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16. Abstract

The current value of freeway capacity and the speed flow relationship have been questioned as a result of observations of flow rates much higher than 2,000 passenger cars per hour per lane (pcphpl) and the recent revision of the multi-lane highway chapter in the Highway Capacity Manual. An analysis of freeway capacity in Texas found greater variability in free-flow flow rates and determined that queue discharge is the best estimate of maximum sustainable flow. Average queue discharge flow rates averaged 2,225 pcphpl but were measured as high as 2,400 pcphpl for individual lanes. Based on the analysis, a speed-flow model was developed and the maximum sustainable flow rate was determined to be 2,200 pcphpl, which is the value recommended for freeway capacity in Texas.

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AN EVALUATION OF FREEWAY CAPACITY IN TEXAS

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Research Report 1196-2F

Research Study Number 2-10-88/1196

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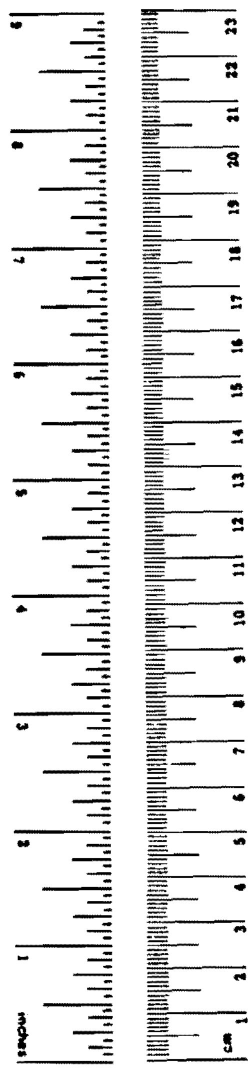
METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	2.54	centimetres	cm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	centimetres squared	cm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha
	MASS (weight)			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	2.53	acres	ac
	MASS (weight)			
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
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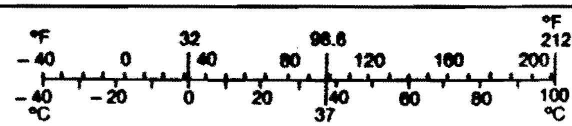
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



These factors conform to the requirement of FHWA Order 5190.1A.

* SI is the symbol for the International System of Measurements

ABSTRACT

The current value of freeway capacity and the speed flow relationship have been questioned as a result of observations of flow rates much higher than 2,000 passenger cars per hour per lane (pcphpl) and the recent revision of the multi-lane highway chapter in the Highway Capacity Manual. An analysis of freeway capacity in Texas found greater variability in free-flow flow rates and determined that queue discharge is the best estimate of maximum sustainable flow. Average queue discharge flow rates averaged 2,225 pcphpl but were measured as high as 2,400 pcphpl for individual lanes. Based on the analysis, a speed-flow model was developed and the maximum sustainable flow rate was determined to be 2,200 pcphpl, which is the value recommended for freeway capacity in Texas.

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The contents of this report reflect the view of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Texas Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. In addition, this report is not intended for construction, bidding, or permit purposes. This report was prepared by John Ringert and Thomas Urbanik II (Texas P.E. registration #42384)

IMPLEMENTATION STATEMENT

This report may be used by individuals involved in the planning, design or operation of freeway facilities. The characteristics of flow on freeways are evaluated and a model is developed for estimating capacity on freeways. The assumptions of the proposed model are summarized and recommendations are presented for the usage of the model. The implementation of this model will provide a revised approach for estimating capacity on freeways for the use in design and operation of such facilities.

EXECUTIVE SUMMARY

Freeway capacity plays an important role in the planning, design and operation of freeways in general and urban freeways in particular. As a result of increasing congestion in urban areas, problems have been identified with the existing freeway capacity of 2,000 pcphpl given in the 1985 Highway Capacity Manual. This report documents the development of an empirical model for estimating the maximum sustainable flow at freeway bottlenecks in Texas. This maximum sustainable flow is the value recommended for consideration as the freeway design capacity in Texas.

Ten sites in Texas were chosen for the study. Out of the initial ten sites, four sites were chosen for a detailed analysis and the model development. The criteria for choosing the sites were the occurrence of congestion on a regular basis, varying geometrics, and that the bottleneck was not affected by downstream congestion. Data were collected using inductive loop detectors at the primary study sites and video cameras at the other sites.

The analysis and validation procedure produced four major results. First, free flow-flow rates have significantly higher variability than queue discharge flow rates. Second, peak flows do not generally occur in all lanes during free-flow conditions because of turbulence caused by an imbalance of traffic. This prematurely transitions the flow from free-flow into queue discharge. Third, queue discharge is the best estimate of maximum sustainable flow. Finally, much lower flow rates occur if the study site is affected by downstream congestion. Although the study sites were selected to be the controlling bottleneck, many were affected by downstream congestion. In reality, it is difficult to identify a site that is always the controlling bottleneck because bottlenecks may appear sporadically at several locations.

Based on the analysis, a speed-flow model was developed and the maximum sustainable flow rate was determined to be 2,200 pcphpl. Although it is possible to sustain flows as high as 2,400 pcphpl in certain lanes, it is not possible in all lanes or over an entire facility. A flow rate of 2,200 pcphpl should be maintainable on most facilities.

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INTRODUCTION

This project is the second part of a two part study of freeway traffic flow. The first part of the study was to develop revised K factors for design hourly volume. This part of the study is to develop revised a model for estimating freeway capacity for use in planning, designing and operating freeway facilities in Texas.

Highway capacity has been a topic of research for over 50 years and dates back to the 1930s when the automobile was beginning to emerge as the dominant form of transportation. The *1950 Highway Capacity Manual* (HCM) states, "Highway capacity has been the subject of careful and painstaking study for more than three decades" (*1*). Since the publication of the 1950 HCM, extensive work has been done to update and improve both the concepts and values in the various chapters of the manual. These improvements were published in the 1965 HCM (*2*) and later in the 1985 HCM (*3*). Although significant changes were incorporated in both editions, the value of capacity remained the same as discussed in the 1950 HCM.

During the past decade, much attention has been given to freeway capacity and the relationships among speed, flow, and density at freeway bottlenecks. Freeway capacity plays a critical role in the planning, design, and operation of freeways in general and urban freeways in particular. As a result of steadily increasing congestion in urban areas throughout the United States, traffic engineers and transportation planners have identified problems associated with existing freeway capacity numbers given in the 1985 HCM and the corresponding speed-flow relationship (*3*). Many studies have concluded that greater flows than the values given as capacity are measured frequently on freeways. Another major issue in the past few years is whether a reduction in capacity occurs when a queue forms. In addition, the revised multilane highway procedure increased the value of capacity for multilane highways to a value greater than that given in the 1985 HCM for freeways. These issues, along with a common belief that the current speed-flow relationship should be re-examined, have increased the need for more detailed study of freeway capacity.

Problem Statement

There are many reasons why it is crucial to study freeway capacity and flow. The most important is that freeways play a prominent role in most urban areas throughout Texas as well as the United States. Freeways in most urban areas perform a multitude of functions from serving interstate trips to allowing access to urban centers. The absence of direct access distinguishes freeways from urban arterials and multilane highways.

As congestion increases in urban areas and facility improvements are needed, the impacts of alternative improvements must be assessed. One problem typically encountered by professionals is determining the effect of widening a freeway facility or installing traffic management systems. For both applications, professionals need to know the capacity of a facility as well as have a good understanding of the characteristics of flow. The current procedures contained in the 1985 HCM do not include the latest data and therefore lack credibility in the engineering and planning community.

Because of the importance of urban freeways in the United States and abroad, accurate and reliable procedures are needed to estimate capacity to aid in the planning, design and operation of such facilities. This research provides new insights into freeway capacity and the relationship between speed and flow. The research focuses on three aspects of freeway capacity. The first is the maximum observed flow rates. The second is the characteristics of pre-queue and queue discharge flows and whether a reduction in flow occurs once a queue forms. The third aspect is the variation in flows between different study sites with varying geometric and traffic conditions.

Research Objectives

The objectives of this research were to:

1. Study the traffic flow characteristics at freeway bottlenecks in Texas.
2. Study the variations resulting from geometric and vehicle characteristics.

3. Develop an empirically based model for estimating maximum sustainable flow.

Although site specific influences were examined, this study was not intended to include a detailed analysis of all geometric influences such as lane widths, shoulders and grades. Although all of these affect traffic flows, this research focuses on the characteristics of flow at freeway bottlenecks and the major influences. Freeway bottlenecks are the primary study sites because they are known locations where flows exceed capacity. Flows in excess of capacity are the only way to know that capacity has been reached.

The product of this research should assist traffic engineers and planners in evaluating the operation of freeways for planning, designing and operating facilities. In addition, this research introduces new insights into the flow processes on freeways.

Research Approach

The research approach was comprised of six phases. Each of these phases is discussed in following sections.

Phase 1: Literature Review

Phase 1 of the research consisted of a review of relevant literature and research. As part of Phase 1, the key components and variables were identified and the procedures for data collection and analysis were specified. The literature review included reviewing past publications regarding freeway capacity and flow as well as data collected by the Texas Transportation Institute. The purpose of this task was to identify useful information on freeway capacity and flow.

Phase 2: Primary Data Collection

Phase 2 consisted of the selection of a primary data collection site and the initial data collection. Sites throughout Texas were evaluated. The following criteria were considered in site selection:

- Frequent occurrence of congestion
- Geometric configuration
- Range of traffic volumes
- Absence of congestion caused by downstream bottlenecks

Data were collected for multiple days at the primary study bottleneck during the peak hours of traffic flow. Samples containing incidents were removed as well as data affected by downstream congestion.

Phase 3: Data Analysis
The data analysis included an evaluation of the characteristics of flow during both uncongested and congested operation. The analysis procedure included:

- An evaluation of geometric and traffic characteristics.
- A comparison of free flow, queue discharge and peak flow rates.
- An evaluation of the variation of flow rates measured during different days at the primary study site.

The purpose of this phase was to determine the characteristics of flow and flow processes at the primary bottleneck.

Phase 4: Formation of Flow Model
Based on the analysis of data, a flow model was developed for the primary study site in the form of maximum sustainable flows for free-flow and queue discharge conditions and a speed-flow relationship. The hypothesized maximum sustainable flow was based on the characteristics of speed and flow during uncongested and queue discharge conditions.

Phase 5: Comparison and Validation Procedure
Phase 5 consisted of validating the proposed flow model. The proposed values for maximum sustainable flow for the primary study site were compared to data collected at other freeway bottleneck sites in Texas. The purpose of applying the proposed model to other bottleneck

areas was to determine if the model is valid for cases other than the primary study site. The comparison also shows how much influence geometric and vehicle characteristics have on traffic operations.

The procedure used for the comparison considered both maximum flows as well as the average flows before and during queue discharge. The averages were compared using two methods:

- By visual inspection to identify major differences and possible causes.
- A statistical test of population means to determine if the differences between locations are statistically significant.

The purpose of this procedure was to determine the variations between different sites.

Phase 6: Prepare Recommendations for Use

The final task was to make recommendations for the use of the proposed value of capacity and the empirical model. This included a discussion of the assumptions, and the limitations of the proposed model. This task also addressed the location of measurement and analysis for freeway bottlenecks.

BACKGROUND AND PREVIOUS WORK

There are five elements of freeway capacity that relate to this study. The first element is the numerical value of freeway capacity. The second is the notion of a capacity drop in congested conditions, also known as the "two capacity" hypothesis. The third element that will be discussed is the relationship between speed and flow. The fourth element is the impact of the measurement location on the data collected. The last element is the meaning of capacity.

The Numerical Value of Freeway Capacity

The numerical value of capacity has been an issue of discussion since the 1930s. To adequately discuss the value of capacity, the issues which have created questions regarding capacity must be discussed. The first issue is the recent decision by the Highway Capacity and Quality of Service Committee to increase the value of capacity for multilane rural highways to 2,200 passenger cars per hour per lane (pcphpl). The second issue is the common measurement of traffic flows much higher than the capacity given by the 1985 HCM. The final issue is the lack of change in the value of freeway capacity since 1950 even though major advances in traffic control, freeway design, vehicle size and driver experience have occurred.

The past three editions of the HCM do not differ on the numerical value of freeway capacity. Since the publication of the 1950 HCM, there has been a continuing consensus on the value of 2,000 pcphpl for capacity. Although all of the manuals recommend 2,000 pcphpl, there are differences in the way it is described. The 1950 HCM stated that the largest number of vehicles that can pass a point in a single traffic lane under the most ideal conditions is between 2,000 and 2,200, but later went on to say that the basic capacity of a multilane road is 2,000 pcphpl (*1*). A similar statement was made in the 1965 HCM which was that the largest number of vehicles that can pass a point averaged between 1,900 and 2,200 pcphpl (*2*). The 1985 HCM summarized the maximum observed volumes, many of which were over 2,000 pcphpl, but stated that values over 2,000 pcphpl still represent unusual occurrences (*3*). Both the 1985 HCM and 1965 HCM give 2,000 pcphpl as the value of capacity.

Furthermore, the 1985 HCM discusses capacity as a "national average" which adds the question of where data should be collected and how much is needed to provide an accurate value. The term national average could be interpreted as being achievable on half of all freeways without consideration of the type of facility. This type of assurance would be difficult to achieve since most capacity studies are based on limited data.

In addition to the data and descriptions given in the HCM, a number of studies have produced results concerning the value of capacity. As far back as the 1940s high flow rates were measured. The 1941 edition of the *Traffic Engineering Handbook* reported an observation of 2,700 vehicles per hour per lane (vphpl) for a 20-minute period on U.S. Route 101 (*4*). During the past few years many extensive studies have been undertaken to determine the value of capacity. A study by Hurdle and Datta in 1983 concluded that the value of 2,000 pcphpl was still a good estimate of capacity (*5*). In contrast, a study by Agyemang-Duah (*6*) concluded that the capacity flow rate was approximately 2,300 pcphpl, which was also a result of a later study by Hall (*7*). Chin and May confirmed that flows in excess of 2,200 pcphpl are possible (*8*). Another study by Urbanik and Hinshaw showed data from four sites in Texas which had peak 15-minute volumes between 2,100 and 2,300 vphpl (*9*).

Based on these findings it is difficult to determine what is a reasonable value for capacity, although it is obvious that in many locations traffic flows in excess of 2,000 pcphpl commonly occur. Even though there is obviously still some speculation on whether the value of capacity is larger than 2,000 pcphpl, it is reasonable to assume that capacity has increased over the past 40 years for two reasons. The first is that with improved freeway designs and freeway traffic management systems, some increase in capacity would be expected. The second is that changes in vehicle size should have some impact on freeway capacity. Woods reported that the large to small car ratio was 3:1 in 1975 and 1.2:1 in 1980 (*10*). Although vehicle length would not be expected to increase capacity substantially, when combined with facility and vehicle improvements, some amount of increase should occur.

The Two Capacity Hypothesis

The second element regards the distinction between different regions of flow which is the basis for the "two capacity" hypothesis. The two capacity concept infers that two separate capacities exist, one during uncongested flow conditions and one during queue discharge. In the past few years many studies have reported a drop in the maximum flow once there is a queue while others have indicated no such drop.

The 1985 HCM discusses the suggestion of a capacity reduction for congested conditions in a brief statement (*3*):

> Some researchers have fit continuous curves through density-flow data, yielding a single maximum flow rate. Others have projected discontinuous curves through data, with one curve treating stable flow points, and another unstable or forced flow points. In these cases two maxima are achieved, one for each curve. All such models indicate that the maximum flow rate for the stable flow curve is considerably higher than that for the unstable flow curve, perhaps as much as 200 vphpl.

One argument against the existence of two capacities concerns the location of the data collected. This is based on the idea that there will be an absence of data if a queue backs into the locations while flow is lower than capacity (*11,12*). Many other studies have attempted to measure the flows in both conditions and have produced varying results.

Another related issue is the requirement for the existence of sufficient demand which is highlighted by McShane and Roess (*13*). Agyemang-Duah and Hall discuss two possible ways to find sufficient demand for capacity to be measured. The first is the existence of a queue. The cause of the queue must also be ascertained to determine if the queue is not being caused by an incident. Sufficient demand may be possible in the absence of a queue, but this is difficult to say without examination (*6*). The existence of sufficient demand and the knowledge of the cause of queued conditions are very important. If a queue is caused by a downstream bottleneck, then the capacity would be limited by the downstream congestion and lower flow rates would occur even though the location would appear to be over capacity. Many studies have considered these issues and have still identified a drop in maximum flow in queue discharge.

Hall and Agyemang concluded that there is a capacity drop in the bottleneck and once a queue has formed upstream the bottleneck location does not handle as many vehicles as it did prior to the queue formation. They estimated the reduction to be about 5-6% (*7*). A later paper by Agyemang and Hall in 1991 concluded that there was a difference between maximum flow before queue discharge and during queue discharge. They reported that capacity dropped by approximately 100 pcphpl from 2,300 to 2,200 in queue discharge conditions (*6*).

In the past 2 years other detailed studies have been done on the characteristics of traffic flow with respect to the two capacity hypothesis and the implications on freeway operations. Persaud and Hurdle studied the standard deviations of flow before and after queue discharge and found less fluctuation after the formation of queues. They concluded that queue discharge flow followed a normal distribution and that the mean queue discharge flow is currently the most suitable for capacity (*14*). Banks also studied the two capacity phenomenon from a theoretical standpoint and related it to the concept of ramp metering to obtain the maximum flow. He concluded that the differential in flow was extremely small and significant benefits would not be gained from ramp metering (*15*). Based on the Banks research there may be varying capacities for different types (or regions) of flow, although the numerical differences are small.

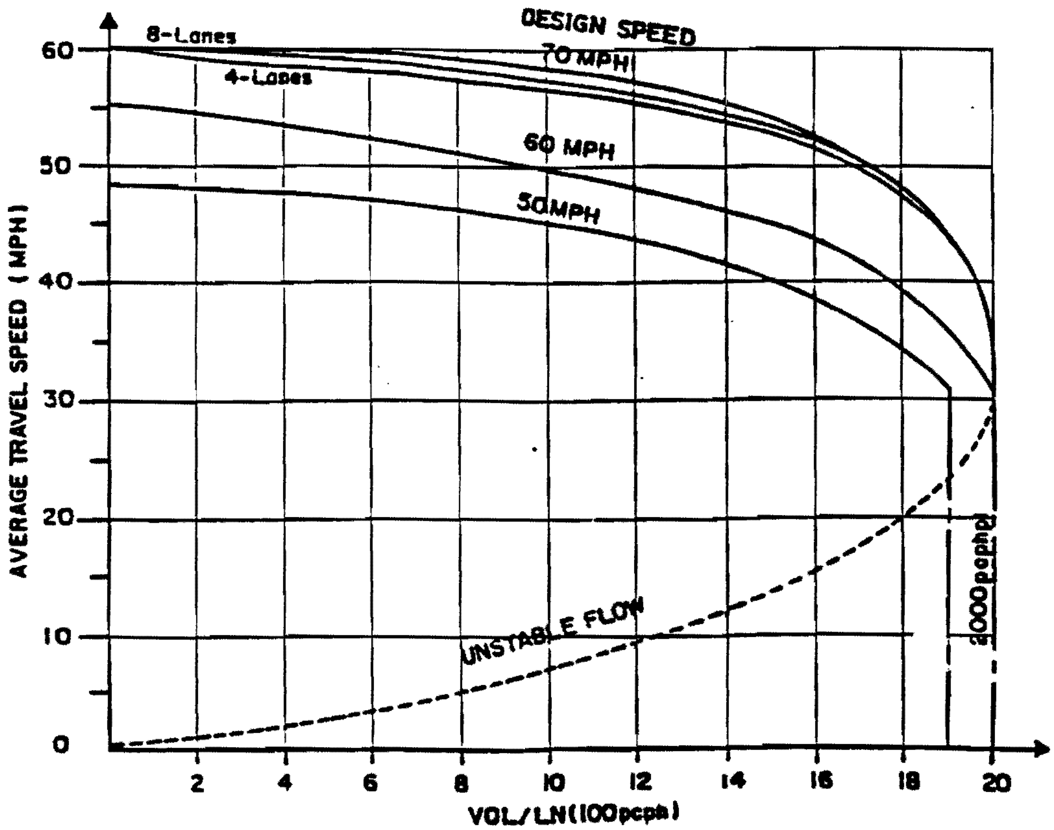
The conclusions of much of the research have indicated that there are two capacities, one during free-flow conditions and one during queue discharge, although the relation of the values is not known completely. It is also clear that the location of data collection is very important and data from locations within a queue are misleading, which will be discussed later.

The implications of two capacities affects the definitions and value of capacity. The 1985 HCM defines capacity as the maximum sustained (15 minute) rate of flow at which traffic can pass a point or uniform section of freeway (*2*). If two capacities exist, a possible problem exists because the peak 15-minute flow rate could possibly contain flows from both regions. This would make the peak 15-minute flow rate impractical for use as capacity because it would give a flow that may not be sustainable at higher demand levels where queue discharge flow occurs.

The Speed-Flow Relationship

The third element is the relationship between speed and flow. As a result of much of the research on capacity and the two capacity hypothesis, the relationship of speed and flow has become a center of debate. Although the relationship of speed and flow has been studied extensively from an analytical approach, this review focuses primarily on empirical findings concerning the relationship of speed and flow.

As with capacity, the relationship of speed and flow has not changed significantly over the last 40 years. A parabolic speed-flow relationship was originally conceived in the 1930s with Greenshields' linear speed-density model and remains the accepted standard (*16*). The 1965 HCM published a basic parabolic speed-flow relationship with the speed at capacity being one-half the free flow speed (*2*). Although some minor modifications were made, the same basic relationship remained in the 1985 HCM. The only difference was that the upper region of the curve was slightly flatter (*3*). The speed-flow relationship given in the 1985 HCM is shown in Figure 1.



