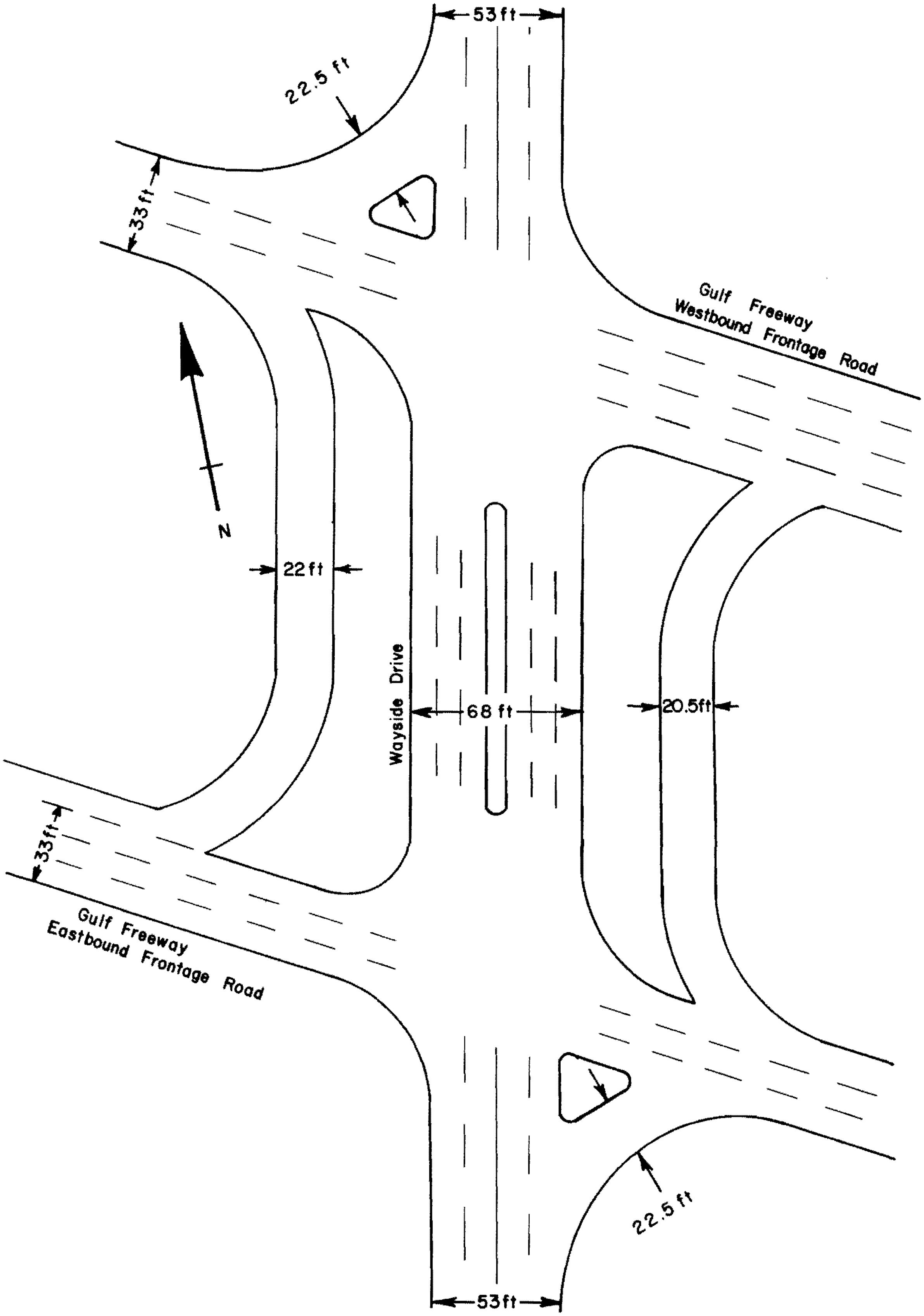

Fig 8.2. Layout of Gulf Freeway frontage roads and Wayside Drive intersection (after changes).

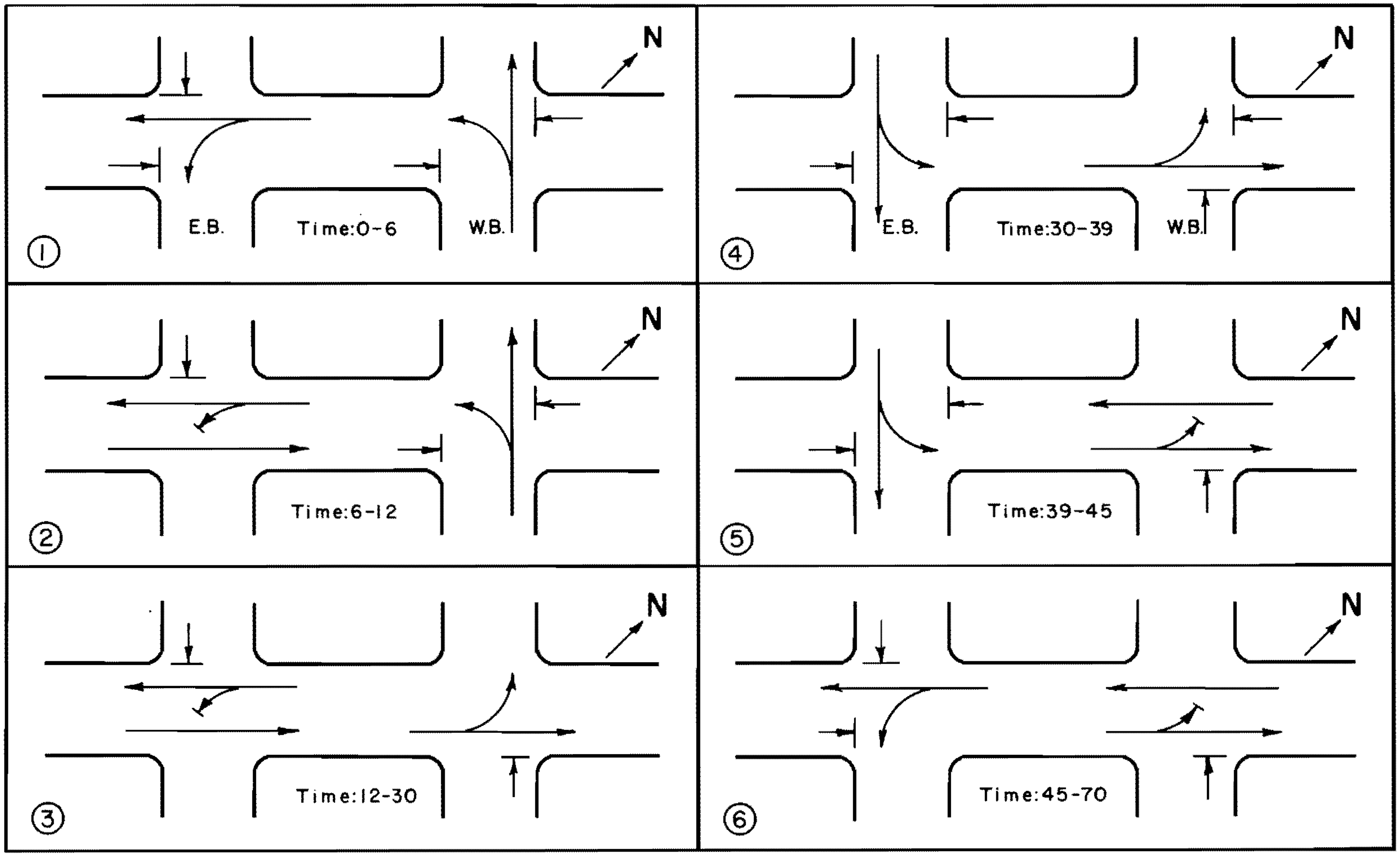

Fig 8.3. Signal phasing as recorded by equipment, 70-second cycle.

2,012 on December 7. Since the total volumes are equal, results of the two studies can be compared.

U-Turn Movements

On October 5, vehicles making a U-turn had to pass through both intersections and some were stored between the intersections for a portion of the signal cycle. U-turn vehicles represented 13 percent of the volume approaching on the westbound frontage road and 9 percent of the total approaching on the eastbound frontage road.

On December 7, 15 percent of the volume approaching from the west made a U-turn, and 7 percent of the vehicles from the east made U-turns. These volumes indicate that the U-turn movements were nearly equal for both studies.

Discussion of Results

Total Vehicle-Seconds of Delay. A comparison of the vehicular delay permits the evaluation of the effect of changes in geometric design. The total vehicle-seconds of delay for all approaches is shown in Fig 8.4. This figure shows that the total delay dropped from 111,000 vehicle-seconds on October 5 to 66,000 vehicle-seconds on December 7. This represents a decrease of 40 percent.

The westbound approach experienced the largest decrease in delay, approximately 73 percent, despite a small increase in the volume of about 11 percent. The delay on the middle lane of this approach was reduced from 21,600 vehicle-seconds to 1,600 vehicle-seconds.

The vehicle delay on the eastbound approach decreased 31 percent; however, the volume decreased 20 percent. Assuming that the delay is proportional to the volume, this represents a 10 percent reduction in delay.

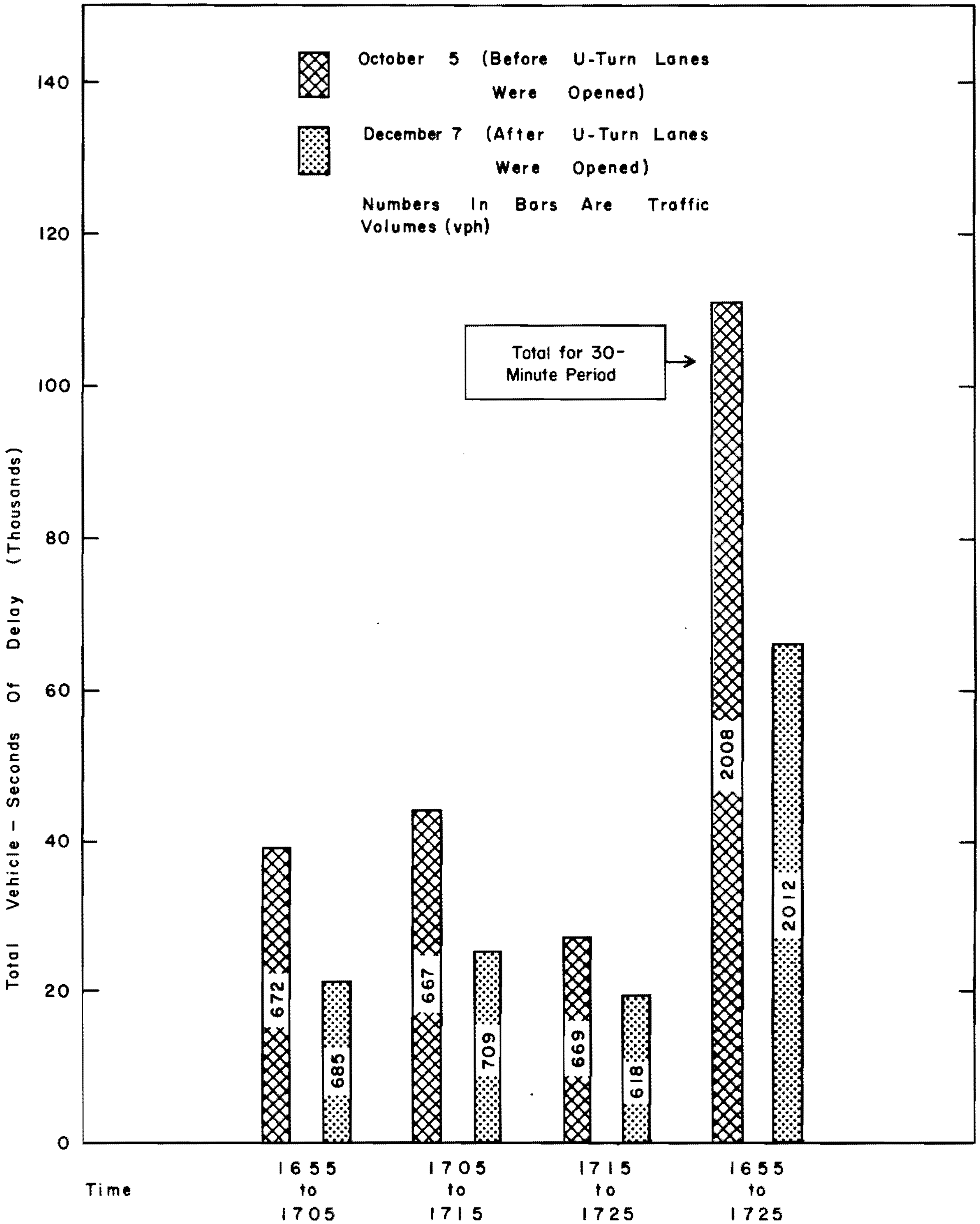

Fig 8.4. Total vehicle-seconds of delay, all approaches, Gulf Freeway and Wayside Drive.

The southbound and northbound approaches, even though only indirectly affected by the U-turn lanes, also showed a decrease in delay. On the southbound approach, a drop of 25 percent in vehicle delay was experienced, despite an increase of 11 percent in the volume. The northbound approach showed an 11 percent decrease in volume and a 13 percent decrease in delay.

Percentage of Vehicles Stopped. During the evening peak this interchange is often overloaded, requiring some of the vehicles to wait for more than one cycle before proceeding through the intersection and some to stop more than once. Figure 8.5 shows a comparison of the percentage of vehicles stopped for the two study periods. Before the U-turn lanes were opened, all vehicles were forced to stop at least once before clearing the intersection. After the lanes were opened, only 83 percent of the vehicles were forced to stop.

The westbound approach was affected most by the change in design. On October 5, all vehicles were required to stop, whereas only 75 percent had to stop on December 7. While the other three approaches experienced a reduction in the percentage of vehicles stopped, it was not as noticeable.

Influence of Signal Timing. The signal phasing and timing were the same for both studies. While this has no influence on the comparative values, the effect of the timing is clearly shown for each study individually.

It was shown earlier in this chapter that an inbalance existed between the green time allowed on the westbound and eastbound frontage roads as compared with the traffic volume on each approach. This imbalance is clearly reflected in the relative values of vehicle-seconds of delay. Four times more delay was recorded on the westbound (39,750 vehicle-seconds) than one the eastbound (10,240 vehicle-seconds) during the 30-minute interval studied, while the westbound approach had 25 percent less traffic than the eastbound. The main

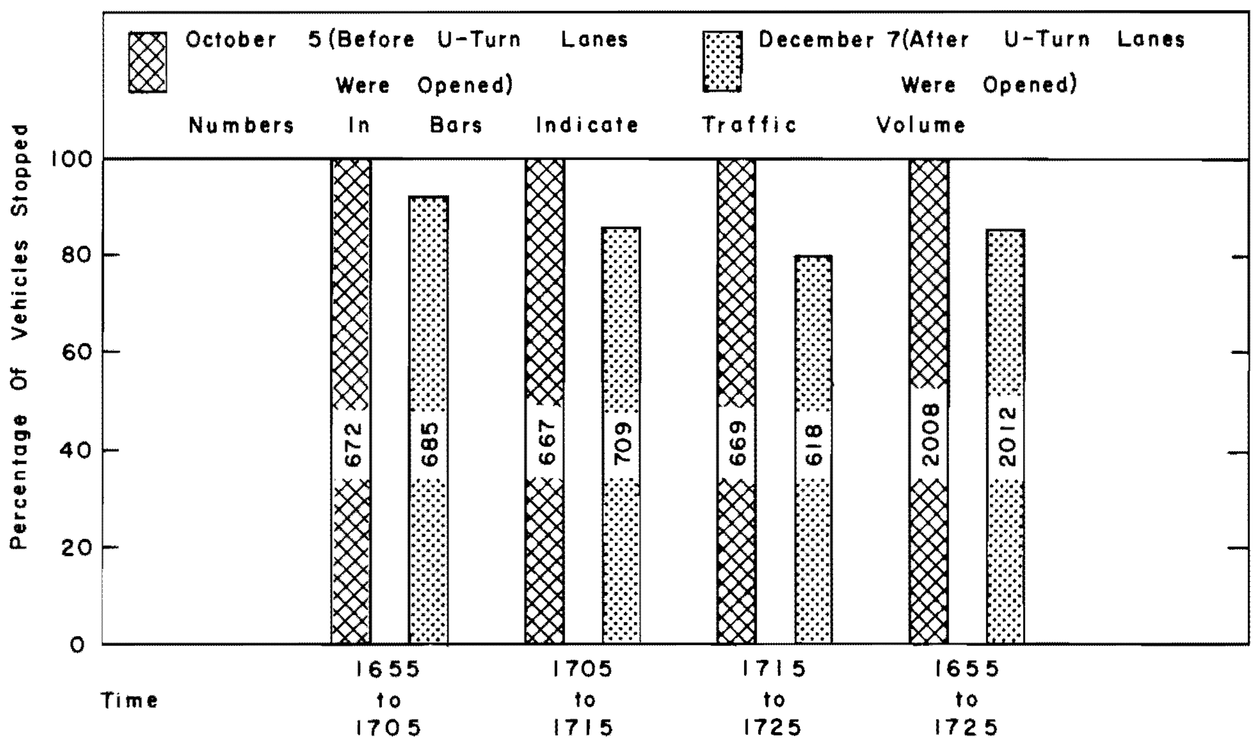

Fig 8.5. Percentage of vehicles stopped, all approaches, Gulf Freeway and Wayside Drive.

reason for this imbalance in delay appears to be the imbalance in the allotted green time.

TRAFFIC CONTROL MODIFICATIONS: 19TH AND CHICON STREETS IN AUSTIN

For the purpose of evaluating an experimental change from four-way to two-way stop-sign control at the intersection of 19th and Chicon Streets in Austin, Texas, two field studies were conducted at that intersection during the summer of 1967. Volume counts indicated that about 75 percent of the traffic was on 19th Street and 25 percent on Chicon. The first study, in June, was made with the intersection operating under its normal four-way stop-sign control; then later, in August, another study was conducted with stop signs located only on Chicon Street. Advance warning signs were installed when the two-way stop experiment was begun in order to alert repeat drivers to the modification. The intersection operated under two-way stop-sign control for ten days before the second study was conducted.

Geometric Features

During both studies, geometrics at the intersection remained essentially unchanged. As shown in Fig A.1 (Appendix A), 19th Street had four-lane approaches, and the approaches on Chicon Street were two-lane. Parking was permitted on both streets, but during the studies the approaches operated as four and two lanes, respectively, and no vehicles were parked near the intersection. Pedestrian traffic was light, and sight distance was restricted slightly by a building with a covered sidewalk on the southwest corner of the intersection (see Fig A.1, Appendix A).

Traffic Volumes

Data were recorded for morning and evening peaks as well as for afternoon periods. For purposes of this comparison, however, segments of the morning

TABLE 8.6. VOLUME AND DATA OF STUDIES AT 19TH AND CHICON

Time	Control	
	Two-Way Stop	Four-Way Stop
0745-0845	1,014 (August 10, 1967)	1,033 (June 28, 1967)
1630-1730	1,250 (August 10, 1967)	1,241 (June 24, 1967)

TABLE 8.7. TOTAL VEHICULAR DELAY FOR STUDIES OF 19TH AND CHICON (SECONDS)

Time	Control	
	Two-Way Stop	Four-Way Stop
0745-0845	5,055	12,518
1630-1730	13,861	15,663

and evening peak studies were chosen. The particular time segments or test periods were chosen on the basis of the equivalency of the total traffic volume passing through the intersection. The segments of the morning peak studies for four-way and two-way stops that were chosen had a 0.6 percent difference in volume, and the evening studies showed a 1.8 percent total volume differential. The dates, times, and volumes are shown in Table 8.6. For all study periods, total traffic volume remained essentially distributed in a ratio of 75 percent on 19th Street to 25 percent on Chicon.

Discussion of Results

When total seconds of vehicular delay experienced by all vehicles passing through the intersection are compared, it is found that during the morning peak test periods (0745 to 0845 hours) two-way stop-sign control resulted in 40 percent less delay than four-way stop signs. Data obtained during evening peak periods showed a similar trend, although less pronounced, with a 10 percent reduction in total delay for the two-way stop. Table 8.7 shows the results of the delay study.

Although two-way stop-sign control reduced total delay substantially during both morning and evening peak periods, certain problems were associated with this type of control. These problems included large increases in average delay per vehicle stopped and hazardous actions by repeat drivers, who were accustomed to four-way stop control at this intersection.

In the morning peak studies, average delay per stopped vehicle increased 100 percent, while evening peaks showed a 400 percent increase. This meant average delays to all vehicles on the minor street (Chicon) of 24.4 seconds for morning peak periods and 52.7 seconds for evening peaks. These values can be compared to an average of 12.5 seconds for both morning and evening peaks under four-way control.

Even though no accidents were reported at the intersection during the two-week experimental period, a number of near misses were observed when repeat drivers on Chicon Street moved boldly into the intersection after stopping, obviously expecting traffic on 19th Street to stop. Special advance warning signs were not completely effective in this period of time.

After reviewing the operational characterisitics of the intersection under experimental two-way stop-sign control, the City of Austin restored four-way stop control and subsequently, in May, installed a traffic signal. Quantitative measures of total delay and average delay per stopped vehicles provided part of the data needed for comparing four-way and two-way control at this intersection.

CONCLUSIONS

The following conclusions may be drawn from the two examples of before-and-after studies in which the D3 delay recording equipment was utilized:

(1) The digital delay data recorder is a useful tool for quantitative evaluation of the effects of changes in geometric design features and in traffic control technique.

(2) Opening of new U-turn lanes at the Wayside interchange on the Gulf Freeway in Houston reduced the percentage of vehicles required to stop at the intersections by 17 percent.

(3) Installation of the U-turn lanes resulted in a 40 percent decrease in total vehicle-seconds of delay during the evening peak-traffic flow period.

(4) Although the two-way stop-sign control reduced total vehicular delay at 19th and Chicon Streets in Austin by 40 percent, as compared with four-way stop control, excessive minor-street delay and potential

accident considerations indicated that two-way stop-sign control was not the best choice for this particular intersection.

CHAPTER 9. SPECIAL STUDIES

The traffic observation and recording techniques described in previous chapters can also be used for other types of studies. Two ancillary investigations are described briefly to illustrate the usefulness of quantitative information in evaluating the effectiveness of traffic control measures. One investigation deals with assessing the effects of minor adjustments to dial settings of actuated controllers and the other with developing factual information concerning traffic arrival patterns.

THE EFFECT OF DIAL SETTINGS OF FULL-ACTUATED SIGNAL CONTROLLERS

If full-actuated control is selected for use at an intersection, it is helpful to the traffic engineer to know the potential influence which dial settings will have on traffic delays. The Manual on Uniform Traffic Control Devices recommends the range of timing adjustments shown in Table 9.1 for normal operating conditions of a full-actuated controller.

In 1967, nine studies were conducted at South First and Oltorf and at Woodrow and Koenig to determine the relative effect of selected controller dial settings on delay at these intersections. Tables 9.2 and 9.3 summarize the conditions included in the studies.

Woodrow and Koenig is a four-leg (two approach lanes per leg) intersection located in northwest Austin. A two-phase, full-actuated signal controls traffic. The intersection is essentially a right-angle crossing, and traffic is not noticeably influenced by other signals in the area. Pedestrian and heavy truck traffic are light. The traffic split at Woodrow and

TABLE 9.1. RANGE OF TIMING ADJUSTMENTS FOR FULL-ACTUATED CONTROL*

Period	Range of Timing Adjustments, seconds
Initial intervals	2-30
Vehicle intervals	2-30
Maximum intervals	10-60
Clearance intervals	Up to 10
Recall switches	On-off

* Manual on Uniform Traffic Control Devices for Streets and Highways, U.S. Bureau of Public Roads, Washington, D.C., June 1961, p 209.

TABLE 9.2. SUMMARY OF DIAL SETTING STUDIES OF FULL-ACTUATED CONTROL AT SOUTH FIRST AND OLTORF

Date	Times	Dial Settings, seconds								Comments
		Initial Interval		Vehicle Interval		Clearance Interval		Maximum Interval		
		N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	
July 26, 1967	0700-0900	8.5	8.5	5.0	6.0	2.5	2.5	60.0	60.0	Normal settings
	1315-1510									
	1600-1800									
August 1, 1967	0700-0900	8.5	8.5	4.5	4.5	2.5	2.5	60.0	60.0	Short vehicle interval
	1315-1515									
	1600-1800									

TABLE 9.3. SUMMARY OF DIAL SETTING STUDIES OF FULL-ACTUATED CONTROL AT WOODROW AND KOENIG

		Dial Settings, seconds								
		Initial Interval		Vehicle Interval		Clearance Interval		Maximum Interval		
Date	Times	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	Comments
July 19, 1967	0730-0930	8.0	6.0	5.0	5.5	3.5	3.5	60.0	60.0	Normal settings
July 21, 1967	0715-0915 1300-1500 1600-1800	8.0	6.0	5.0	5.5	3.5	3.5	60.0	60.0	Normal settings
July 25, 1967	0700-0900 1300-1500 1600-1730	8.0	6.0	3.0	5.5	3.5	3.5	60.0	60.0	Short vehicle interval
August 9, 1967	0715-0915	4.0	4.0	5.0	5.5	3.5	3.5	60.0	60.0	Short initial interval

Koenig is almost even; Woodrow, the minor street, rarely handles less than 40 percent of the total traffic for a given time period. Woodrow and Koenig is similar to South First and Oltorf in all factors (physical conditions, geometry, traffic volumes and movements, etc.) that might have a significant bearing on vehicular delay. Figure A.7 (Appendix A) shows the layout of Woodrow and Koenig and Fig A.8 shows South First and Oltorf.

Delay data were collected at both intersections in July and August of 1967. The intersections were studied in the field with the full-actuated signal controllers operating first under normal dial settings, shown in Tables 9.2 and 9.3; the initial interval and the vehicle interval were then shortened to the the levels shown in the tables, and studies were made during afternoon off-peak hours and morning and evening peak hours.

Figures 9.1 and 9.2 show the results of the dial setting studies. Figure 9.1 shows the average delay per vehicle on all approaches and the vehicular volume relationships for normal dial settings and for short vehicle intervals at South First and Oltorf.

Observers at South First and Oltorf noted that a maximum of four vehicles per lane could be stored between the detectors and the stop lines. Vehicle departure rate studies have indicated that four vehicles stored between the detector and the intersection can normally clear the detector for actuation by a fifth vehicle in less than about 8 seconds; therefore, the initial interval should be at least 8 seconds. The associated minimum green time (initial interval plus vehicle interval) on South First and Oltorf should be at least 11 seconds. Normal dial settings allowed 13.5 seconds on South First and 14.5 seconds on Oltorf.