Note: Adapted from the *1985 Highway Capacity Manual* (*3*).

FIGURE 1 Speed-flow curve given in the *1985 Highway Capacity Manual* (*3*).

In the past decade there has been continuing controversy over the speed-flow relationship as a result of empirical data collected from numerous sites. The catalyst has been the recent adoption of a completely new speed-flow relationship for multi-lane highways. The speed-flow relationship for multilane highways is shown in Figure 2 (*17*). As can be seen in Figure 2, the relationship is much different than the current parabolic shape. Speed remains nearly constant to relatively high volumes and only drops 5 mph at capacity.

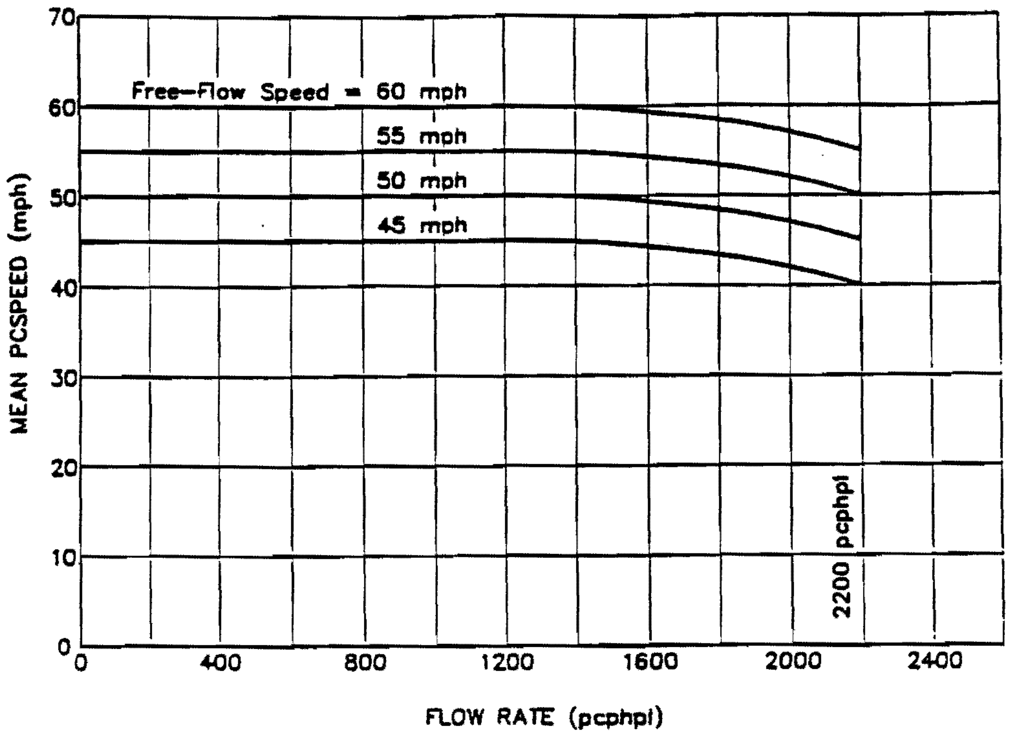


FIGURE 2 Speed-flow relationships for multilane highways (*17*).

The idea of a flatter free-flow region is not a new concept. A paper by Hurdle and Datta in 1983 showed some possible shapes for of the speed-flow curve based on empirical data which is shown in Figure 3 (*5*). This figure was also shown in the 1985 HCM but was only briefly discussed. They also concluded that speeds remain high until flow reached at least 75% of capacity and the speed at capacity flow was approximately 50 mph (*5*). Many other studies have resulted in similar conclusions. A study by Persaud and Hurdle indicated a 25% drop in speed from 95 km/hr (59 mph) to 65-70 km/hr (40-43 mph) and concluded that a precipitous drop in speed at high flow may very well have resulted from misinterpretation of data that arose because the speed of vehicles discharging from a queue varies with the location in the bottleneck (*18*). A study by Hall found a 25% drop from 104 km/hr (65 mph) to approximately 80 km/hr (50 mph) (*19*). Other studies found even less drop in speed at capacity. Chin and May found a drop of about 15% (10 mph) at the Caldecott Tunnel in California and Banks found a smaller drop in speed in San Diego (*8,20*).

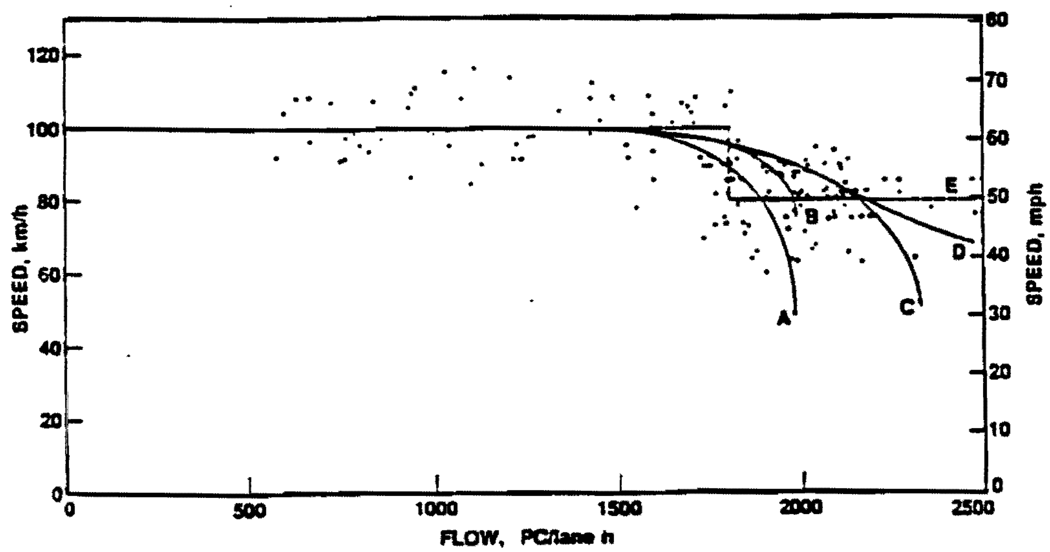


FIGURE 3 Speed-flow data and some possible curve shapes (*5*).

A recent paper by Hall, Hurdle and Banks made many conclusions by pulling together research published during the past 5 years on capacity and the speed flow relationship. One conclusion was that speed remained nearly constant up to approximately two-thirds capacity and a speed drop of only 10-25% occurred between free flow and capacity. As a result of the analysis, they proposed a speed-flow relationship which is shown in Figure 4 (*21*). This speed-flow relationship displays many of the concepts previously discussed. The first is the relatively small speed drop near capacity. The second is the higher flow rate in free-flow conditions than in queue discharge which is illustrated by the vertical points that symbolize the operation at different locations downstream of the queue. Although no attempt was made to determine the speeds and flows associated with the relationship, it has been proposed as the best understanding of the speed-flow relationship.

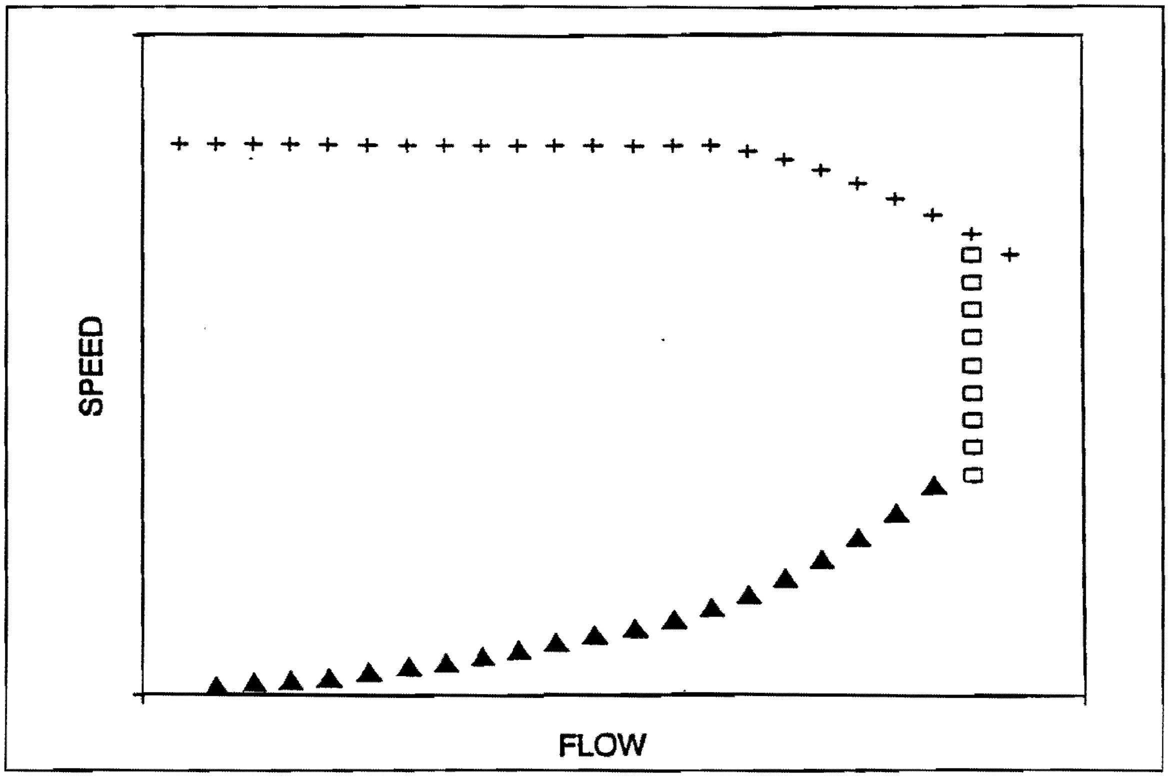


FIGURE 4 Generalized speed-flow relationship (*21*).

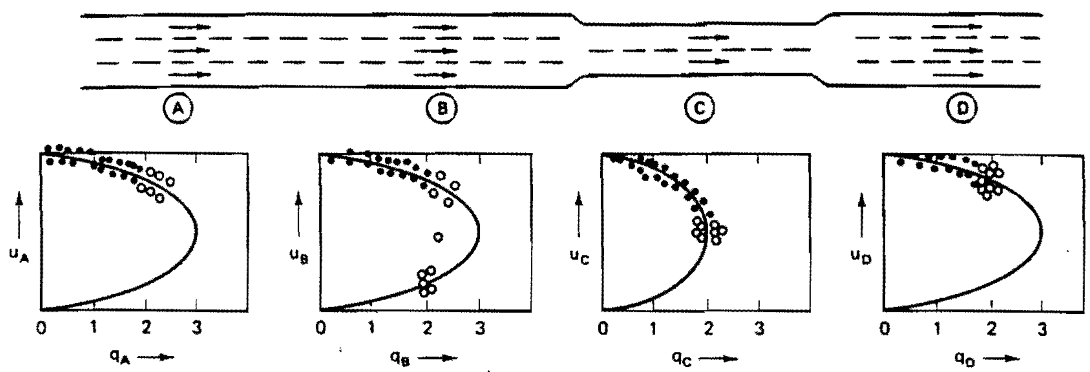
Based on the results of research on capacity, the two-capacity hypothesis and the relationship of speed and flow, there appear to be problems associated with the current values and relationships given in the 1985 HCM. Although most research supports a change, there is very little agreement over what changes should be made and the implications of such changes. With the revision of the multilane highway chapter of the HCM it is necessary to reevaluate these topics in more detail to assist in forming a common consensus on capacity, and the characteristics of speed and flow at freeway bottlenecks.

Impact of Data Collection Location

To accurately evaluate the relationship of speed and flow and flow processes, it is very important to understand the type of flow being observed. The location of data collection with respect to the bottleneck has a significant effect on the type of data being collected. May studied this topic and provides a helpful illustration which is shown in Figure 5 (*22*). Figure 5 depicts a 3-lane section of freeway which is reduced to 2 lanes. Four locations with respect to the bottleneck are labeled. Station A is at the upstream end of the study section. Station B is in the 3-lane section a short distance upstream of the bottleneck. Station C is in the 2-lane section just downstream of the bottleneck and Station D is far downstream in the 3-lane section.

The speed-flow curves shown illustrate the differences in the data collected for each location. When flow is equal or less than 2 lanes of capacity, which is represented by the solid dots, stations A, B and D are operating at below capacity while station C is at capacity. Once the flow has become greater than capacity at C, the flow at station B will be equal to the flow at C and is congested with low speeds. The flow at both B and D are metered by the service rate at station C, although the flow at D will exhibit higher speed as a result of vehicles accelerating from the bottleneck. This means that although stations B and D have more lanes they will exhibit the same maximum flow as station C. Because of this effect, it is very important to determine the cause of the congestion at bottleneck and not assume that because a segment is congested, it is operating at capacity.

It should also be noted than none of the locations cover the complete range of speed and flow values. All locations display similar data in low flow conditions but are extremely different for higher flows. Although the capacity of the 2-lane section occurs at all points, the flow conditions vary. At locations downstream of the bottleneck, the capacity flow will have high speeds due to acceleration for the bottleneck region while the section directly upstream of the bottleneck will display very low speed due to the congested conditions created by the bottleneck.



Note: Adapted from May (*22*)

FIGURE 5 Importance of data collection location (*22*).

In summary, the location of data collection limits the range of flow and speed values, and the data sets used for developing and validating traffic stream models influence the results and the comparison between models. Because of these effects it is critically important that the analyst understand the conditions of flow in which observations are made.

Meaning of Capacity

One issue that is not specifically addressed in this report is the definition of capacity which plays a significant role in the determination of the value to use as capacity. Both the 1965 HCM and the 1985 HCM describe capacity as a maximum flow rate that can be achieved for a given period of time which is 15 minutes in the 1985 HCM (*2,3*). As discussed earlier, if two capacities exist, then the peak 15-minute flow rate may not be sustainable, which is a very important consideration since peak periods in urban areas can be as long as 2 hours.

Since the publication of the 1985 HCM, many papers have been published regarding the definition of capacity. McShane and Roess clarified some of the key points that the definition of capacity should encompass. Some of these are that capacity of a facility is dependent upon prevailing traffic, roadway and control conditions and that capacity should be defined on the basis of "reasonable expectancy" and is a value that can be achieved repeatedly day in and day out (*13*).

For capacity to be useful, it is very important that it is sustainable and can reasonably be achieved repeatedly. This report will focus on the maximum sustainable flow from which a recommendation will be made for a value of capacity.

SITE SELECTION

The site selection process evaluated bottleneck sites throughout Texas. Because congestion occurs primarily in metropolitan areas, the Houston, Dallas, Fort Worth, and San Antonio metropolitan areas were the focus of the site selection process. The major criteria for site selection was the occurrence of congestion on a regular basis. Other factors that were considered were:

- Horizontal and vertical alignment
- Lane and shoulder widths
- Type of bottleneck
- Location with respect to other bottlenecks

Both the type of bottleneck and the geometrics affect the characteristics of flow at bottlenecks. Therefore, it was important to examine a range of sites with varying geometrics.

The location with respect to other bottlenecks was also an important factor in the site selection. In many urban areas congestion extends through many interchanges and merge locations which are not all bottleneck locations. Congestion created by a downstream bottleneck will restrain the flow through upstream bottlenecks. Therefore sites with congestion caused by downstream bottlenecks were avoided.

Based on the factors previously discussed, ten study sites were chosen which are given in Table 1. Traffic counts and operations were analyzed for each of the study sites. Although all of the sites were used in the study, certain sites were chosen as primary study sites for the initial analysis and model development.

Table 1 Study Sites			
City	Highway	City	Highway
Dallas	I-35/US 67	Fort Worth	I-820
	I-635 @ Coit		US 183 @ West of Central Drive
	North Tollway		US 183 @ Central Drive
Houston	US 290 @ Pinemont	San Antonio	I-410 @ McCullough
	US 290 @ Tidwell		I-410 @ West Avenue

The sites chosen for the detailed analysis and model development were on U.S. 290 at Tidwell in Houston, I-35/US 67 in Dallas, U.S. 183 at Central Drive in Fort Worth and I-410 at West Avenue in San Antonio. These primary study sites were determined to be the best sites based on the site selection factors. In addition the U.S. 290 at Tidwell site was chosen for the initial analysis and model development. For these sites, detailed data was collected for the analysis of flow processes.

DATA COLLECTION

Data Collection Procedure

The data for this report were collected by two means. For all of the initial sites, data were collected using video cameras, and reduced lane by lane using computer assisted data reduction. This method of data collection provides for individual vehicle headways and classification by lane. The information corresponding to each vehicle is identified by the time of observation. Based on the headway data, many aspects of the flow characteristics can be investigated. Reliability analysis of the count data suggests that the data reduction errors are less than 0.5 percent (2,400 ± 2). One element that this type of data collection does not provide is speeds. This initial procedure was used due to the difficulties in developing an automated data collection procedure using loop detectors.

Once the necessary automation equipment was developed, a different procedure was used for detailed data at the primary study sites. Data were collected using pairs of inductive loop detectors in each lane which were connected to electronic traffic counters that collected individual actuation information which was then recorded on a laptop computer. Figure 6 shows a schematic of the data collection process. The data in the form of time stamps were then used to calculate flow rates, headways, speeds, and vehicle lengths. The advantage of this type of data collection is that all aspects of the traffic flow can be examined. Because all the data is collected in a microscopic form (individual vehicles), they can be summarized in many different forms to study both microscopic and macroscopic characteristics. These characteristics include individual vehicle speed and headway as well as macroscopic characteristics such as average speed and flow rates.

An additional consideration for using two methods of data collection was to ensure a very detailed data base for the analysis of flow characteristics and development of a flow model ,while at the same time providing a large sample of sites throughout Texas. The cost of installing the loop detector systems is very expensive and therefore is was not practical

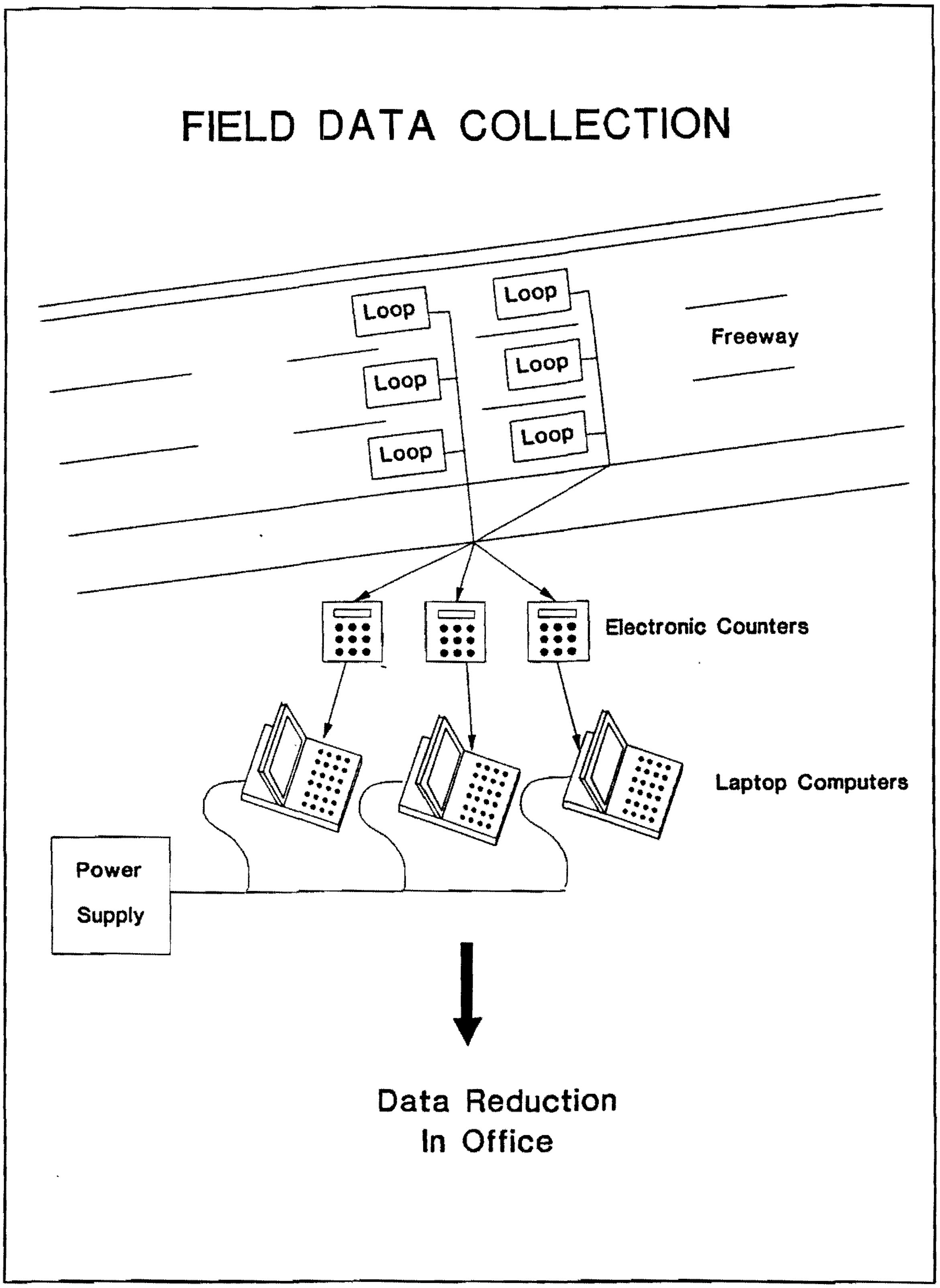


FIGURE 6 Schematic of field data collection using loop detectors.

for use at all study sites.

Experiences with Data Collection

As shown in Figure 6, collecting data using inductive loop detectors is relatively complicated involving installation and maintenance of detectors, electronic traffic counters, laptop computers and a mobile power source. Because of the required accuracy needed for the data, data collection devices normally employed for such work would not produce satisfactory results. Because of these complications, many different approaches were attempted to simplify the data collection process.

Normal traffic counters require a traffic counter for each lane to be used and do not provide enough memory to store the detailed actuation data; therefore, laptop computers were used. Although the normal counters did not provide the necessary capability for data collection, counters were found that did provide adequate memory to independently collect and store data. These counters were furnished by Golden River Co. Use of the Golden River counters revealed a significant flaw which was that they could not scan the loops at a high enough rate to produce accurate results when a single counter was used across all lanes. In addition, if separate counters were used for each lane, the frequencies interfered which corrupted the data. This interference occurred because the counters were designed in Europe and made for use on a limited range of frequencies with detectors separated by large distances.

Use of the Data

Accurate microscopic speed and flow data is very limited in the United States and Canada. Most data such as the type used for this analysis can only be acquired from operating freeway management systems. Although such systems exist in large cites, they are rare and not always designed to collect the required data. For these reasons, the data used for this analysis has also been used by the Highway Capacity and Quality Service Committee of the

Transportation Research Board to study freeway traffic flow and produce revised analysis procedures. In addition, this data will also be used by an upcoming National Cooperative Highway Research Project (NCHRP) to update chapter 3 of the Highway Capacity Manual.

PRELIMINARY ANALYSIS AND MODEL DEVELOPMENT

Study Site

The study site used for the initial development of the model for capacity was on U.S. 290 North in Houston, Texas. A schematic of the study site is shown in Figure 7. This section of U.S. 290 is level with three 11-foot lanes, a 10-foot outside shoulder and a 2-foot inside shoulder. The location for data collection is approximately 350 feet west of the end of the taper of the Tidwell on-ramp and 120 feet east of the taper of the next off-ramp.

The purpose of choosing this study site was the geometric conditions and the frequent occurrence of queues during the p.m. peak period. Other factors that were considered were the ability to find a vantage point for videotaping and moderate on-ramp volumes. Nearly ideal geometric conditions are important to obtain a good understanding of traffic flow at freeway bottlenecks with as little adverse influence by external factors as possible. Although the lane widths and inside shoulder are slightly less than ideal, they are common in many congested urban areas and were not expected to significantly impact the results based on preliminary studies of this and another site. The results were compared to other sites with 12 foot lanes and full width shoulders later in the analysis. The on-ramp volume at the U.S. 290 site was approximately 750 vph from 4:00 p.m. to 5:00 p.m. during the study period and increased to approximately 1,100 vph by the latter part of the study period.

Estimators and Measures of Effectiveness

Several types of estimators were used to understand the characteristics of flow at the primary study site. Vehicle counts, headways, vehicle lengths, and speeds were calculated based on loop detector data which were verified by video taping the flows during the data collection. From these, flow rates for various averaging intervals were calculated and examined. Some of the speeds and flows rates examined are listed below.

- One-minute flow rates and speeds

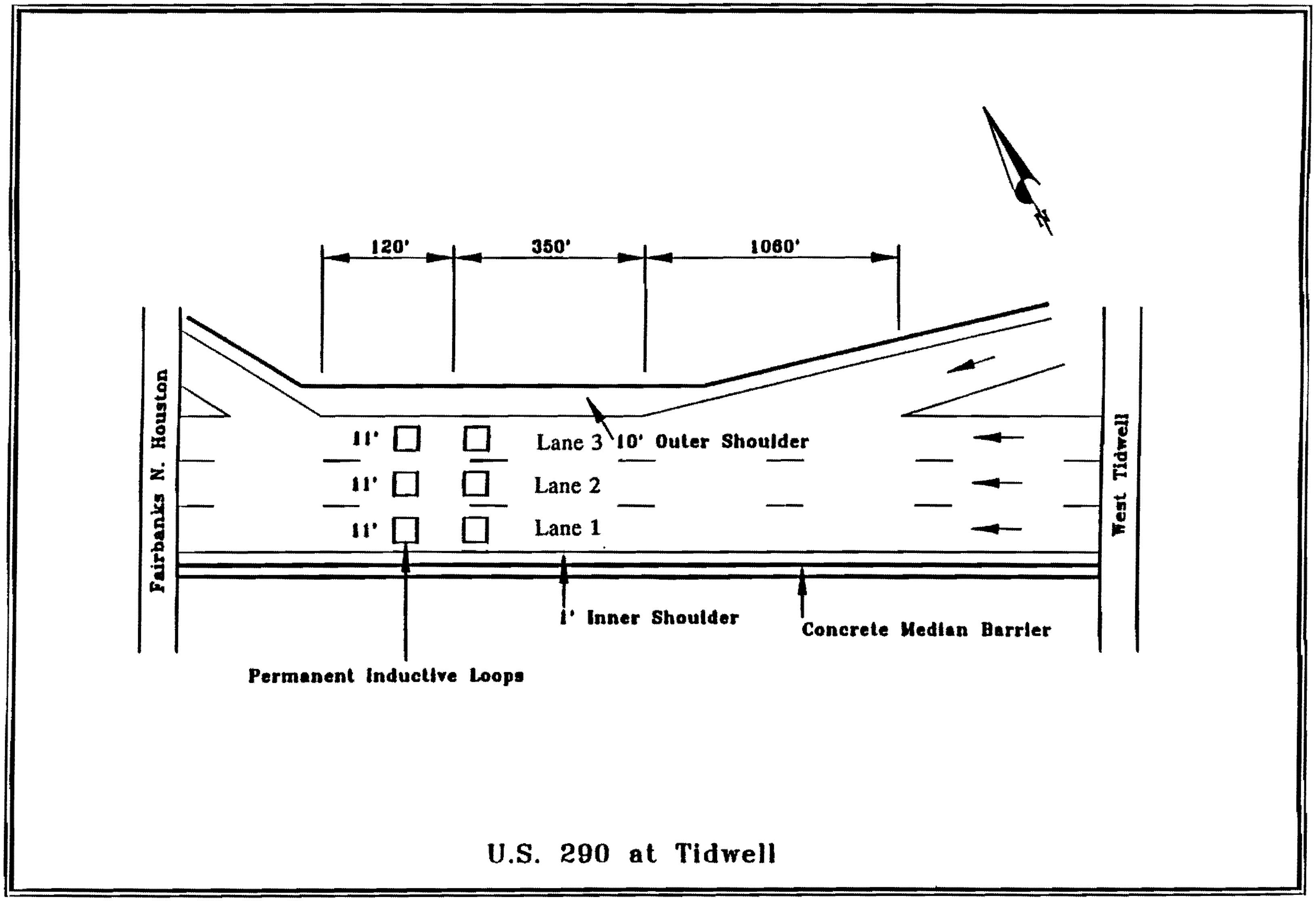


FIGURE 7 U.S. 290 at Tidwell study site.

- Five minute flow rates and speeds
- Peak 15 minute flow rates

Data Collection

Data were collected for 15 days at the U.S. 290 study site. The dates that data were collected are shown in Table 2. Data were collected for 2 to 3 hours for each day during the p.m. peak.

TABLE 2
Data Samples For U.S. 290 at Tidwell in Houston

Data Sample	Date	Data Sample	Date
#1	July 1, 1991	#9	October 16, 1991
#2	July 11, 1991	#10	March 11, 1992
#3	August 7, 1991	#11	March 12, 1992
#4	October 2, 1991	#12	March 25, 1992
#5	October 3, 1991	#13	March 31, 1992
#6	October 8, 1991	#14	April 7, 1992
#7	October 10, 1991	#15	April 8, 1992
#8	October 15, 1991		

Analysis of Flows at the U.S. 290 at Tidwell Study Site

Measured Peak 15-minute Volumes

To be consistent with existing convention, 15-minute peak flows were calculated for each of the sample days. The peak 15-minute flow rates are given in Table 3. The flow rates shown in Table 3 represent the peak 15-minute flow rate across all lanes for 15 consecutive minutes. Individual lanes may have higher peak 15-minute flow rates that occur at different times.

TABLE 3 U.S. 290 at Tidwell Peak 15 Minute Flow Rates					
Highway	Sample	Peak 15-Minute Flow Rate (vphpl)			
		Lane 1	Lane 2	Lane 3	Average
U.S. 290	1	2336	2256	2348	2313
	2	2320	2200	2140	2220
	3	2300	2244	2456	2333
	4	2380	2240	2060	2227
	5	2288	2172	2180	2213
	6	2368	2204	2312	2295
	7	2260	2224	2120	2201
	8	2228	2196	2228	2217
	9	2220	2268	2132	2207
	10	2384	2256	2188	2276
	11	2496	2244	2172	2304
	12	2252	2220	2348	2273
	13	2408	2344	2156	2303
	14	2248	2356	2028	2211
	15	2312	2240	2068	2207
	Average	2320	2244	2196	2253

As can be seen in Table 3, the average 15-minute flow rates across all three lanes ranged from 2,201 vphpl to 2,333 vphpl. Therefore, all of the measurements exceeded the 2,000 pcphpl value given in the 1985 HCM. Also shown in Table 3 are the average traffic volumes by lane which indicate that the inside lane (lane 1) has the highest average flow rate of 2320 vph, while the middle (lane 2) and outside (lane 3) lanes have more similar flow rates of 2244 vph and 2196 vph respectively, during the peak 15 minutes. Adjusting for heavy vehicle percentages, which are shown in Table 4 and discussed in the next section, the traffic volumes are very similar between lanes. The adjusted volumes (assuming an E_t of 2.0) are 2,337 pcphpl, 2,310 pcphpl, and 2,311 pcphpl for lanes 1, 2, and 3, respectively.

Although the peak 15-minute flow rates reported in Table 3 follow the existing convention detailed by the 1985 HCM, it is likely that they are impractical to use as maximum sustainable flow. As mentioned in the background, a potential problem with the use of peak 15-minute flow rates are the implications of the two capacity hypothesis. If two capacities (or maximum flows) exist, then it is necessary to know when free flow and forced flow conditions occur because averages that do not segregate the data are corrupted by the two flow regimes. This means that it is possible to have flows higher than queue discharge but under free-flow capacity. The result may be a flow that is not sustainable at higher demand levels where queue discharge flows occur. For this reason, the characteristics of flow during free-flow and forced flow conditions were examined in considerable depth to obtain a practical estimate of the maximum sustainable flow.

Heavy Vehicles

Although truck traffic was present, no attempt was made to adjust for truck passenger car equivalents (PCEs) in the preliminary analysis. The heavy vehicles percentages for each lane during the peak period are shown in Table 4. Truck percentages averaged 3.0% for all lanes combined. The highest percentages were in the outside lane which averaged 5.3% trucks, while the inside lane had the lowest average truck percentage with 0.7%. The reported truck percentages reflect the truck traffic for the study periods.

TABLE 4 Heavy Vehicle Percentages at U.S. 290 at Tidwell					
Highway	Sample	Truck Percentage			
		Lane 1	Lane 2	Lane 3	Average
U.S. 290	1	.82	3.49	5.66	3.35
	2	.71	3.68	5.7	3.39
	3	.75	3.26	4.87	2.98
	4	.90	3.48	5.78	3.30
	5	.74	3.34	5.72	3.33
	6	.85	2.77	5.48	3.07
	7	.38	3.59	6.30	3.39
	8	.54	2.81	6.06	3.11
	9	.58	2.99	4.33	2.66
	10	1.09	2.34	4.85	2.81
	11	.94	1.84	4.73	2.55
	12	.98	2.87	4.86	2.86
	13	Equipment Failure	Equipment Failure	Equipment Failure	Equipment Failure
	14	.35	1.72	4.22	2.14
	15	Equipment Failure	Equipment Failure	Equipment Failure	Equipment Failure
	Average	.74	2.94	5.28	3.00

Characteristics of Flow

To study the characteristics of flow, speed versus time and flow versus time plots were made for each day. Figures in Appendix A show plots of speed and flow versus time for each sample day. It is important to determine the correct time in which queue discharge begins and ends in a flow process, which calls for the shortest time interval possible. A 1-minute interval was considered accurate for illustration purposes. For analysis, 30-second data were used.

The speed-time plots given in the figures in Appendix A show many important flow characteristics. The first characteristic is change in average speed. At the beginning of the time period the average speed of the traffic stream is very high, between 60 and 70 mph for the median lane (lane 1). As time progresses and flows increase, the average speed lowers, but very gradually. Suddenly, a substantial and nearly instantaneous drop in speed occurs and speeds stabilize at about 50 mph (for lane 1).

To distinguish between free-flow and queue discharge conditions, the point at which the demand exceeded the service rate was identified. Demand is considered to exceed the service rate when vehicle speed is controlled by the service rate of the bottleneck, thus making vehicles wait to resume their desired speed. The excess demand generates queues producing queue discharge flow. The point at which a rapid drop in 30-second average speed occurs, which is shown conceptually in Figure 8, was determined to be the beginning of queue discharge because vehicles are forced to a lower speed than previously desired under prevailing conditions.

As mentioned, even before the speed drop, average speeds are lower than truly free-flow conditions and therefore, some argument may be made that they are not in fact "free-flow." Although this interpretation may be correct, the most important element for determining maximum sustainable flow is the differentiation between congested and uncongested operation. Therefore, all data before the speed drop were considered free-flow.

A similar argument can be made for flows after the speed drop. Because speeds have dropped to a much lower level, signifying a change in operational characteristics, it is not evident in the plots themselves that queue discharge conditions exist immediately subsequent to the speed drop. Although this is a relevant argument, initially flows after the speed drop will be considered queue discharge in nature. One reason for using the speed

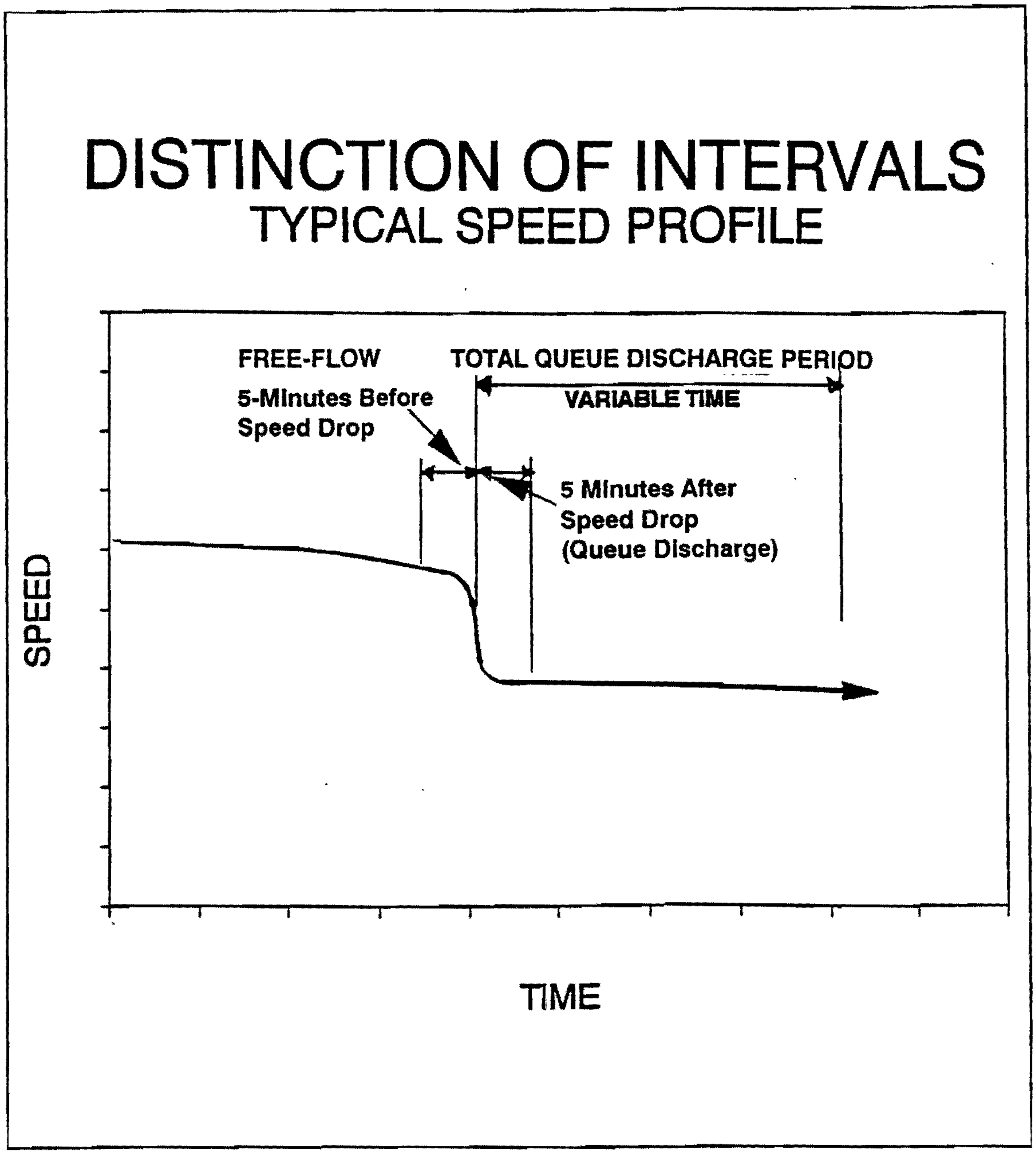


FIGURE 8 Time intervals used for analysis.

drop as an exact boundary between free-flow and queue discharge is that the speed drop is a consistent procedure for distinguishing between regions of flow. Visual determination of queue discharge is valuable, but extremely subjective. Further study of flow in this region was performed to determine if this assumption is valid and is addressed in later sections.

The second important characteristic is the relation between lanes. Although each lane has many of the same general characteristics, the average speeds differ substantially. For example, the average free-flow speeds in lane one are between 65 mph and 75 mph for most days while the free-flow speeds for lane two are between 60 mph and 65 mph and the average free-flow speeds for lane three are about 50 mph to 55 mph. These differences are extremely important because if the speeds are significantly different for each lane then the corresponding speed-flow relationship is different for each lane, and aggregation of all the lanes is misleading. For this reason, the analysis is by lane and comparisons are made to determine if significant differences exist and how they might be explained.

Time Periods Used For Comparison of Flow Rates

To compare the characteristics of flow before and during queue discharge, three time periods were used:

1. The five minutes directly before the speed drop (free-flow period).
2. The five minutes directly after the speed drop (5-minute queue discharge period).
3. The entire time period after speed drop until end of queue discharge, formation of a downstream queue or the end of data collection (queue discharge).

These time periods in relation to a typical speed profile are illustrated in Figure 8.

The 5-minute periods before and after the speed drop were used to determine whether the characteristics of flow changed significantly after the speed drop. Five-minute intervals were chosen for two reasons. The first and most important is that before the speed drop the speed is influenced by the flow rate, and therefore the larger the time interval, the lower the flow and the higher the speed. This introduces error because all samples cannot be considered to be from the same population if speed is a function of flow, which is increasing. The second factor is the need for a long enough time period to get an adequate number of samples. Based on these considerations, a 5-minute interval was chosen. This interval was assumed to be short enough to not introduce substantially different flows and speeds than those seen in the vicinity of the speed drop, but long enough to make an adequate estimate of flow.

Calculation of flows for the three time periods was based on the average headways measured for the time period. The equation for flow is shown below:

$$Flow\ Rate = \frac{3600}{\bar{h}} \qquad \bar{h} = Average\ \ headway\ \ (seconds)$$

Table 5 shows the average calculated flow rates for the three time periods. As can be seen in Table 5, there is much variation in flow rates during all three intervals for different samples. Because of the variation within individual samples between regions, it was necessary to perform a statistical analysis to determine the significance of the speed drop on flow characteristics.

Statistical Analysis of Speed Drop for Individual Samples

The statistical analysis of the capacity drop was done in two main steps. The comparison of standard deviations of the 5-minute periods before and after the speed drop was done first using an F-test for each sample day. This test determines if the standard deviations of vehicle headways after queue formation are significantly different and also is the basis for determining which test statistic to use in the analysis of the statistical differences in means between the two periods.

In using the F-test, it was assumed that the 5-minute periods before queue discharge and at the beginning of queue discharge were independent and the sample mean headways within them have a normal distribution. Because of the use of a large sample of headways,

TABLE 5
Comparison of Flow Rates Before and After the Speed Drop at U.S. 290 at Tidwell

Sample	5 Minutes Before Speed Drop (vph)				5 Minutes After Speed Drop (vph)				Entire Queue Discharge Period After Speed Drop (vph)			
	Lane 1	Lane 2	Lane 3	Avg	Lane 1	Lane 2	Lane 3	Avg	Lane 1	Lane 2	Lane 3	Avg
1	2142	1951	2109	2067	2365	2128	2231	2241	2328	2213	2254	2265
2	2170	2125	2449	2248	2177	2063	1964	2068	2238	2135	2083	2152
3	1993	2124	2290	2135	2212	2112	2113	2146	2201	2191	2163	2185
4	2139	1863	2262	2088	2356	2035	1931	2107	2302	2114	1987	2134
5	1990	1918	2267	2058	2316	2037	2080	2144	2250	2120	2129	2166
6	2013	2207	2469	2230	2433	2209	2091	2244	2280	2224	2159	2221
7	1903	1910	1933	1916	2017	2041	1885	1981	2177	2111	2039	2109
8	2099	1836	2191	2042	2008	1607	1991	1869	2047	2000	2065	2037
9	2228	2289	2298	2271	2132	1960	2094	2062	2194	2141	2152	2162
10	1929	1690	1916	1845	2311	2052	1985	2116	2339	2233	2126	2233
11	1849	1883	2167	1966	2525	2080	2049	2218	2312	2248	2090	2217
12	2033	2063	1862	1986	2209	2122	2019	2116	2241	2114	2146	2167
13	2085	2059	2372	2172	2381	2076	2167	2208	2366	2252	2150	2256
14	2340	2123	2368	2277	2311	1967	1824	2034	2227	2182	1978	2129
15	2232	1985	2197	2138	2243	2044	1405	1897	2187	2140	1835	2054

this assumption is considered valid based on the central limit theorem, which states that as sample size increases, the distribution of sample means approaches normality. The results of the F-test are shown in Table 6.