In the August 1 study, the vehicle interval was shortened to 4.5 seconds on each street. Minimum green time then became 13 seconds on both streets,

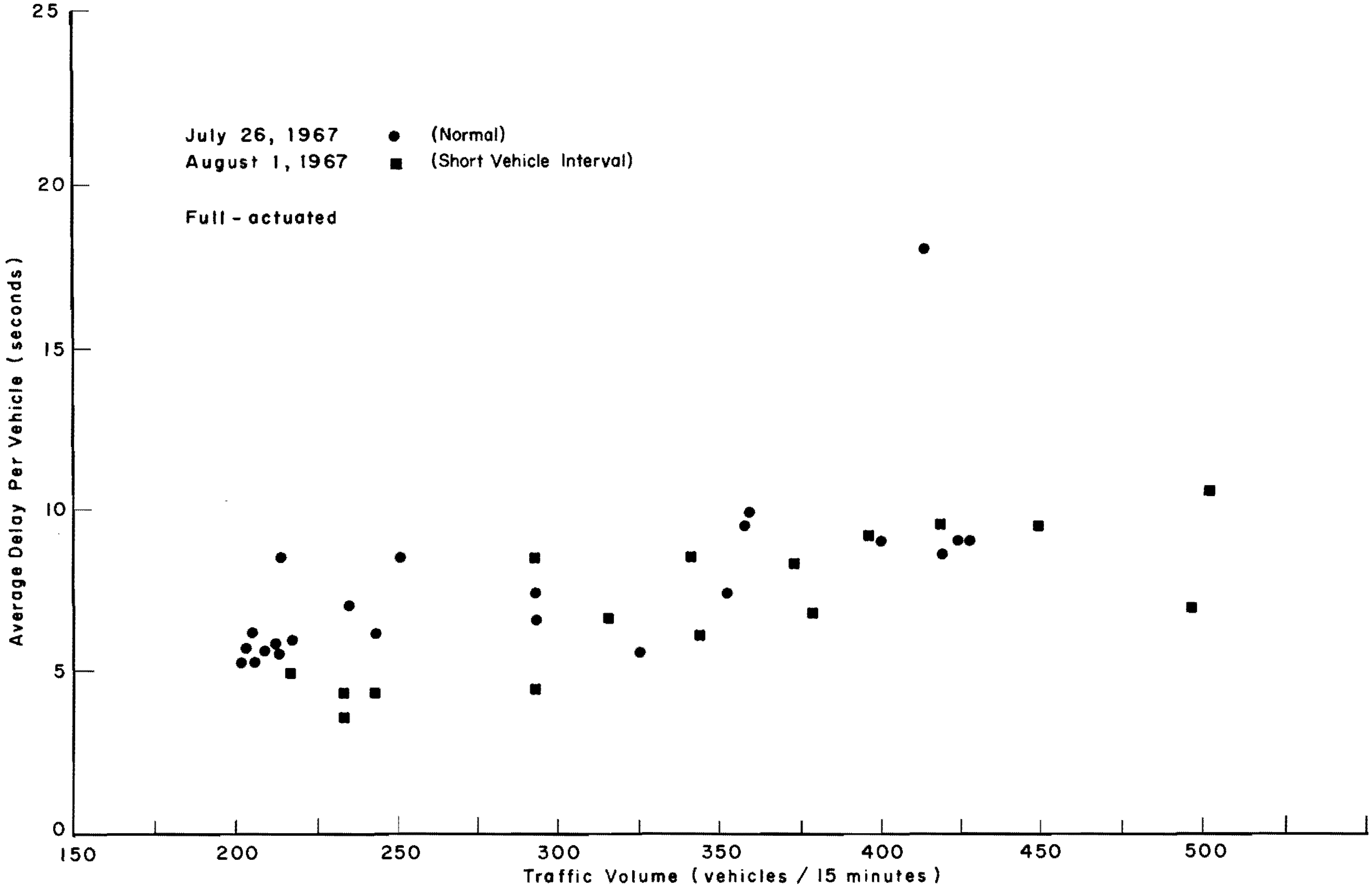

Fig 9.1. Average delay per vehicle, all approaches, South First and Oltorf.

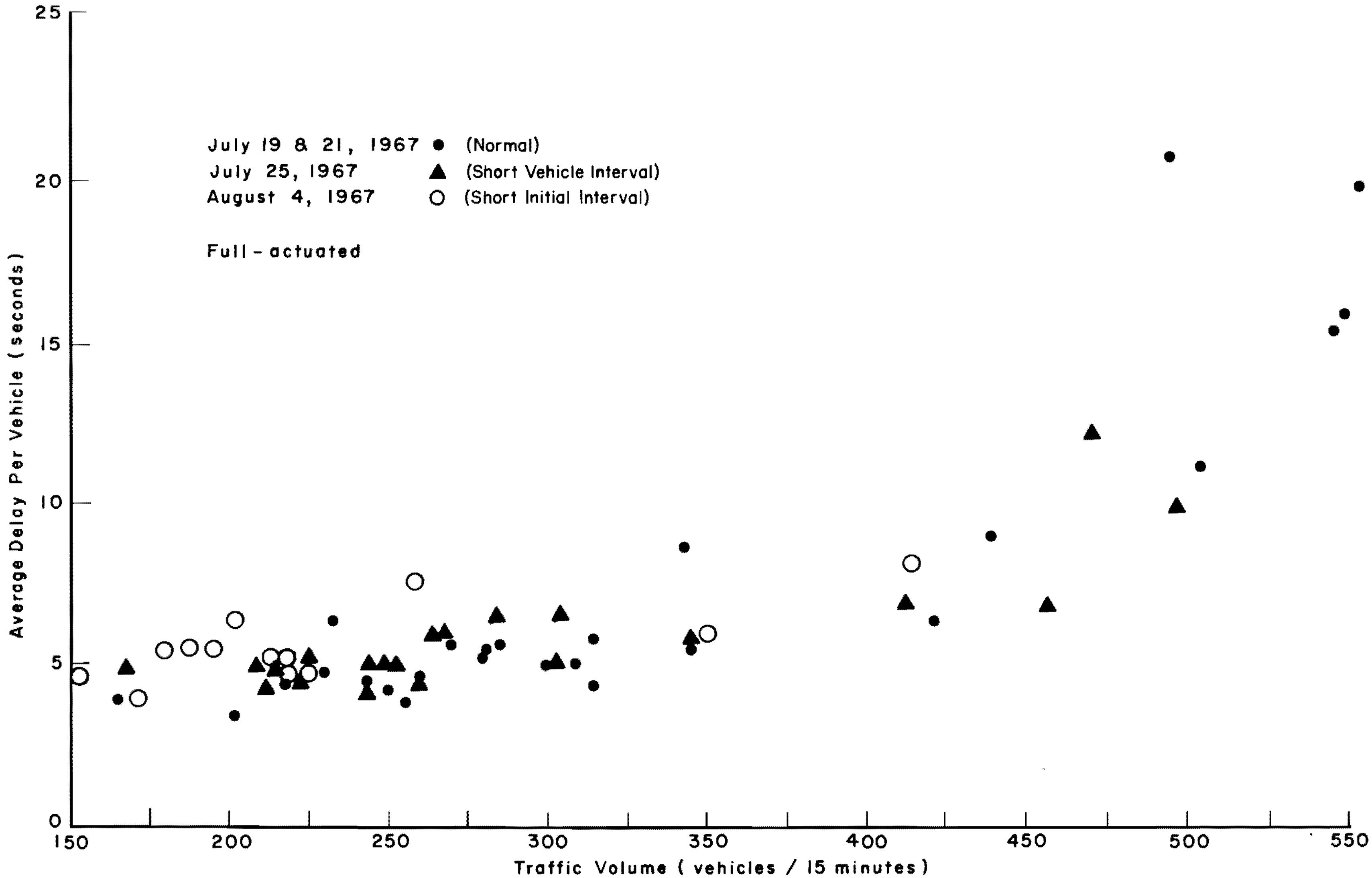

Fig 9.2. Average delay per vehicle, all approaches, Woodrow and Koenig.

still greater than the nominal minimum of 11 seconds. A bold adjustment was not deemed desirable until the sensitivity of the adjustment could be evaluated.

During periods of light traffic, the changed dial settings produced slightly less delay than the normal dial settings as shown in Fig 9.1. At volumes of less than 300 vehicles per 15 minutes, the shortened vehicle interval produced less average delay per vehicle than the normal settings. At higher volumes, there is little apparent effect of the shorter interval on average delay. Delays to the first vehicles in queues were reduced; average phase lengths also became shorter than under normal conditions. During periods of light volume, traffic was rarely queued beyond the detectors; the minimum green time, being less with the shorter vehicle interval, was adequate to clear the stored vehicles and in less time than was allotted under normal dial settings, thereby conserving what may have been wasted green time.

At Woodrow and Koenig, the detectors are about 130 feet from the stop lines. An average minimum of 12 seconds is required for the fifth vehicle to enter the intersection, according to past studies. Normal dial settings allotted a minimum green time of 13 seconds for Woodrow and 11.5 seconds for Koenig.

Average delay per vehicle increased considerably at total traffic volumes over 400 vehicles per 15 minutes, both when the normal dial settings were employed and when the shortened vehicle interval on Woodrow was introduced (see Fig 9.2). Shortening the vehicle interval from 5 to 3 seconds did not affect the average delay per vehicle for the volumes observed.

Reducing the initial interval to approximately half the time normally required for the detector placement pattern at Woodrow and Koenig had no pronounced effect on average delay per vehicle as shown in Fig 9.2. Similar effects are indicated by total volume versus total delay relationships.

Within the range of dial settings for the actuated controllers studied, there were no dramatic effects on vehicular delay. Settings were not varied over extreme ranges, since there are obvious practical limits to such settings for given intersection and detector placement situations. Quantitative measures of delay, however, provided a basis for comparing the subtle variations in performance resulting from changes within practical limits. It may be concluded from these limited observations that for the conditions studied dial settings of the actuated controllers are not extremely critical when kept within normally recognized ranges.

FIELD DATA FOR VALIDATING SIMULATION MODELS

Computer simulation of traffic flow at intersections is potentially a very powerful tool of traffic engineering. In field studies, the range over which controllable parameters can be varied is limited by practical, economic, and safety considerations, and traffic flow patterns must be accepted as they exist. But with simulation, no such restrictions exist. Parameters can be selected and varied at will, and traffic can be generated in many varied patterns. Real time can be compressed greatly.

Even though a considerable amount of work has been done on the development of traffic simulation models, the state of the art is still rather primitive. The primary restriction on significant advances in simulation today probably results from the almost complete lack of sufficient field data with which to validate the computer models. Before a model can be accepted for practical use in traffic studies, the validity of the assumptions and the relationships upon which it is built must be proven by comparing the results of simulation with the observed real-world phenomena which are being simulated.

Data collected in the traffic studies described in this report include several types of information needed for verifying simulation models. Some of the recorded or computed information is directly applicable; other relationships can be deduced. Arrival rates of vehicles are an example of this.

Assuming that vehicles decelerate at the same rate, headways computed by using the difference between the times that successive vehicles stop on an intersection approach can be expected to relate favorably to headways observed at an arbitrary distance away from an intersection in a field traffic survey. In the studies described previously, observers, using counting modules, recorded the time each vehicle stopped on each approach at an intersection; these data were scanned and recorded on punched paper tape every 1.44 seconds. Vehicle stoppage headways on a selected approach were then computed for all times when the signal indication was red merely by finding the time gap between the stoppage times of successive vehicles.

Figures 9.3 through 9.7 show some of the results of vehicle arrival-pattern studies at the South First and Oltorf intersection. Figure 9.3 shows the number of vehicles arriving at various vehicle stoppage headways for the July 26, 1967, off-peak traffic study on the northbound approach. During this study, full-actuated traffic control was being utilized.

Approximately 36 percent of the vehicle stoppage headways calculated were 2.88 seconds or less, and approximately 26 percent were 1.44 seconds or less. This relationship indicates the possibility of a random arrival pattern.

Figure 9.4 shows the number of vehicles arriving at various vehicle stoppage headways for the northbound approach at the South First and Oltorf intersection for the morning peak traffic study on July 26, 1967. The traffic controller used at the intersection during the study was the full-actuated type. Approximately 59 percent of the vehicle stoppage headways were 2.88

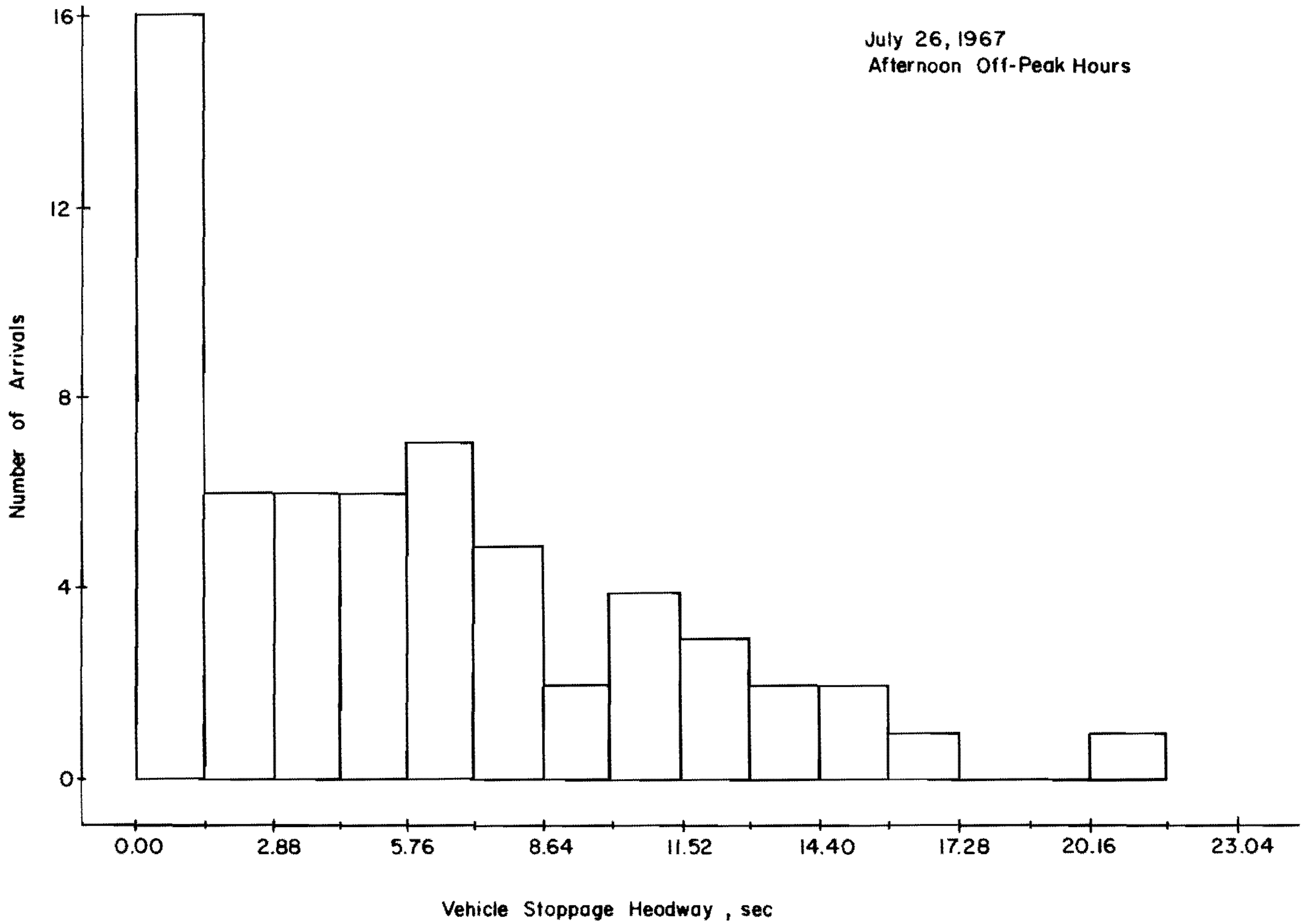

Fig 9.3. Arrival patterns, northbound approach, South First and Oltorf.

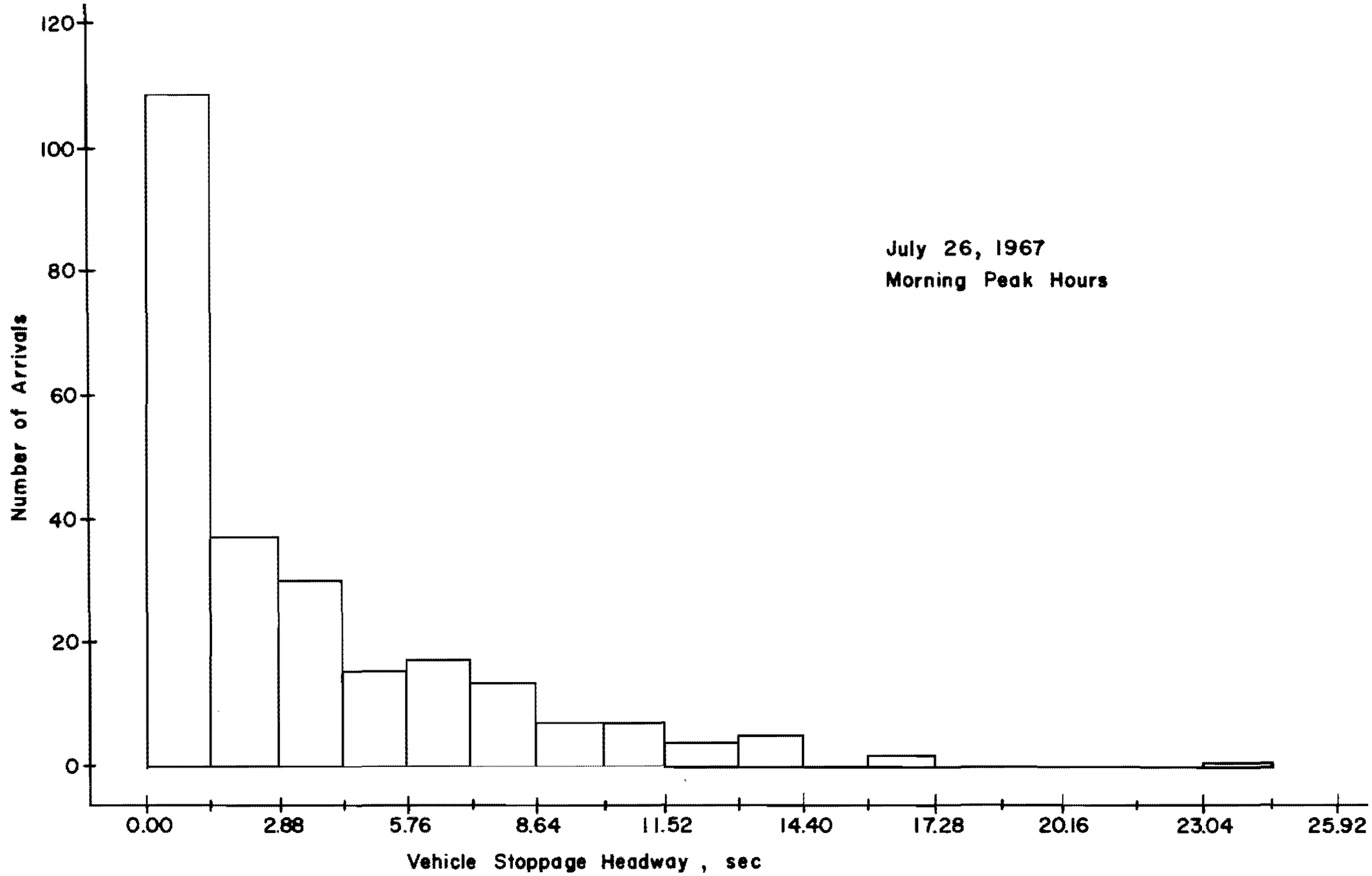

Fig 9.4. Arrival patterns, northbound approach, South First and Oltorf.

seconds or less and approximately 44 percent of the vehicle stoppage headways were 1.44 seconds or less. The traffic volumes were higher during the morning peak traffic study than during the afternoon off-peak traffic study; moreover, the percentage of vehicles having vehicle stoppage headways of 2.88 or 1.44 seconds or less is higher during the morning traffic study than those in the afternoon traffic study.

It may then be concluded from the relationships shown in Figs 9.3 and 9.4 that as approach volumes increase, the average vehicle stoppage headway decreases, or the percentage of vehicles with restricted headways increases as approach volume increases. Thus, the distribution of headways becomes less nearly random.

Similar vehicle stoppage headway calculations have been made for traffic behavior on the northbound approach at the South First and Oltorf intersection for pretimed traffic control. Figure 9.5 shows the relationship of the number of arrivals versus the vehicle stoppage headway for the northbound approach at the South First and Oltorf intersection when traffic was controlled by the pretimed method. Approximately 58 percent of the vehicle stoppage headways are 2.88 seconds or less, and approximately 42 percent of the headways are 1.44 seconds or less. Since these percentages compare favorably with those obtained when traffic was controlled by a full-actuated type signal, the distribution of vehicle stoppage headways seems to be dependent on approach volumes and independent of traffic controller type.

Studies have also been made to determine the relationship between the time elapsed after the red indication has been displayed and the frequency of single vehicle stoppages. Figure 9.6 shows the number of vehicle arrivals which occurred in the various time intervals after the beginning of the red when pretimed control was in use. Figure 9.7 shows comparable data for

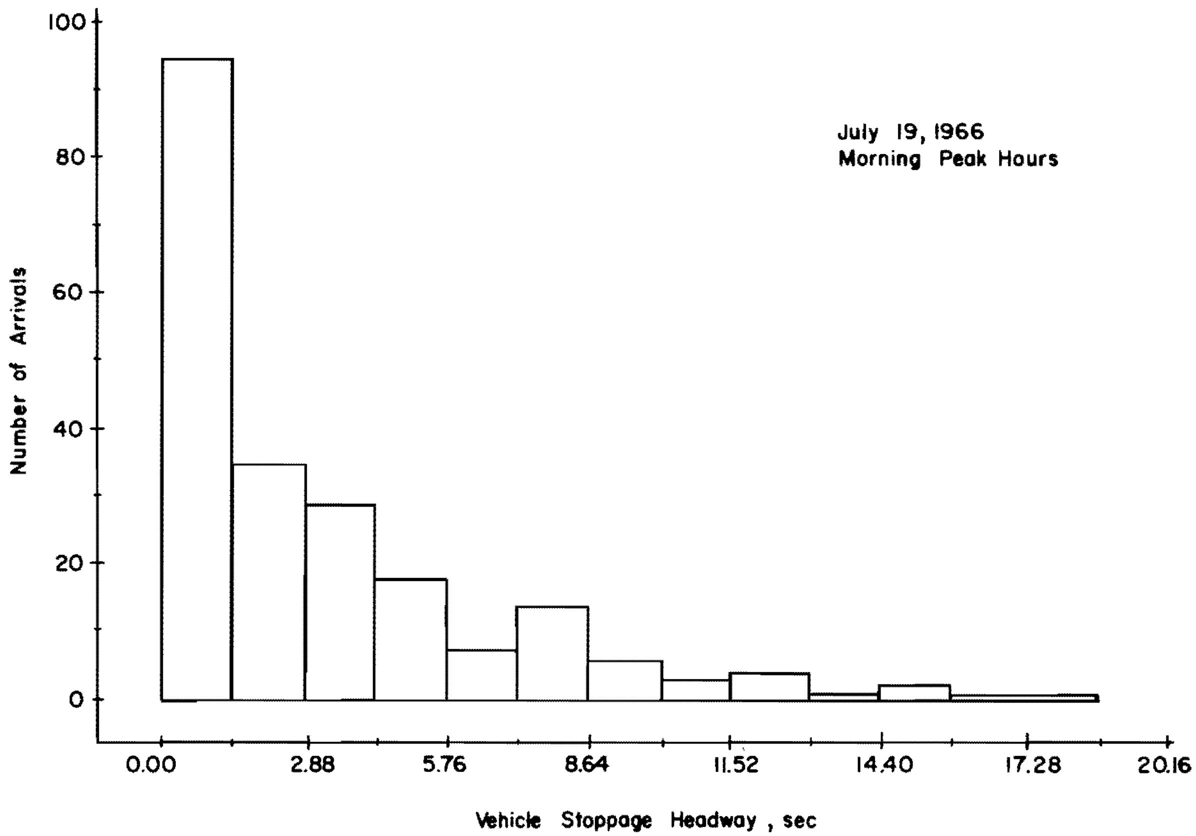

Fig 9.5. Arrival patterns, northbound approach, South First and Oltorf.

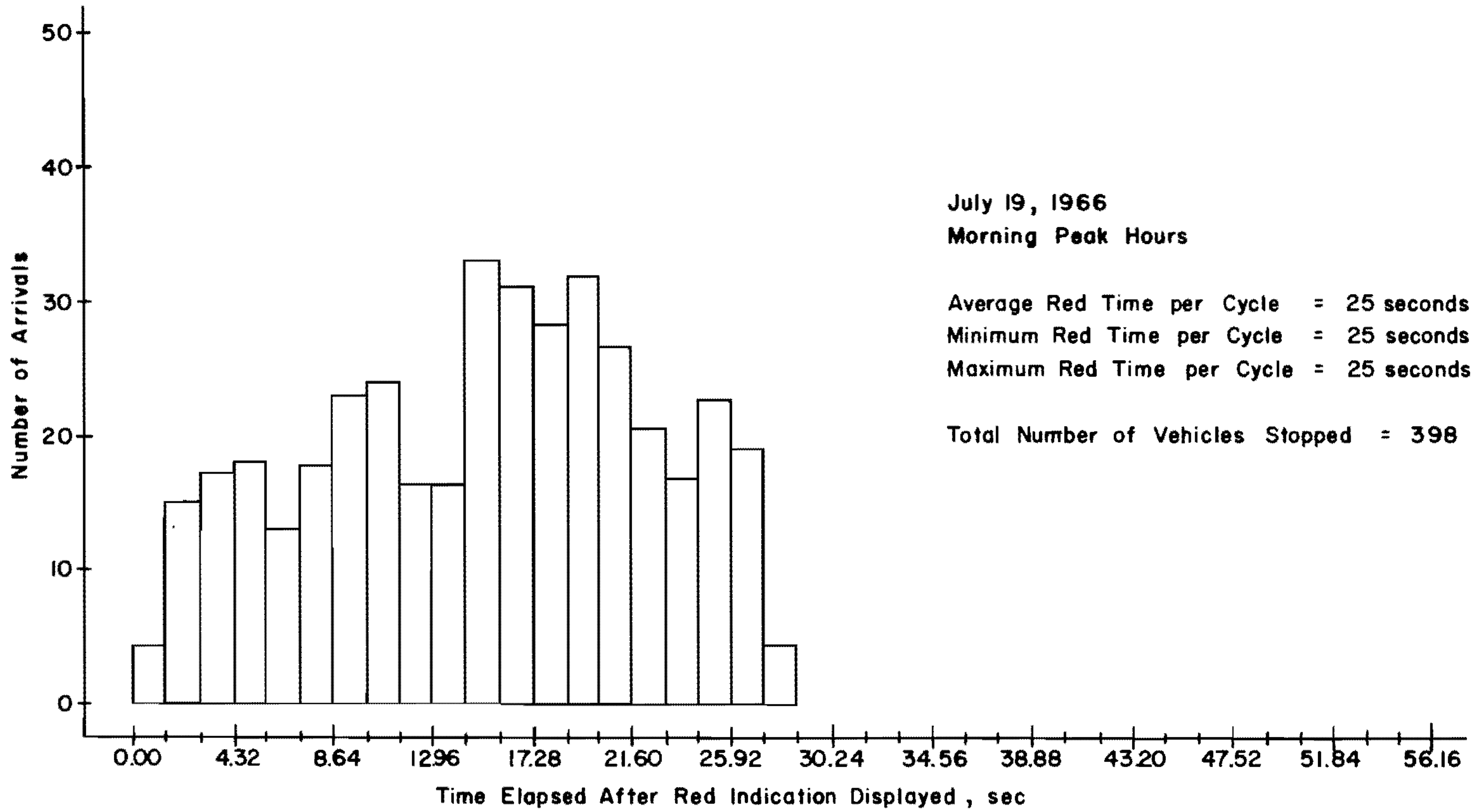

Fig 9.6. Arrival patterns, northbound approach, South First and Oltorf.