TABLE 6
Test of Headway Variances for 5-Minute Intervals at U.S. 290 at Tidwell

Sample	Lane 1		Lane 2		Lane 3	
	F-stat	Significant	F-stat	Significant	F-stat	Significant
1	2.82	Yes	2.05	Yes	2.79	Yes
2	3.78	Yes	2.00	Yes	.66	Yes
3	3.84	Yes	1.14	No	1.04	No
4	4.79	Yes	3.88	Yes	1.36	No
5	7.38	Yes	1.24	No	2.69	Yes
6	5.66	Yes	1.28	No	.82	No
7	2.30	Yes	1.93	Yes	2.26	No
8	2.50	Yes	.79	No	1.40	Yes
9	2.50	Yes	.79	No	1.40	Yes
10	3.73	Yes	2.56	Yes	.85	No
11	8.55	Yes	3.30	Yes	1.78	Yes
12	2.35	Yes	1.87	Yes	2.83	Yes
13	4.95	Yes	1.72	Yes	1.01	No
14	2.28	Yes	1.37	No	.4	Yes
15	1.66	Yes	601	Yes	.06	Yes

As can be seen in Table 6, the statistical significance of the standard deviations vary by lane within a single sample and between samples for lanes 2 and 3. One interesting result is that in all samples for lane 1 the variances (and standard deviations) were significantly different before queue discharge than during queue discharge. In all cases the variance before queue discharge was larger than after queue discharge. This result was not as evident in lanes 2 and 3. For lane 2, 9 out of the 15 samples had significantly different variances and in lane 3, 10 out of the 15 samples had significantly different variances. All lanes had one thing in common, which was at least half of the samples had statistically different variances for pre-queue and queue discharge conditions.

The results of the F-test indicate that for the most part, the two time periods have unequal variances. Because the variances were significantly different in a number of cases, the mean headways before and during queue discharge were compared using a t-test approximation for two means with different variances. This test also assumes that the distribution of sample mean headway is normal. Although the distribution of mean headways may be slightly skewed, based on the central limit theorem the assumption of normality is assumed valid because of the large sample sizes. Table 7 shows the results of the statistical test of mean headways for the 5-minute periods before and during queue discharge.

Based on the results given in Table 7, it is difficult to conclude that the mean headways and therefore, the mean flow rate, decreased for the 5 minutes after the start of queue discharge as would be predicted by the two capacity hypothesis. One factor that should be noted is that the standard deviations are large and therefore, a very large difference in headway is required to make the difference statistically significant. Also shown in Table 7 are the differences in flow rates. To further illustrate the high standard deviations of the 5-minute samples, confidence intervals were constructed at a 5 percent level of significance for the means before and during queue discharge. The results for lane 1 are shown in Table 8.

The large amount of variability in vehicle headways also shows the problems with attempting to evaluate the impact of trucks and other vehicles types on headways. For instance the average headways for the 5-minute period before queue discharge ranged from 1.53 seconds to 1.94 seconds with standard deviations ranging from .91 to 1.97. Because of

TABLE 7
Test of the Difference in Mean Headways For U.S. 290 at Tidwell

Day	Lane 1		Lane 2		Lane 3		Avg. Flow Change per Lane
	Flow Change (vph)	Significant at 5%	Flow Change (vph)	Significant at 5%	Flow Change (vph)	Significant at 5%	
1	223	No	177	No	122	No	174
2	7	No	-61	No	-485	Yes	-180
3	219	No	-12	No	-176	No	10
4	217	No	171	No	-332	Yes	19
5	325	No	119	No	-188	No	86
6	420	Yes	2	No	-378	Yes	15
7	113	No	131	No	-50	No	65
8	-91	No	-230	No	-199	No	-173
9	-96	No	-329	Yes	-204	No	-210
10	382	Yes	362	Yes	69	No	271
11	676	Yes	197	No	-118	No	252
12	175	No	59	No	157	No	130
13	296	No	17	No	-205	Yes	36
14	-29	No	-156	No	-544	Yes	-243
15	12	No	59	No	-792	Yes	-241
Total # Significant		3		2		6	

the high variation within the traffic flow itself, evaluation of the impacts of truck and other vehicles on headways and traffic operations is difficult and would contain a great deal of error. In addition the truck volume was very low. Therefore, the impacts of trucks were only considered on a macroscopic level.

TABLE 8
Confidence Intervals for Lane 1 at U.S. 290 at Tidwell

Sample	5-min Before Queue Discharge		5-min During Queue Discharge		Queue Discharge	
	Mean Flow	95% CI (+/-)	Mean Flow	95% CI (+/-)	Mean Flow	95% CI (+/-)
1	2142	298	2365	206	2328	78
2	2170	423	2177	200	2238	59
3	1993	355	2212	207	2201	72
4	2139	363	2356	186	2302	53
5	1990	350	2316	158	2250	57
6	2013	307	2433	175	2280	60
7	1903	256	2017	182	2177	47
8	2099	329	2008	179	2047	54
9	2228	248	2132	202	2194	53
10	1929	302	2311	212	2339	51
11	1849	361	2525	209	2312	70
12	2033	327	2209	240	2241	46
13	2085	280	2381	154	2366	52
14	2340	326	2311	202	2227	47
15	2232	201	2243	155	2187	47

Before queue discharge the mean flow rates have confidence intervals ranging from ± 201 vph to ± 423 vph. The initial 5-minute intervals during queue discharge also have very large confidence intervals, but lower than before queue discharge. This concurs with the test on variances which found that the variances significantly dropped in queue discharge. The small confidence intervals for the entire queue discharge period are partially the result of large sample sizes but show that a reasonably accurate sample can be obtained from a single day. The high variability within each sample before queue discharge indicates the potential problem with using a maximum flow in free-flow conditions. With such high variability, operation in free-flow conditions would likely be difficult to maintain assuming the mean flow was used.

Some important observations are that the mean flow rate increased in queue discharge in 12 of the 15 samples for lane 1, 10 of the 15 samples in lane 2, and only in 3 samples in lane 3. This indicates that although many of the differences are not statistically significant at the 95 percent level of confidence, some changes in flow did occur. Lanes 1 and 2 tended to increase in flow from free-flow conditions to queue discharge while the flow in lane 3 decreased. Although the individual lanes show some trends, the averages across all lanes do not. As shown in Table 7, while some samples decreased in queue discharge, many increased indicating that in aggregate the flow does not necessarily increase or decrease during queue discharge.

Analysis of Average Flows for all Samples Combined

Up to this point the focus of the statistical analysis has been on the characteristics of each individual sample. The statistical analysis of individual samples have indicated that the variances decrease in queue discharge conditions and that each lane has slightly different characteristics. In order to estimate the maximum sustainable flow, all the days were evaluated together. To evaluate the characteristics of flows between days, statistics were calculated for the mean flows given in Table 5. The statistics for average daily flows are shown in Table 9.

Table 9 shows the standard deviation and 95% confidence interval on the mean of all 15 samples for each time period. The confidence interval assumes that the distribution of sample means follows a normal distribution, which should be applicable since a relatively large sample size was used. The frequency distributions of flow rates for the average flow across all lanes during free-flow and queue discharge are shown in Figures 9 and 10. As can be seen, the distributions appear to be normal especially in queue discharge. Using a 50 vph grouping interval, the mean, median and modes are the same for the samples in both study intervals.

TABLE 9
Statistics for Daily Averages for U.S. 290 at Tidwell

Time Period	Statistic	Lane 1	Lane 2	Lane 3	Average
5-min Before Speed Drop (Free-Flow)	Average Flow (vph)	2076	2002	2210	2096
	Std Dev.	136	159	187	132
	95% CI (+/-)	75	88	104	65
5-min After Speed Drop	Average Flow (vph)	2266	2035	1989	2097
	Std. Dev.	145	134	193	115
	95% CI (+/-)	80	74	107	64
Total Queue Discharge Period	Average Flow (vph)	2246	2161	2090	2166
	Std. Dev.	81	68	101	67
	95% CI (+/-)	45	38	56	37

The average flows show some interesting trends. While the mean average flow rate across all lanes does not change much for the 5 minutes before the speed drop compared to the 5 minutes after the speed drop, the distribution of traffic across the lanes does change. This is especially evident in lanes 1 and 3. In lane 1 the mean traffic volume

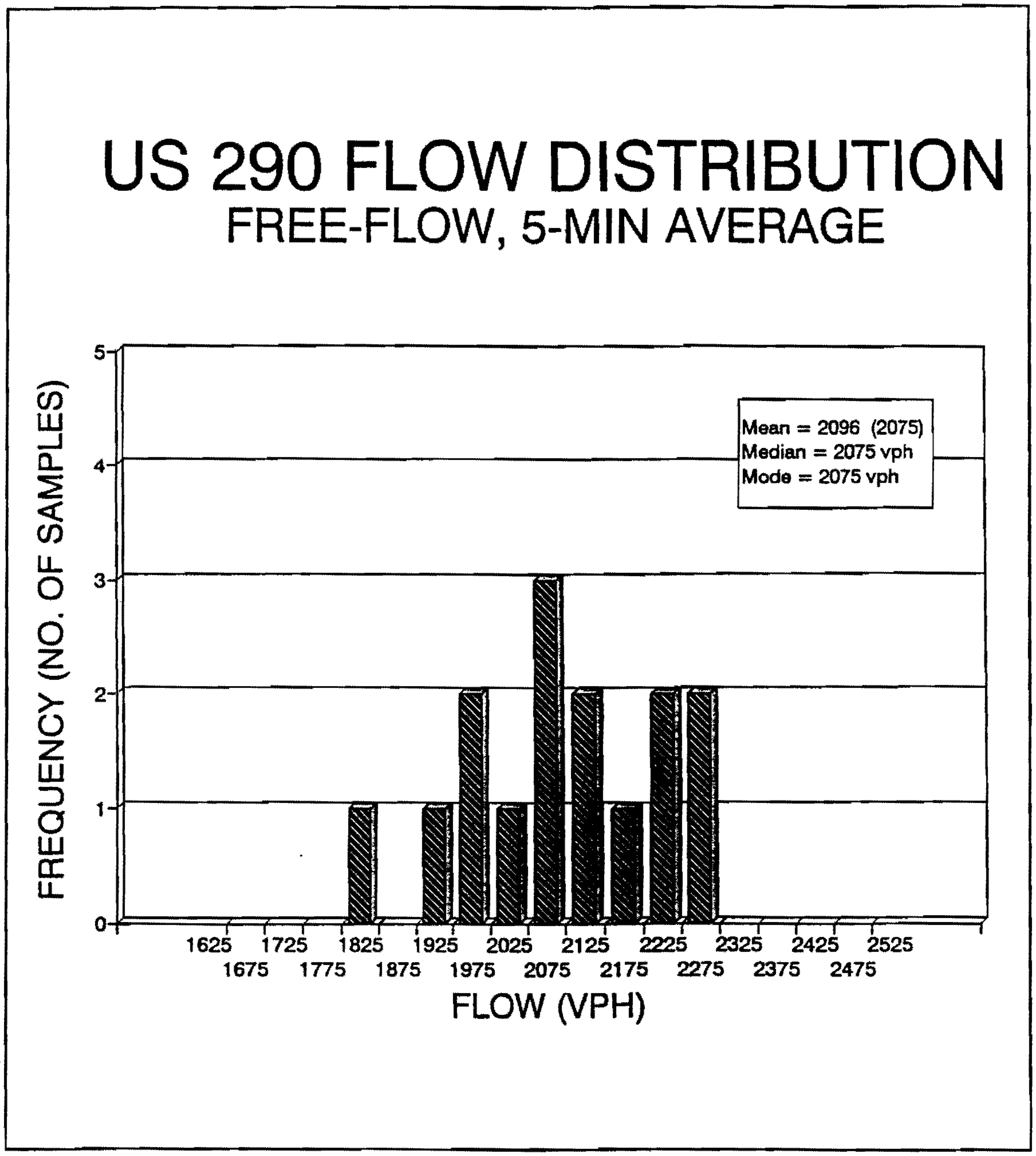


FIGURE 9 Distribution of average free-flow flow rates over all lanes before speed drop.

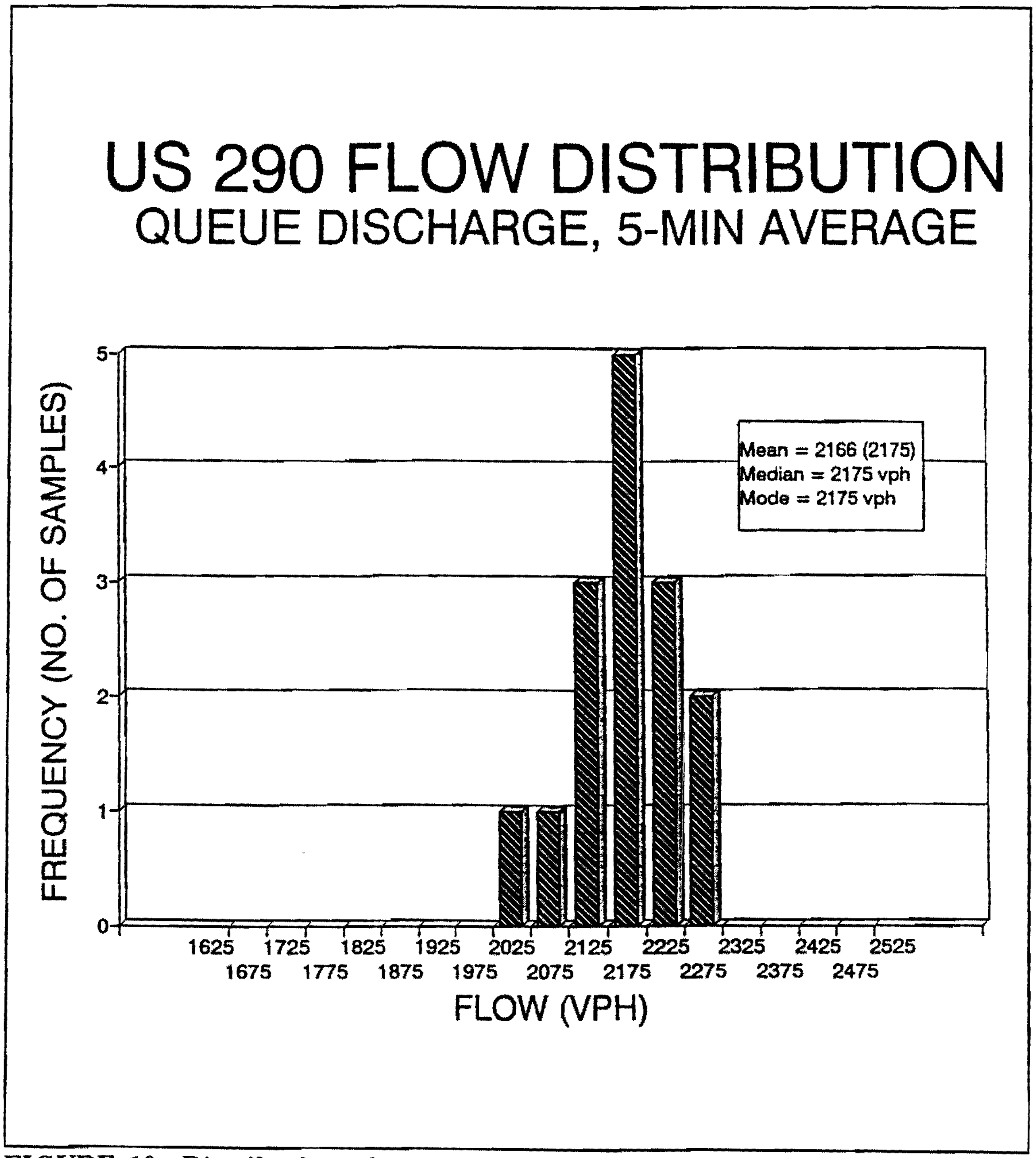


FIGURE 10 Distribution of average queue discharge flow rates over all lanes after speed drop.

substantially increased after the speed drop while the mean flow in lane 3 decreased after the speed drop. This could be explained by the fact that in free-flow conditions lane 1 has not reached its maximum flow and operates at very high speeds, while lane 3 is already becoming congested because of high merge volumes associated with relatively high freeway volumes. As reported earlier, during the study hour (4:00 p.m. to 5:00 p.m.) the ramp merge volume was 750 vph, which when combined with even a moderate freeway volume in lane 3 causes lane 3 to break down. Once lane 3 becomes congested and slows down, traffic merges into lanes 2 and 1 and subsequently drops the speed and transitions the flow into queue discharge conditions. Therefore, if the theoretical maximum flow occurs in free-flow conditions, it is not seen in lane 1 since lane 1 transitioned from free-flow, with a flow rate below maximum flow, directly to queue discharge.

The analysis of means points out another problem with using free-flow conditions to obtain a maximum sustainable flow rate. Although a maximum flow rate in free-flow conditions exists, in certain lanes it may never be reached and therefore, it would be impossible to achieve. For example, say lane 1 had a maximum sustainable flow of 2,400 vph, but because of the turbulence created by lanes 2 and 3, the flow rate in lane 1 transitioned directly from 2,076 vph to 2,266 vph in queue discharge as the data would suggest. The flow rate of 2,400 vph would never be reached and therefore could not be considered the maximum flow rate. Of course, this assumes that the maximum flow rate occurs in free-flow conditions, which according to this data only occurs in lane 3.

The standard deviations for the 5-minute periods before and after the speed drop show some unexpected results. Although it appears that for individual samples the variance decreased in the 5-minutes after the speed drop, some of the sample variances shown in Table 9 did not. In lanes 1 and 3 the standard deviations actually increased slightly which would not be expected. One explanation for this occurrence is the way in which the boundary of the free-flow and queue discharge conditions were chosen. As shown in Figure 8 the boundary was determined based on the speed drop. As can be seen in Appendix A, a large speed drop into what would appear to be queue discharge conditions does not occur in all samples. Although some type of speed drop does occur in all samples, the extent of time over which it occurs is not the same. The samples for times 3, 6, 8, 14 and 15 do not show a rapid speed drop down to approximately 50 mph which indicates queue discharge conditions. Many of these samples show a steady or sporadic transition to queue discharge which decreases the quality of the comparison of the 5 minutes before and 5 minutes after the speed drop, especially if the speed in one sample may be 55 mph and the average speed in the other sample may be 50 mph. Because speed drops occur in these samples, the differences in variances for the individual samples before and after each speed drop are significant, but because all the samples are not dropping to the same operational conditions the variance between samples remained high.

Another important thing to note about the flows directly after the speed drop is that they are lower than those for the entire period of queue discharge for lanes 2 and 3. It is difficult to determine the cause of this phenomenon, but the way the boundary between queue discharge and free-flow was chosen could explain the differences. During the first 5 minutes, all the lanes are still in a state of change as traffic moves from one to another to maximize speed. Once all lanes are in queue discharge, traffic volumes begin to stabilize and become more evenly distributed.

To investigate the effect of the samples that did not appear to drop directly into queue discharge, the means and variances before and during queue discharge were calculated for samples 2, 5, 9, 10, and 13. These samples had rapid speed drops to queue discharge. Table 10 gives the statistics calculated for these samples.

As would be expected, for all lanes the standard deviations dropped in the 5 minutes after the speed drop. This concurs with the previous explanation for the lack of change in the standard deviations given in Table 8. Although all samples experience some type of speed drop, the samples that experience a rapid speed drop to a constant speed queue discharge condition show a reduction in variance in queue discharge conditions, while all samples together do not. The important result of this is that queue discharge is not always reached via a nearly instantaneous speed drop. Therefore, the region labeled queue discharge in this analysis may not be completely in queue discharge in some samples for an initial period, but rather somewhere between free-flow and queue discharge conditions.

TABLE 10
Statistics for Samples with a Rapid Speed Drop at U.S. 290 at Tidwell

Time Period	Statistic	Lane 1	Lane 2	Lane 3	Average
5-min Before Speed Drop (Free-Flow)	Average Flow (vph)	2080	2016	2260	2119
	Std Dev.	123	226	205	174
	95% CI (+/-)	171	313	284	242
5-min After Speed Drop	Average Flow (vph)	2263	2038	2058	2120
	Std. Dev.	104	46	83	60
	95% CI (+/-)	145	64	116	83
Total Queue Discharge Period	Average Flow (vph)	2278	2176	2128	2194
	Std. Dev.	72	61	28	47
	95% CI (+/-)	100	85	38	65

Especially reassuring is the small confidence interval values for the large samples during the total queue discharge period. The means and confidence intervals for lanes 1, 2, and 3 were 2,246 ± 45 vph, 2,161 ± 38 vph, and 2,090 ± 56 vph respectively. The confidence interval for all lanes averaged was 2,166 ± 37 vph. The low end of all of these values is well over the 2,000 pcphpl given by the HCM. The small confidence intervals also show the advantage of using a queue discharge flow to determine maximum sustainable flow, which is the ability to obtain large samples with low variability and therefore predict a relatively accurate mean flow rate.

Based on the analysis to this point some basic conclusions can be made:

- The characteristics of speed and flow vary by lane.
- The variance of headways and flows is greater during free-flow conditions than during queue discharge for each individual sample.
- The peak flow in lane 3 occurs during free-flow conditions, while the peak flow for lanes 1 and 2 occurs during queue discharge conditions. This indicates that once lane 3 becomes congested due to high merge volumes and slows down, lanes 1 and 2 are transitioned into queue discharge conditions.
- Not all samples experience the same magnitude of speed drop; and therefore, the region labeled queue discharge does not always display the same flow characteristics directly after the speed drop.
- The average of hourly flow rates for the 15 days during the entire queue discharge period produces a relatively accurate estimate of the mean queue discharge flow rate.

Comparison of Peak 15-Minutes to Other Flows

The one issue that has not been addressed directly is the relation between the free-flow and queue discharge regions and the peak 15-minute flow rates. The previous analysis showed that two distinct regions of flow exist and not all samples experienced the same type of speed drop. Therefore, peak 15-minute flows containing data from both free-flow and queue discharge are not necessarily sustainable, particularly in lanes which reach their peak flows in free-flow.

The relation of the peak 15-minute flow rates to both the free-flow and queue discharge regions is shown for lane 1 of all samples in Appendix A. In all samples the peak 15 minutes for both lane 1 and all the lanes combined occurred after the speed drop. This is a very important finding since most speed-flow models assume that the maximum flow occurs in the free-flow region, not in queue discharge.

To examine the relationship between queue discharge and peak 15-minute flow, the five samples which had a rapid speed drop were used. These data sets were considered the most likely to represent stable queue discharge. The queue discharge flows and peak 15 minute flows for these samples are shown in Table 11.

The peak 15-minute flows in Table 3 are similar to the average queue discharge flows for the samples in Table 11. The difference in the averages over all lanes is only 50 vph (2,194 vph compared to 2,244 vph). Based on the five samples, the 95% confidence interval is 2,194 ± 65 vph; therefore, the difference of 50 vph is not significant. What this indicates is that in constant queue discharge conditions, the peak 15-minute flow rate is very similar to the average flow rate during the entire queue discharge period. Since the peak 15 minute flow rate is simply the largest flow rate for 15 consecutive minutes, it would be expected to be slightly larger than the average.

TABLE 11
Comparison of Average Queue Discharge and Peak 15-Minute Flows at U.S. 290 at Tidwell

Sample	Lane 1		Lane 2		Lane 3		All Lanes	
	Queue	Peak 15	Queue	Peak 15	Queue	Peak 15	Queue	Peak 15
2	2238	2320	2135	2200	2083	2140	2152	2220
5	2250	2288	2120	2172	2129	2180	2166	2213
9	2194	2220	2141	2268	2152	2132	2162	2207
10	2339	2384	2233	2256	2126	2188	2233	2276
13	2366	2408	2252	2344	2150	2156	2256	2303
Avg	2278	2324	2176	2248	2128	2159	2194	2244

Comparing the average for all samples to the peak 15-minute flow produces slightly different results. The overall average for all samples was 2,166 vph with a confidence interval of ± 37 vph. The average of peak 15-minute flows was 2,253. Based on all of the samples, the difference is significant. This is primarily a function of the larger sample size which increases the accuracy of the estimate. The slow transition to queue discharge may also affect the values, but because the queue discharge averages were based on a long interval, the effect should be small.

Based on these findings, the peak 15-minute flow rate across all lanes is larger than the average queue discharge flow rate, although the difference is small in many cases. This indicates that although the peak 15-minute flow rate may occur during queue discharge it does not provide a reasonable estimate of average queue discharge flow in all cases. Since it has been shown that individual lanes react differently it may be better to compare the peak 15 minutes for each individual lane to the average queue discharge flow rate for each individual lane. The major problem with this exercise is that the peak 15-minute flow rates by lane do not occur at the same time and therefore would give an inaccurate estimate of the maximum flow possible on the facility. The queue discharge flow rates occur at the same time and therefore account for lane interaction as do the peak 15-minute flow rates.

Relationship Between Speed and Flow

Figures 11 through 13 show the speed-flow plots for all samples combined for each lane. Five minute averages were used for the plots. As can be seen in the speed-flow figures, the peak flows in lanes 1 and 2 occur in queue discharge while the peak flows in lane 3 occur in free-flow. This supports the previous conclusion that because of lane interaction, once lane 3 transitions into queue discharge the other lanes quickly follow without reaching their theoretical maximum flow. The lines drawn through the points represent the estimated speed-flow relationship based on the data. It should be noted that some data points include downstream congestion, which is shown in Figure 11.

The transition from free-flow to queue discharge can also be observed for each sample in the figures in Appendix B. The Figures in Appendix B show the speed-flow plots for lane 1 and all lanes together for 5-minute moving averages. The purpose for using 5-

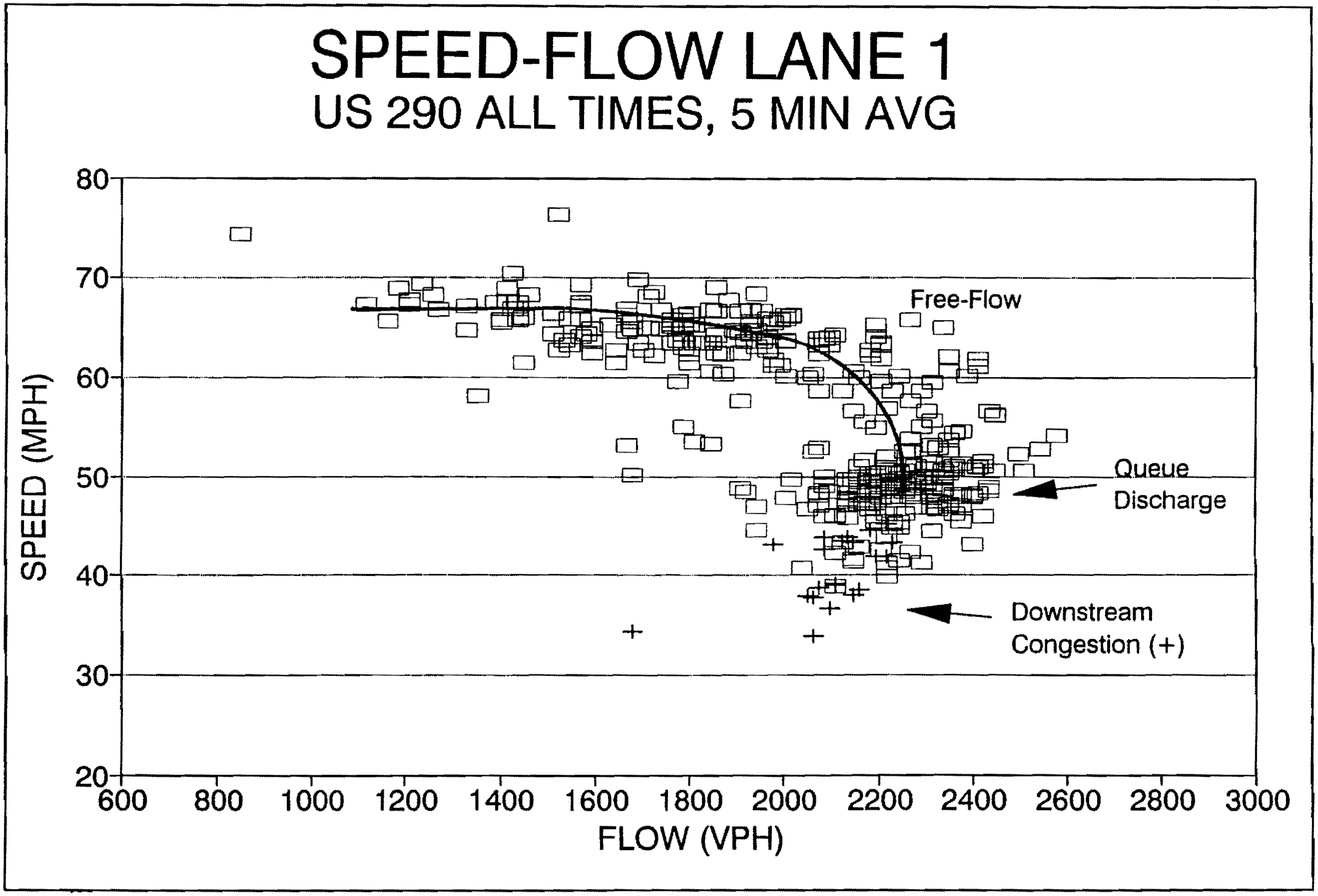


FIGURE 11 Speed-flow for lane 1 at U.S. 290 at Tidwell using 5-minute averages.

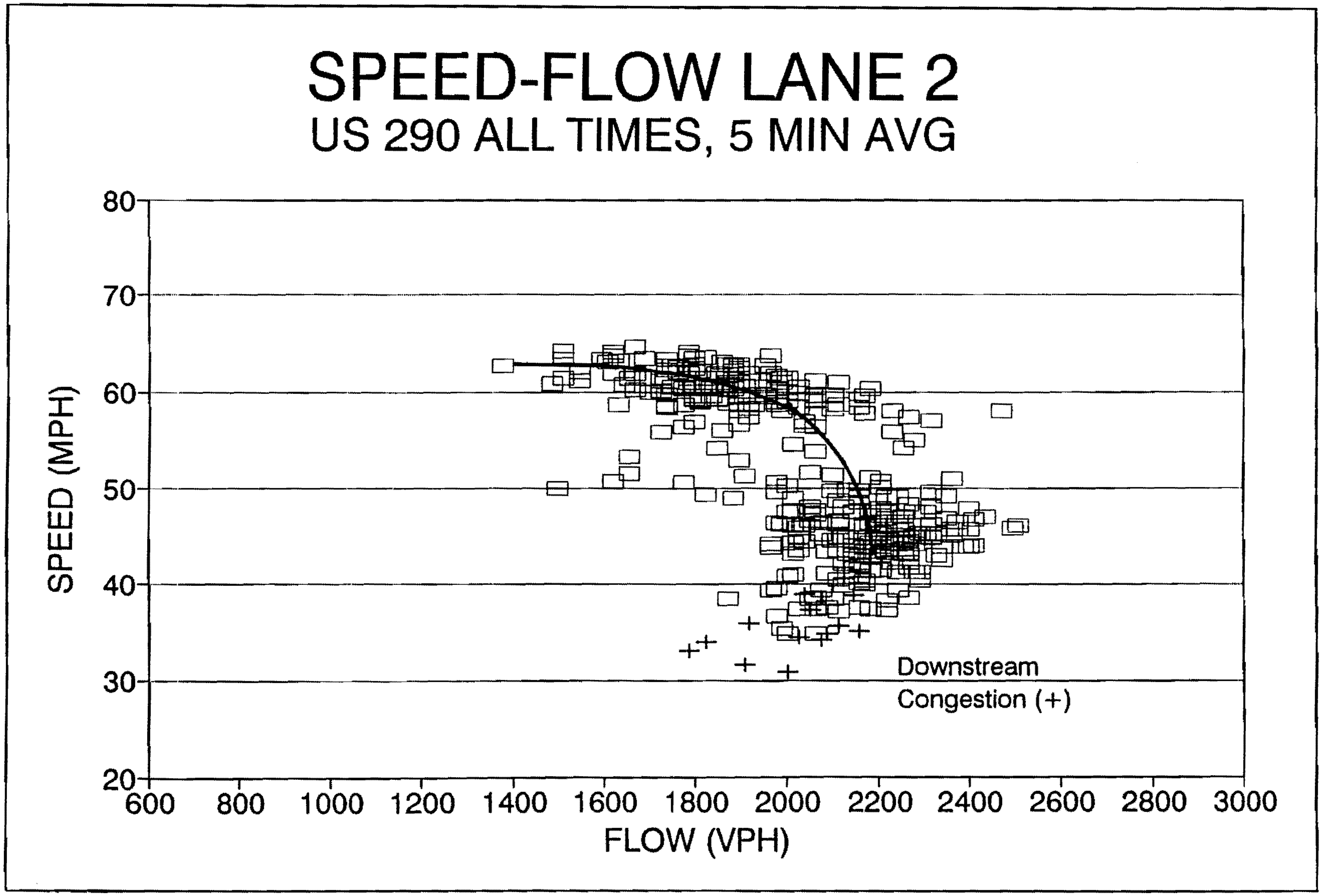


FIGURE 12 Speed-flow for lane 2 at U.S. 290 at Tidwell using 5-minute averages.

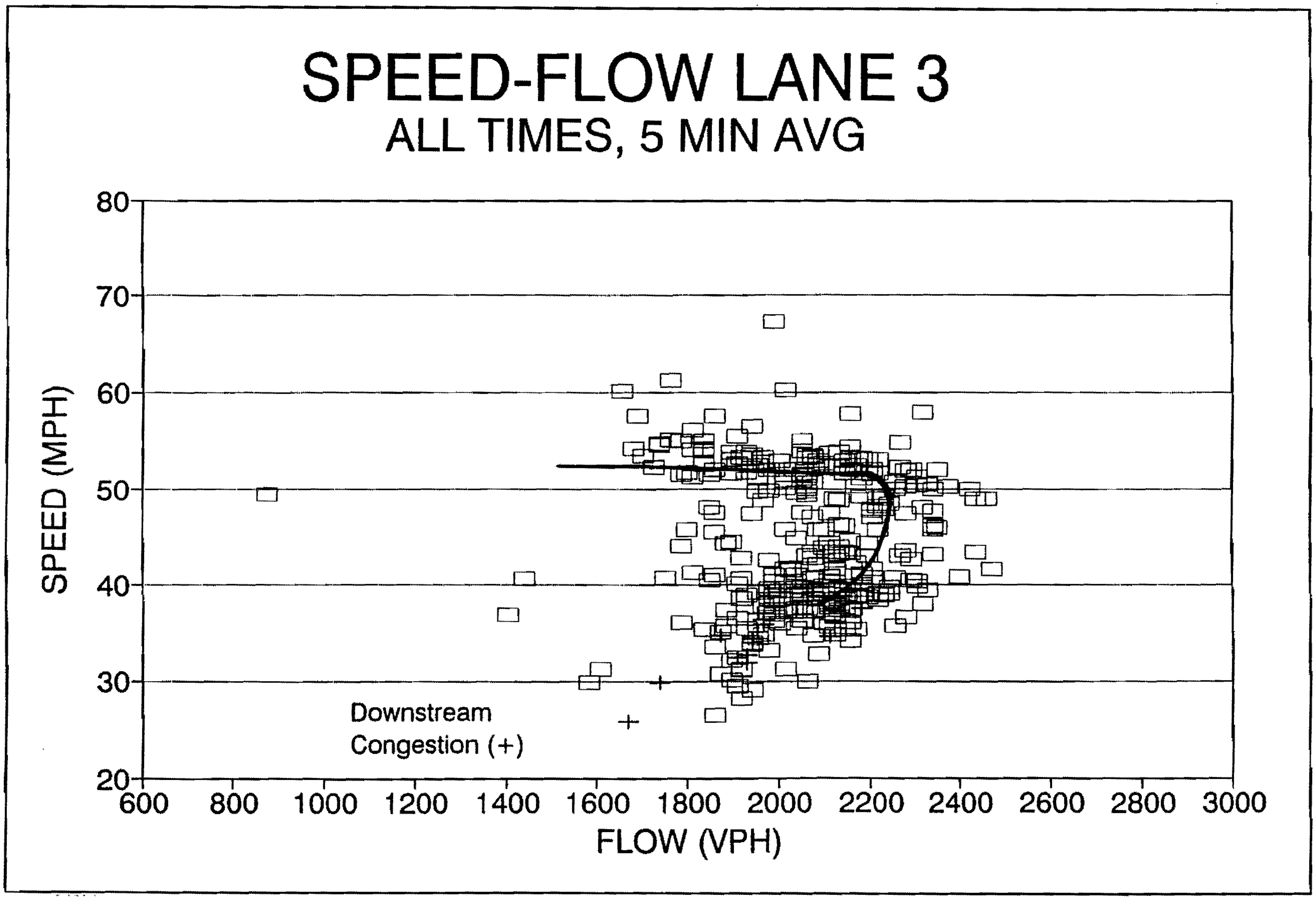


FIGURE 13 **Speed-flow for lane 3 at U.S. 290 at Tidwell using 5-minute averages.**

minute moving averages was to more accurately illustrate the transition of flow from free-flow to queue discharge. A moving average allows the average 5-minute flow for each sequential minute to be shown and therefore, all 5-minute periods are represented. Because the speed-flow plots were produced directly from the profiles in Appendix A, some of them also contain downstream congestion effects. An example of these effects is shown in Figure 14.

The interaction between lanes is a very important consideration because it shows that a single speed flow curve does not represent all lanes. One of the most significant problems is when lanes are combined to form a overall speed-flow relationship. If all lanes are combined, the flow rates and speeds are significantly different than those in individual lanes since all lanes do not break down the same. Although the entire relationship for a facility (all lanes together) is not similar to the individual lanes, during free-flow conditions it should be similar. Figure 15 shows the speed-flow plot for the average of all lanes for all samples at the U.S. 290 site during free-flow conditions. The relationship for the facility during free-flow conditions is similar to those across the individual lanes except for the speeds. This shows that in free-flow conditions the relationships derived from individual lanes can be used for the entire freeway, although each lane will likely have different free-flow speed characteristics than the average for the facility.

Results of the Preliminary Analysis

Based on the analysis of flow characteristics at the U.S. 290 at Tidwell study site, the following conclusions can be made:

1. The variances in flow decrease after the speed drop.
2. The flow rate in lane 3 decreased after the speed drop while flow rates in lanes 1 and 2 increased, indicating that once lane 3 transitions into queue discharge, lanes 1 and 2 also transition to queue discharge.
3. The transition to queue discharge can occur nearly instantaneously or over a longer period of time.

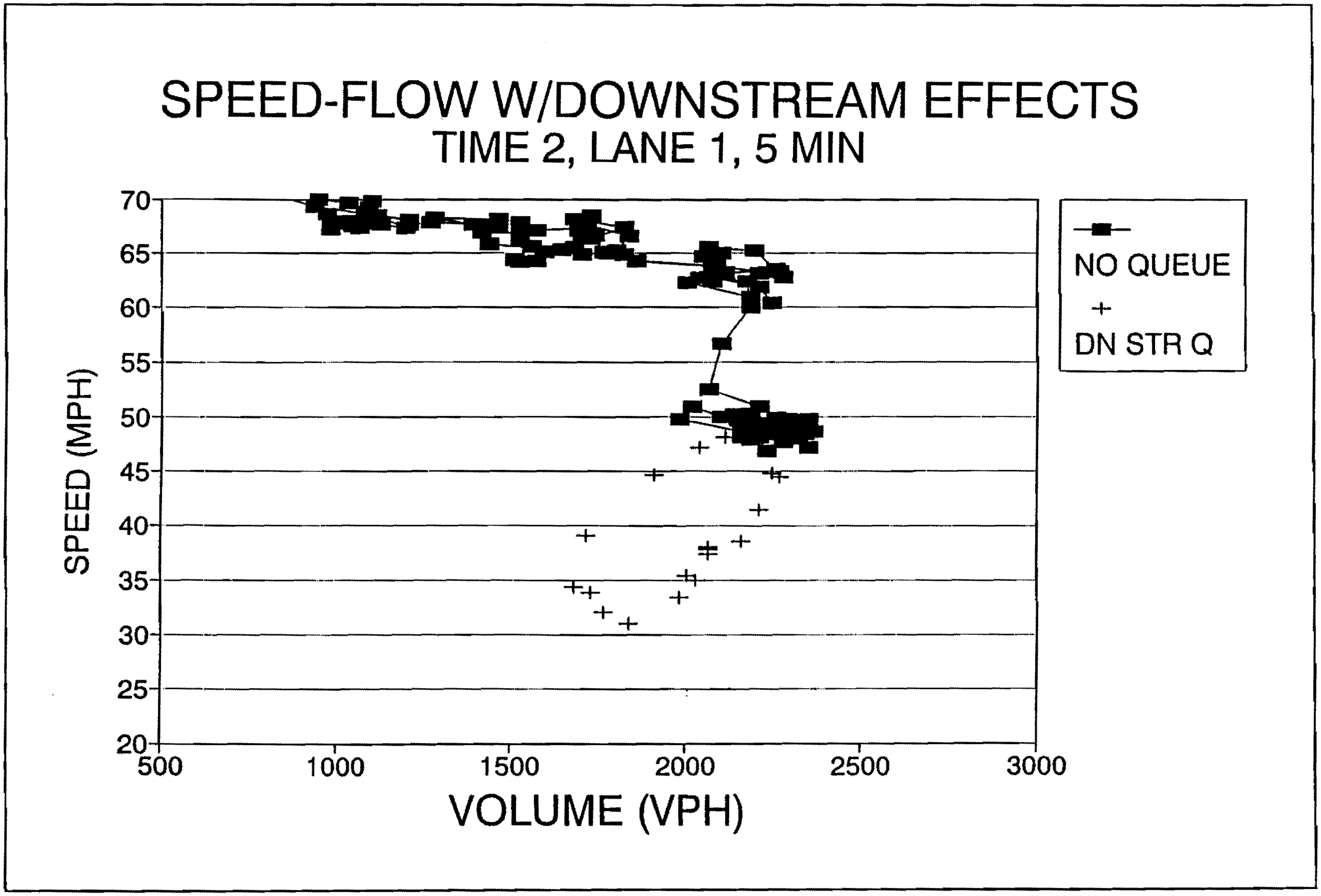


FIGURE 14 Effects of downstream congestion in sample 2 at U.S. 290 at Tidwell.