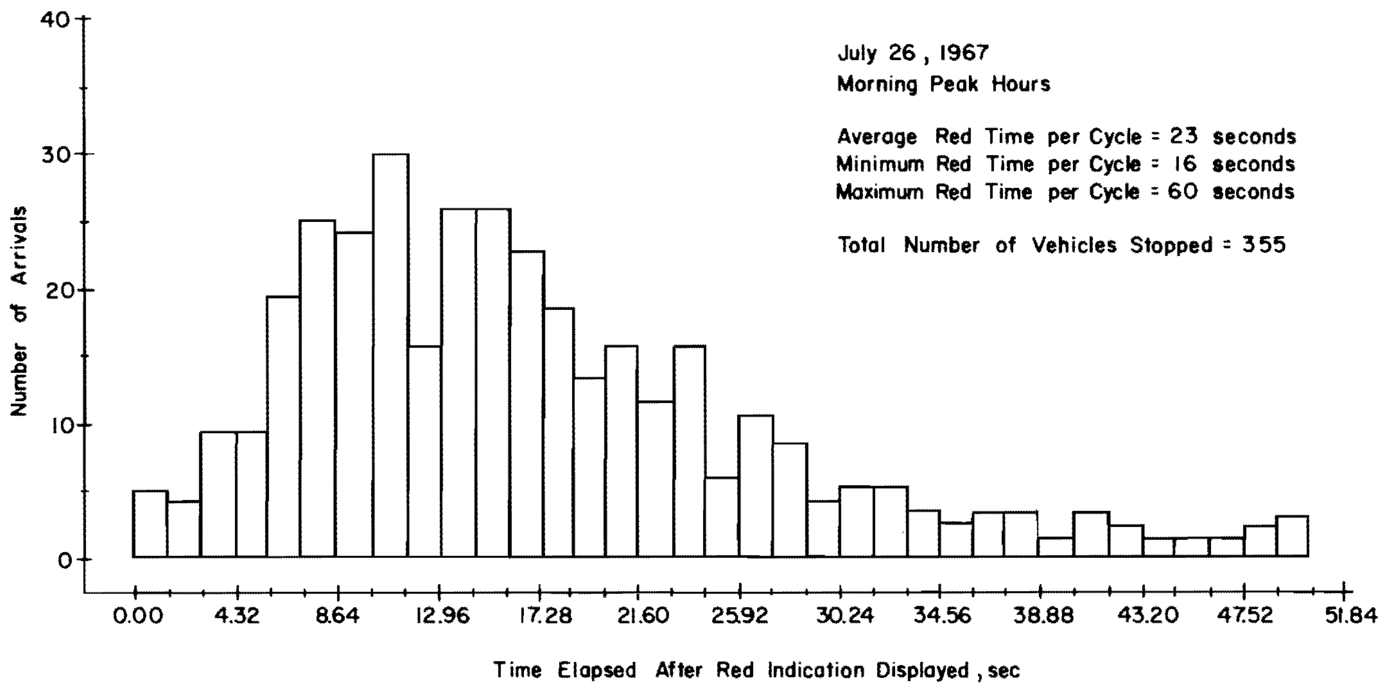

Fig 9.7. Arrival patterns, northbound approach, South First and Oltorf.

full-actuated control. The distributions are generally similar, but the duration of the red indication was extended by traffic actuations in the latter case.

These examples serve to illustrate the type of information that can be deduced from data which were perhaps recorded for another specific purpose. By designing field observation and recording practices appropriately, much valuable data needed for the validation of simulation models can be procured by the D3 digital delay data recorder.

CHAPTER 10. SUMMARY AND RECOMMENDATIONS

All study objectives as outlined in the first chapter of this report have been met.

DELAY RECORDING EQUIPMENT

The Digital Delay Data Recorder (D3 Recorder) was developed and used successfully at everal intersection sites for the collection of large quantities of data pertaining to the delay experience of vehicles interrupted by various intersection control mechanisms. The principal advantage of the D3 Recorder is its ability to record multiple observations in a form immediately suitable for computer processing with a minimum of encoding and manual data handling. In fact, six hours of field data, which frequently involved as many as 360,000 individually recorded data items, could normally be processed and fully analyzed, with all required summaries, on an overnight basis.

The principal disadvantage is the large number of observers necessary to provide input data which is then automatically recorded on punched paper tape. This feature perhaps detracted slightly from the overall efficiency of traffic movement during the studies because of the resulting gaper's block, but utilization of a sign reading "Traffic Survey", at the intersection seemed to minimize the effect.

The D3 Recorder appears to be a practical tool for the comprehensive analysis and evaluation of intersection phenomena, including some which are discussed below.

COMPUTER PROGRAMS

Computer programs were written for summarizing delay data in various time intervals. It was these data summaries that were used extensively in the analysis portion of this research. Methods were developed for analyzing the data and for calculating appropriate traffic parameters. Traffic parameters which were calculated for each approach over a selected time period were:

(1) traffic volume,

(2) total vehicle-seconds of delay,

(3) total number of vehicles stopped,

(4) average delay per vehicle,

(5) average delay per vehicle stopped,

(6) percent of vehicles stopped,

(7) total green time,

(8) number of complete cycles,

(9) average green time per cycle, and

(10) average cycle length.

The first six items were summarized for the intersection as a whole. Items seven and nine were characteristic of a given direction, while items eight and ten were characteristic of the intersection control. Only the first six items were calculated for stop-sign controlled intersections.

TIME PERIOD FOR ANALYSIS

Although the hour is generally the time unit employed in planning, design, and operation of highway facilities, there are significant variations in the short-term flow rates within an individual hour which are important from an operational standpoint. The largest 15-minute flow on a given intersection approach may range from the uniform value of 25 percent to more than 50 percent of the total hourly flow.

Thus, the time base used in a delay study should be some period of less than an hour's duration to take some of the short-term fluctuation into account. Plots were made of several volume versus delay relationships calculated for various time intervals. After analyzing these, it was concluded that 15-minute time periods provided the most representative results in terms of the smoothness and regular appearance of the curves and were used throughout the study.

FIELD STUDIES

A total of 19 intersections were selected for the collection of field data. In all, 124 individual studies, including 240 hours of observed data, were performed, mainly during the summer months of 1966 and 1967.

Intersections studied ranged from a low-volume, two-way stop-sign controlled intersection to a high-volume, signalized diamond interchange. Virtually every type of traffic signal control used in Austin, Texas, was included in this study. Some intersections were studied under several types of control in an attempt to gain additional insight into delay characteristics.

Except for the diamond interchange, all of the intersections had four approaches and were essentially right-angle crossings. The sites generally were situated in suburban areas which were classified as either outlying business districts or residential fringe areas. Parking was prohibited on all approaches in virtually all instances. Sight distances were generally adequate. The volume of pedestrian and truck traffic at each intersection location was considered to be negligible.

STOP-SIGN CONTROLLED INTERSECTIONS

The performance of traffic at seven different stop-sign controlled intersections was studied and evaluated. Several graphic displays were developed which illustrated the relationship of both average and total delay to several traffic parameters. A number of mathematical expressions for delay as a function of such values as approach volume and intersection volume were also developed.

It was observed that delays began to increase very rapidly at total traffic volumes of about 200 to 250 vehicles per 15-minute interval at two-way stops but at volumes of 300 vehicles per 15-minute interval at four-way stops. It also was observed that, for a given total intersection volume, the average delay for stopped vehicles was much higher for two-way stop control than for four-way stop control, but that the total delay was greater for four-way stop control than for two-way stop control.

The following model was developed for two-way stop-sign controlled intersections:

$$y = 456 - 6.89x_1 + 0.08464x_2 - .0712x_3$$

where

y = total vehicle-seconds of delay on the stop-sign controlled approaches for 15-minute intervals,

x_1 = the total volume (on all four approaches),

x_2 = the square of the total volume,

x_3 = the square of the through volume.

This particular model was used in developing the volume versus delay relationships illustrated in Figs 5.3 and 5.12. These relationships lead directly to the set of minimum volume warrants for the installation of four-way stop signs (Table 5.2).

The following model was developed for four-way stop-sign controlled intersections:

$$y = (18.95 + .00044x^2)^2$$

where

y = the total vehicle-seconds of delay per 15-minute interval,

x = the total vehicular volume per 15-minute interval.

This model for four-way stop control is especially interesting, not only because of the high correlation between the delay values predicted by the model and the actual delays observed in field studies ($R^2 = 0.984$), but mainly because it was developed by using delay data from five different intersections. These intersections included two 4 X 2 and three 4 X 4 type intersections. One 4 X 4 had a wide median opening for one direction of flow. Traffic splits ranged from 75/25 to 50/50. The percentage of left-turn movements varied widely. Data were collected during the morning, afternoon, and evening periods. Thus, although the conditions varied quite widely, this particular model is excellent for explaining the variability in the volume versus delay relationships of four-way stop-sign controlled intersections.

While it appeared that no traffic parameter other than total volume influenced the delay characteristics at four-way stop controlled intersections, the traffic split was a factor at two-way stop controlled intersections. For a given total intersection volume, the total delay increased steadily as the percentage of stopped vehicles increased from about 20 percent ot slightly more than 40 percent, beyond which point the magnitude of total delay showed indications of leveling out or even decreasing.

It is recommended that the minimum volume warrants in Table 5.2 for the installation of four-way stop signs be validated by field testing and considered for adoption by the Texas Highway Department.

SIGNALIZED INTERSECTIONS

Traffic signals are used to assign the right-of-way alternately to vehicles or queues of vehicles passing through an intersection. For maxiumum efficiency, the signals should be timed so that

(1) the total delay to all traffic using the intersection is minimized,

(2) no individual vehicle experiences excessive delay, and

(3) the average delay per vehicle is tolerable for the circumstances.

Studies of stopped-time delay at eight isolated signalized intersections which were operated under pretimed, semiactuated, and full-actuated control indicated that traffic-actuated control generally resulted in less delay than pretimed control for the range of conditions observed. Apportioning of the green time was found to have a pronounced effect on delay for pretimed control. Semiactuated control was most effective at locations where less than about 40 percent of the total traffic was consistently carried on the street equipped for detection of vehicles. Full-actuated control resulted in less delay than either of the other types when the total traffic was split approximately 50/50 on the two streets or where short-time demands fluctuated on various approaches during the day.

An economic analysis of a representative intersection showed that the higher equipment, maintenance, and operating costs of actuated control could be easily compensated for in less than two years by the lower stopping, idling, and time costs that would accrue to road users from the more efficient traffic control.

Warrants for traffic signals developed by D-18T, Texas Highway Department, were evaluated and found to provide good guidelines for selecting actuated equipment for locations where traffic volumes do not warrant pretimed signals. Delay studies at three intersections which met the suggested warrants for actuated control, but not for pretimed control, showed that actuated control consistently resulted in less delay than pretimed equipment up to total volumes of about 450 vehicles per 15-minutes.

Studies of the effect of dial settings of actuated signal controllers on delay indicated that these settings were not extremely critical over the rather limited ranges considered to be practicable. If long loop-type detectors (40 to 80 feet long) or other suitable vehicle presence detectors which have become available since these studies were conducted are used, problems associated with detector placement, initial intervals, and vehicle intervals are virtually eliminated. Very precise controller response can be achieved by setting initial and vehicle intervals to minimum values.

SPECIAL STUDIES

The practical feasibility of using multichannel digital recording equipment in the field for comparative delay studies was demonstrated. The recording and data analysis techniques that were developed are useful for many types of before-and-after evaluation studies. Minor modifications to the observation and analysis techniques will make it possible to use equipment similar to the D3 Recorder for studying traffic phenomena such as headways, gaps, arrival patterns. and intersection capacity.

MODERNIZED EQUIPMENT

Recent spectacular advancements in electronic instrumentation have rendered the electromechanical hardware, but not the concept, of the digital data delay recorder obsolete. Development of a new instrument system with the same basic capabilities as the D3 Recorder is recommended. It is now possible to have a portable unit the size of a small suitcase with all the features needed to conduct field traffic studies at the most complex intersections. This unit would overcome most of the limitations such as bulk, scanning rate, and complex operation associated with the D3 Recorder.

TRAFFIC SIMULATION

Computer simulation of traffic flow at intersections is potentially a powerful tool for studying intersection efficiency, but up to now very little adequate field data have been available for validating simulation models. Data collected in the traffic studies described in this report include extensive amounts of several types of information needed for verifying such models. Some of the recorded or computed information is directly applicable; other relationships can be deduced.

It is recommended that serious consideration be given to developing computer simulation models that can be used to evaluate traffic flow at isolated intersections and on street networks. Once properly verified models are available, wide ranges of traffic patterns, intersection configurations, and control techniques can be evaluated rapidly and conveniently without resorting to cut-and-try field techniques. Delay recording equipment can be used to establish quantitative information concerning realistic ranges of parameters to be evaluated by simulation.

CHAPTER 11. IMPLEMENTATION

Warrants for traffic-actuated signals in urban areas as proposed by D-18T, Texas Highway Department, have been evaluated in terms of volume versus delay relationships developed from several field studies conducted under this research project. The urban portion of the proposed warrants can now be applied with confidence in that actuated equipment selected in accordance with these warrants, even though slightly more expensive initially, will consistently result in less delay than pretimed equipment. Use of these warrants by the Texas Highway Department should be continued. Sections of the warrants proposed for rural area conditions should be evaluated as soon as is feasible.

The warrants proposed herein for the installation of four-way stop-sign control at intersections should be evaluated by the Texas Highway Department and cities in the State and, if found suitable, adopted for general use.

The characteristic volume versus delay relationships for stop-sign and signal control as described in this report will prove useful in guiding the judgment and subsequent decisions of traffic engineers in regard to the suitability of certain types of control for specific situations.

The usefulness of multichannel data recording equipment for field traffic studies has been demonstrated in this study. Consideration should be given to developing a new generation of highly portable hardware with the same basic capabilities as the D3 Recorder. This equipment combined with the data collection and analysis techniques developed for this study can yield quantitative information needed for before-and-after evaluation studies of improvements made under programs such as TOPICS. It can also be used to develop badly needed information concerning intersection and network capacities.

Extensive amounts of recorded field data that are suitable for validating traffic simulation models have been developed in this project. A study of the formulation of valid simulation models useful for intersection and network flow analysis should be undertaken.

REFERENCES

1. Alroth, Williard A., and Paul C. Box, "Warrants for Traffic Control Signals (Parts I, II, and III)," Traffic Engineering, November 1967, pp 32-41; December 1967, pp 22-29; and January 1968, pp 14-21.

2. Berry, D. S., "Field Measurement of Delay at Signalized Intersections," Proceedings, Vol 35, Highway Research Board, 1956, pp 505-522.

3. Berry, D. S., and J. H. Kell, "Stopped-Time Delay at Three-Phase Signalized Intersections," Proceedings, California Street and Highway Conferences, 1955, pp 114-121.

4. Berry, D. S., and C. J. Van Til, "A Comparison of Three Methods for Measuring Delay at Intersections," Proceedings, California Street and Highway Conferences, 1954, pp 71-79.

5. Biomedical Computer Programs, "BMD-2R - Stepwise Multiple Regression," University of California at Los Angeles, 1964.

6. Branden, Bruno Van Den, "A Study of Stop Warning Systems," Ohio State University, Columbus, Ohio, March 1967.

7. Cooner, Harold D., "An Analysis of Vehicular Delay at Signalized Intersections," Master's Thesis, The University of Texas at Austin, January 1970.

8. Cunningham, Frank N., "Traffic Delay at Stop Sign Controlled Intersections," Master's Thesis, The University of Texas at Austin, May 1969.

9. Dyas, D. E., "Traffic Engineering and the Highway Engineer," Traffic Engineering and Control, July 1963, pp 171-175.

10. "Effect of Control Devices on Traffic Operations," Interim Report, National Cooperative Highway Research Program Report 11, Highway Research Board, 1964, pp 1-86.

11. Hall, E. M., "Intersection Delay - Signal vs. Four Way Stop," Proceedings, California Street and Highway Conferences, 1952, pp 107-111.

12. Hebert, Jacques, "A Study of Four-Way Stop Intersection Capacities," Highway Research Record No. 27, Highway Research Board, 1963, pp 130-147.

13. Highway Capacity Manual, U. S. Bureau of Public Roads, Washington, D. C., 1950.

14. Hood, Louis E., "A Technique for Evaluating Vehicular Delay at Intersections," Master's Thesis, The University of Texas atAustin, June 1967.

15. Kell, James H., "Analyzing Vehicular Delay at Intersections Through Simulation," Bulletin 356, Highway Research Board, 1962, pp 28-39.

16. Kell, James H., "Intersection Delay Obtained by Simulating Traffic on a Computer," Highway Research Record No. 15, Highway Research Board, 1963, pp 73-97.

17. Lewis, Russell M., and Harold L. Michael, "Simulation of Traffic Flow to Obtain Volume Warrants for Intersection Control, Highway Research Record No. 15, Highway Research Board, 1963, pp 1-43.

18. Manual on Uniform Traffic Control Devices for Streets and Highways, U. S. Bureau of Public Roads, Washington, D. C., June 1961.

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

20. McCasland, William R., "Capacity-Demand Analysis of the Wayside Interchange on the Gulf Freeway," Research Report No. 24-9, Texas Transportation Institute, Texas A&M University, College Station, September 1960.

21. McCracken, Daniel D., and William S. Dorn, Numerical Methods and Fortran Programming, John Wiley and Sons, Inc., New York, 1964, pp 262-275.

22. Mitchell, H. R., "A Digital Recorder for Vehicular Delay Studies," Master's Thesis, The University of Texas, Austin, August 1962.

23. Model Traffic Ordinance, National Committee on Uniform Traffic Laws and Ordinances, Washington, D.C., 1962, pp 22-23.

24. Oglesby, Clarkson H., and Lawrence I. Hewes, Highway Engineering, John Wiley and Sons, Inc., New York, 1964, 71 pp.

25. Oppenlander, J. C., and Per Solberg, "Lag and Gap Acceptances at Stop Controlled Intersections," Highway Research Record No. 118, Highway Research Board, 1966, pp 48-67.

26. Raff, Morton S., A Volume Warrant for Urban Stop Signs, The ENO Foundation for Highway Traffic Control, Saugatuck, Connecticut, 1950. 188 pp.

27. Research on Road Traffic, Road Research Laboratory, London, 1965.

28. Sagi, George S., and Lowell D. Campbell, "Vehicle Delay at Signalized Intersections," Traffic Engineering, Vol 39, No. 5, February 1969, pp 32-40.

29. Traffic Engineering Handbook, Institute of Traffic Engineers, Washington, D. C., 1965.

30. Texas Motor Vehicle Laws, State of Texas, Texas Highway Department, 1965-1966.

31. Uniform Vehicle Code, National Committee on Uniform Traffic Laws and Ordinance, Washington, D. C., 1962.

32. Vernez, Georges, "Vehicular Delay at Signalized Intersections," Master's Thesis, The University of Texas, Austin, August 1963.

33. Volk, N. Wayne, "Effect of Type of Control on Intersection Delay," Proceedings, Vol 35, Highway Research Board, 1965, p 523.

34. Wagner, Frederick A., Jr., "An Evaluation of Fundamental Driver Decisions and Reactions at an Intersection," Highway Research Record No. 118, Highway Research Board, 1966, pp 68-83.

35. Wagner, F. A., D. L. Gerlough, and F. C. Barnes, "Improved Criteria for Traffic Signal Systems on Urban Arterials," Research Report 73, Planning Research Corporation, Los Angeles, California.

36. Webster, F. V., "Traffic Signal Settings," Road Research Technical Paper 39, Road Research Laboratory, 1958, pp 1-44.

37. Webster, F. V., "Traffic Signals," Road Research Technical Paper 56, Road Research Laboratory, 1966, pp 1-106.

38. Winfrey, Robley, Economic Analysis for Highways, International Textbook Company, Scranton, Pennsylvania, 1969.

APPENDIX A

GEOMETRIC LAYOUT OF INTERSECTIONS AND VOLUME INFORMATION

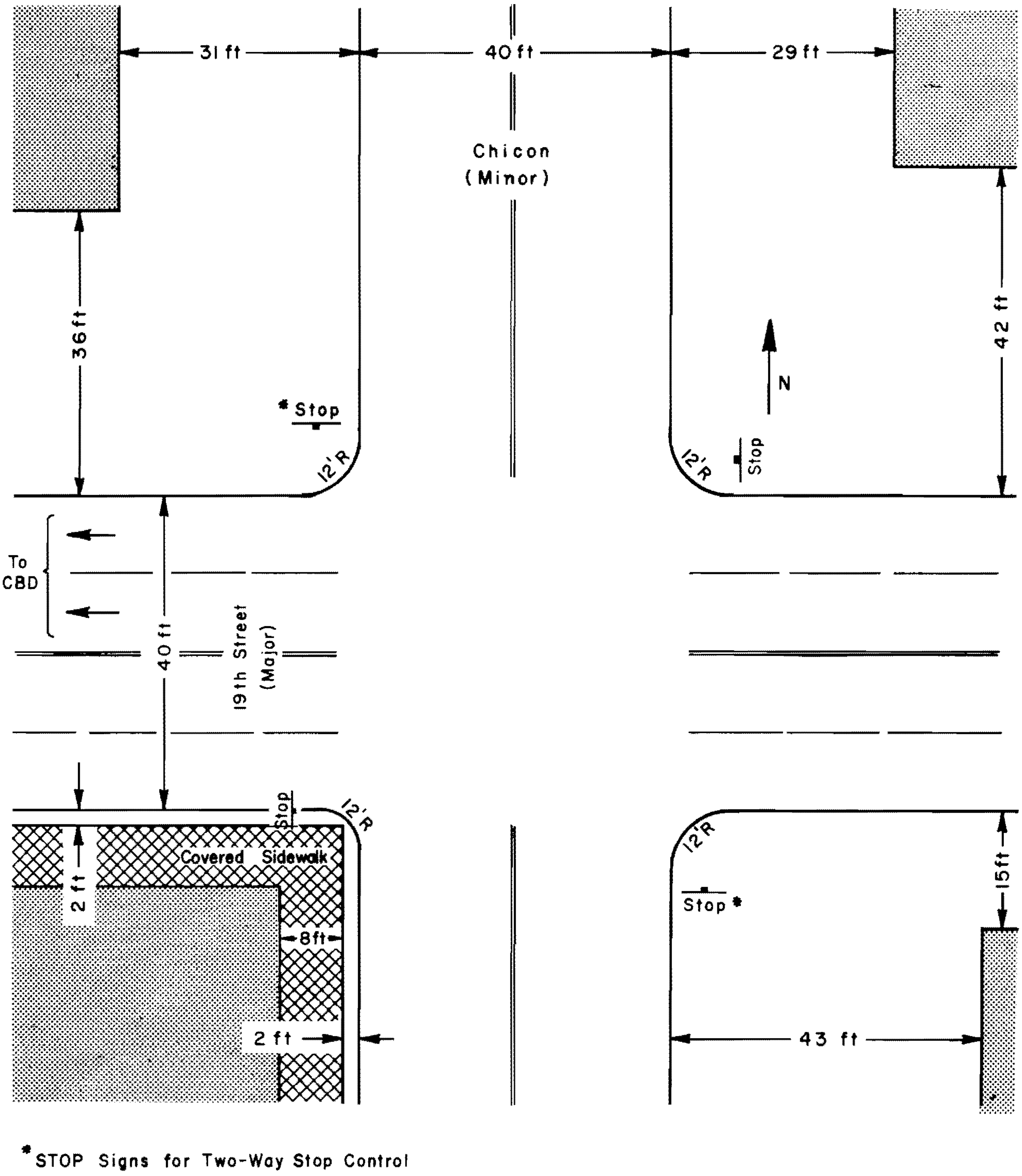

Fig A.1. Geometric layout of 19th and Chicon intersection (two-way and four-way stop).

TABLE A.1. 15-MINUTE VOLUME - 19TH AND CHICON

Intersection	Type of Control	Date	Time
19th and Chicon	4-Way Stop	June 15, 1967	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	25	18	12	55	6	24	29	59	0	73	7	80	10	154	3	167	361
0800	19	21	7	47	5	31	24	60	4	61	3	68	17	173	2	192	367
0815	10	23	9	42	2	20	9	31	4	68	9	81	12	105	4	121	275
0830	14	17	7	38	2	6	7	15	4	54	6	64	7	74	4	87	204
0845	16	14	3	33	1	3	3	7	4	63	11	78	2	57	3	62	180
0900	9	26	6	41	6	15	3	24	3	44	10	57	8	72	2	82	204

Intersection	Type of Control	Date	Time
			1330-1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	5	15	11	31	7	17	4	28	3	52	8	63	11	51	3	65	187
1400	10	19	10	39	5	18	3	26	2	65	12	79	14	52	4	70	214
1415	5	13	7	25	2	12	1	15	4	55	15	74	12	58	2	72	186
1430	13	10	2	25	8	14	0	22	3	62	6	71	6	56	2	64	182
1445	10	17	10	37	0	14	1	15	5	62	13	80	7	45	2	54	186
1500	9	13	14	36	2	15	0	17	11	58	11	80	11	61	1	73	206
1515	9	20	2	31	7	16	2	25	2	72	10	84	9	46	2	57	197
1530	13	14	8	35	4	17	1	22	3	65	9	77	15	62	2	79	213

(Continued)

TABLE A.1. (CONTINUED)

Intersection	Type of Control	Date	Time
19th and Chicon	4-Way Stop	June 15, 1967	1630-1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	10	14	10	34	8	16	1	25	5	95	7	107	10	71	6	87	253
1700	11	27	9	47	7	18	3	28	6	103	10	119	11	87	5	103	297
1715	5	24	20	49	7	23	6	36	6	134	12	152	6	92	1	99	336
1730	4	15	17	36	6	20	2	28	8	156	6	170	15	72	3	90	324
1745	12	22	16	50	5	34	1	40	15	102	15	132	18	69	6	93	315
1800	7	17	12	36	7	14	4	25	3	63	12	78	11	77	6	94	233
1815	8	19	11	38	7	15	3	25	4	76	8	88	11	46	8	65	216
1830	7	29	14	50	5	13	2	20	6	49	16	71	13	48	5	66	207

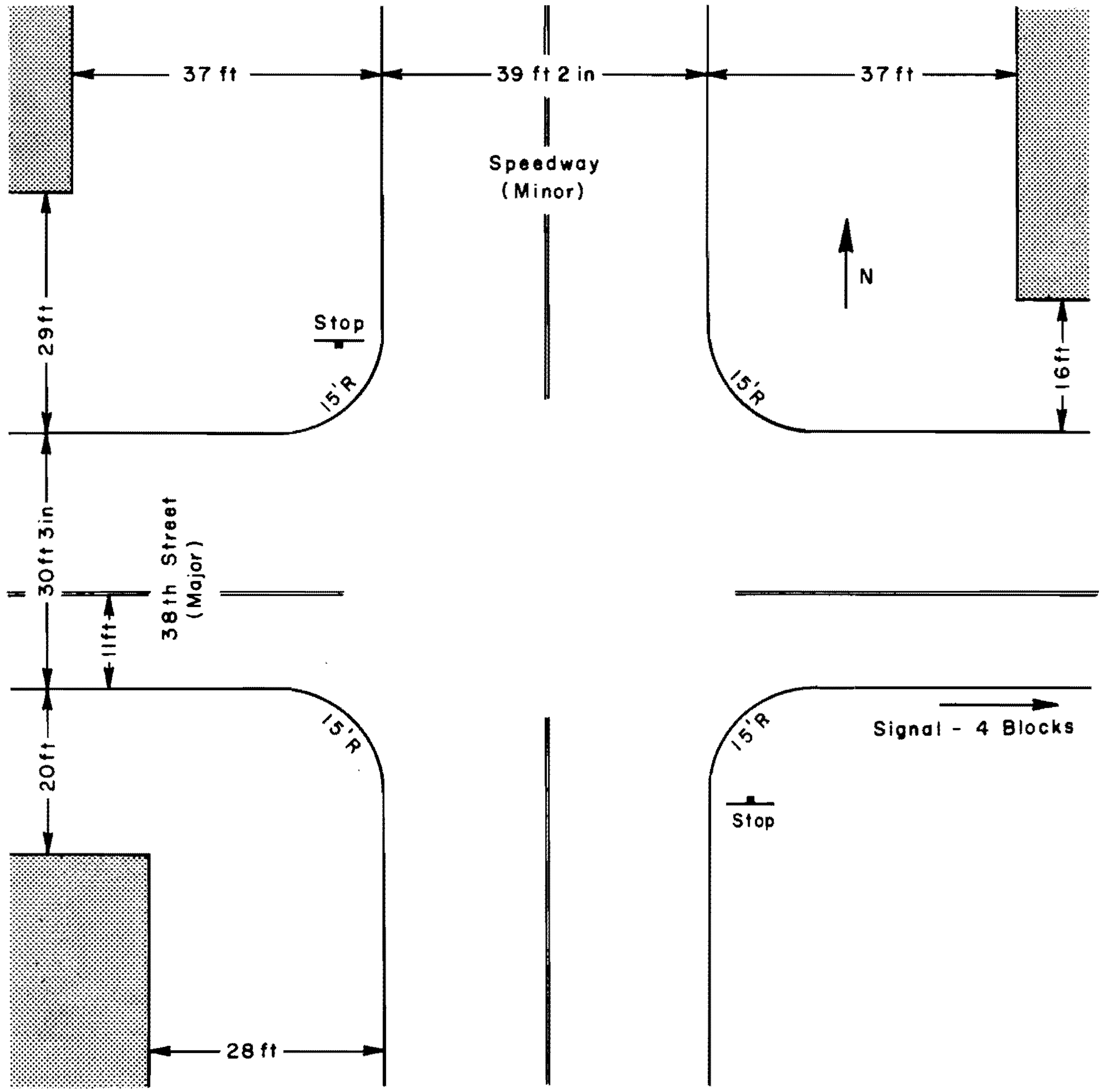

Fig A.2. Geometric layout of 38th and Speedway intersection (two-way stop).

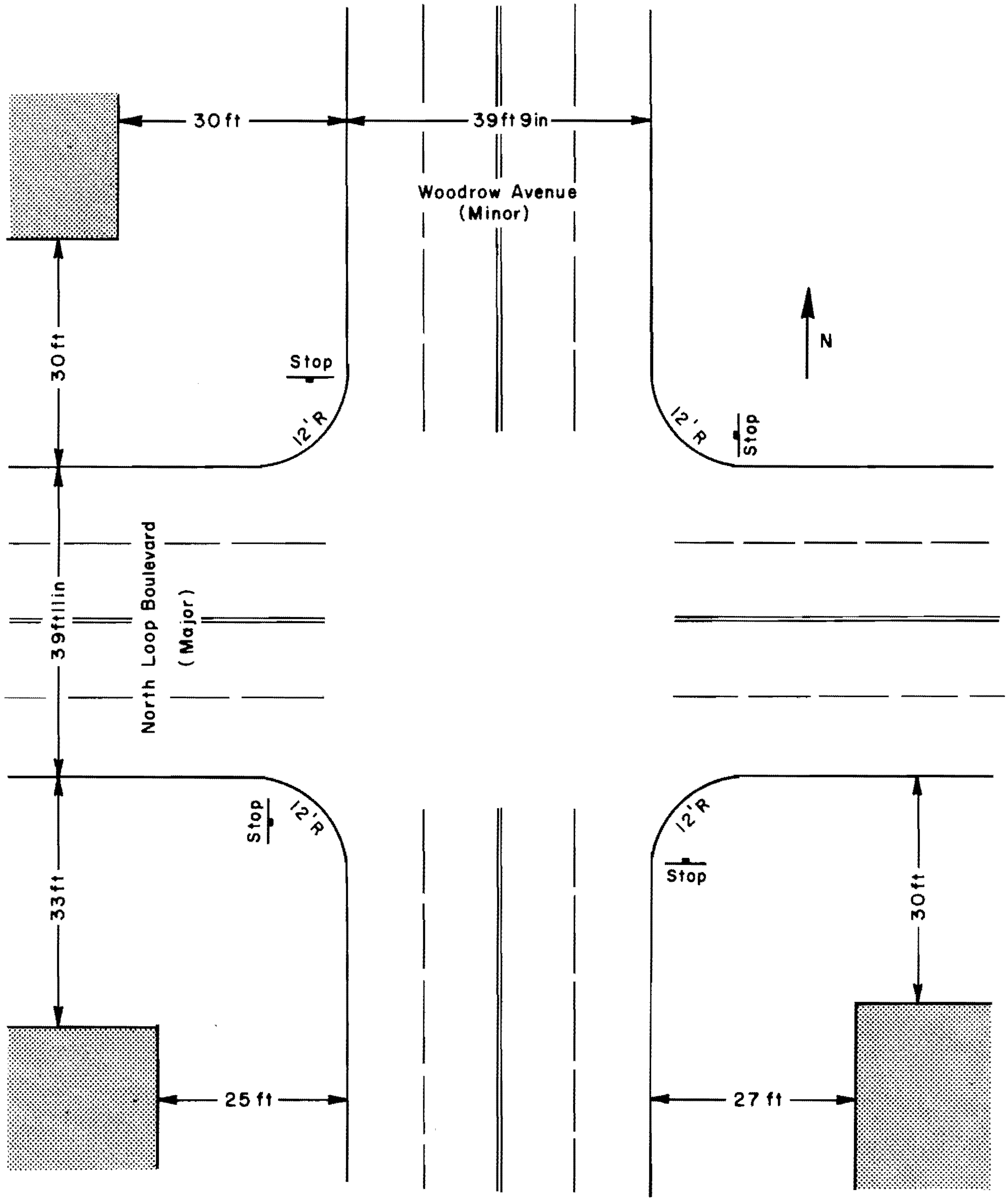

Fig A.3. Geometric layout of North Loop and Woodrow intersection (four-way stop).

TABLE A.3. 15-MINUTE VOLUME - NORTH LOOP AND WOODROW

Intersection	Type of Control	Date	Time
North Loop and Woodrow	4-Way Stop	June 8, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	0	19	18	37	33	66	21	120	3	88	2	93	2	73	11	86	336
0800	1	15	16	32	35	56	19	110	9	85	3	97	11	76	7	94	333
0815	0	16	5	21	18	37	2	57	4	39	1	44	6	43	9	58	180
0830	3	9	1	13	12	21	2	35	3	28	0	31	7	48	7	62	141
0845	2	12	7	21	13	23	4	40	3	25	0	28	4	34	7	45	134
0900	1	15	3	19	11	16	5	32	5	26	1	32	4	42	2	48	131

Intersection	Type of Control	Date	Time
			1335-1520

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1350	1	16	8	25	6	18	3	27	6	52	0	58	4	35	5	44	154
1405	0	14	4	18	8	19	7	34	5	50	0	55	4	46	4	54	161
1420	5	17	3	25	1	14	4	19	3	25	0	28	3	36	6	45	117
1435	0	14	5	19	4	17	5	26	5	32	1	38	2	48	7	57	140
1450	2	16	3	21	8	21	7	36	6	39	1	46	2	42	5	49	152
1505	2	20	3	25	2	12	3	17	4	39	2	45	2	38	8	48	135
1520	4	17	4	25	6	11	2	19	6	32	3	41	1	37	3	41	126

(Continued)

TABLE A.3. (CONTINUED)

Intersection	Type of Control	Date	Time
North Loop and Woodrow	4-Way Stop	June 8, 1966	1630-1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	5	22	4	31	3	23	7	33	13	66	1	80	3	56	12	71	215
1700	0	48	14	62	3	18	9	30	10	57	3	70	7	42	21	70	232
1715	3	98	9	110	11	23	13	47	28	76	1	105	9	84	31	124	386
1730	4	102	5	111	4	18	7	29	19	50	2	71	3	74	26	103	314
1745	3	58	5	66	10	21	10	41	9	53	2	64	5	64	21	90	261
1800	1	52	9	62	7	23	9	39	8	58	0	66	3	66	13	82	259
1815	0	36	8	44	5	17	4	26	10	42	1	53	5	64	10	79	198
1830	4	22	7	33	6	16	7	29	9	36	4	49	5	60	5	70	181

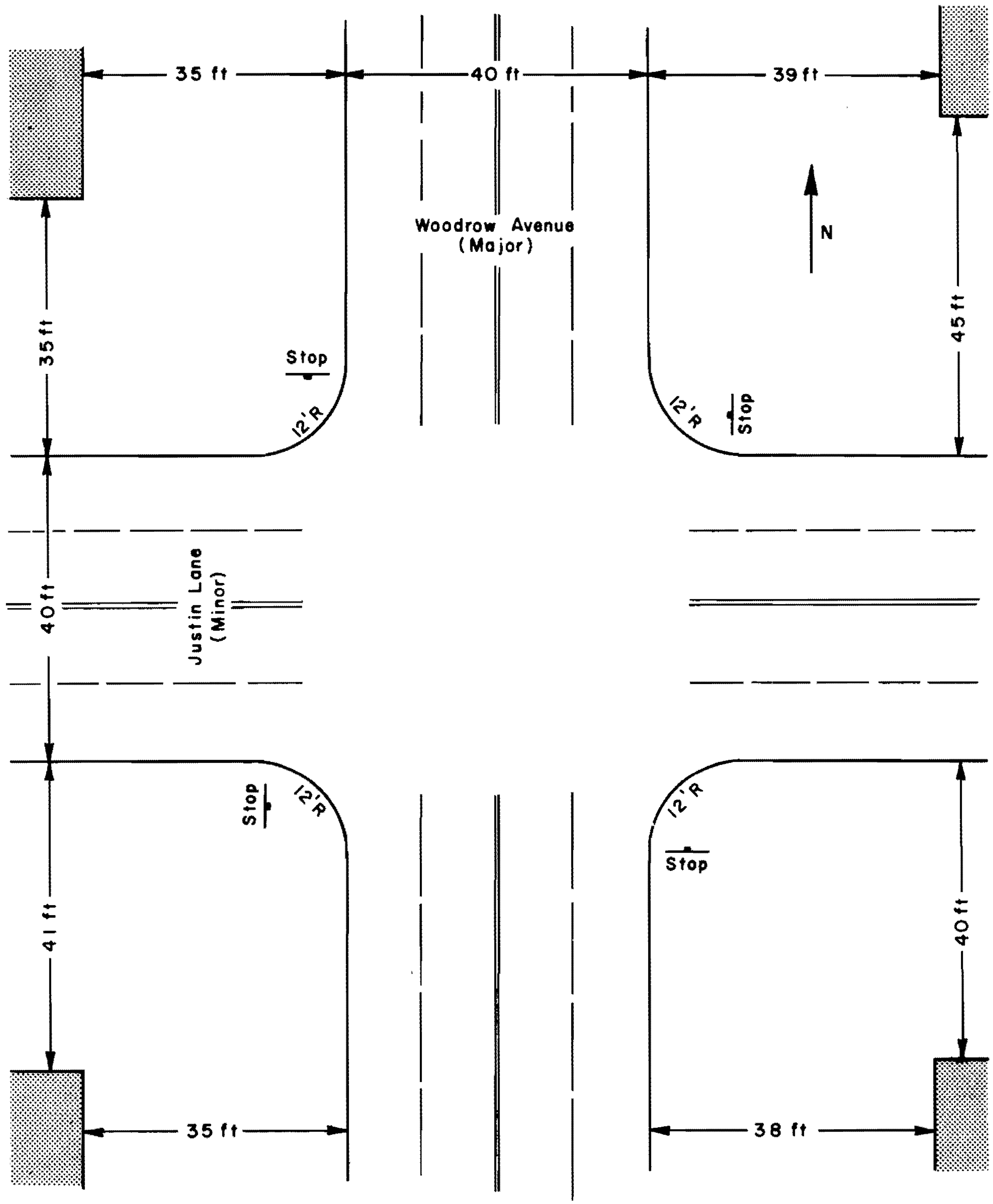

Fig A.4. Geometric layout of Justin and Woodrow intersection (four-way stop).

TABLE A.4. 15-MINUTE VOLUME - JUSTIN AND WOODROW

Intersection	Type of Control	Date	Time
Justin and Woodrow	4-Way Stop	June 2, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	6	18	6	30	28	85	3	116	2	91	6	99	5	23	7	35	280
0800	2	18	8	28	20	39	0	59	0	81	10	91	2	30	7	39	217
0815	0	6	3	9	13	31	0	44	0	31	6	37	1	22	5	28	118
0830	1	6	4	11	10	28	1	39	2	28	1	31	3	24	6	33	114
0845	1	9	5	15	14	20	4	38	3	27	5	35	3	40	8	51	149
0900	3	9	1	13	19	24	2	45	1	34	3	38	7	21	9	37	133

Intersection	Type of Control	Date	Time
			1330-1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	2	18	3	23	6	17	3	26	0	23	4	27	1	29	10	40	116
1400	4	17	2	23	11	17	1	29	3	32	3	38	1	27	5	33	123
1415	0	16	2	18	11	15	3	29	2	26	12	40	5	35	11	51	138
1430	4	18	1	23	7	19	1	27	2	28	5	35	1	24	14	39	124
1445	8	20	3	31	9	16	0	25	0	30	6	36	3	30	7	40	132
1500	2	14	3	19	7	19	1	27	1	26	3	30	7	37	7	51	127
1515	6	20	3	29	8	21	4	33	8	30	2	40	9	23	4	36	138
1530	4	24	3	31	7	23	1	31	3	24	10	37	4	39	12	55	154

(Continued)

TABLE A.4. (CONTINUED)

Intersection	Type of Control	Date	Time
Justin and Woodrow	4-Way Stop	June 2, 1966	1630-1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	6	35	5	46	14	28	2	44	1	48	4	53	6	47	15	68	211
1700	9	49	9	67	5	27	1	33	4	28	6	38	6	58	22	86	224
1715	11	89	12	112	11	33	3	47	3	35	4	42	6	83	37	126	327
1730	5	77	5	87	6	33	10	49	3	42	3	48	2	58	34	94	278
1745	8	44	7	59	9	35	3	47	4	44	6	54	3	68	21	92	252
1800	11	38	8	57	7	25	2	34	7	40	5	52	6	40	24	70	213
1815	8	31	4	43	8	28	3	39	2	21	5	28	3	35	19	57	167
1830	2	27	5	34	7	24	0	31	3	27	8	38	4	45	15	64	167

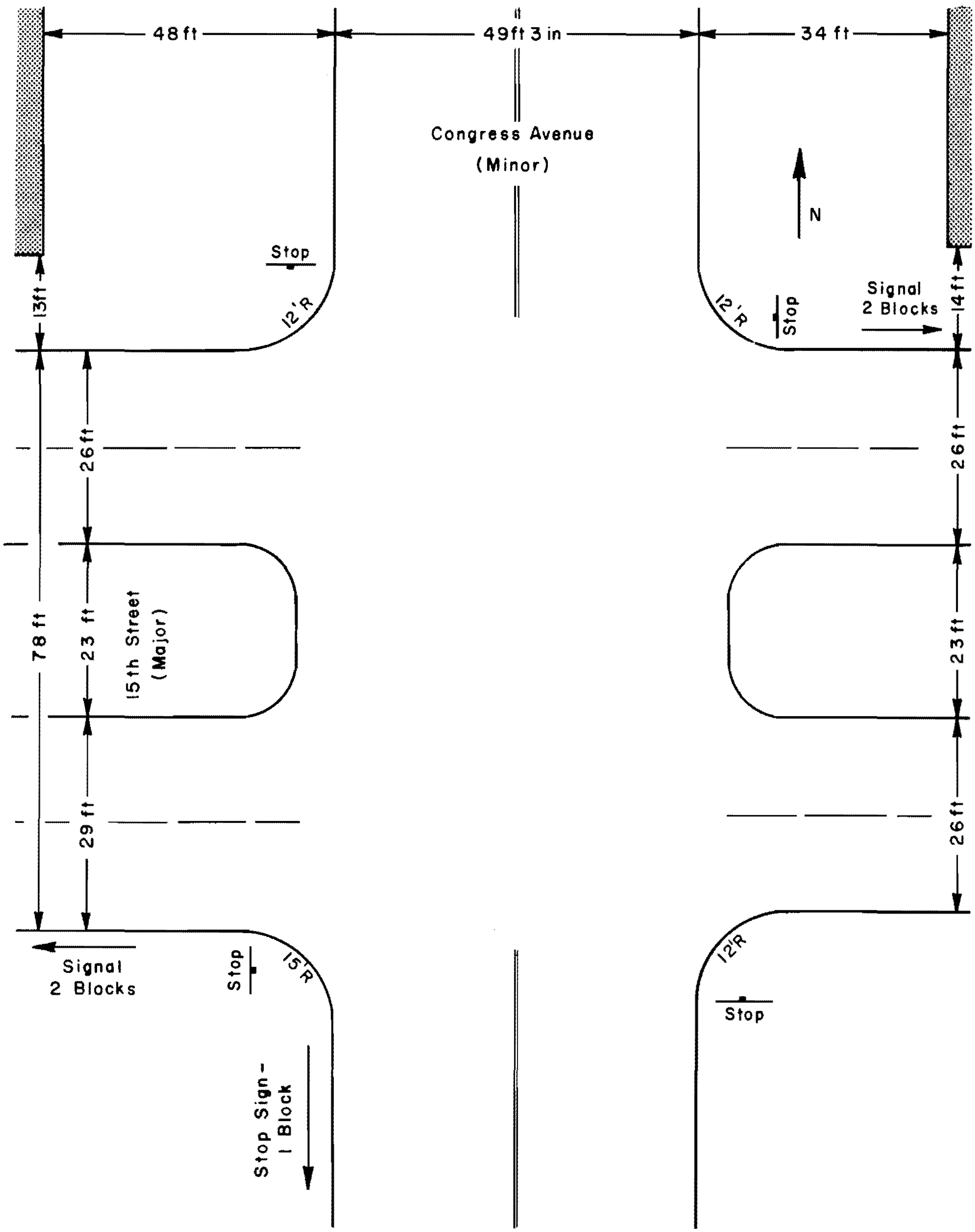

Fig A.5. Geometric layout of 15th and Congress intersection (four-way stop).

TABLE A.5. 15-MINUTE VOLUME - 15TH AND CONGRESS

Intersection	Type of Control	Date	Time
15th and Congress	4-Way Stop	June 13, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	8	33	9	50	3	23	10	36	6	70	16	92	12	92	14	118	296
0800	9	49	17	75	7	18	9	34	13	106	10	129	10	112	17	139	377
0815	17	31	6	54	1	9	4	14	3	53	2	63	7	68	10	85	216
0830	7	11	7	25	3	15	5	23	1	50	2	23	1	36	4	41	112
0845	7	12	5	24	4	8	4	16	1	29	6	36	2	34	1	37	113
0900	2	10	5	17	3	8	1	12	2	26	3	31	2	45	0	47	107

Intersection	Type of Control	Date	Time
			1330-1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	5	21	6	32	8	15	4	27	3	44	3	50	11	55	2	68	177
1400	2	13	0	15	2	14	3	19	1	37	0	38	2	39	4	45	117
1415	6	12	4	22	1	8	6	15	4	41	8	53	2	40	3	45	135
1430	8	14	5	27	3	11	2	16	3	51	2	56	2	34	9	45	144
1445	4	9	3	16	3	19	2	24	2	43	6	51	5	42	1	48	139
1500	7	6	3	16	2	10	1	13	2	42	4	48	7	44	1	52	129
1515	3	13	3	19	2	11	3	16	2	36	9	47	6	36	0	42	124
1530	3	9	6	18	0	18	7	25	2	38	3	43	3	33	1	37	123

TABLE A.5. (CONTINUED)

Intersection	Type of Control	Date	Time
15th and Congress	4-Way Stop	June 13, 1966	1630-1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	7	10	5	22	10	18	3	31	3	71	13	87	6	52	3	61	201
1700	9	28	35	72	7	31	25	63	5	92	9	106	13	87	6	106	347
1715	41	36	31	108	17	36	31	84	13	110	12	135	12	96	13	121	448
1730	10	17	9	36	5	20	4	29	3	71	12	86	2	53	4	59	210
1745	5	8	7	20	1	13	3	17	0	57	1	58	0	30	2	32	127
1800	4	9	6	19	0	4	6	10	1	45	2	48	2	21	1	24	101
1815	1	10	2	13	0	6	4	10	1	32	3	36	0	19	0	19	78
1830	0	7	4	11	1	5	1	7	1	25	1	27	1	19	0	20	65

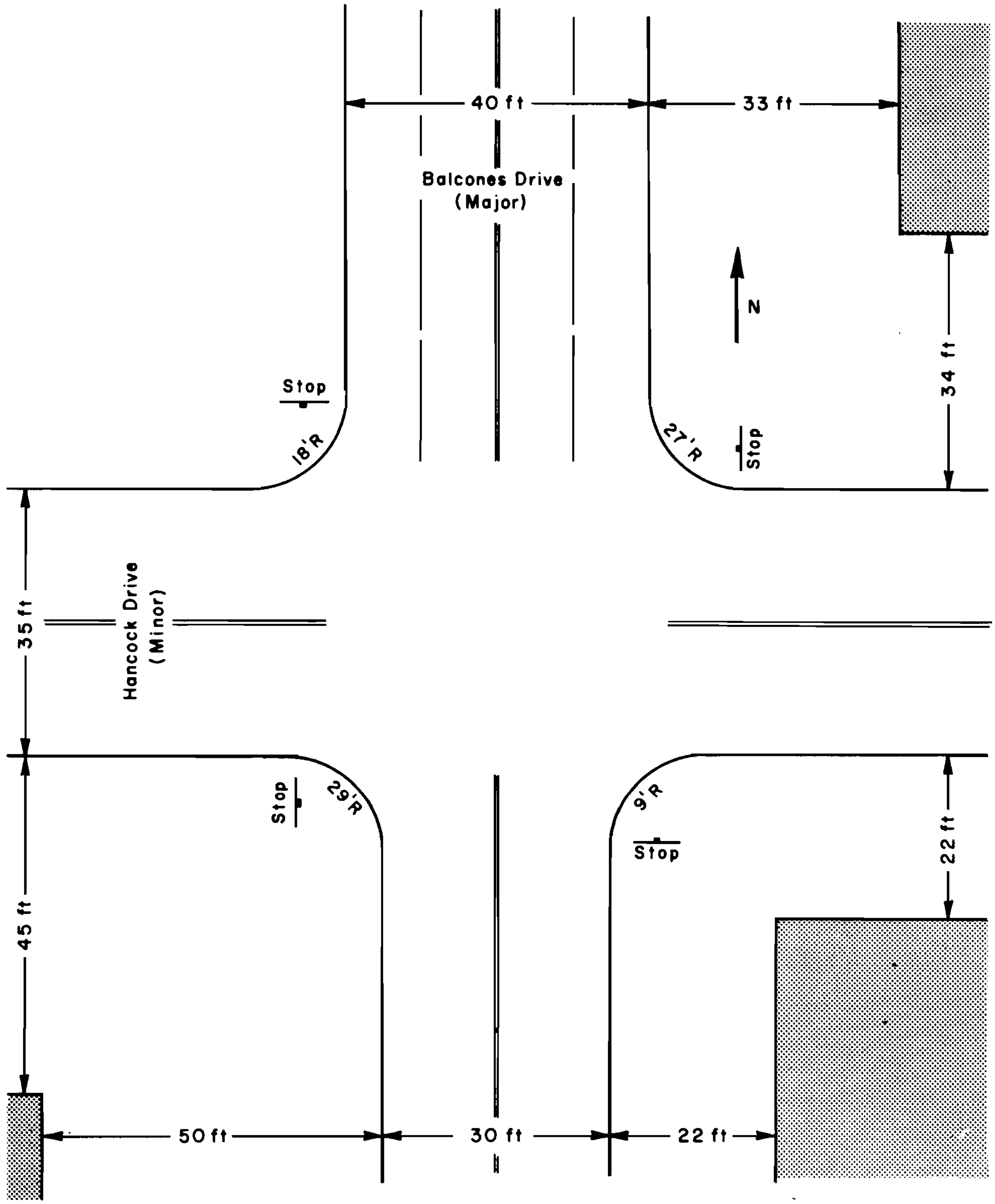

Fig A.6. Geometric layout of Hancock and Balcones intersection (four-way stop).

TABLE A.6. 15-MINUTE VOLUME - HANCOCK AND BALCONES

Intersection	Type of Control	Date	Time
Hancock and Balcones	4-Way Stop	June 28, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	0	40	7	47	52	97	1	150	2	9	0	11	2	2	25	29	237
0800	0	41	10	51	47	76	1	124	1	5	0	6	5	5	32	42	223
0815	1	26	7	34	37	60	0	97	0	8	1	9	3	9	29	41	181
0830	2	34	4	40	34	48	1	83	1	7	0	8	1	6	23	30	161
0845	1	19	4	24	43	41	1	85	1	8	4	13	4	2	23	29	151
0900	0	22	7	29	26	39	0	65	1	2	0	3	1	8	31	40	137

Intersection	Type of Control	Date	Time
			1330-1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	0	21	4	25	26	25	1	52	0	1	0	1	4	5	25	34	112
1400	2	23	3	28	39	23	3	65	1	2	0	3	5	4	30	39	135
1415	0	26	8	34	39	32	1	72	3	4	0	7	2	4	24	30	143
1430	2	24	7	33	22	21	4	47	0	1	1	2	2	4	16	22	104
1445	1	25	2	28	40	25	2	67	1	4	0	5	1	1	20	22	122
1500	0	30	3	33	24	23	0	47	2	1	0	3	4	4	25	33	116
1515	1	30	5	36	27	29	2	58	2	8	1	11	6	10	28	44	149
1530	0	28	4	32	25	27	0	52	1	3	0	4	5	5	38	48	136

(Continued)

TABLE A.6. (CONTINUED)

Intersection	Type of Control	Date	Time
Hancock and Balcones	4-Way Stop	June 28, 1966	1630-1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	1	57	6	64	43	34	6	83	3	4	1	8	10	10	54	74	229
1700	2	59	9	70	31	38	3	72	0	6	0	6	2	9	49	60	208
1715	9	78	10	97	43	49	2	94	2	2	0	4	7	8	85	100	295
1730	2	83	5	90	51	43	2	96	4	7	0	11	3	1	108	112	309
1745	1	53	5	59	32	38	2	72	2	10	0	12	9	10	137	156	299
1800	1	65	5	71	32	26	1	59	7	11	1	19	12	11	123	146	295
1815	0	46	3	49	28	34	0	62	3	2	1	6	6	6	72	84	201
1830	0	42	5	47	34	26	2	62	1	2	1	4	6	6	42	54	167

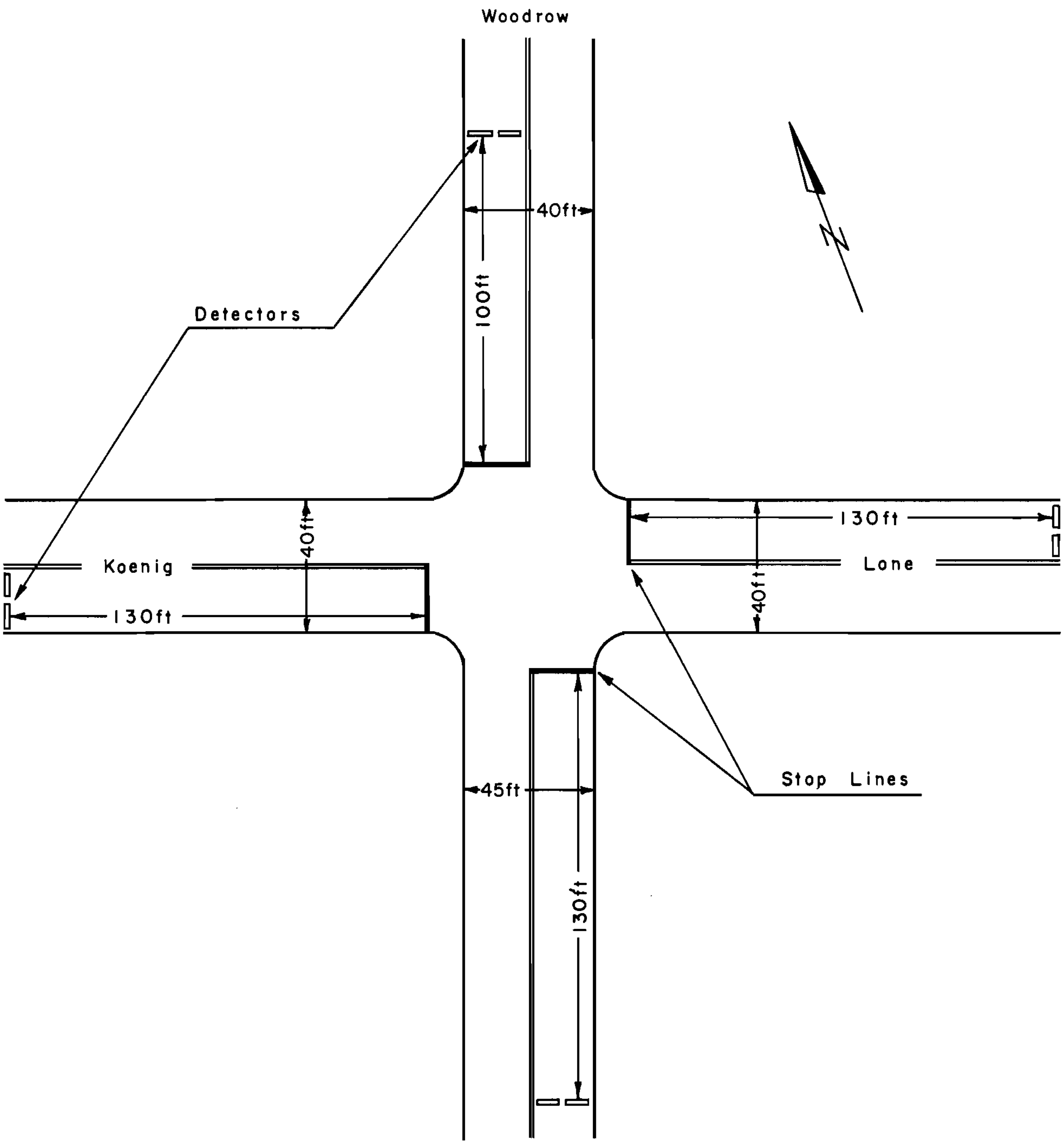

Fig A.7. Geometric layout of Woodrow and Koenig intersection (signalized).