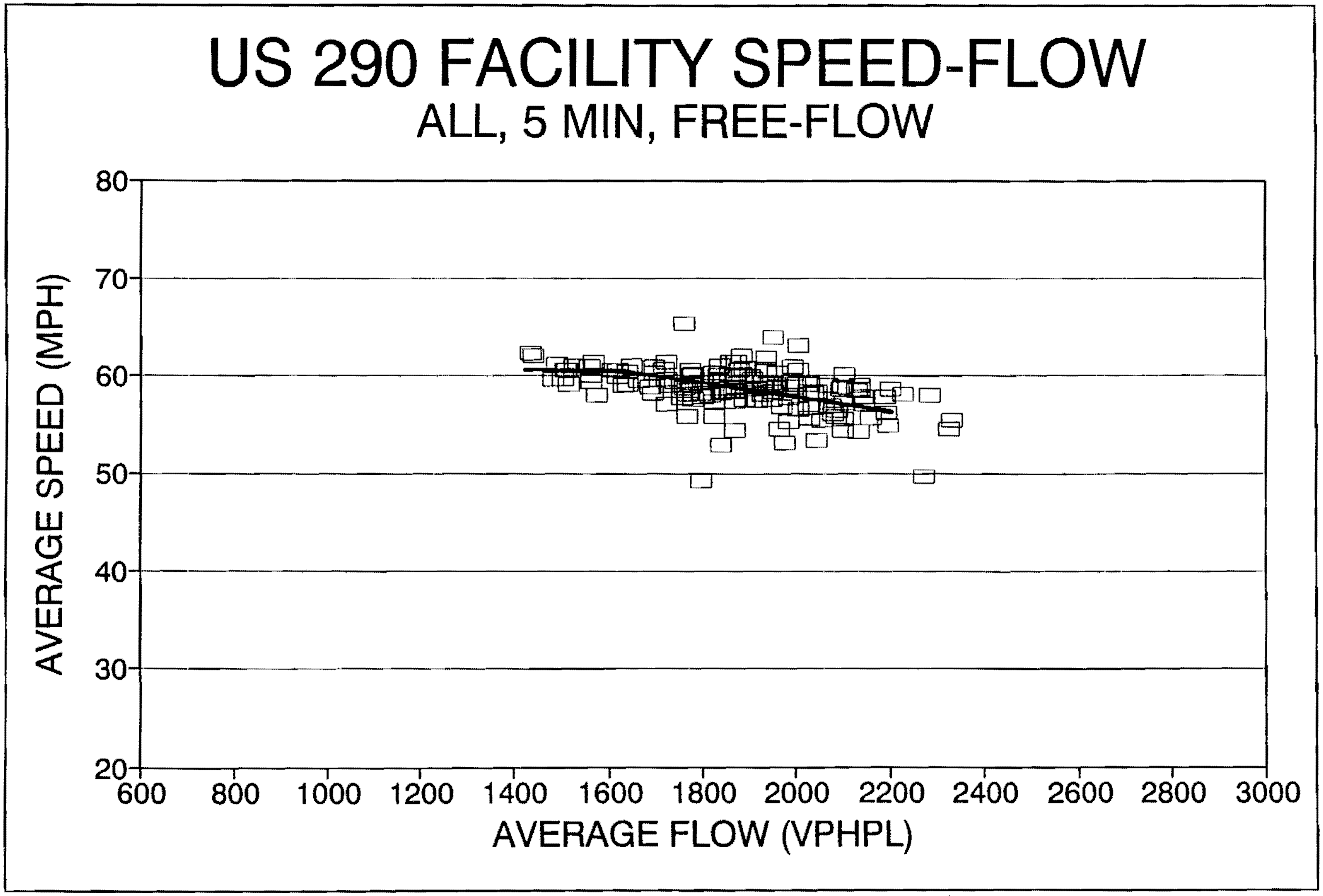


FIGURE 15 **Average speed-flow curve for U.S. 290 at Tidwell during free-flow using 5-minute averages**

4. Impacts of heavy vehicles on headways cannot easily be analyzed because of the high variability of individual vehicle headways.
5. Because of the variability and instability of flow during free-flow conditions and in the direct vicinity of the speed drop, use of these regions as maximum sustainable flow is not currently practical.
6. Peak 15-minute flows do not provide a reasonable estimate of the average maximum sustainable flow in all cases.

As a result of the statistical analysis of the two capacity hypothesis and the characteristics of flow in the queue discharge region, queue discharge is proposed as the best estimate of capacity. Because of the variability in flow beyond the speed drop, the average queue discharge flow rate for each lane is considered a conservative estimate. The overall averages for lanes 1, 2 and 3 were 2,246 vph, 2,161 vph, and 2,090 vph respectively. The averages for the five samples considered to have more stable queue discharge characteristics were 2,278 vph, 2,176 vph, and 2,128 vph for lanes 1, 2, and 3 respectively. These flows are slightly higher but do not occur in most samples and therefore cannot be considered average values. The overall averages are what commonly occur due to freeway demand fluctuations or changes in ramp demand and therefore illustrate a more practical maximum queue discharge flow rate.

Development of Preliminary Empirical Flow Model

Maximum Sustainable Flow

Based on the analysis of the freeway bottleneck on U.S. 290 at Tidwell in Houston the model shown in Figure 16 best represents the flow conditions during free flow and queue discharge. Queue discharge appears to be the best estimate for maximum sustainable flow and is therefore considered the best estimate of capacity. Shown in Table 12 are the queue discharge flow rates for each lane, the average for the facility, and the approximate truck percentages. The average queue discharge flow rate across all lanes was 2166 vphpl. Applying a truck equivalency factor (E_t=2.0 for nearly level sections) given in the 1985

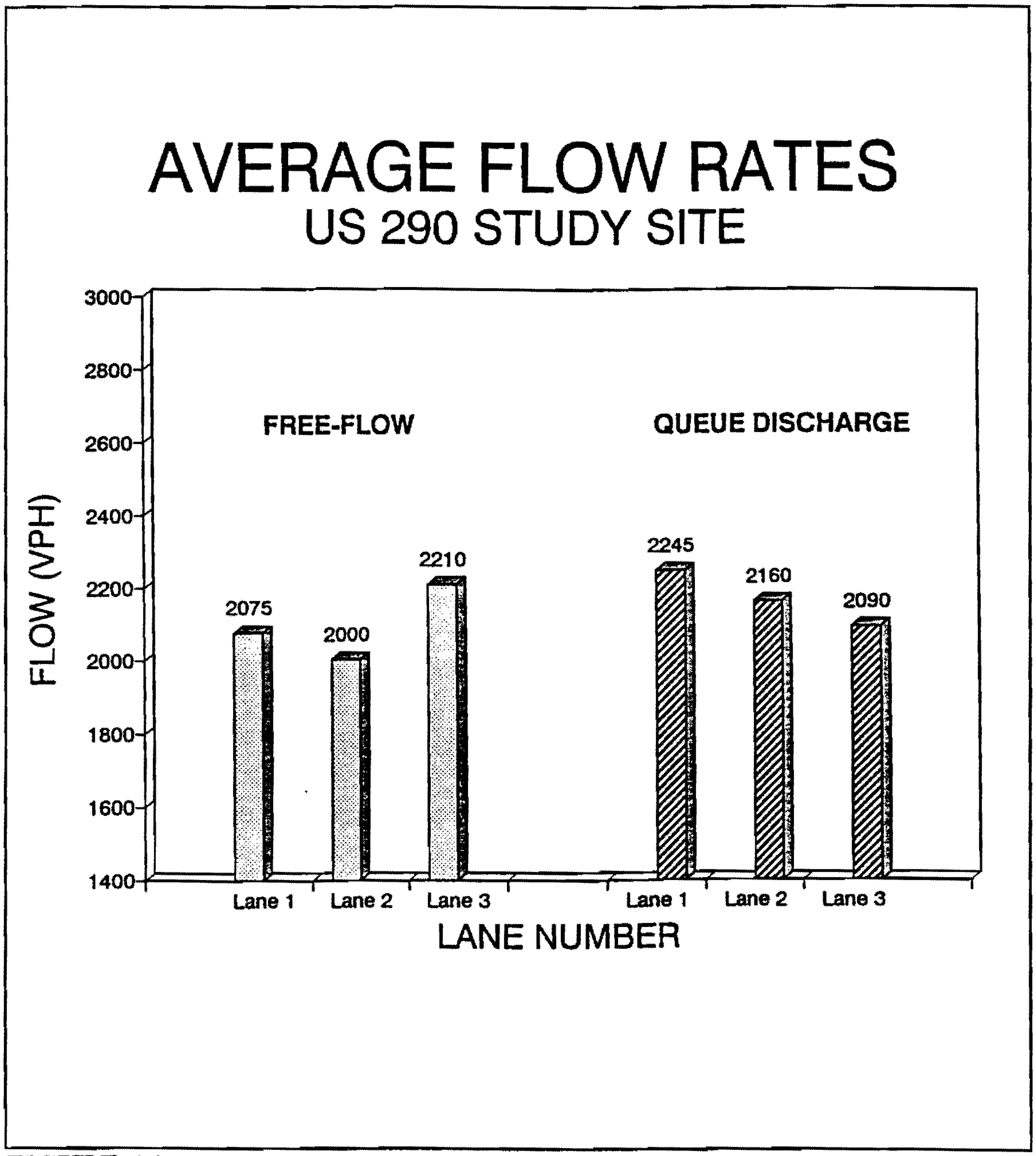


FIGURE 16 Average flow rates at U.S. 290 at Tidwell study site.

HCM the resulting flows were calculated which are also shown in Table 12.

TABLE 12
Adjusted Queue Discharge Flow Rates for U.S. 290 at Tidwell

Lane Number	Max Queue Discharge Flow	Approximate % Heavy Vehicles	Adjustment 1985 HCM
Lane 1	2245 vph	0.7 %	2260 pcphpl
Lane 2	2160 vph	2.9%	2220 pcphpl
Lane 3	2090 vph	5.3%	2200 pcphpl
Average	2165 vph	3.0%	2230 pcphpl

The adjusted flows were approximately 2,260 pcphpl for lane 1, 2,220 pcphpl for lane 2, 2,200 pcphpl for lane 3 and 2,230 pcphpl for the average across all lanes. Adjusting for trucks lowered the differential between lanes bringing the queue discharge flows much closer. Although lane 1 remained the highest flow lane, lanes 2 and 3 have similar flows. Based on these results, a flow rate of approximately 2,200 pcphpl can be maintained in individual lanes and on average over the entire facility.

Speed-Flow Model

The hypothesized speed flow relationships resulting from the analysis are shown in Figure 17. The solid lines indicate the actual data while the dashed lines show the hypothesized shape of the entire speed-flow relationship. As shown in Figure 17, the operational characteristics of the site determine the observed speed-flow relationship.

The right ends of the curves for lanes 1 and 2 in Figure 17 were projected to illustrate the speed-flow relationship assuming that they did not prematurely break down and transition to queue discharge. The theoretical maximum flows for lanes 1 and 2 were estimated and are only for illustration purposes. The reason for the higher maximum flow in the inside lane is that the turbulence from the on-ramp is greatest in the outside lane and

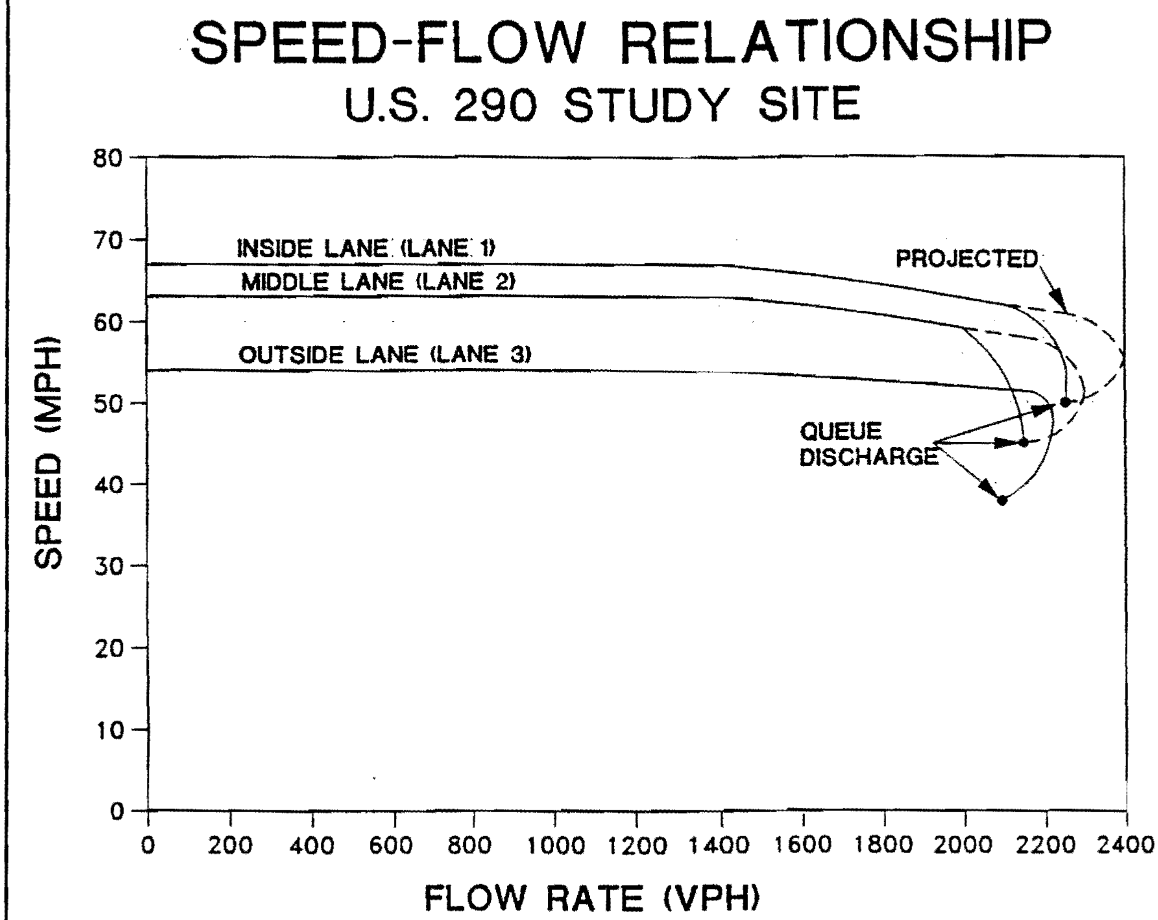


FIGURE 17 Speed-flow relationship for US 290 study site.

least in the inside lane, which is shown even in queue discharge. As a result of merging and driver characteristics, the flow in the outside lane is lower during queue discharge and would also have a lower maximum flow rate.

Based on the speed flow relationship shown in Figure 17, if lane 3 did not break down, it is logical to assume that lanes 1 and 2 would have followed the projected paths instead of dropping to queue discharge. Figure 18 shows the curves for all three lanes in free-flow conditions. Also shown is the average for all lanes combined. The ends of the curves are equal to the queue discharge flow which is considered to be the maximum sustainable flow. Although much higher free-flow flow rates may occur for individual lanes during certain times, they are unstable and not sustainable if the facility breaks down and transitions to queue discharge flow.

It should be noted that the free-flow curves shown in Figure 18 assume that the lane distribution is such that lane 3 does not break down. In reality, the lane distribution causes this site to prematurely break down and therefore, such flows will not be obtained in free-flow conditions for this particular site. Even so, the flow rates can be sustained in queue discharge.

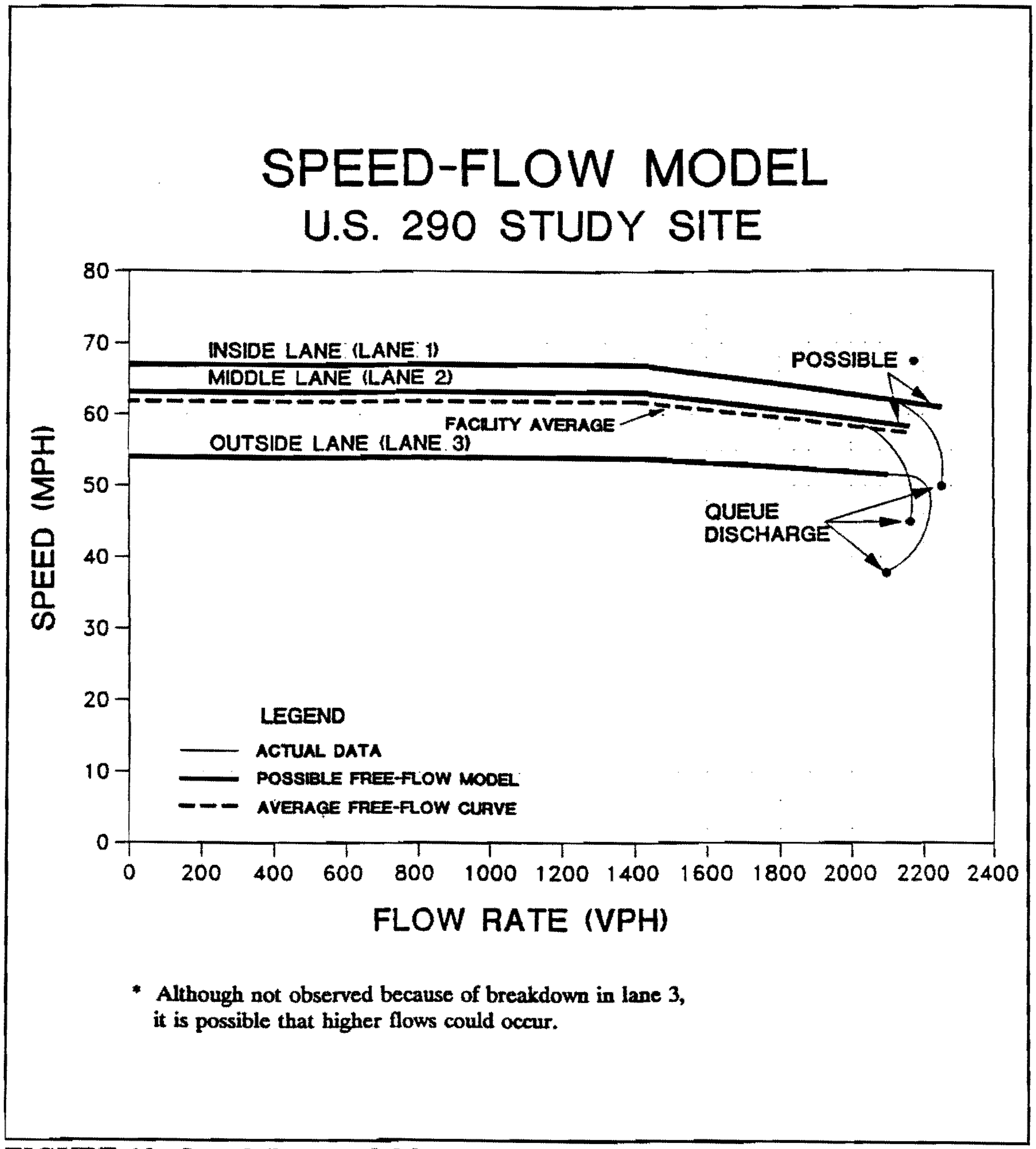


FIGURE 18 Speed-flow model based on US 290 at Tidwell study site.

COMPARISON AND VALIDATION WITH OTHER BOTTLENECK SITES

To verify the results of the analysis for U.S. 290 at Tidwell, data were collected at other sites in Texas. The purpose of the comparison and validation procedure was to determine if the results and conclusions of the analysis for the site on U.S. 290 are valid for other bottlenecks which may have different geometric characteristics and traffic conditions. Because geometric characteristics as well as traffic conditions are different at almost every freeway bottleneck, it is important to include multiple sites to ensure the model represents the operation of more than a single site.

Study Sites

Three study sites were chosen for validation of the design flow model. Each of the sites experiences traffic congestion during the peak hours. The first site is just downstream of the intersection of I-35, a 6-lane (3 lanes in each direction) radial interstate highway, and U.S. 67, a 4-lane U.S. highway in southern Dallas, Texas. A schematic of this site is shown in Figure 19. As shown in Figure 19, this site is significantly different than the U.S. 290 site. At this location, I-35 with a 3-lane cross-section, merges with U.S. 67 which has two lanes into a section with four 12 foot lanes. The inductive loops are located approximately 500 feet north of the gore area. The purpose of choosing this location was the occurrence of congestion during the morning peak period and non-typical geometry. Although not a typical bottleneck, the study section exhibits the characteristics of on-ramp bottlenecks. I-35 operates like a 2 lane on-ramp and causes extensive congestion on U.S. 67. The site has 12 foot lanes, a 14 foot inside shoulder and a 11 foot outside shoulder.