TABLE A.7. 15-MINUTE VOLUME - WOODROW AND KOENIG

Intersection	Type of Control	Date	Time
Woodrow and Koenig	Full-Actuated	Aug. 31, 1966	0730 - 0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	2	13	7	22	54	96	5	155	2	179	3	184	7	81	9	97	458
0800	1	20	8	29	36	85	4	125	2	162	5	169	3	74	4	81	404
0815	2	17	6	25	14	46	7	67	1	92	4	97	5	79	11	95	284
0830	2	12	14	18	13	23	5	41	2	80	2	84	6	80	5	91	234
0845	4	8	0	22	8	23	7	38	1	86	3	90	3	72	12	87	237
0900	2	18	0	12	9	29	9	47	1	69	2	72	9	81	8	98	229

Intersection	Type of Control	Date	Time
Woodrow and Koenig	Full-Actuated	July 15, 1966	1330 - 1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	5	7	4	16	11	13	8	32	3	93	0	96	2	111	13	126	270
1400	1	9	5	15	11	27	4	42	5	87	3	95	3	79	14	96	248
1415	3	18	1	22	10	15	7	32	4	80	5	89	3	99	12	114	257
1430	4	15	6	25	14	11	2	27	7	98	2	107	4	84	8	96	255
1445	3	17	3	23	8	35	8	51	4	106	3	113	3	78	11	92	279
1500	3	23	2	28	9	17	3	29	5	80	7	92	2	92	13	107	256
1515	6	20	3	29	6	16	11	33	6	100	2	108	4	96	10	110	280
1530	5	18	4	27	10	17	5	32	4	76	3	83	1	97	8	106	248

(Continued)

TABLE A.7. (CONTINUED)

Intersection	Type of Control	Date	Time
Woodrow and Koenig	Full-Actuated	Aug. 31, 1966	1630 - 1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	4	40	8	52	14	17	2	33	6	104	2	112	3	121	18	142	339
1700	12	59	8	79	14	21	5	40	2	112	2	116	9	111	20	140	375
1715	8	128	6	142	13	36	14	63	2	129	5	136	6	191	34	231	572
1730	18	139	8	165	14	29	10	53	8	100	3	111	5	172	36	213	542
1745	7	65	7	79	15	31	13	59	6	97	2	105	6	160	46	212	455
1800	13	56	7	76	16	22	7	45	8	93	4	105	5	97	27	129	355
1815	11	61	5	77	13	19	5	37	7	91	1	99	7	121	23	151	364
1830	5	39	9	53	16	19	3	38	16	74	4	94	5	107	18	130	315

Intersection	Type of Control	Date	Time
Woodrow and Koenig	Semiactuated	July 22, 1966	0715 - 0915

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0730	5	7	3	15	37	87	5	129	1	108	5	114	5	75	6	86	344
0745	2	19	8	29	72	125	6	203	3	189	7	199	4	105	2	111	542
0800	5	40	9	54	73	101	5	179	1	238	6	245	0	102	10	112	590
0815	2	25	9	36	17	53	8	78	3	128	2	133	6	94	10	110	357
0830	7	9	3	19	14	48	4	66	6	79	0	85	4	66	10	80	250
0845	4	20	3	27	15	37	4	56	1	56	3	60	0	55	6	61	204
0900	3	13	1	17	9	27	3	39	1	73	3	77	0	60	5	65	198
0915	1	10	7	18	9	22	4	35	4	65	2	71	6	53	7	66	190

(Continued)

TABLE A.7. (CONTINUED)

Intersection	Type of Control	Date	Time
Woodrow and Koenig	Semiactuated	July 18, 1966	1330 - 1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	6	13	3	22	14	18	6	38	8	100	2	110	3	73	8	84	254
1400	5	14	7	26	8	20	5	33	5	77	2	84	2	63	8	73	216
1415	6	15	0	21	12	25	4	41	6	63	7	76	0	71	9	80	218
1430	4	21	9	34	12	19	7	38	4	84	4	92	7	105	9	121	285
1445	4	19	5	28	9	37	9	55	3	90	3	96	1	66	5	72	251
1500	2	14	3	19	4	13	2	19	8	63	3	74	3	76	6	85	197
1515	4	24	5	33	10	19	3	32	2	84	4	90	9	72	8	89	244
1530	6	23	5	34	5	15	8	28	2	58	0	60	2	68	11	81	203

Intersection	Type of Control	Date	Time
Woodrow and Koenig	Semiactuated	July 18, 1966	1630 - 1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	9	32	4	45	8	26	9	43	4	93	2	99	3	115	24	142	329
1700	8	70	6	84	11	21	6	38	6	91	3	100	3	140	19	162	384
1715	12	157	13	182	13	25	10	48	4	122	2	128	5	205	36	246	604
1730	7	97	4	108	13	23	7	43	5	75	5	85	8	143	35	186	422
1745	12	56	7	75	16	38	15	69	10	100	4	114	7	136	28	171	429
1800	8	49	5	62	17	23	5	45	7	72	3	82	10	119	18	147	336
1815	4	47	4	55	18	23	8	49	4	73	3	80	7	87	19	113	297
1830	7	25	5	37	14	17	8	39	5	71	2	78	7	87	21	115	269

(Continued)

TABLE A.7. (CONTINUED)

Intersection	Type of Control	Date	Time
Woodrow and Koenig	Pretimed	July 15, 1966	0715 - 0915

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0730	4	8	5	17	53	101	4	158	1	138	4	143	5	87	6	98	416
0745	4	20	9	33	72	128	5	205	3	227	14	244	5	92	13	110	592
0800	7	36	4	47	64	93	8	165	2	225	9	236	2	130	16	148	596
0815	3	15	2	20	22	32	6	60	5	102	4	111	4	95	16	115	306
0830	0	22	2	24	15	24	8	47	4	80	2	86	2	73	11	86	243
0845	4	15	4	23	13	25	3	41	2	82	0	84	3	69	7	79	227
0900	2	13	3	18	13	21	6	40	2	75	4	81	3	56	9	68	207
0915	4	15	2	21	9	17	6	32	3	68	1	72	2	56	9	67	192

Intersection	Type of Control	Date	Time
Woodrow and Koenig	Pretimed	July 6, 1966	1345 - 1530

Time	Northbound				Southbound				Eastbound				West bound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1400	2	11	5	18	12	21	6	39	6	78	6	90	6	67	8	81	228
1415	5	13	2	20	3	6	7	16	5	65	1	71	6	83	8	97	204
1430	6	15	5	26	11	15	5	31	6	85	1	92	3	84	12	99	248
1445	4	17	4	25	12	25	4	41	3	82	2	87	4	85	10	99	252
1500	2	12	4	18	15	27	6	48	1	74	3	78	1	92	9	102	246
1515	5	20	1	26	14	16	8	38	4	64	4	72	5	70	6	81	217
1530	4	22	6	32	9	13	6	28	5	86	3	94	0	59	8	67	221

(Continued)

TABLE A.7. (CONTINUED)

Intersection	Type of Control	Date	Time
Woodrow and Koenig	Pretimed	July 6, 1966	1700 - 1845

Time	Northbound				Soutbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1715	13	133	5	151	9	23	10	42	6	114	2	122	11	166	36	213	528
1730	15	111	11	137	11	22	4	37	5	82	4	91	8	68	33	209	474
1745	5	49	5	59	15	23	6	44	6	74	5	85	9	139	36	184	372
1800	9	49	6	64	16	15	5	36	6	76	3	85	15	111	27	153	338
1815	7	43	9	59	18	16	11	45	7	81	4	92	5	106	22	133	329
1830	5	38	3	46	7	27	9	43	4	54	4	62	4	112	18	134	285
1845	4	22	8	34	18	22	10	50	4	77	1	82	3	78	15	96	262

TABLE A.8. SIGNAL CONTROLLER SETTINGS FOR WOODROW AND KOENIG

Date	Time	Full-Actuated			
		Woodrow		Koenig	
Aug 31	0730 - 0900	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	8 6 off 60 4	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 6 off 60 3
July 15	1330 - 1530	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	8 6 off 60 4	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 6 off 60 3
Aug 31	1630 - 1830	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	8 6 off 60 4	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 6 off 60 3

Date	Time	Semiactuated			
		Woodrow		Koenig	
July 22	0715 - 0915	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	8 5 off 60 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 29 3
July 14	1330 - 1530	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	8 5 off 60 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 29 3
July 14	1630 - 1830	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	8 5 off 60 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 27 3

(Continued)

TABLE A.8. (CONTINUED)

Date	Time	Pretimed*			
		Woodrow		Koenig	
July 15	0715 - 0915	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	0 30 off 25 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	0 30 off 29 3
July 6	1340 - 1540	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	0 30 off 25 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	0 30 off 29 3
July 6	1650 - 1850	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	0 30 off 20 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	0 30 off 29 3

* These settings were set to force the controller to operate on a pretimed plan. In this case the maximum interval controls the length of green on each phase.

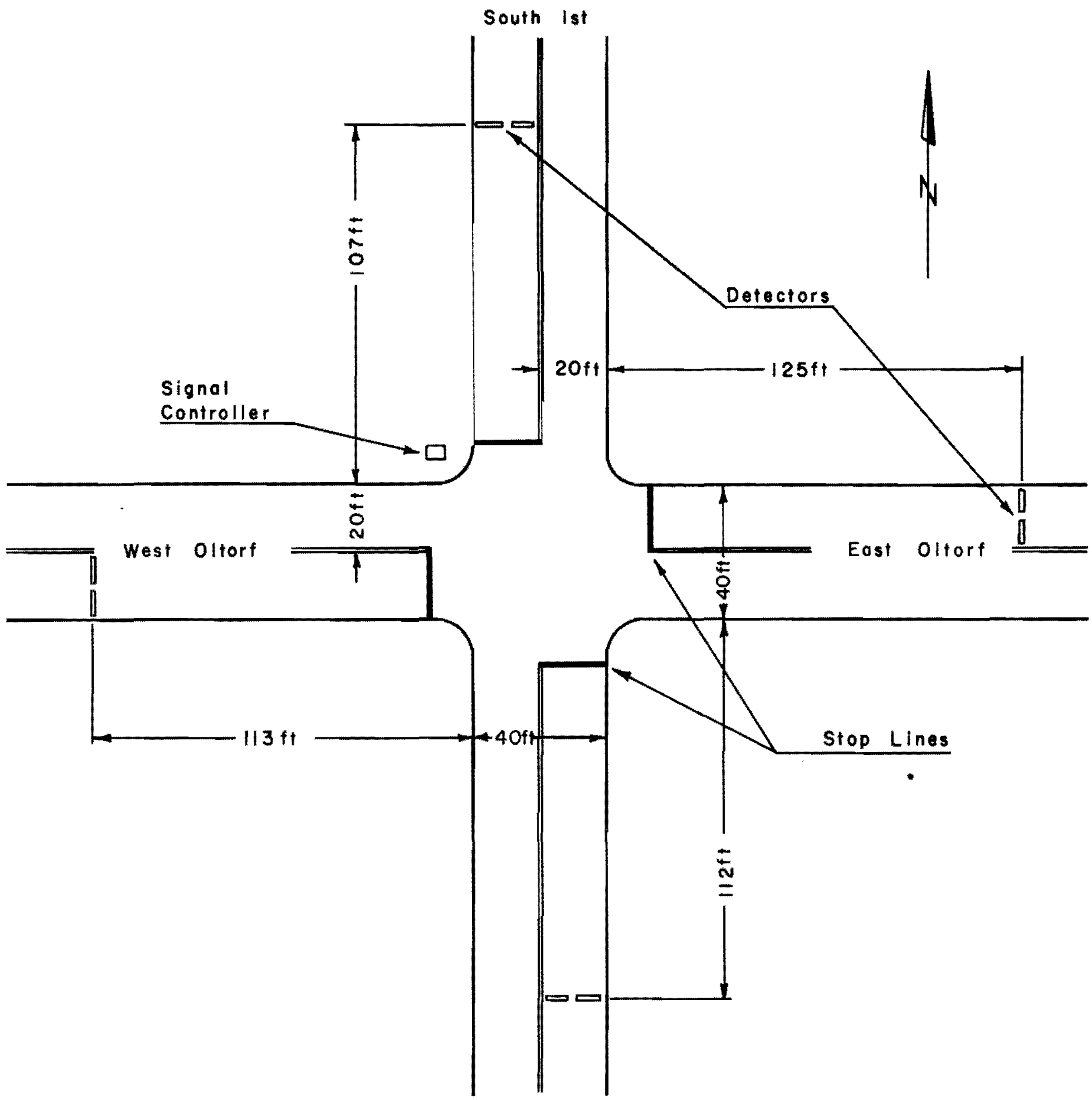

Fig A.8. Geometric layout of South First and Oltorf intersection (signalized).

TABLE A.9. 15-MINUTE VOLUME - SOUTH FIRST AND OLTORF

Intersection	Type of Control	Date	Time
South 1st and Oltorf	Full-Actuated	July 14, 1966	1330 - 1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	3	20	15	38	11	36	8	55	7	33	3	43	11	49	5	65	201
1400	6	38	12	56	19	24	6	49	8	48	5	61	3	38	11	52	218
1415	2	35	7	44	18	32	4	54	4	58	4	66	10	35	17	62	226
1430	4	22	6	32	13	27	7	47	3	39	4	46	2	42	9	53	178
1445	4	33	4	41	9	31	8	48	4	27	7	38	9	32	10	51	178
1500	3	22	13	38	10	26	5	41	5	53	1	59	9	39	13	61	199
1515	3	27	4	34	15	33	5	53	2	45	5	52	10	49	16	75	214
1530	5	21	2	28	14	35	6	55	3	52	6	61	11	50	10	71	215

Intersection	Type of Control	Date	Time
South 1st and Oltorf	Full-Actuated	July 14, 1966	1630 - 1800

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	14	55	14	83	26	76	17	119	5	53	13	71	12	85	19	116	389
1700	9	49	16	74	21	87	23	131	12	47	11	70	15	81	19	115	390
1715	9	56	13	78	32	141	27	201	6	44	13	63	22	95	14	131	473
1730	12	61	22	96	36	118	22	176	7	59	17	83	20	97	25	142	497
1745	11	45	17	74	26	88	22	136	10	83	13	106	20	95	17	132	448
1800	7	45	16	68	11	59	9	79	8	58	14	80	20	71	9	100	327

(Continued)

TABLE A.9. (CONTINUED)

Intersection	Type of Control	Date	Time
South 1st and Oltorf	Semiactuated	July 20, 1966	0715 - 0915

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0730	12	127	8	147	8	35	3	46	21	63	5	89	4	32	13	49	331
0745	12	149	11	172	4	28	4	36	33	71	4	108	4	30	16	50	366
0800	7	112	17	136	12	42	2	46	24	66	7	97	4	50	9	63	342
0815	5	75	12	92	9	39	8	56	16	57	10	83	3	25	13	41	272
0830	3	45	9	57	14	23	4	51	8	43	8	59	5	27	6	38	205
0845	3	49	8	60	8	25	5	38	16	26	8	50	3	36	8	47	195
0900	5	50	10	65	12	33	6	51	11	41	6	58	11	30	9	50	224
0915	9	44	7	60	19	31	6	56	6	44	5	55	4	32	17	53	224

Intersection	Type of Control	Date	Time
South 1st and Oltorf	Semiactuated	July 20, 1966	1330 - 1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	6	26	9	41	17	25	3	45	4	36	7	47	11	47	17	75	208
1400	4	29	6	39	25	31	3	59	9	41	8	58	16	43	10	69	225
1415	5	32	15	52	10	21	3	34	4	30	5	39	5	49	16	70	195
1430	2	32	6	40	11	31	8	50	10	43	5	58	7	36	13	56	204
1445	3	30	9	42	10	28	5	43	4	41	8	53	11	33	17	61	199
1500	8	31	12	51	13	25	5	43	5	54	6	65	7	44	14	65	224
1515	12	21	12	45	12	44	9	65	9	48	9	66	8	38	12	58	234
1530	5	26	12	43	17	14	5	36	6	35	8	49	6	55	8	69	197

(Continued)

TABLE A.9. (CONTINUED)

Intersection	Type of Control	Date	Time
South 1st and Oltorf	Semiactuated	July 20, 1966	1630 - 1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	14	52	11	77	24	73	10	107	10	65	11	86	13	83	26	122	392
1700	10	55	16	81	25	91	16	132	10	69	11	90	10	98	22	130	433
1715	7	41	15	63	35	133	25	193	4	67	12	83	24	91	12	127	466
1730	12	40	16	68	20	118	23	161	5	59	15	79	25	113	16	154	462
1745	10	39	22	71	25	92	13	130	6	42	11	59	18	67	14	109	369
1800	11	36	9	56	17	73	15	105	2	59	21	82	22	77	14	113	356
1815	13	27	10	50	18	63	17	98	3	73	7	83	30	94	22	146	377
1830	8	46	10	64	15	59	13	87	6	45	7	58	17	77	16	110	319

Intersection	Type of Control	Date	Time
South 1st and Oltorf	Pretimed	July 19, 1966	0705 - 0905

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0720	5	91	11	107	6	41	7	54	10	55	9	74	5	35	9	49	284
0735	12	142	6	160	12	28	6	46	27	85	2	114	3	36	11	50	370
0750	7	151	10	168	15	46	3	64	39	88	9	136	3	52	17	72	440
0805	7	114	11	132	10	53	11	74	22	62	16	100	1	37	15	53	359
0820	11	100	9	120	11	30	6	47	18	45	5	68	3	35	15	53	288
0835	4	73	10	87	13	24	9	46	13	41	5	59	6	30	4	40	232
0850	2	47	6	55	11	28	6	45	11	52	5	68	1	33	7	41	209
0905	4	47	11	62	13	28	7	48	8	62	3	73	3	33	4	40	223

(Continued)

TABLE A.9. (CONTINUED)

Intersection	Type of Control	Date	Time
South 1st and Oltorf	Pretimed	July 19, 1966	1330 - 1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	7	43	11	61	22	30	8	60	12	47	7	66	8	46	13	67	254
1400	6	48	8	62	14	38	2	54	11	47	13	71	11	40	12	63	250
1415	6	45	17	68	15	29	4	48	9	40	8	57	9	60	14	83	256
1430	3	36	8	47	17	31	17	65	5	42	7	54	9	51	13	73	239
1445	4	34	12	52	12	29	9	50	3	40	10	53	12	41	7	60	215
1500	6	37	12	55	12	36	14	62	2	37	12	51	7	38	14	59	227
1515	7	23	13	45	17	49	12	78	11	39	14	64	3	48	18	69	256
1530	10	37	13	60	12	31	12	55	8	57	3	68	13	44	11	68	251

Intersection	Type of Control	Date	Time
South 1st and Oltorf	Pretimed	July 19, 1966	1630 - 1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	9	41	9	59	17	65	17	99	8	50	2	60	12	77	22	111	329
1700	8	50	10	68	17	103	16	136	3	74	1	78	20	102	18	140	422
1715	9	43	8	60	25	129	24	178	4	58	10	72	23	95	17	135	445
1730	10	47	18	75	32	123	29	174	7	71	12	90	26	118	26	170	509
1745	7	40	14	61	31	104	28	163	5	59	12	76	19	78	15	112	412
1800	6	46	12	64	19	80	9	108	6	41	14	61	13	89	15	107	340
1815	6	52	12	70	11	66	17	94	5	56	9	70	17	71	10	98	332
1830	10	38	6	54	15	56	13	84	2	50	3	55	15	66	7	88	281

TABLE A.10. SIGNAL CONTROLLER SETTINGS FOR SOUTH FIRST AND OLTORF

Date	Time	Semiactuated			
		South First		Oltorf	
July 20	0715 - 0915	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 22 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 6 off 60 3
July 20	1330 - 1530	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 22 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 6 off 60 3
July 20	1630 - 1830	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 22 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 6 off 60 3

Date	Time	Pretimed			
		South First		Oltorf	
July 19	0700 - 0900	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 22 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 22 3
July 19	1330 - 1530	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 22 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 22 3
July 19	1630 - 1830	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 22 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 22 3

Note: See p 253 for full-actuated controller settings at South First and Oltorf.

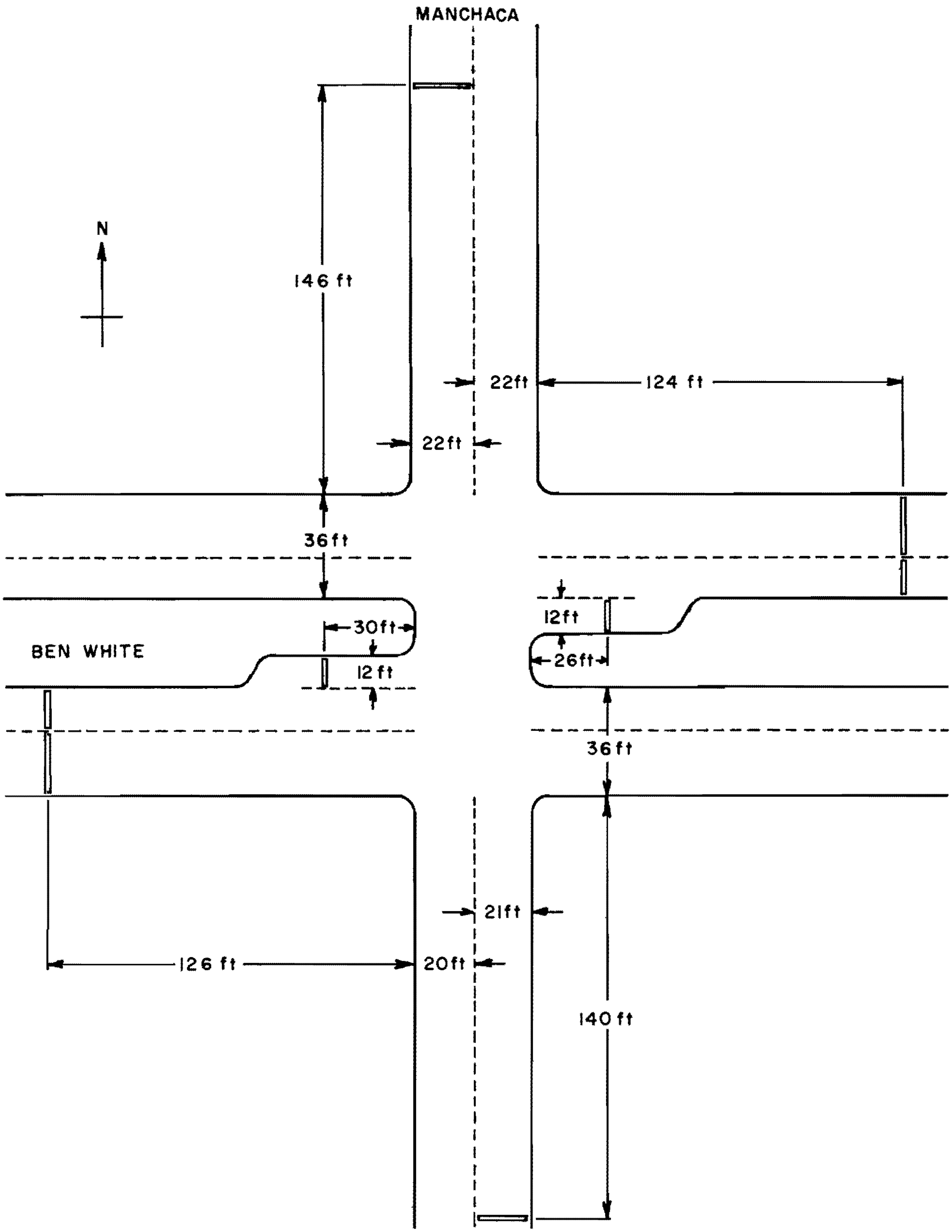

Fig A.9. Geometric layout of Ben White and Manchaca intersection (signalized).

TABLE A.11. 15-MINUTE VOLUME - BEN WHITE AND MANCHACA

Intersection	Type of Control	Date	Time
Ben White and Manchaca	Full-Actuated	July 28, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	3	64	59	126	8	14	3	25	1	88	2	91	12	29	8	49	291
0800	5	35	33	73	14	14	0	28	2	62	0	64	15	38	7	60	225
0815	1	35	19	55	9	13	2	24	5	43	2	50	12	32	5	49	178
0830	3	31	27	61	11	7	1	19	5	39	4	48	13	28	10	51	179
0845	1	31	17	49	7	13	2	22	4	33	2	39	9	30	2	41	151
0900	1	20	19	40	10	11	4	25	4	39	3	46	12	24	7	43	154

Intersection	Type of Control	Date	Time
			1330-1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	0	21	13	34	11	27	7	45	6	72	3	81	9	46	9	64	224
1400	2	29	15	46	12	23	7	42	4	59	4	67	16	52	13	81	236
1415	4	19	19	42	6	31	6	43	4	58	2	64	14	45	7	66	215
1430	1	25	16	42	7	26	6	39	6	46	2	54	17	67	8	92	227
1445	0	29	25	54	9	28	2	39	3	50	2	55	17	47	7	71	219
1500	1	16	20	37	11	32	5	48	2	50	6	58	22	55	10	87	230
1515	4	29	19	52	12	32	5	49	6	53	4	63	16	48	11	75	239
1530	1	19	13	33	5	28	7	40	4	50	1	55	29	37	9	75	203

(Continued)

TABLE A.11. (CONTINUED)

Intersection	Type of Control	Date	Time
Ben White and Manchaca	Full-Actuated	July 28, 1966	1630-1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	5	34	19	58	9	44	9	62	2	48	4	54	26	69	8	103	277
1700	5	26	16	45	18	55	7	80	7	40	11	58	50	84	8	142	325
1715	2	35	17	54	13	68	5	86	5	63	6	74	66	104	16	186	400
1730	3	24	15	42	13	87	12	112	6	42	4	52	49	73	12	134	340
1745	2	41	19	62	12	70	12	94	3	48	11	62	43	65	11	119	337
1800	5	34	21	60	19	49	8	76	4	49	8	61	30	82	19	131	328
1815	4	26	14	44	20	37	3	60	6	40	11	57	37	62	11	110	271
1830	0	30	12	42	17	27	9	53	1	47	6	54	24	45	14	83	232

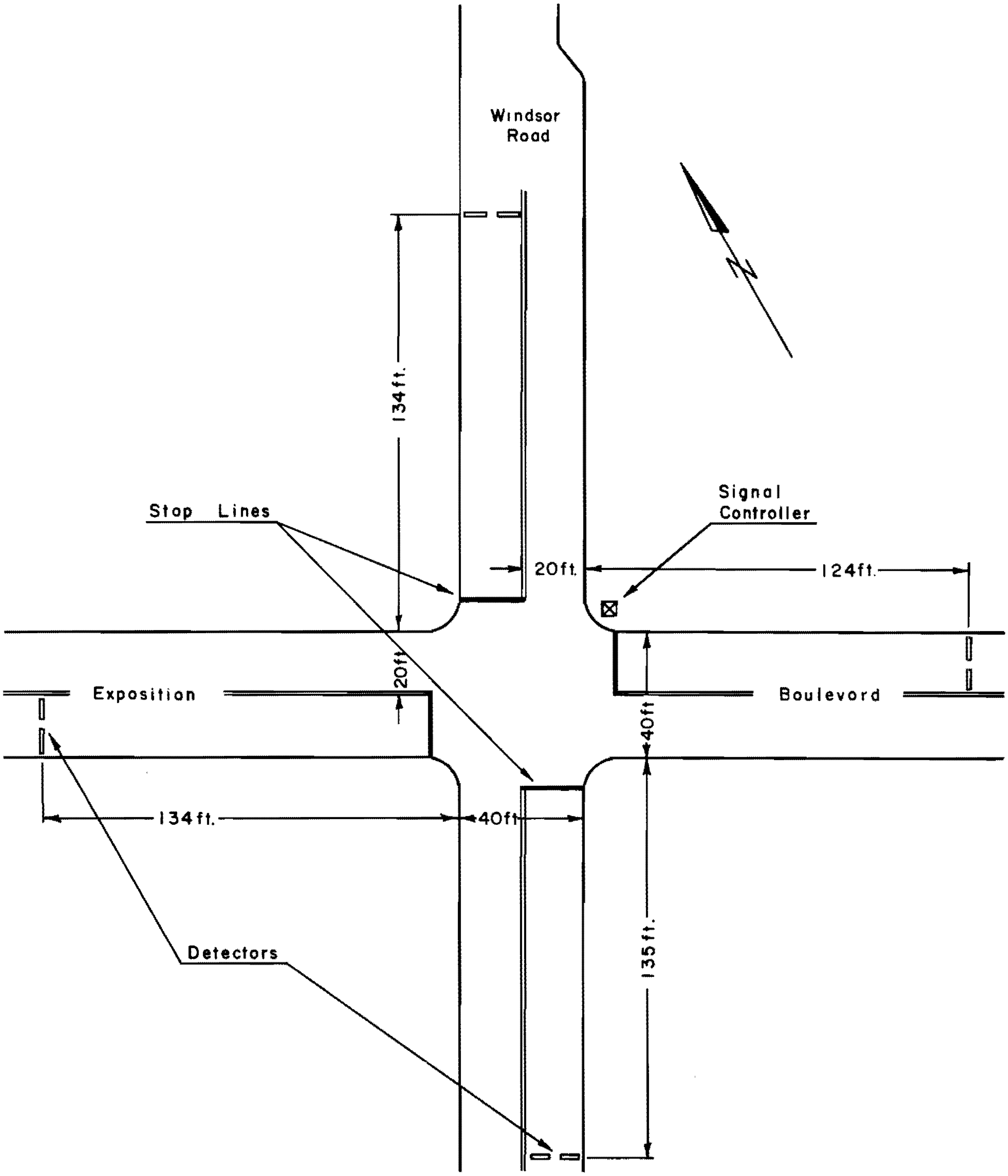

Fig A.10. Geometric layout of Exposition and Windsor intersection (signalized).

TABLE A.12. 15-MINUTE VOLUME - EXPOSITION AND WINDSOR

Intersection	Type of Control	Date	Time
Exposition and Windsor	Full-Actuated	July 27, 1966	0715 - 0845

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0730	2	55	23	80	17	68	3	88	4	29	5	38	8	7	5	20	226
0745	4	101	27	132	18	60	8	86	8	38	10	56	10	12	5	27	301
0800	0	103	33	136	15	80	2	97	4	46	3	53	14	13	20	47	333
0815	3	52	27	92	16	50	5	71	5	52	3	60	12	15	12	39	262
0830	2	34	21	57	25	54	7	86	6	25	2	33	10	21	11	42	218
0845	2	44	14	60	19	45	7	71	4	29	8	41	11	16	8	35	207

Intersection	Type of Control	Date	Time
Exposition and Windsor	Full-Actuated	July 21, 1966	1330 - 1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	1	41	14	56	19	49	10	78	10	25	5	40	12	22	21	55	229
1400	0	46	8	54	20	39	12	71	9	20	2	31	12	20	14	46	202
1415	3	43	16	62	13	49	6	68	9	23	3	35	12	25	15	52	217
1430	3	59	11	73	12	45	9	66	3	22	2	27	15	14	15	44	210
1445	1	51	11	63	13	33	4	50	5	15	2	22	18	17	7	42	177
1500	1	37	8	46	7	31	6	44	9	12	2	23	16	16	14	46	159
1515	6	38	6	50	18	44	18	80	8	19	5	32	18	21	18	57	219
1530	4	38	10	52	10	55	6	71	10	9	4	23	9	22	14	45	191

(Continued)

TABLE A.12. (CONTINUED)

Intersection	Type of Control	Date	Time
Exposition and Windsor	Full-Actuated	July 21, 1966	1630 - 1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	1	73	12	86	18	73	9	100	9	24	8	41	24	46	21	91	318
1700	5	93	17	115	7	38	1	133	7	31	5	43	24	46	25	95	386
1715	5	91	19	115	11	164	10	185	14	24	4	42	51	59	25	135	477
1730	7	106	18	131	20	77	13	110	12	24	2	38	37	54	32	123	402
1745	5	85	26	116	14	93	10	117	9	24	9	42	25	64	25	114	389
1800	1	65	12	78	8	95	14	117	12	16	10	38	22	48	18	88	321
1815	11	83	14	108	12	94	13	119	5	19	2	26	19	44	21	84	337
1830	4	48	14	66	19	53	11	83	4	23	4	31	17	35	11	63	243

Intersection	Type of Control	Date	Time
Exposition and Windsor	Semiactuated	July 26, 1966	0715 - 0915

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0730	0	52	20	72	12	48	4	64	6	21	2	29	4	7	2	13	178
0745	2	106	30	138	17	79	1	97	6	35	7	48	6	9	4	19	302
0800	3	122	33	158	31	66	4	101	6	53	5	64	11	16	8	35	358
0815	1	56	27	84	14	66	7	87	7	44	1	52	13	11	12	36	259
0830	3	44	18	65	17	71	4	92	5	40	5	50	7	11	11	29	236
0845	6	53	14	73	13	57	1	71	7	31	4	42	8	19	8	35	221
0900	1	51	12	64	11	46	6	63	8	43	7	58	8	22	12	42	227
0915	0	44	10	54	16	23	7	46	6	31	6	43	9	20	4	33	176

(Continued)

TABLE A.12. (CONTINUED)

Intersection	Type of Control	Date	Time
Exposition and Windsor	Semiactuated	July 26, 1966	1330 - 1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	2	49	12	63	19	44	6	69	12	24	2	38	10	15	12	37	207
1400	0	48	16	64	16	41	7	64	6	15	2	23	12	28	21	61	212
1415	1	46	9	56	15	32	7	54	11	20	3	34	18	21	18	57	201
1430	1	36	11	48	16	38	0	54	3	20	2	25	13	13	17	43	170
1445	3	46	11	60	17	46	7	70	8	23	4	34	14	22	17	53	217
1500	2	46	9	57	21	38	3	62	7	19	5	31	11	21	12	42	192
1515	7	42	14	63	16	46	8	70	9	15	7	31	17	27	13	57	221
1530	3	27	12	42	7	32	7	46	9	18	3	30	18	21	15	44	172

Intersection	Type of Control	Date	Time
Exposition and Windsor	Semiactuated	July 26, 1966	1630 - 1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	2	71	10	83	15	54	6	75	12	30	6	48	14	24	20	58	264
1700	3	71	14	88	13	72	9	94	8	20	10	38	18	37	15	70	290
1715	4	109	21	134	14	139	8	161	10	20	6	36	24	38	12	74	405
1730	7	100	11	118	15	100	20	135	13	29	7	49	35	60	29	124	426
1745	2	72	8	82	13	56	14	83	8	15	8	31	26	68	24	118	314
1800	8	93	19	120	16	61	4	81	7	26	1	34	31	63	18	112	347
1815	3	68	13	84	6	51	1	58	7	16	2	25	19	68	13	100	267
1830	2	57	17	76	2	58	6	66	11	27	5	43	21	32	9	62	247

(Continued)

TABLE A.12. (CONTINUED)

Intersection	Type of Control	Date	Time
Exposition and Windsor	Pretimed	Aug. 4, 1966	0700 - 0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0715	1	30	9	40	4	43	2	49	5	12	2	19	6	4	2	12	120
0730	1	64	13	78	11	48	2	61	4	18	4	26	4	5	3	12	177
0745	1	112	38	151	23	70	5	98	11	43	5	59	14	9	8	31	339
0800	1	107	33	141	28	79	3	110	8	46	3	57	11	22	13	46	354
0815	2	39	34	75	21	56	4	81	6	36	3	45	12	11	7	30	231
0830	5	46	19	70	25	46	5	76	8	34	2	44	13	18	12	43	233
0845	2	42	8	52	11	40	4	55	5	26	3	34	7	16	8	31	172
0900	2	33	17	52	13	43	3	59	7	30	6	43	15	15	11	41	195

Intersection	Type of Control	Date	Time
Exposition and Windsor	Pretimed	July 25, 1966	1330 - 1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	4	43	11	58	16	44	11	71	14	23	5	42	15	22	23	60	231
1400	5	43	11	59	16	33	7	56	9	26	4	39	9	22	20	51	205
1415	2	52	10	64	26	45	11	82	15	25	4	44	17	25	22	64	254
1430	5	53	9	67	19	34	10	63	15	27	4	46	12	22	19	53	229
1445	6	39	4	49	21	34	4	59	5	28	2	35	13	20	17	50	193
1500	3	54	13	70	16	47	9	72	10	21	6	37	10	22	16	48	227
1515	5	46	10	61	12	40	11	53	5	26	4	35	10	24	23	57	206
1530	0	40	6	46	17	33	6	56	7	22	3	32	14	28	15	57	191

(Continued)

TABLE A.12. (CONTINUED)

Intersection	Type of Control	Date	Time
Exposition and Windsor	Pretimed	July 25, 1966	1630 - 1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	2	55	13	70	21	69	10	100	12	18	7	37	18	28	28	74	281
1700	3	81	23	107	20	91	10	121	10	21	4	35	23	33	21	77	340
1715	3	95	21	117	21	151	14	186	6	18	7	31	39	54	31	124	458
1730	7	101	17	125	20	88	14	122	14	25	10	49	32	62	22	116	412
1745	3	82	11	96	13	55	11	79	8	24	2	34	35	54	24	113	322
1800	4	77	20	101	8	55	8	71	11	25	1	37	30	42	17	89	298
1815	4	59	13	76	20	70	9	99	11	21	3	35	25	34	22	81	291
1830	3	62	13	78	17	54	18	79	13	14	1	28	14	28	13	55	240

Intersection	Type of Control	Date	Time
South 1st and Oltorf	Full-Actuated	July 29, 1966	0700 - 0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0715	6	86	15	107	14	32	4	55	13	52	11	76	7	25	6	38	276
0730	8	120	9	137	7	39	4	50	26	64	8	98	8	33	12	53	338
0745	6	140	17	163	11	40	5	56	31	78	3	112	3	35	13	51	382
0800	13	126	14	153	9	33	6	48	36	115	6	157	6	43	14	63	421
0815	8	63	10	81	14	32	7	53	12	50	7	69	4	31	17	52	255
0830	2	45	9	56	10	20	7	37	13	44	8	65	5	24	10	39	197
0845	4	50	9	63	11	22	6	39	8	5[illegible]	6	69	4	27	9	40	211
0900	1	41	9	51	13	24	5	42	11	57	8	76	6	24	20	50	219

TABLE A.13. SIGNAL CONTROLLER SETTINGS FOR EXPOSITION AND WINDSOR

Date	Time	Full-Actuated			
		Exposition		Windsor	
July 27	0715 - 0845	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 3 off 60 4	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 3 on 60 4
July 21	1330 - 1530	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 5 off 55 4	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 3 on 55 4
July 21	1630 - 1830	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 6 off 52 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 3 on 52 4

Date	Time	Semiactuated			
		Exposition		Windsor	
July 26	0715 - 0915	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 18 4	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 3 off 60 3
July 26	1330 - 1530	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 18 4	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 3 off 60 3
July 26	1630 - 1830	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 18 4	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 3 off 60 3

TABLE A.13. (CONTINUED)

Date	Time	Pretimed			
		Exposition		Windsor	
Aug 4	0700 - 0900	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 18 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 17 3
July 25	1330 - 1530	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 18 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 17 3
July 25	1630 - 1830	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 18 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 17 3

Date	Time	Full-Actuated			
		South First		Oltorf	
July 29	0700 - 0900	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	10 6 off 50 5	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	9 7 off 50 5
July 14	1330 - 1530	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	10 6 off 50 5	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	9 7 off 50 5
July 14	1630 - 1800	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	10 6 off 50 5	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	9 7 off 50 5

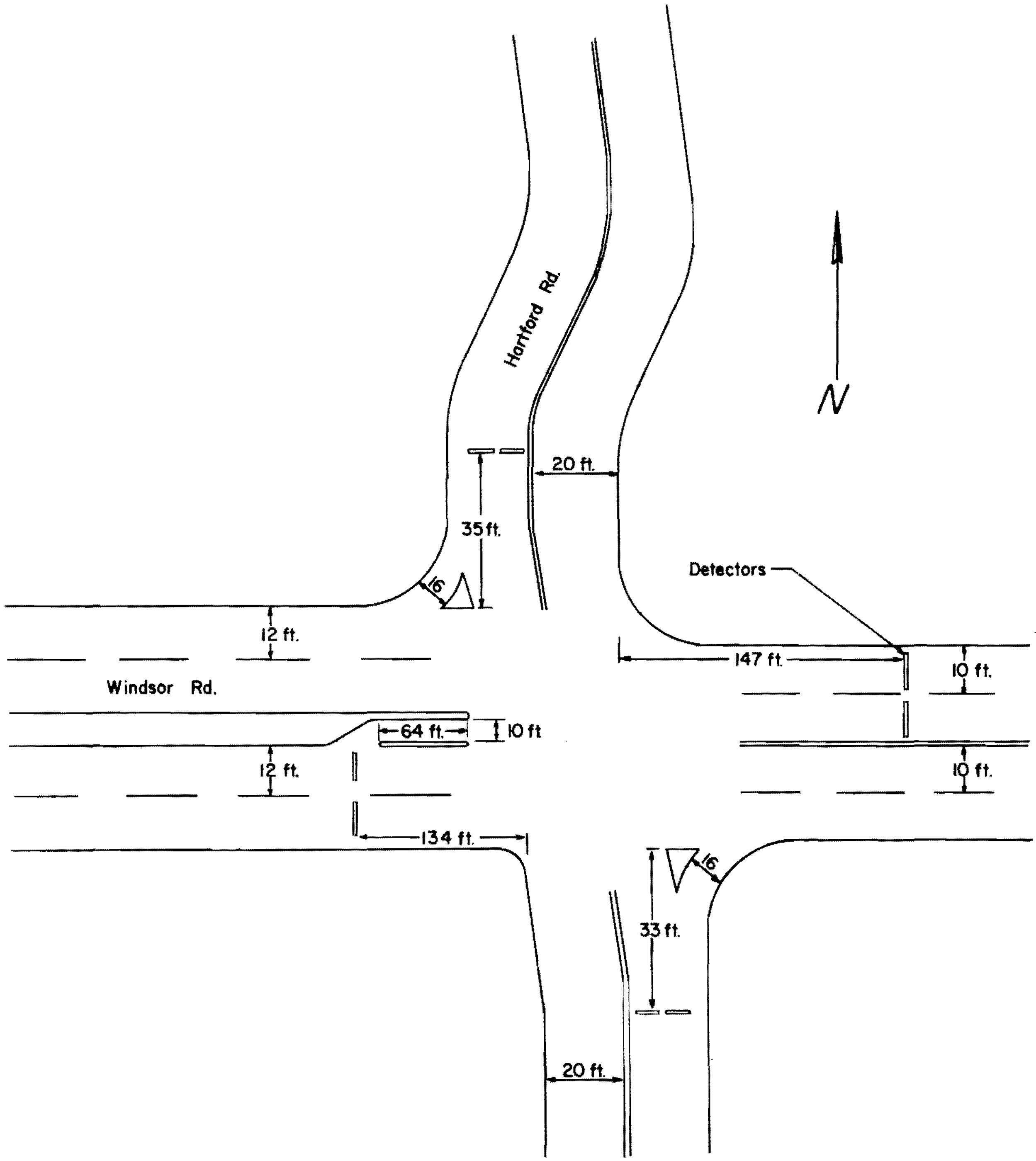

Fig A.11. Geometric layout of Hartford and Windsor intersection (signalized).

TABLE A.14. 15-MINUTE VOLUME - HARTFORD AND WINDSOR

Intersection	Type of Control	Date	Time
Hartford and Windsor	Full-Actuated	June 21, 1967	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	3	38	16	67	37	56	1	94	12	172	8	192	0	24	8	22	375
0800	1	47	21	69	51	75	4	130	71	185	7	199	1	41	21	63	461
0815	2	24	10	36	34	42	4	80	3	143	9	155	2	58	11	71	343
0830	5	19	9	33	29	38	4	71	7	118	10	135	3	62	7	72	311
0845	5	20	5	30	27	42	6	75	11	89	7	107	5	46	8	59	271
0900	1	19	7	27	23	26	2	51	3	85	10	98	2	41	10	53	229

Intersection	Type of Control	Date	Time
			1330-1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	8	19	1	28	13	16	10	39	6	54	12	72	3	51	9	63	202
1400	6	21	4	31	11	31	8	50	8	54	12	74	0	52	12	64	219
1415	6	16	2	24	10	10	6	26	7	65	7	79	5	59	8	72	201
1430	2	7	1	10	10	50	14	74	6	70	11	87	1	59	9	69	240
1445	7	20	1	28	6	37	5	48	7	79	9	95	3	40	3	46	217
1500	3	21	3	27	8	37	9	54	7	44	7	58	4	54	9	67	206
1515	10	13	3	26	9	19	3	31	8	59	8	73	2	70	5	77	209
1530	4	12	0	16	5	14	7	26	6	50	5	61	0	52	8	60	163

(Continued)

TABLE A.14. (CONTINUED)

Intersection	Type of Control	Date	Time
Hartford and Windsor	Full-Actuated	June 21, 1967	1630-1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	11	32	6	49	17	35	10	62	7	60	6	73	3	95	22	120	304
1700	12	37	4	53	16	40	5	61	10	68	10	88	8	89	27	124	326
1715	4	75	3	82	20	51	17	88	11	51	8	70	5	150	49	204	444
1730	12	59	4	75	16	34	17	67	15	45	8	68	5	178	37	220	430
1745	19	39	7	65	11	27	11	49	3	36	4	43	7	120	25	152	318
1800	7	41	3	51	7	31	16	54	4	64	3	71	8	102	16	126	302
1815	8	37	0	45	1	36	18	55	4	34	6	44	1	69	18	88	232
1830	7	19	5	31	7	3	7	51	11	51	9	71	2	58	14	74	227

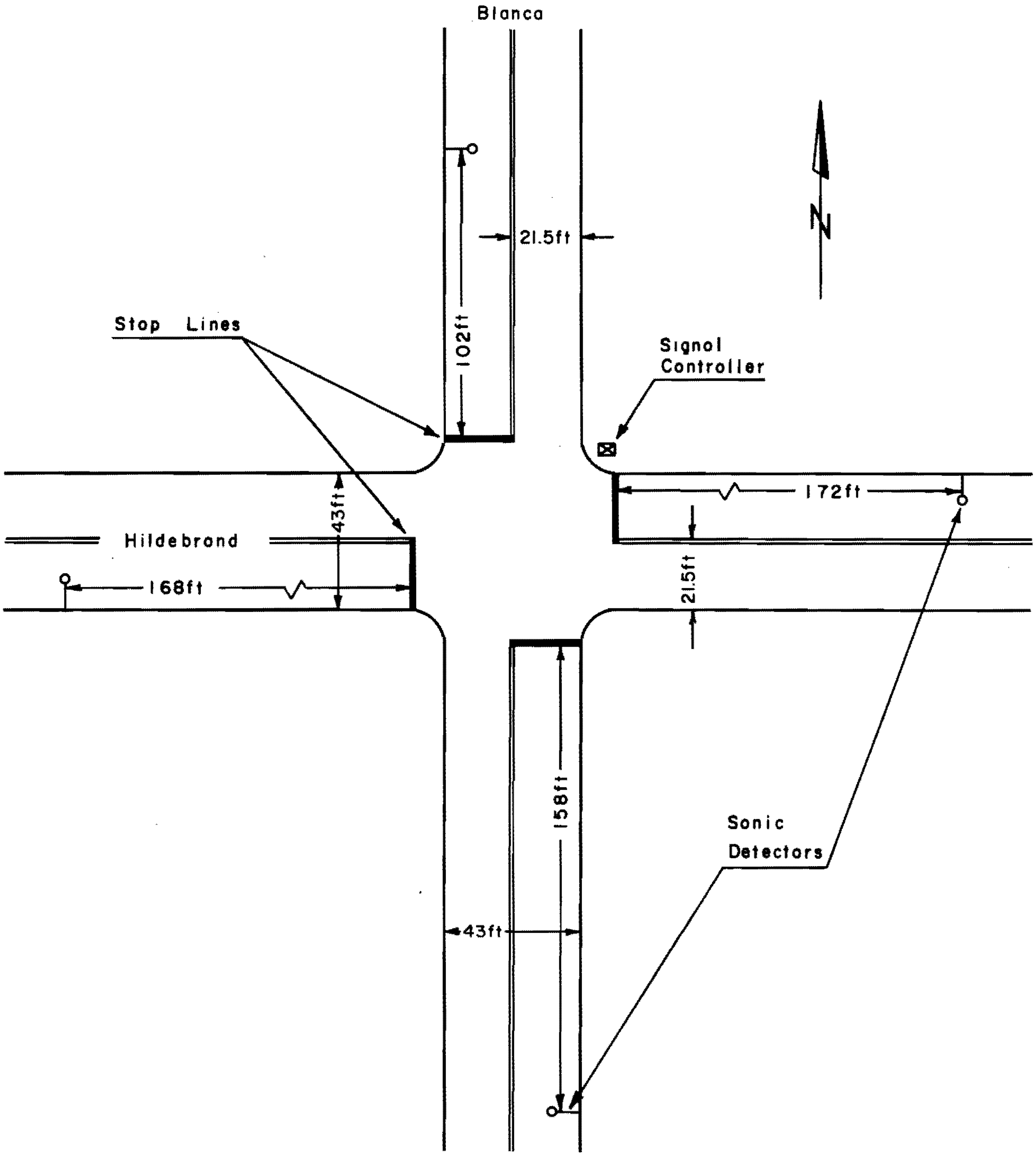

Fig A.12. Geometric layout of Hildebrand and Blanco intersection (signalized).

TABLE A.15. 15-MINUTE VOLUME - HILDEBRAND AND BLANCO

Intersection	Type of Control	Date	Time
Hildebrand and Blanco	Full-Actuated	Aug. 23, 1966	0800 - 0930

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
815	5	57	8	70	11	94	20	125	28	124	11	163	5	68	7	80	438
830	7	48	11	66	10	88	23	121	18	138	6	162	4	66	7	77	426
845	4	48	7	59	12	62	18	92	17	113	8	138	6	69	5	80	369
900	11	51	16	78	14	62	21	97	27	108	4	139	9	63	19	91	405
915	9	47	9	65	12	51	27	90	18	94	8	120	11	68	9	88	363
930	6	63	9	78	13	48	17	78	21	99	10	130	6	73	12	91	377

Intersection	Type of Control	Date	Time
Hildebrand and Blanco	Full-Actuated	Aug. 23, 1966	1330 - 1515

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1330																	
1345	15	42	11	68	12	54	24	90	29	88	15	132	11	103	4	118	408
1400	8	63	12	83	16	62	19	97	21	89	8	118	6	90	12	108	406
1415	13	55	11	79	20	50	22	92	20	102	16	138	5	97	10	112	421
1430	8	37	8	53	15	54	22	91	20	99	13	132	7	104	16	127	403
1445	7	33	7	47	6	38	16	60	19	88	10	117	5	92	17	117	341
1500	11	45	5	61	9	41	18	68	15	87	9	111	5	121	5	141	381
1515	12	40	12	64	11	44	34	89	19	102	9	130	8	123	17	148	431

(Continued)

TABLE A.15. (CONTINUED)

Intersection	Type of Control	Date	Time
Hildebrand and Blanco	Full-Actuated	Aug. 23, 1966	1600 - 1800

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1615	18	52	7	77	10	57	22	89	18	102	9	129	13	139	23	175	470
1630	11	98	10	119	12	65	25	102	21	105	13	139	6	146	19	171	531
1645	10	89	9	108	11	86	27	124	15	108	10	133	7	190	18	215	580
1700	14	105	7	126	12	84	37	133	17	86	17	120	18	208	21	247	626
1715	13	123	5	141	9	69	38	116	14	112	10	136	10	196	33	233	626
1730	19	187	8	214	10	77	33	120	23	118	13	154	11	148	23	184	672
1745	17	124	7	148	12	63	26	102	15	88	14	117	16	160	23	199	566
1800	10	120	10	140	13	60	31	104	19	86	10	115	11	135	24	170	529

Intersection	Type of Control	Date	Time
Hildebrand and Blanco	Pretimed	Aug. 24, 1966	0700 - 0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
715	7	58	9	74	26	99	18	143	26	143	6	175	3	46	2	51	443
730	6	55	7	68	24	131	14	169	22	176	15	213	1	65	7	73	523
745	4	70	9	83	16	189	12	217	32	161	13	206	4	70	4	78	584
800	7	76	12	95	13	151	15	179	36	182	10	228	10	71	13	94	596
815	10	41	12	63	14	104	11	129	36	126	9	171	3	64	8	75	438
830	3	45	9	57	14	63	20	97	28	127	7	162	5	53	11	69	385
845	12	64	3	79	18	70	19	107	45	111	12	168	4	87	15	106	460
900	10	69	11	90	9	83	16	108	42	111	11	164	6	65	18	89	451

(Continued)

TABLE A.15. (CONTINUED)

Intersection	Type of Control	Date	Time
Hildebrand and Blanco	Pretimed	Aug. 24, 1967	1330 - 1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	11	57	12	80	9	76	34	118	20	99	9	128	9	97	9	115	441
1400	8	43	5	56	12	46	16	75	15	80	12	107	8	90	9	107	345
1415	8	50	6	64	13	54	19	86	16	72	13	101	6	98	10	114	365
1430	10	49	12	71	11	44	16	71	21	79	16	116	10	87	13	110	368
1445	10	48	13	71	14	60	18	92	10	95	9	114	7	101	13	121	398
1500	6	71	13	80	9	62	24	95	17	100	6	123	13	116	18	147	445
1515	14	44	15	73	6	50	39	95	26	100	7	133	6	115	13	134	435
1530	4	46	5	55	10	63	16	89	18	93	7	118	9	94	19	122	384

Intersection	Type of Control	Date	Time
Hildebrand and Blanco	Pretimed	Aug. 24, 1967	1600 - 1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1615	6	74	8	88	9	47	26	82	21	101	13	135	14	130	15	159	464
1630	12	88	11	111	12	65	35	112	15	112	7	134	13	139	29	181	538
1645	12	89	12	113	10	85	40	135	16	108	13	137	8	190	14	217	602
1745																	
1800	12	115	14	141	9	80	27	116	18	95	10	123	10	122	16	148	528
1815	18	63	14	95	6	58	18	82	16	76	13	105	9	147	20	176	458
1830	10	72	10	92	13	38	15	66	17	83	9	109	12	109	18	139	406

TABLE A.15. (CONTINUED)

Date	Time	Full-Actuated			
		Hildebrand		Blanco	
Aug 23	0800 - 0930	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	5 4 off 40 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	5 4 off 40 3
Aug 23	1330 - 1525	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	5 4 off 40 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	5 4 off 40 3
Aug 23	1600 - 1800	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	5 4 off 40 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	5 4 off 40 3

Date	Time	Pretimed			
		Hildebrand		Blanco	
Aug 24	0700 - 0900	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 24 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 30 3
Aug 24	1330 - 1530	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 24 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 30 3
Aug 24	1745 - 1830	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 24 3	Initial Interval Vehicle Interval Recall Switch Maximum Interval Clearance Interval	6 30 off 30 3

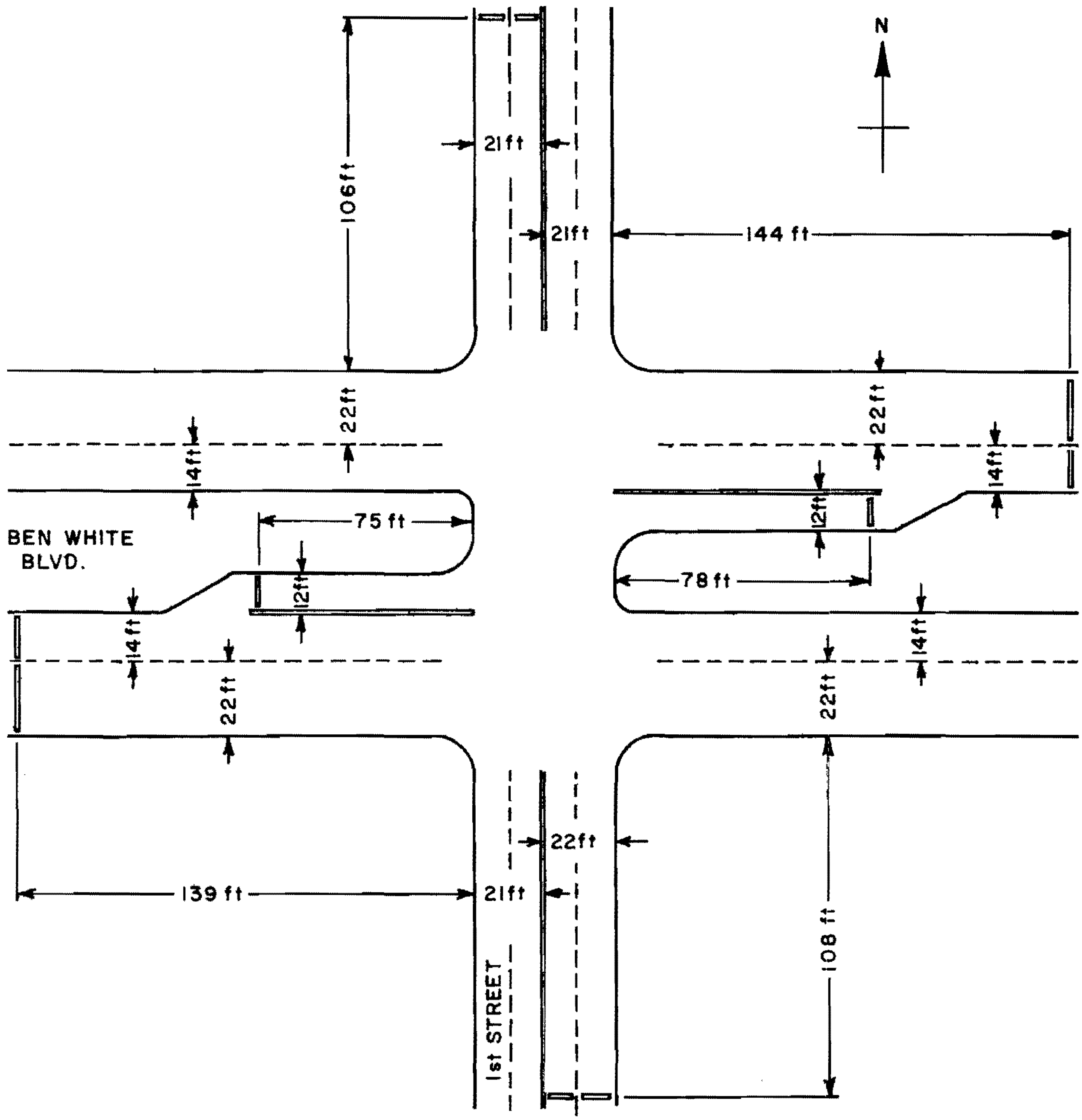

Fig A.13. Geometric layout of South First and Ben White intersection (signalized).