The second site is on interstate 410 (I-410) in San Antonio, Texas. The I-410 site is located on the eastbound side of I-410 just downstream of a moderate volume entrance ramp. The I-410 site is shown in Figure 20. This site was chosen based on the frequent occurrence of congestion and the geometry of the study area. It was believed to be removed from the effects of the downstream I-10 interchange located approximately 1.2 miles west

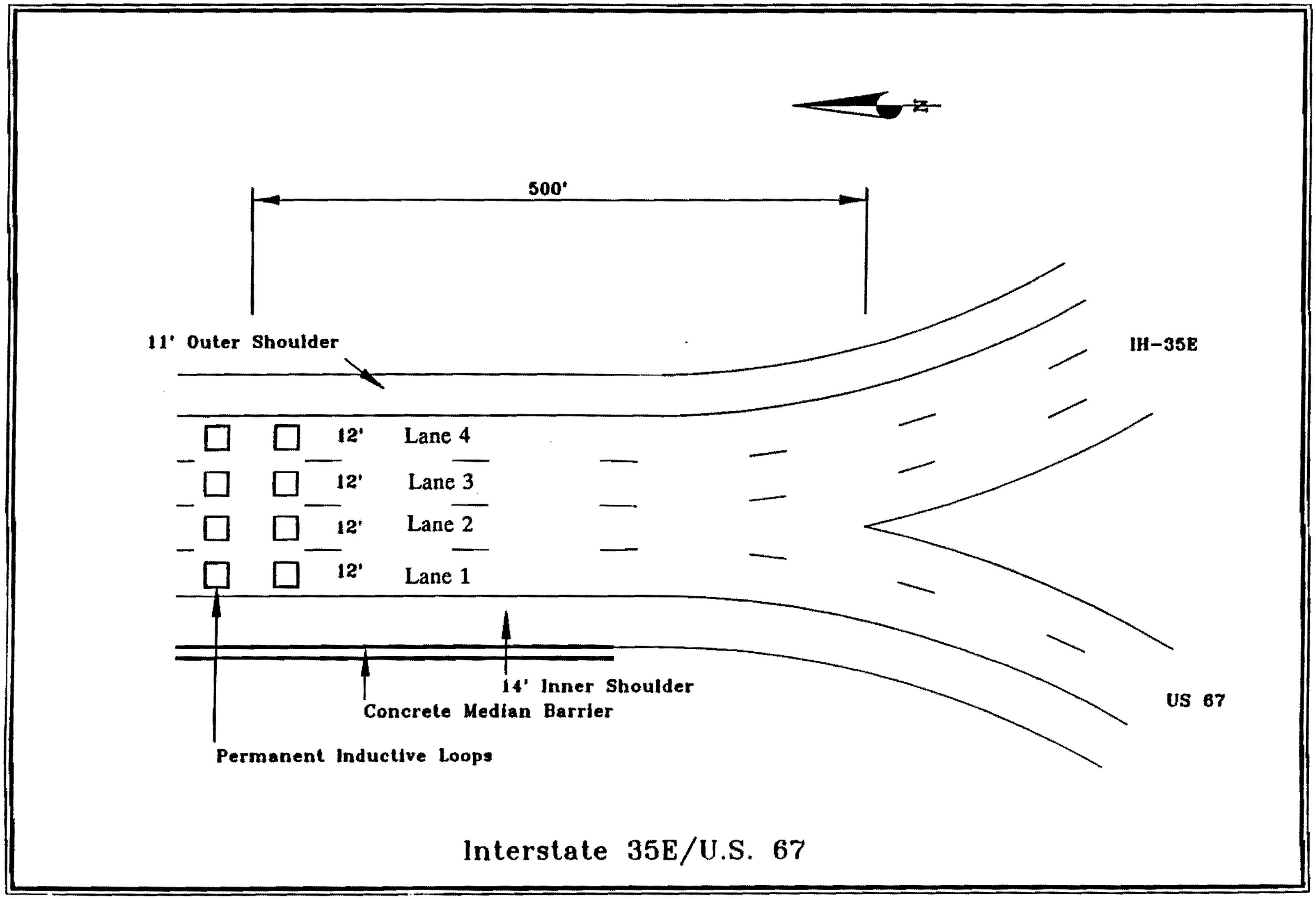


FIGURE 19 I-35/US 67 study site.

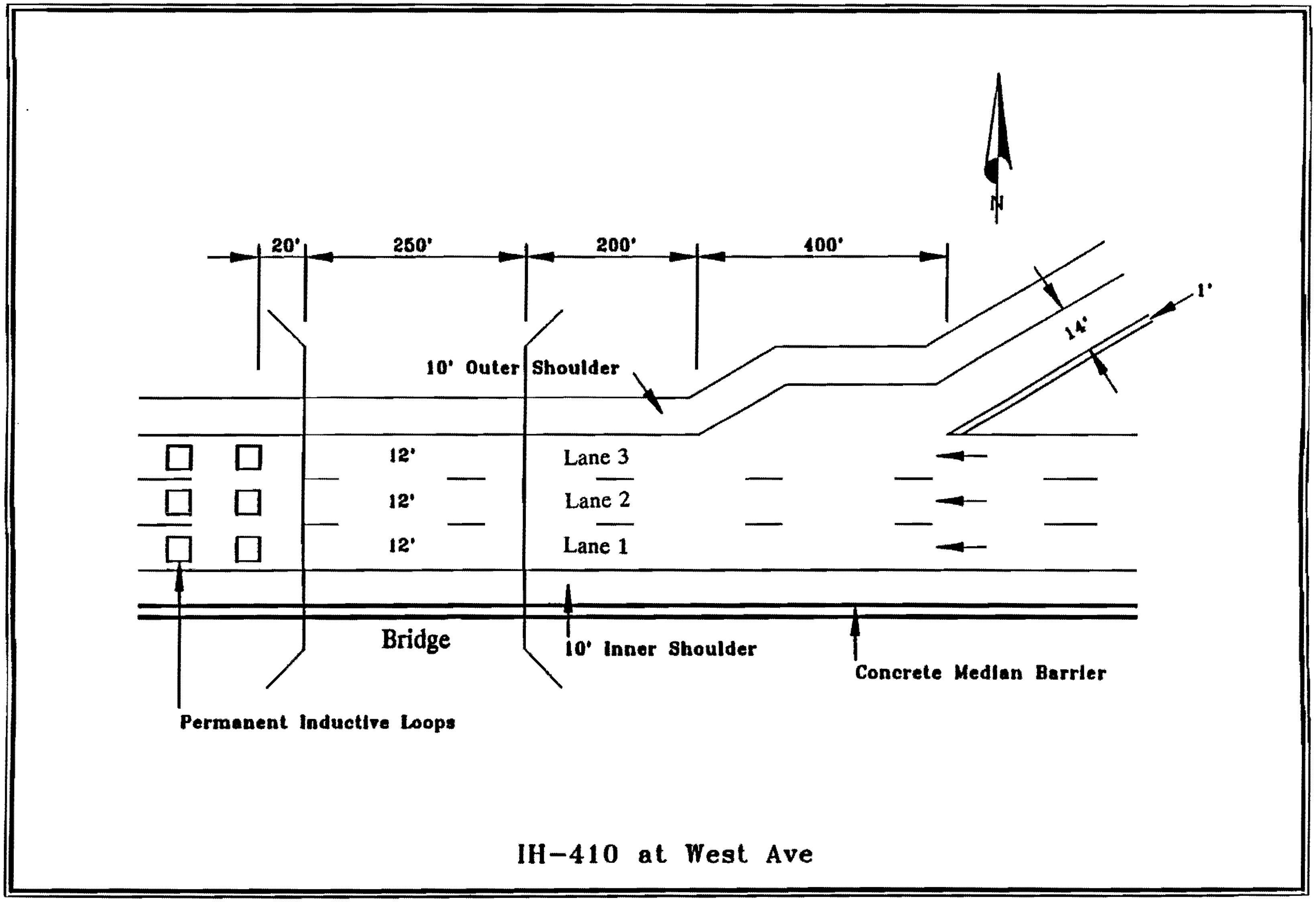


FIGURE 20 I-410 study site.

of the site. The a.m. peak hour on-ramp volume at this site was estimated to be approximately 700 vph.

The third site chosen was on U.S. 183 in Fort Worth, Texas. This site is located downstream of a high volume entrance ramp and a low volume exit ramp. This site is shown in Figure 21. At this bottleneck location very high traffic volumes enter from the upstream on-ramp into an auxiliary lane, which is then dropped at a low volume off-ramp, merging the traffic into three lanes. The purpose for choosing this site was the common occurrence of congestion at the study site and the high volumes measured downstream. Data at the next downstream on-ramp from Central Drive showed very high volumes although congestion did not occur, indicating the bottleneck was upstream at the subject site. Because this site is located downstream of a weaving section, it is not a typical bottleneck location.

Data Collection at Validation Sites

Data were collected for 3 days at the I-410 site, 3 days at the I-35/US 67 site and 4 days at the U.S. 183 site. The dates that data was collected for each site are shown in Table 13. Data were collected for 2 to 3 hours each day.

As shown in Table 13, the fourth sample at I-410 was taken during a rainy day and therefore was not included in the primary analysis. This sample will be discussed later to show some of the impacts of adverse weather on traffic flow characteristics.

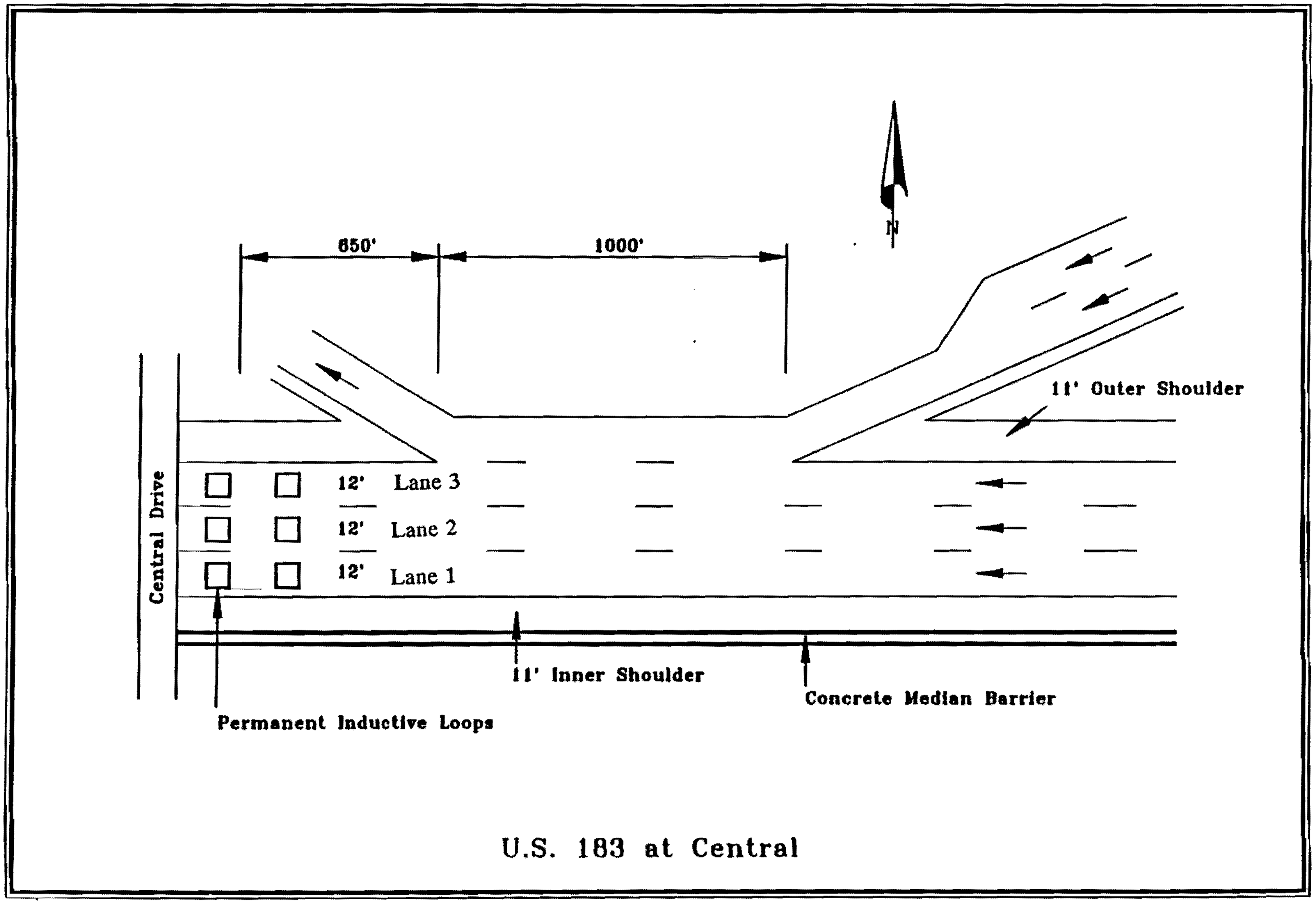


FIGURE 21 U.S. 183 study site.

TABLE 13
Data Samples from Validation Sites

I-35/US 67 Dallas		I-410 San Antonio		U.S. 183 Fort Worth	
Data Sample	Date	Data Sample	Date	Data Sample	Date
#1	May 2, 1991	#1	August 26, 1991	1	May 2, 1991
#2	May 15, 1991	#2	April 15, 1992	2	May 15, 1991
#3	December 4, 1991	#3	May 7, 1992	3	February 19, 1992
		#4*	October 31, 1991	4	April 30, 1992

* Taken during a rainy day

Analysis of Flows at Validation Sites

The comparison and validation procedure was performed in three areas which are listed below:

1. A comparison of flow rates before and after the speed drop and during the peak 15 minutes.
2. An evaluation of the similarities and differences in flow characteristics.
3. An examination of the flow processes and the speed-flow relationship at the validations sites.

Peak 15-Minute Flow Rates

Although peak 15-minute flow rates are not directly applicable as maximum sustainable flow, which was discussed in the development of the maximum design flow model, they do help in understanding some of the traffic characteristics. Some of these are the level of flow reached and traffic distribution across lanes. Table 14 shows the measured peak 15-minute flow rates for the three sites.

The peak 15-minute flow rates show some apparent differences between all four study sites. A comparison of flow rates in individual lanes indicates that the median lane (lane 1) is the highest flow lane but is more predominant at the I-410 and the I-35/US 67 sites. Two of the sites have very large peak 15-minute flows of 2,496 vph and 2,492 vph. The lowest flows are in the outside lanes for all three sites. The major difference is that the outside lanes are the merge lanes for all but the I-35/US 67 site. The U.S. 183 site is downstream of a major weaving section which is likely the cause of the low flow rates measured in the outside lane.

TABLE 14
Peak 15-Minute Flow Rates Across All Lanes

Highway	Sample	Peak 15 Minute Flow Rate				
		Lane 1	Lane 2	Lane 3	Lane 4	Average
U.S. 290	Average	2320	2244	2196	-	2253
I 410	1	2376	2212	1856	-	2148
	2	2476	2196	1856	-	2176
	3	2636	2096	1688	-	2140
	Average	2496	2168	1800	-	2155
I-35/US 67	1	2480	2180	2144	2124	2232
	2	2588	2320	2224	1856	2247
	3	2408	2240	2324	2172	2286
	Average	2492	2247	2231	2051	2255
U.S. 183	1	2340	2032	1876	-	2083
	2	2288	2108	1748	-	2048
	3	2432	2152	1828	-	2137
	4	2352	2204	1672	-	2076
	Average	2353	2124	1781	-	2086

The overall averages across all lanes are similar for the U.S. 290 and the I-35/US 67 sites but lower for the I-410 site and the U.S. 183 site. The major difference for both of these sites is the low volume in the outside lane. The difference at the U.S. 183 site is clearly due to the weaving section, although the cause at the I-410 site is unknown. It should be noted, however, that the I-410 site is affected by downstream congestion to a greater extent than was known when the site was selected.

Heavy Vehicle Percentages

The average heavy vehicle percentages for the analysis period are shown in Table 15. All of the sites had low heavy vehicle percentages.

TABLE 15
Average Heavy Vehicle Percentages

Observation	Lane				
	1	2	3	4	Average
U.S. 290	.7	2.9	5.3	---	3.0
I-35/US 67	.3	2.2	1.9	2.4	1.7
I-410	1.0	2.6	3.0	---	2.1
U.S. 183	1.4	4.9	5.3	---	3.7

Comparison of Speed and Flow Characteristics

To study the characteristics of free flow and queue discharge conditions, the same procedure was used as in the U.S. 290 analysis. The speed-flow-time plots for the three study sites are shown in Appendix C.

The speed and flow profiles in Appendix C show some interesting trends. The first trend is the range of free-flow speeds. Whereas the U.S. 290 site had free-flow speeds for the highest speed lane above 65 mph, the I-410 site had free-flow speeds in lane 1 between 60 mph and 65 mph. Both the I-35/US 67 and U.S. 183 sites have even lower free-flow speeds of approximately 60 mph. It should be noted that the merge point at the I-35/US 67 site is at the ends of curves for both roadways, and the I-410 site is located on a 2.6 percent grade before a horizontal curve, which could account for the lower speeds at these sites.

Another interesting phenomenon is the low speeds after the speed drop. The difference in queue discharge speeds is partially the result of the location of the detectors with respect to the bottleneck. Higher speeds are measured further from bottleneck due to acceleration beyond the bottleneck. Although the placement of the loops at the I-35/US 67 site is slightly closer to the actual merge area than the loops at the U.S. 290 site, the loops at the I-410 site are nearly the same distance from the end of the merge area and therefore should show similar speeds as a result of vehicles accelerating back up to their free-flow speed downstream of the bottleneck. Field observations found that queues from downstream congestion backed up through the study site causing the lower speeds and low flow rates which will be discussed in greater detail in later sections.

The location of the speed drop was determined using 30-second average speeds in lane one at all sites. Although in most cases all of the lanes experience a speed drop at the same time, for comparison a single time interval is required. Therefore, the highest flow lane was used to determine the time of the drop because this lane tends to experience the most dramatic speed drop.

The calculated flow rates for the 5 minutes before the speed drop, 5 minutes after the speed drop, and entire time during queue discharge conditions are shown in Table 16.

Statistical Analysis of Speed Drop

As with the U.S. 290 analysis, the analysis of flows before and after the speed drop was done in two steps. The comparison of standard deviations of the pre-queue and queue discharge headways for the 5-minute intervals was done using an F-test for each sample day. The mean headways were then tested using a t-test. Finally, the statistics for all the sample

TABLE 16
Comparison of Flow Rates Before and After Speed Drop at Validation Sites

Sample	5 Minutes Before Speed Drop (vph)					5 Minutes After Speed Drop (vph)					Entire Queue Discharge Period After Speed Drop (vph)				
	Lane 1	Lane 2	Lane 3	Lane 4	Avg	Lane 1	Lane 2	Lane 3	Lane 4	Avg	Lane 1	Lane 2	Lane 3	Lane 4	Avg
I-410															
1	2214	2204	1932	-	2117	2028	1805	1535	-	1789	2059	1962	1628	-	1883
2	2401	2064	1642	-	2036	1742	1778	1588	-	1703	1746	1857	1653	-	1752
3	2774	2230	1993	-	2333	1746	1787	1633	-	1722	2058	1772	1718	-	1850
Avg.	2463	2166	1856	-	2162	1839	1790	1585	-	1738	1954	1864	1667	-	1828
I-35/US 67															
1	2724	2409	2411	1855	2350	2438	2152	2190	1762	2136	2383	2161	2145	2089	2194
2	2534	2361	1912	1494	2075	2587	2278	2295	1875	2259	2357	2094	2152	1931	2133
3	2778	2254	2078	1713	2205	2599	2284	2209	1931	2256	2449	2206	2220	2032	2227
Avg	2679	2341	2134	1687	2210	2542	2238	2231	1856	2217	2396	2154	2172	2017	2185
U.S. 183															
1	2151	2226	1652	-	2010	2449	2050	1960	-	2153	2181	1942	1757	-	1960
2	2680	2508	1809	-	2332	2156	1981	1834	-	1990	2044	1874	1692	-	1870
3	2397	2100	1732	-	2076	1987	1897	1752	-	1879	2161	1957	1669	-	1929
4	2594	2302	1633	-	2176	1869	1773	1487	-	1710	2039	1845	1647	-	1844
Avg.	2455	2284	1707	-	2149	2115	1925	1758	-	1933	2106	1905	1691	-	1901

days were evaluated.

The results of the F-test for the 5-minute free-flow and queue discharge intervals are shown in Table 17. In general, the results correspond to those found at the U.S. 290 site. All of the sites had statistically different standard deviations in at least half of the samples for all lanes except lane 3 at the I-410 site. Although for the most part the results corresponded to the results found at U.S. 290, the differences between lanes is not as pronounced. At U.S. 290 all the samples in lane 1 had statistically different variances while only part of the samples in the other lanes had statistically different variances. At the U.S. 183 site 50 percent of the samples had statistically different variances in all lanes. At the I-35/US 67 site all of the samples in lane 2 had different variances and two-thirds of the samples in all the other lanes had statistically different variances.

TABLE 17
Test for Difference in Headway Variances at Validation Site During 5-Minute Intervals Before and After the Speed Drop

Site	Sample	Lane 1	Lane 2	Lane 3	Lane 4
		Significant	Significant	Significant	Significant
I-410	1	Yes	Yes	Yes	-
	2	Yes	No	No	-
	3	Yes	Yes	No	-
I-35/US 67	1	Yes	Yes	No	No
	2	Yes	Yes	Yes	Yes
	3	No	Yes	Yes	Yes
U.S. 183	1	Yes	No	No	-
	2	No	Yes	No	-
	3	No	No	Yes	-
	4	Yes	Yes	Yes	-

Based on the results for the F-test, it can be concluded that the variance in headways decreases after the speed drop. The significance of the decrease is affected by the characteristics of the site and the specific lane.

To test the differences in mean headways before and after the speed drop, a t-test approximation was performed between the 5-minute intervals. Table 18 shows the results of the statistical test of mean headways for the 5-minute periods before and during queue discharge. The results at the validation sites were similar to the those at the U.S. 290 site. Although the mean headways and flow rates in some lanes experienced a statistically significant decrease or increase, many did not. Shown in Table 19 are the differences between the free flow rates and the 5-minute queue discharge flow rates.