TABLE A.16. 15-MINUTE VOLUME - SOUTH FIRST AND BEN WHITE

Intersection	Type of Control	Date	Time
South 1st and Ben White	Volume Density	Aug. 5, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	7	50	27	84	7	10	6	23	49	199	3	251	14	72	10	96	454
0800	5	33	19	57	12	11	8	31	35	123	2	160	15	60	9	84	332
0815	5	22	10	37	10	17	10	37	25	111	3	139	12	74	12	98	311
0830	5	18	4	27	5	13	3	21	16	92	3	111	6	60	7	73	232
0845	8	25	12	45	5	7	8	20	18	83	2	103	9	55	11	75	243
0900	10	11	6	27	5	8	7	20	18	68	3	89	10	60	5	75	211

Intersection	Type of Control	Date	Time
			1330-1530

Time	Northbound				Southbound				Eastbound				Westbound				
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Total
1345	5	18	7	30	4	14	12	30	20	85	6	111	8	83	9	100	271
1400	5	14	10	29	7	18	13	38	13	71	5	89	3	69	9	81	237
1415	1	21	9	31	12	17	5	34	13	76	4	93	8	80	6	94	252
1430	1	14	7	22	12	23	14	49	18	67	5	90	10	79	3	92	253
1445	3	10	4	17	9	13	12	34	11	79	1	91	12	73	3	88	230
1500	7	19	5	31	7	23	15	45	13	73	5	91	8	90	12	110	277
1515	2	9	6	17	11	15	15	41	11	70	7	88	6	84	9	99	245
1530	6	18	4	28	11	22	17	50	15	63	3	81	10	83	12	105	264

(Continued)

TABLE A.16. (CONTINUED)

Intersection	Type of Control	Date	Time
South 1st and Ben White	Volume Density	Aug. 5, 1966	1700-1900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1715	5	20	7	32	12	56	41	109	17	76	10	103	21	202	16	239	483
1730	4	16	5	25	19	65	82	166	14	84	7	105	22	244	27	293	589
1745	4	13	4	21	13	50	35	98	14	74	10	98	21	135	15	171	388
1800	6	30	3	39	10	38	30	78	20	81	6	107	20	123	18	161	385
1815	6	15	5	26	8	30	35	73	8	49	3	60	15	89	10	114	273
1830	6	18	5	29	13	23	21	57	15	37	3	55	21	85	18	124	265
1845	5	17	6	28	5	20	12	37	18	53	8	79	22	101	7	130	274
1900	4	17	10	31	12	29	10	51	7	58	10	75	8	91	6	105	190

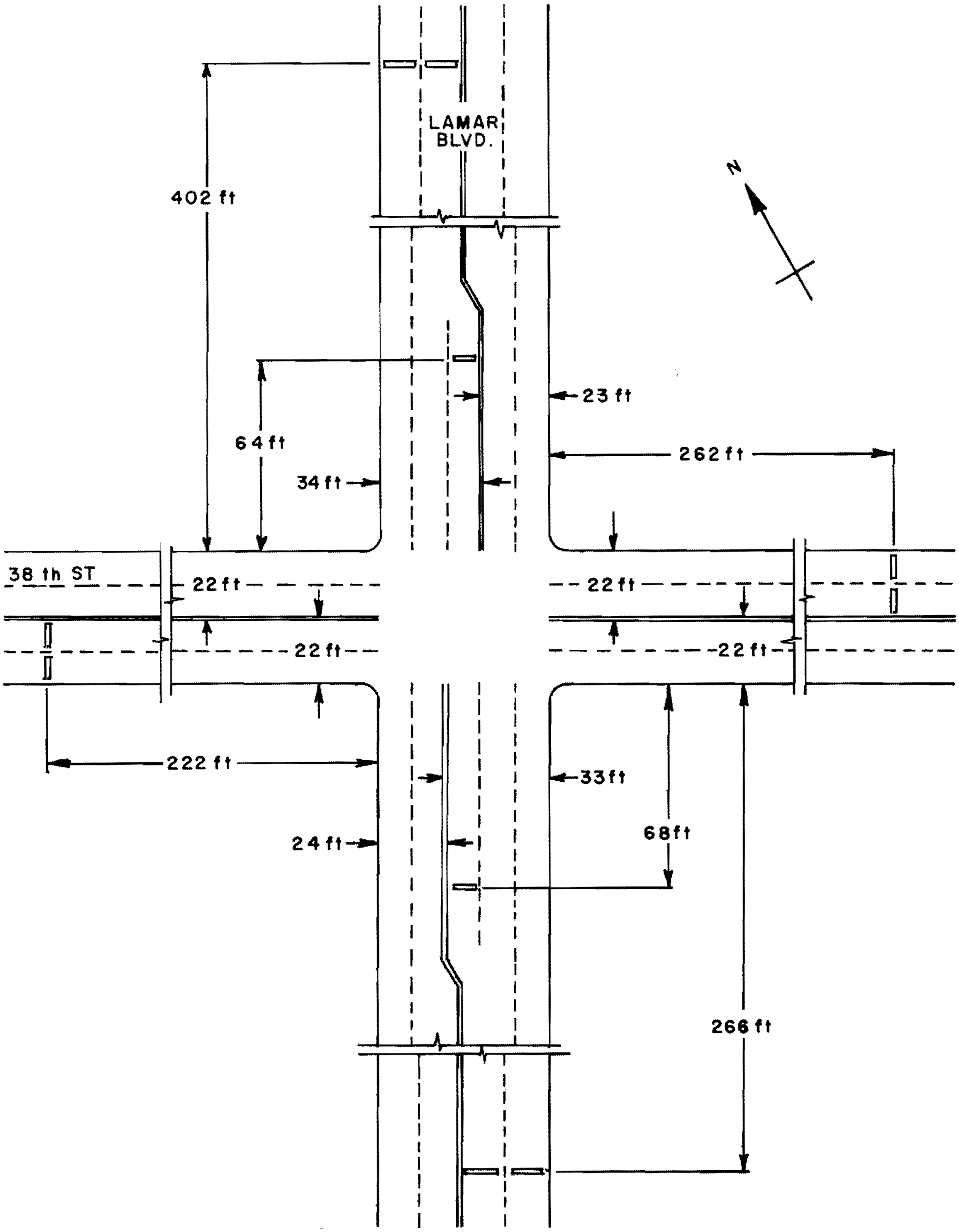

Fig A.14. Geometric layout of 38th and Lamar intersection (signalized).

TABLE A.17. 15-MINUTE VOLUME - 38TH AND LAMAR

Intersection	Type of Control	Date	Time
38th and Lamar	Volume Density	Aug. 10, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	15	87	9	111	28	300	23	351	17	73	38	128	10	103	10	123	713
0800	25	107	16	148	22	272	22	316	13	110	44	167	7	90	17	114	745
0815	12	99	10	121	35	204	4	243	7	92	33	132	8	46	9	63	559
0830	17	82	12	111	6	152	13	171	7	71	35	113	8	43	8	59	454
0845	12	94	8	114	19	106	6	131	5	68	19	92	6	37	9	52	389
0900	10	71	10	91	9	102	9	120	4	58	22	84	9	39	8	56	351

Intersection	Type of Control	Date	Time
			1330-1530

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1345	17	113	15	145	21	104	22	147	19	37	34	90	4	57	13	74	456
1400	21	118	13	152	31	109	17	157	15	66	23	104	14	45	11	70	483
1415	15	113	11	159	15	112	11	138	10	43	18	71	7	43	8	58	426
1430	20	141	14	175	15	102	18	135	8	29	24	61	9	45	18	72	443
1445	16	134	13	163	19	126	14	159	9	43	25	77	18	56	16	90	489
1500	30	142	15	187	12	118	8	138	11	41	20	72	7	42	13	62	459
1515	23	143	6	172	14	116	9	139	13	63	6	82	10	45	18	73	466
1530	27	153	7	187	23	100	10	133	6	37	19	62	19	60	21	100	482

(Continued)

TABLE A.17. (CONTINUED)

Intersection	Type of Control	Date	Time
38th and Lamar	Volume Density	Aug. 10, 1966	1630-1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	38	224	13	275	20	125	26	171	21	52	24	97	14	66	21	101	649
1700	38	266	11	315	18	127	15	160	12	95	22	129	9	84	16	109	713
1715	58	340	13	411	18	162	23	203	9	85	17	111	11	109	19	139	864
1730	49	329	9	387	14	108	20	142	5	52	20	77	12	149	24	185	791
1745	62	253	19	334	16	112	23	151	12	40	18	70	10	82	15	107	662
1800	45	177	13	235	30	89	13	132	12	45	16	73	10	81	17	108	548
1815	41	139	12	192	20	116	14	150	14	36	15	65	12	76	9	97	504
1830	49	182	14	245	13	94	16	123	9	44	16	69	13	54	10	77	514

TABLE A.17. (CONTINUED)

Intersection	Type of Control	Date	Time
38th and Lamar	Volume Density	Aug. 9, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	13	115	11	139	31	336	25	392	15	77	38	130	7	86	11	104	765
0800	19	111	13	143	22	212	23	257	23	93	51	167	3	95	19	117	684
0815	11	109	8	128	22	179	15	216	11	100	51	162	3	57	9	69	575
0830	13	95	10	118	17	159	10	186	11	52	43	106	9	39	9	57	467
0845	20	101	4	125	14	138	6	158	9	37	30	76	6	44	10	60	419
0900	12	76	12	100	18	158	11	187	10	53	34	97	4	41	4	49	433

Intersection	Type of Control	Date	Time
			1335-1535

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1350	28	118	13	159	15	141	12	168	8	63	20	91	16	34	16	66	484
1405	18	130	17	165	12	126	9	147	17	44	23	84	11	24	16	61	457
1420	14	142	11	167	13	105	10	128	18	42	23	83	13	46	18	77	455
1435	26	126	20	172	20	122	8	150	13	48	19	80	11	41	10	62	464
1450	41	156	16	213	15	129	9	153	11	38	16	65	11	58	15	84	515
1505	20	98	11	129	13	111	10	134	11	56	30	97	14	37	11	62	422
1520	26	122	5	153	17	158	15	190	12	58	32	102	14	40	22	76	521
1535	14	184	10	218	13	101	14	128	20	31	25	76	16	54	14	84	506

(Continued)

TABLE A.17. (CONTINUED)

Intersection	Type of Control	Date	Time
38th and Lamar	Volume Density	Aug. 9, 1966	1630-1815

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	40	212	14	266	22	147	16	185	12	58	24	94	13	65	15	93	638
1700	46	228	8	282	24	113	20	157	11	78	19	108	8	102	20	130	677
1715	51	346	13	410	22	173	33	228	10	93	16	119	10	130	20	160	917
1730	47	349	13	409	15	108	22	145	12	54	17	83	7	123	23	153	790
1745	54	244	19	317	17	112	16	145	6	59	16	81	10	86	18	114	657
1800	41	215	7	263	17	112	28	157	14	29	13	56	14	75	12	101	577
1815	32	176	14	222	20	98	20	138	8	28	22	58	13	74	14	101	519

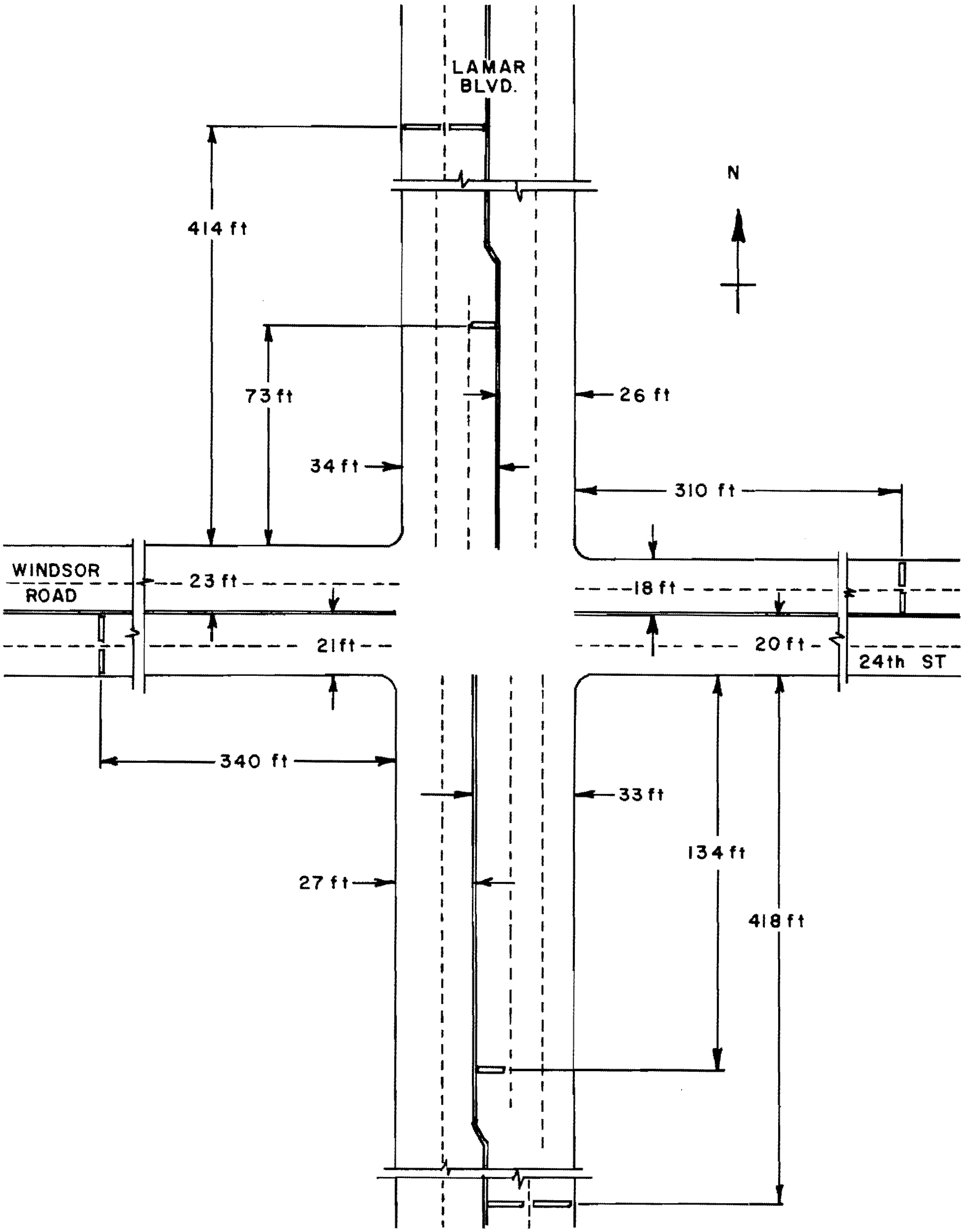

Fig A.15. Geometric layout of 24th and Lamar intersection (signalized).

TABLE A.18. 15-MINUTE VOLUME - 24TH AND LAMAR

Intersection	Type of Control	Date	Time
24th and Lamar	Volume Density	Aug. 17, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	17	117	2	136	1	268	6	275	9	87	112	208	6	17	7	30	649
0800	28	127	3	158	6	250	5	261	14	157	158	329	3	27	6	36	784
0815	26	116	2	144	6	230	12	248	23	131	75	229	3	28	2	33	654
0830	22	117	3	142	4	167	4	175	11	132	99	242	3	47	10	60	619
0845	10	76	0	86	5	143	12	160	4	89	49	142	2	28	4	34	422
0900	23	76	3	102	5	137	6	148	5	88	51	144	6	25	11	42	436

Intersection	Type of Control	Date	Time
			1330-1530

Time	Northbound				Southbound				Eastbound				Westbound				
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Total
1345	34	113	4	151	4	137	7	148	9	50	36	95	8	38	6	52	446
1400	38	107	3	148	9	140	10	159	8	44	46	98	7	31	9	47	452
1415	23	123	4	150	11	129	14	154	13	50	44	107	11	35	7	53	464
1430	32	100	2	134	6	111	9	126	11	47	29	87	9	43	7	59	406
1445	28	118	4	150	11	135	11	157	3	55	22	80	8	27	9	44	431
1500	12	123	2	137	5	116	12	133	7	45	24	76	9	38	9	56	402
1515	39	155	2	196	9	115	10	134	6	50	23	79	10	51	3	64	473
1530	26	115	3	144	3	122	14	139	12	32	23	77	9	37	6	52	412

(Continued)

TABLE A.18. (CONTINUED)

Intersection	Type of Control	Date	Time
24th and Lamar	Volume Density	Aug. 17, 1966	1630-1800

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	66	187	2	255	4	169	13	186	4	52	37	93	18	72	14	104	638
1700	64	267	4	335	3	143	10	156	7	46	38	97	5	99	9	113	695
1715	113	359	1	473	4	194	26	224	5	47	33	85	21	127	15	163	945
1730	89	276	1	366	4	175	19	198	13	35	24	72	22	160	13	195	831
1745	66	224	1	290	6	120	23	149	15	45	35	95	21	118	7	146	680
1800	56	170	2	228	5	100	17	122	16	38	15	69	15	61	9	85	504

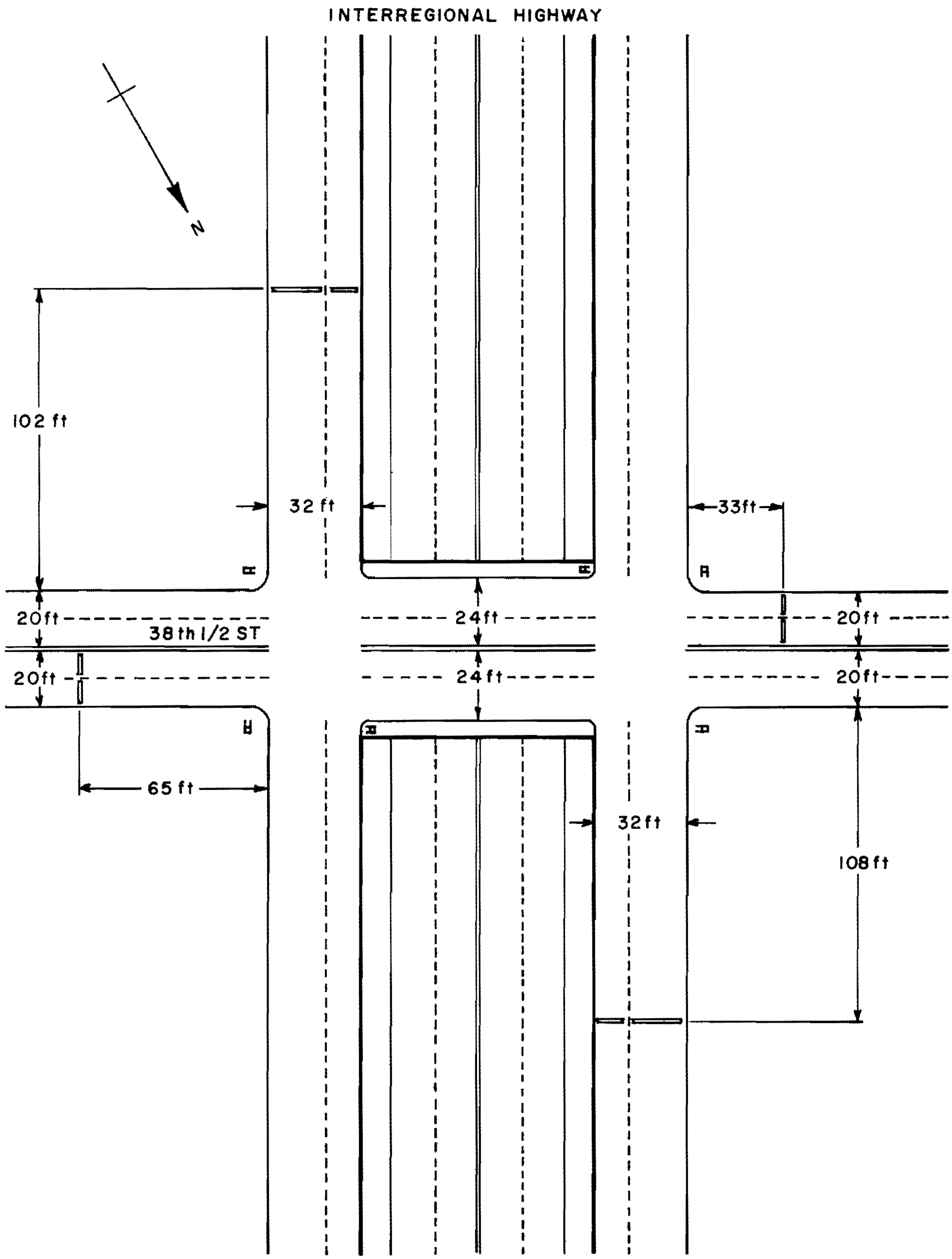

Fig A.16. Geometric layout of 38-1/2 and Interregional intersection (signalized).

TABLE A.19. 15-MINUTE VOLUME - 38-1/2 AND INTERREGIONAL

Intersection	Type of Control	Date	Time
38-1/2 and Interregional	Actuated and Diamond Interchange	Aug. 16, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745	57	13	7	77	17	118	7	142	0	40	7	47	0	98	10	108	374
0800	52	16	4	72	24	74	10	108	0	43	8	51	0	99	10	109	340
0815	34	20	7	61	25	52	9	86	0	39	9	48	0	58	5	63	258
0830	26	15	10	51	16	44	9	69	0	25	11	36	0	48	6	54	210
0845	33	23	5	61	20	26	6	52	0	23	10	33	0	48	6	54	200
0900	36	26	5	67	20	28	9	57	0	30	9	39	0	71	3	74	237

Intersection	Type of Control	Date	Time
			1425-1555

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1440	47	37	3	87	41	57	3	101		37	7	44		38	11	49	281
1455	45	28	4	77	42	52	6	100		38	6	44		35	8	43	264
1510	40	29	9	78	38	63	15	116		38	9	47		44	3	47	288
1525	32	35	8	75	48	59	8	115		68	7	75		31	8	39	304
1546	34	35	7	76	42	57	6	105		43	11	54		44	2	46	281
1555	36	31	7	74	44	64	7	115		35	15	50		46	6	52	288

(Continued)

TABLE A.19. (CONTINUED)

Intersection	Type of Control	Date	Time
38-1/2 and Interregional	Actuated and Diamond Interchange	Aug. 16, 1966	1630-1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645	45	38	10	93	56	59	12	127	0	51	12	63	0	61	14	75	358
1700	51	73	6	130	48	64	12	124	0	85	19	104	0	62	8	70	428
1715	57	145	7	209	66	64	11	141	0	83	13	96	0	65	15	80	526
1730	48	146	14	208	79	58	11	148	0	108	11	119	0	62	11	73	548
1745	36	44	11	91	88	61	15	164	0	75	12	87	0	48	11	59	401
1800	40	50	19	109	53	50	8	111	0	63	7	70	0	53	11	64	354
1815	34	49	8	91	90	50	11	151	0	59	12	71	0	59	17	76	389
1830	22	37	9	68	42	59	9	110	0	37	9	46	0	40	4	44	268

TABLE A.19. (CONTINUED)

Intersection	Type of Control	Date	Time
38-1/2 and Interregional (Over Bridge)	Actuated and Diamond Interchange	Aug. 16, 1966	0730-0900

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
0745									18	40		58	35	112		147	205
0800									18	46		64	37	102		139	203
0815									16	48		64	39	45		84	148
0830									16	26		42	30	54		84	126
0845									18	26		44	26	50		76	120
0900									22	22		44	39	68		107	151

Intersection	Type of Control	Date	Time
			1425-1555

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt UT	Tot	Lft	St	Rt UT	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1440			6				17		24	40		64	10	57		67	154
1455			3				21		24	37		61	26	44		70	155
1510			10				9		33	36		69	21	50		71	159
1525			3				23		24	72		96	14	43		57	179
1540			3				18		18	45		63	18	47		65	149
1555			9				18		21	38		59	18	56		74	160

(Continued)

TABLE A.19. (CONTINUED)

Intersection	Type of Control	Date	Time
38-1/2 and Interregional (Over Bridge)	Actuated and Diamond Interchange	Aug. 16, 1966	1630-1830

Time	Northbound				Southbound				Eastbound				Westbound				Total
	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	Lft	St	Rt	Tot	
1645									24	51	9	84	18	72	12	102	186
1700									37	83	15	135	22	73	7	102	237
1715									28	83	30	141	25	89	12	126	267
1730									41	103	15	159	18	113	13	144	303
1745									34	82	13	129	23	76	18	117	246
1800									46	58	9	113	21	68	4	93	206
1815									14	110	25	149	14	74	10	98	247
1830									9	56	19	84	9	58	11	78	162

APPENDIX B

PROPOSED WARRANTS FOR TRAFFIC-ACTUATED SIGNALS BY THE TEXAS HIGHWAY DEPARTMENT

APPENDIX B. HIGHWAY TRAFFIC SIGNALS

REVIEW OF WARRANTS FOR PRETIMED AND TRAFFIC-ACTUATED SIGNALS

The purpose of this discussion is to describe the procedure and guidelines used by File D-18T in reviewing traffic study data to determine if the installation of a traffic signal is justified. The method described has provided a good indicator in determining if the traffic volumes and accident conditions are such that a traffic signal could be warranted and is being submitted for general use by the Department.

It should be noted initially that the warrants and warrant factors used are minimum volumes and that a traffic signal may not be needed or even be desirable though the warrant volumes are met. Even when the warrants for a traffic signal are met, therefore, consideration should be given to such factors as

(1) traffic volume patterns and movements,

(2) approach speeds,

(3) accident conditions,

(4) intersection and approach conditions,

(5) sight-distance restrictions, and

(6) existing traffic control devices

to determine if there are less restrictive traffic control measures and/or improvements which can be installed that will provide a more efficient and safer operation at the intersection than that which can be obtained with a traffic signal. A traffic signal should be installed at an intersection only when the warrants or warrant factors are met and the results of the study show the operating conditions on a major street are such that the minor street suffers undue delay or hazard in entering or crossing the major street.

The vehicular volumes and the cross-traffic warrant factors are considered to be satisfied when, for each of any eight hours of the average day, the plotted points representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) all lie above the curve in Fig B.1 or Fig B.2 for the particular existing combination of approach lanes. The major-street and minor-street volumes are for the same eight hours. During those eight hours, the direction of higher traffic volume on the minor street may be on one approach during some hours, and on the opposite approach during other hours. The bottom of the three curves in Fig B.1 (and in Fig B.2) is applicable when both streets have one lane on each approach. The center curve of the three is to be used when one street has two or more lanes on each approach and the other street has one lane on each approach, irrespective of whether the major street or the minor street has the wider (more lanes) approach. The top curve of the three curves in Fig B.1 (and in Fig B.2) is to be used when both streets have two or more lanes on each approach.

When the 85-percentile speed of major-street traffic exceeds 40 miles per hour, or when the intersection lies within the built-up area of an isolated community having a population less than 10,000 (latest Federal Census), the curves in Fig B.2 are to be used. In all other cases, the curves in Fig B.1 are to be used.

It should be noted, as a matter of information, that when the plotted points representing the vehicles per hour on the major street and the minor street given in warrants 1, 2, and 6 are plotted on Figs B.1 and B.2 these

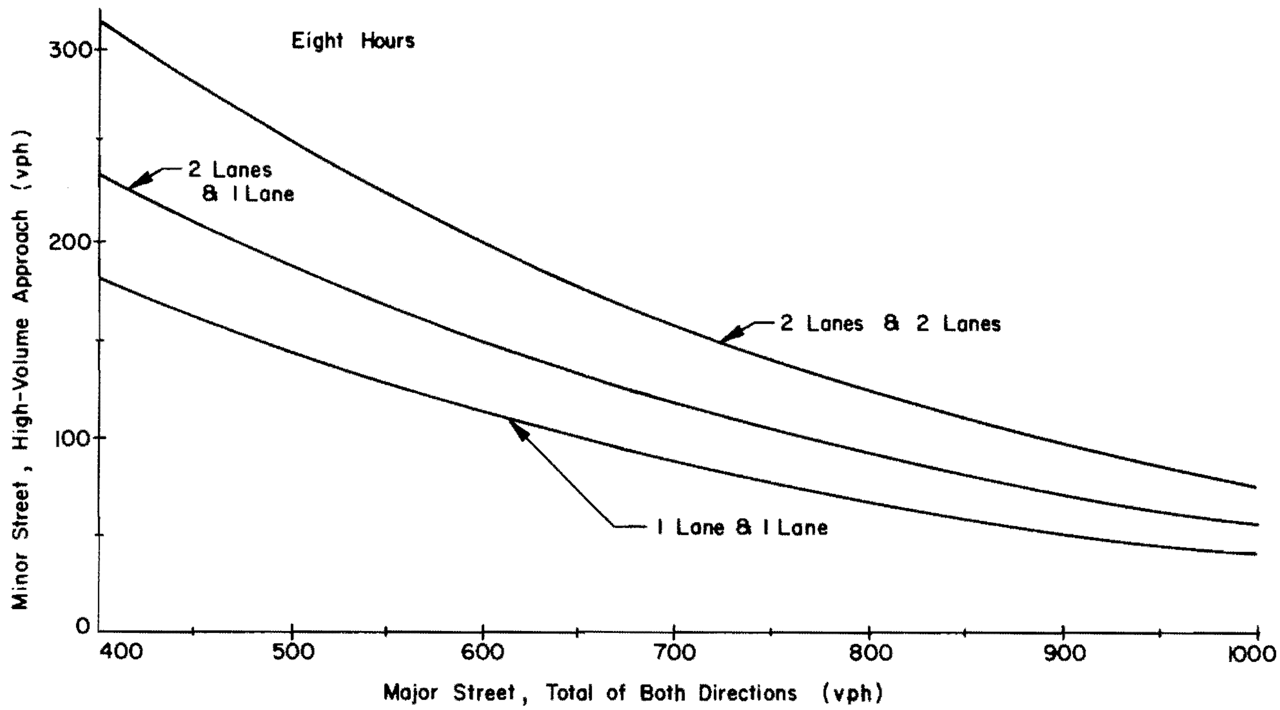

Fig B.1. Vehicular volume and cross-street warrant factor for traffic-actuated signals, urban area conditions.

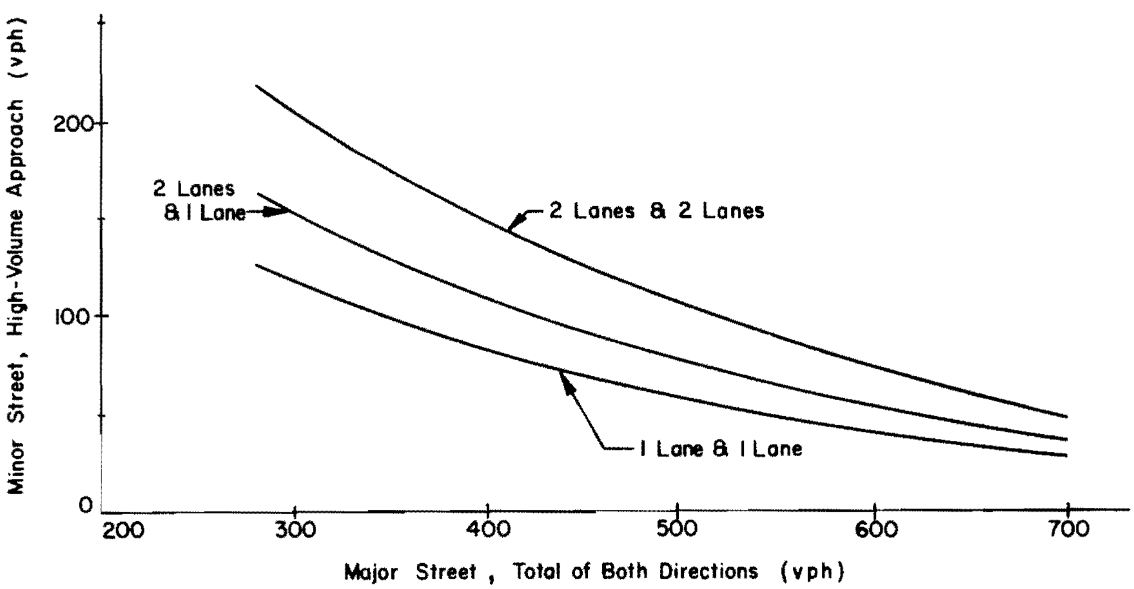

Fig B.2. Vehicular volume and cross-street warrant factor for traffic-actuated signals, rural area conditions.

points will fall close to the appropriate traffic-volume curve for the vehicular volumes and cross-traffic warrant factors.

When traffic control signals are required at an intersection during only a small part of the day, such as during peak traffic hours, traffic-actuated signals may be installed if economically justified, since they will not unduly delay traffic at other times. Three levels of the peak-hour volumes warrant are given for the highest hour, for the two highest hours, and for the four highest hours, all of an average day.

The highest hour level of the peak-hour volumes warrant is considered to be satisfied when, for one hour (any four consecutive 15-minute periods on an average day, the plotted point representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) falls above the curve in Fig B.3 or Fig B.4 for the particular existing combination of approach lanes. The major-street and minor-street volumes are for the same hour.

When the 85-percentile speed of major-street traffic exceeds 40 miles per hour, or when the intersection is within the built-up area of an isolated community having a population less than 10,000 (latest Federal Census), the curves in Fig B.4 are to be used. In all other cases the curves in Fig B.3 are to be used.

The two highest hours of the day shall consist of the four consecutive 15-minute periods having the highest volume of traffic and the four consecutive 15-minute periods having the second highest volume of traffic. The two highest hours level of the peak-hour volumes warrant can be considered to be satisfied when for each of the two hours of the day described above the plotted points representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-

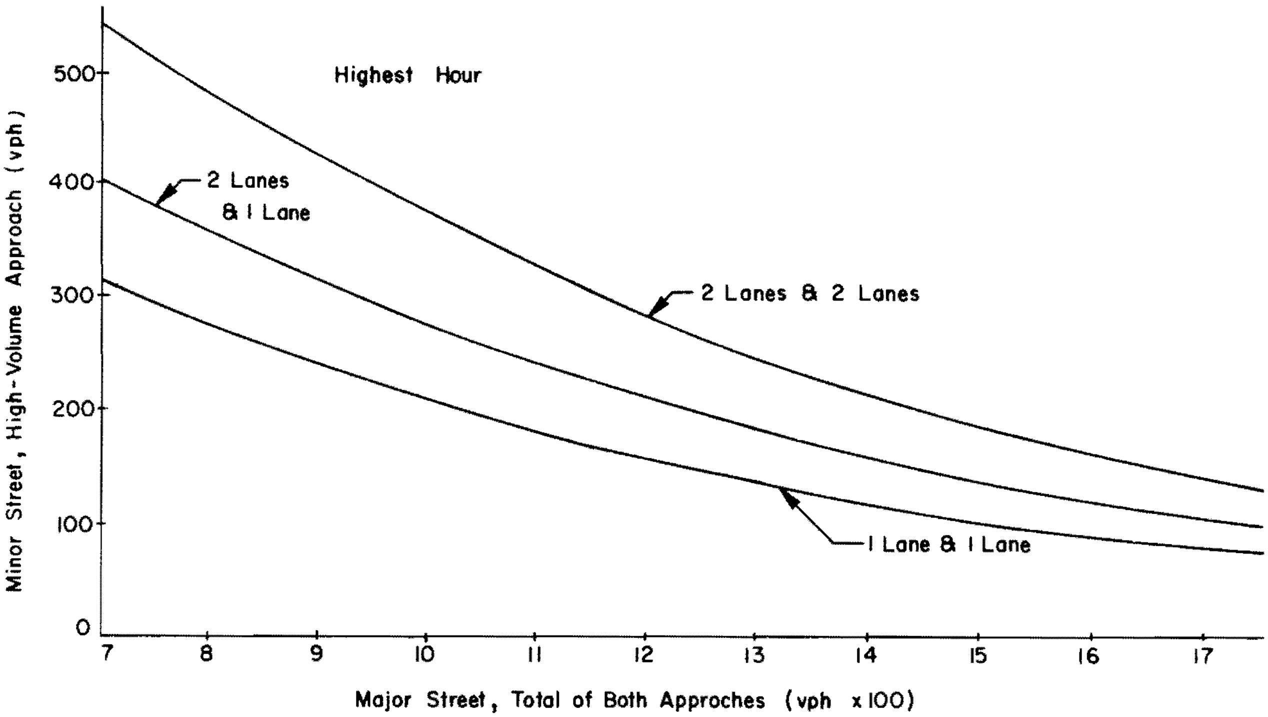

Fig B.3. Peak-hour volumes warrant factor volumes for traffic-actuated signals, highest hour, urban area conditions.

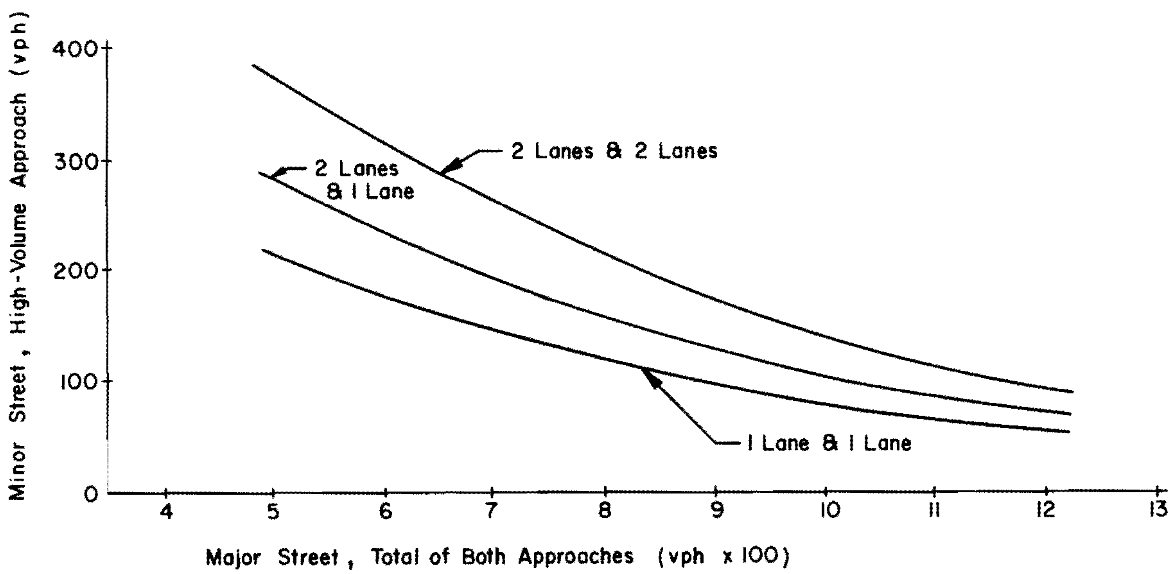

Fig B.4. Peak-hour volumes warrant factor volumes for traffic-actuated signals, highest hour, rural area conditions.

volume minor-street approach (one direction only) both fall above the curve in Fig B.5 or Fig B.6 for the particular existing combination of approach lanes. The major-street and minor-street volumes are for the same two hours. During those two hours the direction of higher traffic volume on the minor street may be on one approach during one hour and on the other approach during the other hour.

When the 85-percentile speed of major-street traffic exceeds 40 miles per hour, or when the intersection lies within the built-up area of an isolated community having a population of less than 10,000 (latest Federal Census), the curves in Fig B.6 are to be used. In all other cases the curves in Fig B.5 are to be used.

The four highest hours of the day shall consist of the four consecutive 15-minute periods having (1) the highest volume of traffic, (2) the second highest volume of traffic, (3) the third highest volume of traffic, and (4) the fourth highest volume of traffic. The four highest hours level of the peak-hour volumes warrant can be considered to be satisfied when, for each of the four hours of the day described above, the plotted points representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher-volume minor-street approach (one direction only) all lie above the curve in Fig B.7 or Fig B.8 for the particular existing combination of approach lanes. The major-street and minor-street volumes are for the same four hours. During those four hours the direction of higher traffic volume on the minor street may be on one approach during some hours and on the opposite approach during the other hours.

When the 85-percentile speed of major-street traffic exceeds 40 miles per hour, or when the intersection lies within the built-up area of an isolated community having a population less than 10,000 (latest Federal Census), the

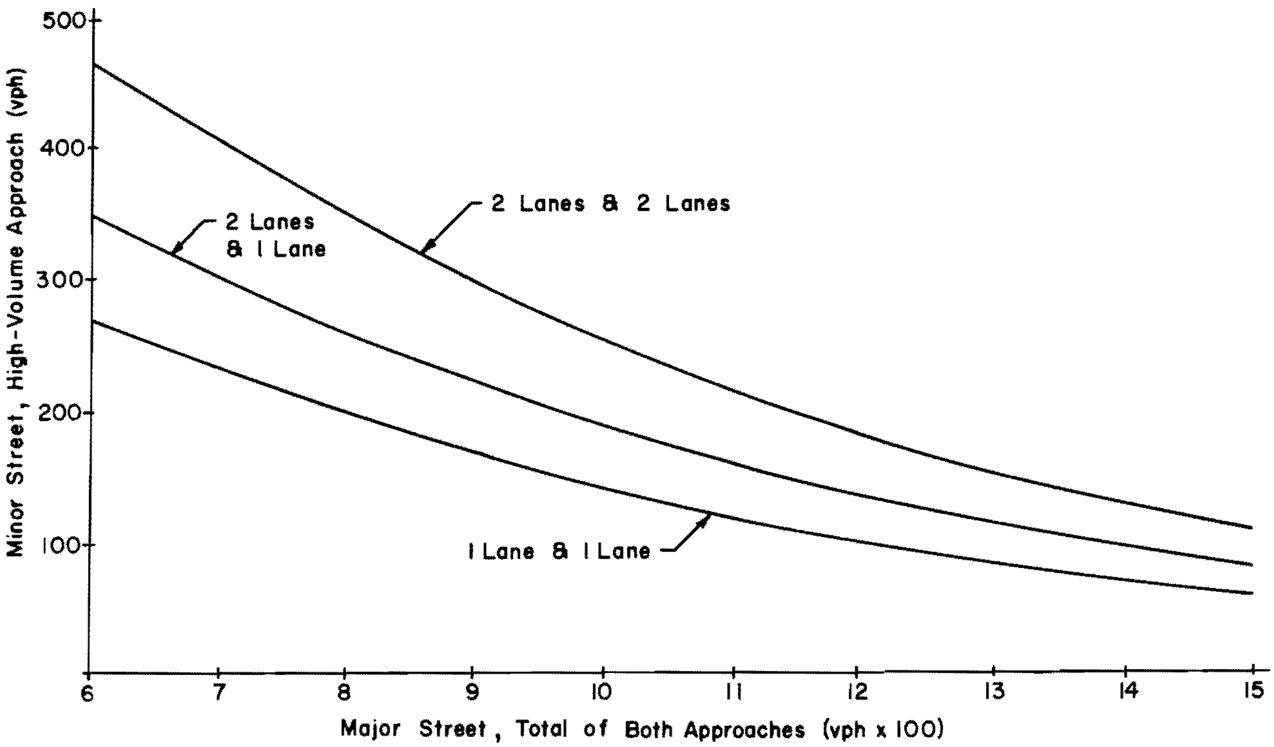

Fig B.5. Peak-hour volumes warrant factor volumes for traffic-actuated signals, two highest hours, urban area conditions.

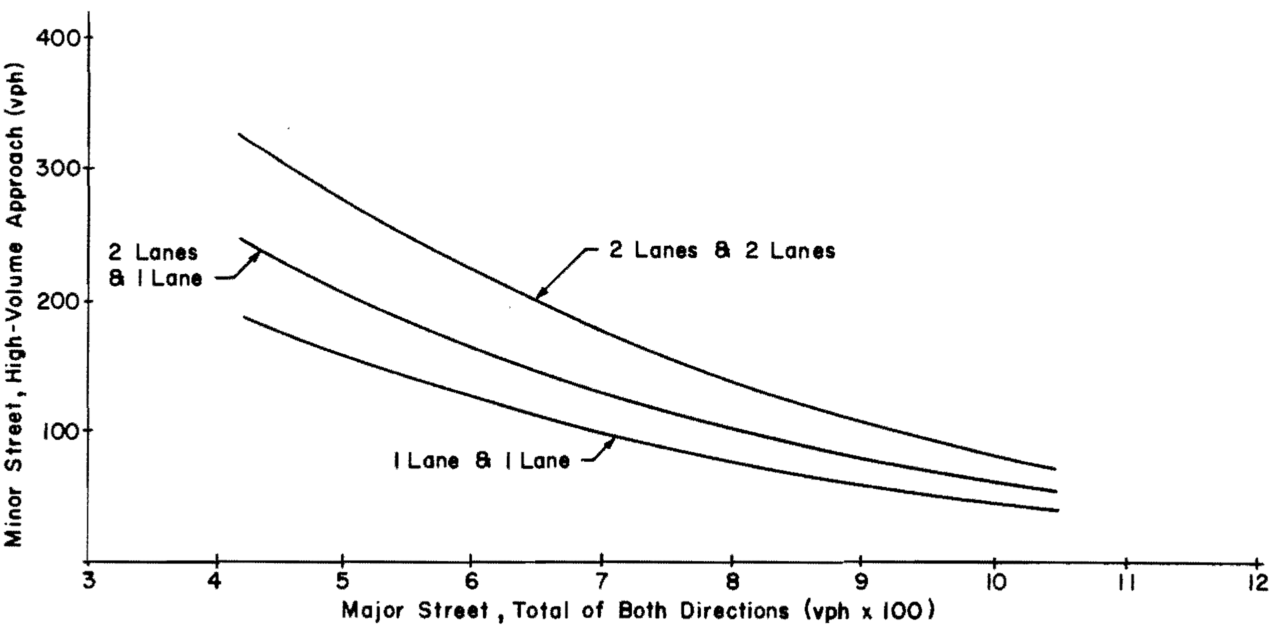

Fig B.6. Peak-hour volumes warrant factor volumes for traffic-actuated signals, two highest hours, rural area conditions.

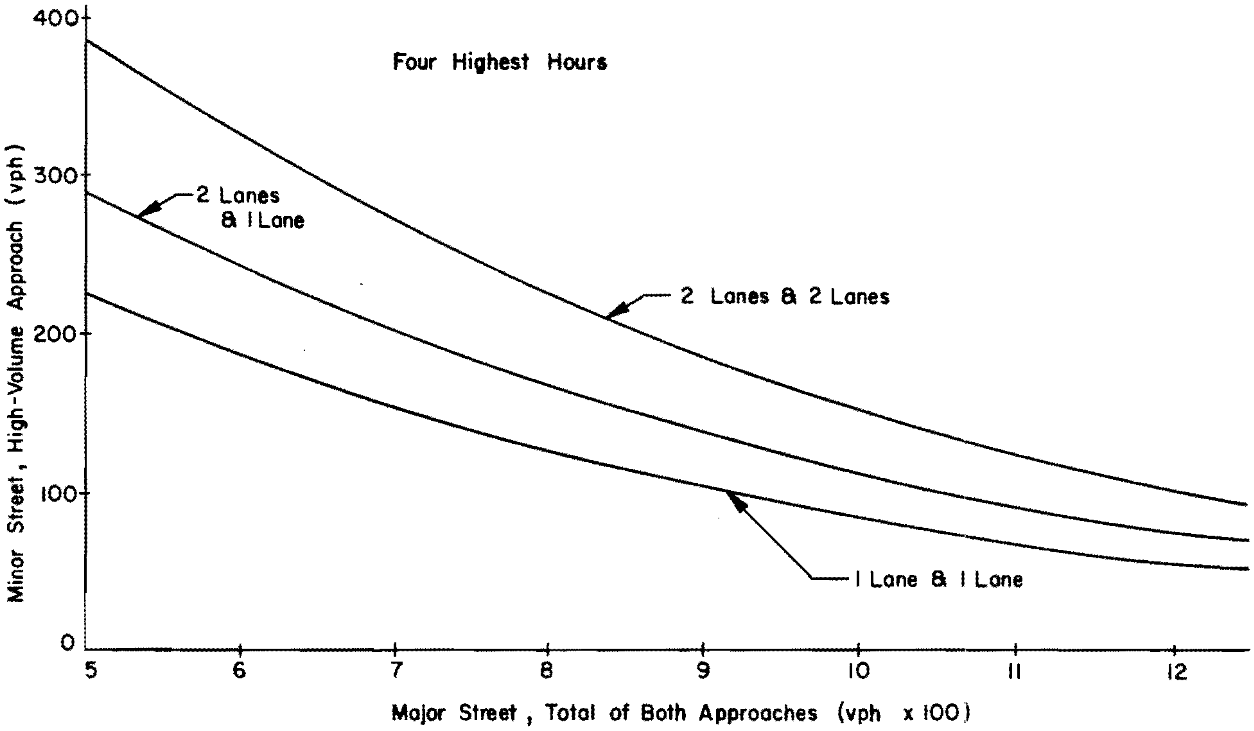

Fig B.7. Peak-hour volumes warrant factor volumes for traffic-actuated signals, four highest hours, urban area conditions.

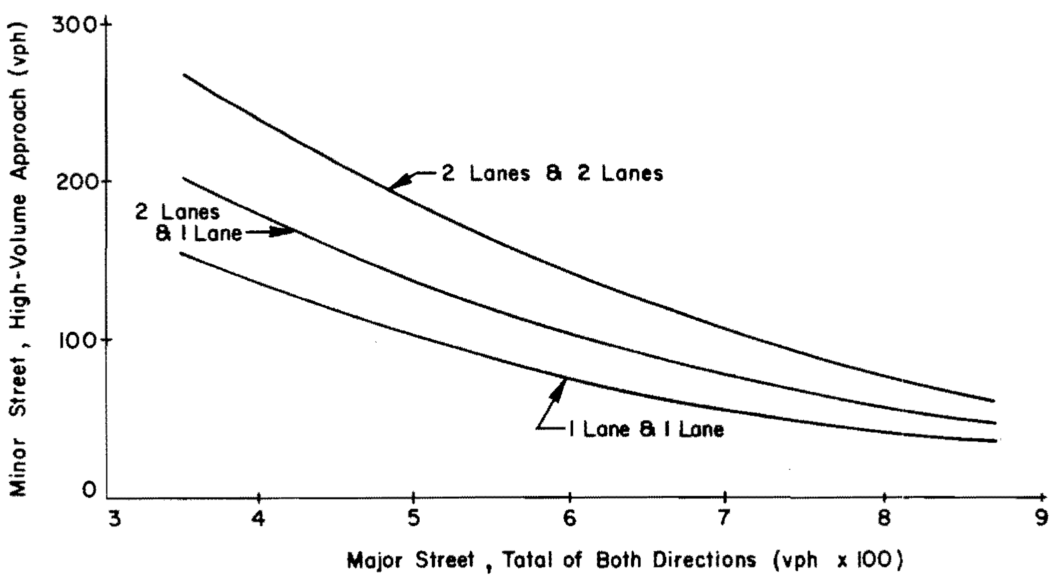

Fig B.8. Peak-hour volumes warrant factor volumes for traffic-actuated signals, four highest hours, rural area conditions.

curves in Fig B.8 are to be used. In all other cases the curves in Fig B.7 are to be used.

It should be noted that the computer-processed Vehicular Volume Summary Sheets and Count Analysis Sheets prepared by File D-19 include the traffic volumes for the four highest hours of traffic to be used in the three levels of the peak-hour volumes warrant factor.

The accident-hazard warrant factor refers back to the accident experience warrant for pretimed signals. The last sentence of this requirement states, however, that signals may be justified at locations where the accident experience is less than that warranting pretimed signals, but adds that careful analysis should be made to assure effective results. In our view a minimum of four reported accidents of a type susceptible to correction by a traffic signal should have occurred during a one-year period before the accident-hazard warrant factor is considered. It should be noted that the traffic volume requirement for the accident-experience warrant also applies to the accident-hazard warrant factor as does the requirement that less restrictive remedies be applied first. It should also be remembered that the following passage on page 189 of the 1961 AASHO Manual applies by inference to the accident-hazard warrant factor as well as to the accident-experience warrant:

> "Not infrequently there are more accidents with signals in operation than before installation. Hence if none of the warrants except the accident-experience warrant is fulfilled, the initial presumption should be against signalization."

The 1961 AASHO Manual also provides the following passage on page 200 under the vehicular-volumes warrant factor:

> "At intersections where the volume of vehicular traffic is not great enough to warrant pretimed signals, traffic-actuated signals may be applied if other conditions indicate the need for traffic control signals and justify the cost of the installation."

This provision is available for application under special conditions when none of the above-mentioned warrants and warrant factors are applicable and where the need for the traffic control signal justifies the cost of installation. Since the above warrant requirements will, in almost all cases, enable personnel to determine if the installation of a traffic signal is warranted, this provision is applicable only in special cases where unusual conditions prevail.

THE AUTHORS

Clyde E. Lee is the Director of the Center for Highway Research and a Professor of Civil Engineering at the University of Texas at Austin. His research experience includes work with the Mississippi Highway Department, the Texas Highway Department, and the Center for Highway Research at The University of Texas at Austin. He is associated with a number of professional societies, and he is a member of the Board of Directors of the American Academy of Transportation and is Liaison Representative for the Highway Research Board. His current research is concerned with (1) in-motion weighing and dimensioning of highway vehicles, (2) dynamic loading of highway structures, (3) simulation of traffic flow on street networks, and (4) micro-surface profile characteristics related to skidding. He is the author of a number of publications.

Walter C. Vodrazka is currently an Associate Professor of Civil Engineering at Wisconsin State University, Platteville, Wisconsin. While at The University of Texas at Austin (1968-70), he was an Assistant Professor of Civil Engineering and Research Engineer at the Center for Highway Research, where his research was focused upon the evaluation of intersection control. He has had a wide variety of other research experience at Purdue University, with the Corps of Engineers, and in private industry. His teaching experience includes a number of years at both Mississippi State University and Purdue University. He is a member of several professional societies and the author of a number of publications.