TABLE 18
Test for Difference in Mean Headways at Validation Sites During 5-Minute Intervals Before and After the Speed Drop

Site	Sample	Lane 1	Lane 2	Lane 3	Lane 4
		Significant	Significant	Significant	Significant
I-410	1	No	Yes	Yes	-
	2	Yes	Yes	No	-
	3	Yes	Yes	Yes	-
I-35/US 67	1	Yes	Yes	No	No
	2	No	No	Yes	Yes
	3	No	Yes	No	No
U.S. 183	1	No	No	No	-
	2	Yes	Yes	No	-
	3	Yes	No	No	-
	4	Yes	Yes	No	-

TABLE 19 Differences in Mean Flow Rates at Validation Sites for 5-Minute Intervals Before and After the Speed Drop						
Site	Sample	Difference in Flow Rates				
		Lane 1	Lane 2	Lane 3	Lane 4	Average
I-410	1	-186	-400	-397	-	-327
	2	-660	-287	-54	-	-333
	3	-1029	-443	-360	-	-611
I-35/US 67	1	-285	-257	-221	-93	-214
	2	53	-83	382	381	183
	3	-178	30	132	218	50
U.S. 183	1	298	-176	309	-	143
	2	-524	-527	25	-	-342
	3	-410	-203	20	-	-198
	4	-724	-529	-147	-	-466

Because of the high variability in the 5 minutes before and after the speed drop, very large flow differences are required to make a statistically significant result. In general, the volume in lane 1 decreased in flow during queue discharge while the outer lanes increased in flow during queue discharge, except at the I-410 site, where the flow in all lanes decreased. The fact that all lanes always decreased in flow at the I-410 site is consistent with the observations that downstream congestion was controlling flow. At the U.S. 183 site lanes 1 and 2 decreased in flow in most samples after the speed drop, while lane 3 increased in flow in 3 out of 4 samples. Because lane 3 is likely to be the most significantly impacted by the weaving and the exit-ramp, the increase in flow may be a result of the stabilizing effect of queue discharge. During queue discharge, gaps that open up as a result of exiting traffic are soon filled by vehicles from the other lanes, increasing the flow in lane 3.

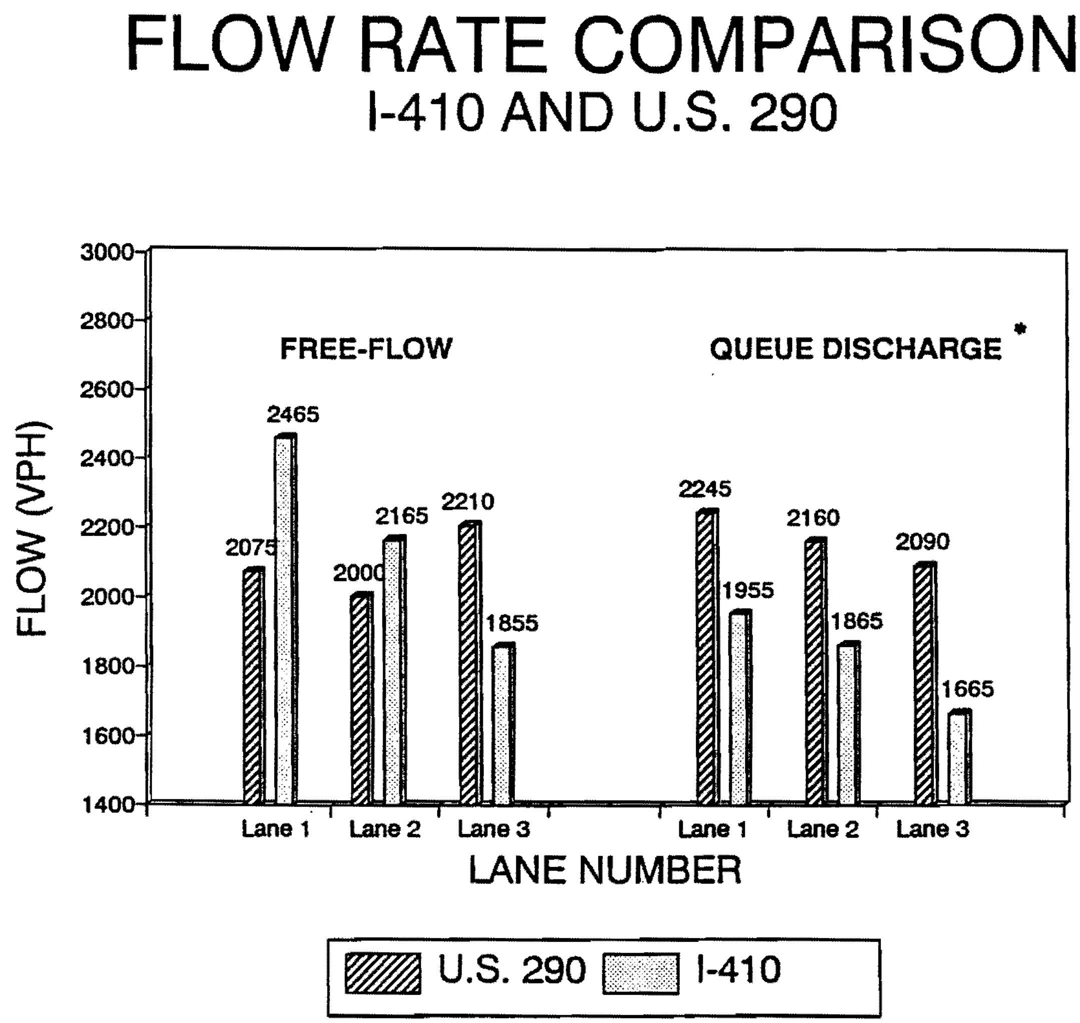
Based on the analysis of means and variances of headways some basic conclusions can be made:

- The variance of headways is greater during free flow conditions than during queue discharge for each individual sample.
- Mean flows directly before the transition to queue discharge are not statistically different than mean flows after the speed drop.

Comparison of Average Flow Rates

A comparison of the average flow rates at the validation sites found many interesting similarities and differences. Figure 22 shows a graphical comparison of the flow rates for I-410 and U.S. 290. The I-410 site, which is geometrically similar to the U.S. 290 site and also has moderate on-ramp volumes, had much different flow rates and distribution of traffic across lanes. At the I-410 site during free-flow conditions, the inside lane had a very high flow before the speed drop. In fact, the third sample at I-410 had a calculated hourly flow rate of 2,774 vph in lane 1, which is much higher than the maximum flow of 2,340 vph in sample 14 for U.S. 290 (in Table 3). The largest flow rate measured at the U.S. 290 site occurred in sample 11 directly after the speed drop with 2,525 vph, which is still smaller than the third sample at the I-410 site, but greater than the other samples. After the speed drop, all of the lanes decreased in flow which is primarily an effect of the downstream congestion.

The I-410 data shows that it is very possible to reach much higher flow rates under free-flow conditions than measured at the U.S. 290 site in the inside lane. This supports the hypothesis that the inside lane at the U.S. 290 site prematurely dropped into queue discharge before reaching the maximum flow under free-flow conditions. In addition, the absence of a left shoulder, presence of a median high occupancy vehicle lane and the 11 foot lane widths may discourage some traffic from the left lane under free-flow conditions at the U.S. 290 site.



* Due to downstream comgestion, the I-410 queue discharge flow rates are not applicable for comparison.

FIGURE 22 Flow rate comparison between I-410 at West and U.S. 290 at Tidwell.

The downstream effects are a significant problem with the data from this site. Because of the downstream slowdown, the study site did not break down independently and the peak free-flow volumes may not be the maximum possible flow under free-flow conditions. A more significant effect occurs during queue discharge because the flow is metered by the downstream flow. During the 5 minutes after the speed drop, the highest flow occurred in lane 1 with an average flow of 1839 vph.

The importance of the I-410 site is that in all respects it appeared to be the a primary bottleneck location, but the transition into queue discharge is caused by downstream slowing produced by downstream ramps. This shows the impact of a congested freeway system on individual bottlenecks. While individual locations may be isolated bottlenecks during certain periods and flow conditions, other bottleneck locations downstream may take control during very high flow conditions. For instance, consider two bottlenecks located 1 mile from each other. Until the breakdown of either bottleneck, both function independently. Once the downstream bottleneck breaks down, the upstream bottleneck remains operating independently. Later, the queuing effects of the downstream bottleneck reach the upstream bottleneck and reduce its flow rate.

The I-35/US 67 site produced the most interesting results. At this site traffic volumes were substantially higher than those measured at the U.S. 290 site. The average flows before the speed drop were 2,679 vph, 2,341 vph, 2,134 vph, and 1,687 vph for lanes 1, 2, 3 and 4 respectively. In the 5-minute interval after the speed drop, the flow in lanes 1 and 2 decreased and lanes 3 and 4 increased. This corresponds to the finding at U.S. 290 that once a lane (or lanes) break down, the other lanes are also subsequently broken down. In this case lanes 1 and 2 reached very high flows which caused them to transition into queue discharge and experience reductions in flow, while lanes 3 and 4 were operating at much lower flows and were forced into queue discharge conditions as a result of the turbulence created by lanes 1 and 2. It should be noted that lane 4 at this site was observed not to be a preferred lane which may explain the low flow rates. This may be because of the curvature of the roadway. A graphical comparison of the average flow rates for the I-35/US 67 site and the U.S. 290 site is shown in Figure 23.

The flow rates in lanes 1 and 2 show that high flows are possible in free flow conditions. What is particularly interesting is that lane 1 is on a curve with a lower free-flow speed and closer to the merge lane than lane 1 at the U.S. 290 site, but has a much higher volume. This further supports that lane 1 at the U.S. 290 site was not at capacity before the speed drop and was prematurely transitioned into queue discharge by the breakdown of the other lanes. Another factor may be the impact of free-flow speed on maximum flow. The theory behind such an occurrence is that at lower speeds, the drivers are more comfortable with higher densities. Assuming the speed-density relationship is different for different free flow speeds in the upper regions, increasing the speed could reduce the maximum obtainable flow. This would cause a lower free-flow speed to have a higher maximum possible flow, because the higher speed free-flow condition transitions from a higher speed toward queue discharge and never experiences the high flow, lower speed conditions. This phenomenon is not particularly new. In the past few years, the idea of maximizing flow by changing the speed limits on roadways has been implemented in Europe.

The U.S. 183 site had a distribution of average flow rates across lanes similar to the I-410 site. The free-flow flow rates for lane 1 ranged from 2,151 vph to 2,594 vph, with an average flow of 2,455 vph. Lane 2 also had high flow rates with an average of 2,285 vph and one sample reaching 2,508 vph. Although lanes 1 and 2 had high flow rates, lane 3 had a very low flow rate under free flow conditions. A comparison of flow rates is shown in Figure 24.

The flow decreased in all lanes at the U.S. 183 site in queue discharge, which does not correspond to the results at the U.S. 290 and I-35/US 67 sites. The reason for this occurrence is that this site is downstream of a weaving section and therefore has different characteristics than the other bottleneck sites. Before this site was chosen, a site downstream of the next on-ramp was studied and had higher flows but did not break down.

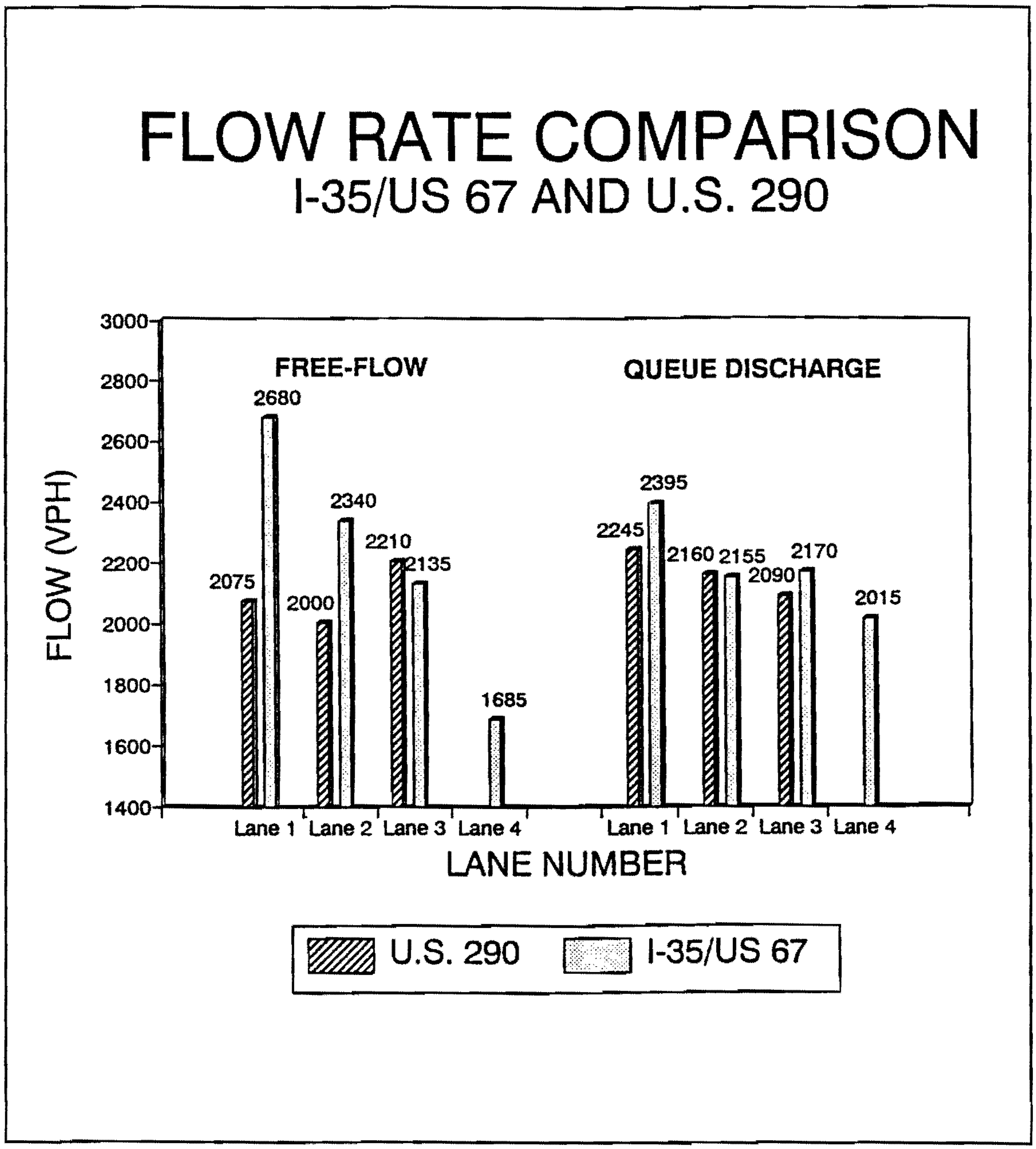


FIGURE 23 Flow rate comparison between I-35/US 67 and U.S. 290 at Tidwell.

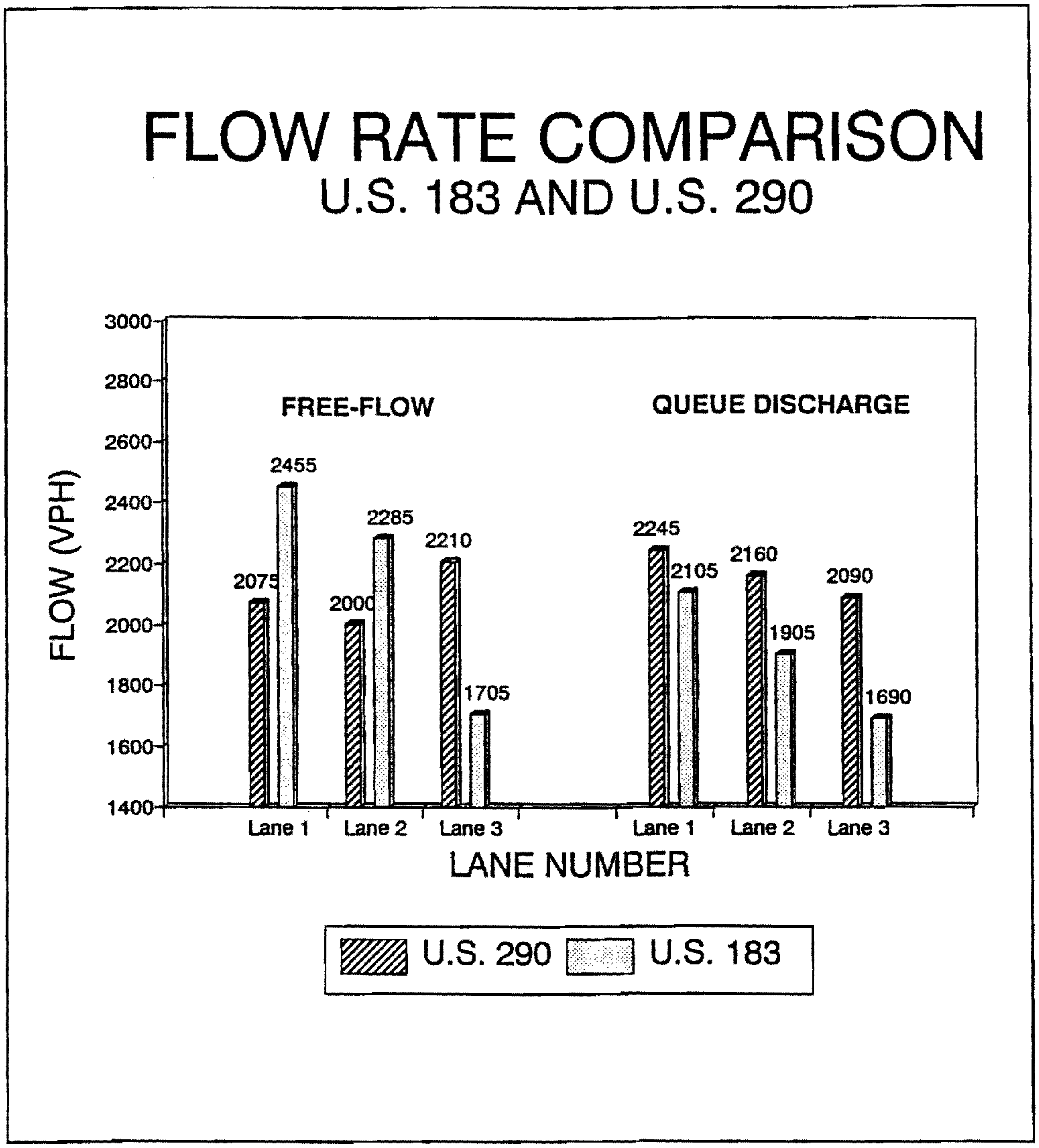


FIGURE 24 Flow rate comparison between U.S. 183 at Central and U.S. 290 at Tidwell.

This indicates that the weaving configuration at the U.S. 183 sites substantially influences the characteristics of flow. Some reduction in maximum flow would be expected since the operation of the weaving section is more complicated than a simple merge location.

Statistics for Average Flow Rates

To evaluate the characteristics of flows between samples, statistics were calculated for the mean flows given in Table 13. Table 20 shows the statistics for the samples from each site.

The flow rates for I-35/US 67 show the same trends as the U.S. 290 data. The standard deviations in mean flows between days are lower after the speed drop than before the speed drop. This further supports the conclusion that flows in the free flow regions contain a large amount of variability, while the flows during queue discharge are not nearly as variable. Because only 3 samples were used as compared to 15 samples from the U.S. 290 site, the corresponding confidence intervals are much larger. Even with only three samples, the confidence intervals for the queue discharge flow rates are relatively small with 2,396 ± 117, 2,154 ± 140, 2,172 ± 103, and 2,017 ± 199 for lanes 1, 2, 3, and 4 respectively. The average across all lanes was 2,185 ± 118 for the three samples. Although the individual lanes differ significantly, the average across all lanes during queue discharge was not significantly different than at U.S. 290, which had an average flow of 2,166 ±37 vph. For these reasons, queue discharge appears to be the most consistent flow for estimating the maximum design flow of a facility, although much higher flows are obviously possible at least in some lanes.

Even in queue discharge, lane 1 at the I-35/US 67 site continually had very high flows which averaged 2,542 vph. It is difficult to determine the reason for these very high queue discharge flow rates although some speculation can be made. As mentioned previously, the free-flow speed may influence the maximum obtainable flow because of the density or close headways that drivers are willing to accept. Although this may be the cause of the difference in free-flow flow rates, it would not be expected to influence queue

TABLE 20
Statistics for Average Flow Rates for All Primary Sites

Sample	5 Minutes Before Speed Drop (vph)					5 Minutes After Speed Drop (vph)					Entire Queue Discharge Period After Speed Drop (vph)				
	Lane 1	Lane 2	Lane 3	Lane 4	Avg	Lane 1	Lane 2	Lane 3	Lane 4	Avg	Lane 1	Lane 2	Lane 3	Lane 4	Avg
U.S. 290															
Average	2076	2002	2210	-	2096	2266	2035	1989	-	2097	2246	2161	2090	-	2166
Std Dev	136	159	187	-	132	145	134	193	-	115	81	68	101	-	67
95% CI	75	88	104	-	65	80	74	107	-	64	45	38	56	-	37
I-410															
Average	2463	2166	1856	-	2162	1839	1790	1585	-	1738	1954	1864	1667	-	1828
Std Dev	285	89	188	-	153	164	14	49	-	46	180	95	46	-	68
95% CI	709	222	466	-	381	400	34	121	-	113	448	236	115	-	169
I-35/US 67															
Average	2679	2341	2134	1687	2210	2542	2238	2231	1856	2217	2396	2154	2172	2017	2185
Std Dev	128	79	254	182	137	90	75	56	86	70	47	56	41	80	47
95% CI	318	197	631	452	341	222	185	139	213	174	117	140	103	199	118
U.S. 183															
Average	2455	2284	1707	-	2149	2115	1925	1758	-	1933	2106	1905	1691	-	1901
Std Dev	235	171	81	-	140	252	119	200	-	187	75	53	47	-	53
95% CI	373	273	128	-	223	400	189	319	-	297	187	133	118	-	132

discharge flow rates. Another explanation for the high flow rates during queue discharge in lane 1 could be the difference in types of bottlenecks. The I-35/US 67 is a merge of two highways while the U.S. 290 site is a simple ramp merge location.

At the I-410 and U.S. 183 sites, the standard deviation between samples generally decreased in queue discharge, except for the first 5-minute interval at the U.S. 183 site. At the U.S. 183 site, the standard deviations increased in the 5-minute interval after the speed drop. This may be a result of the transition to queue discharge or because the U.S. 183 site is downstream of a weaving section and therefore exhibits different flow characteristics.

Speed-Flow Relationship at Validation Sites

Because the I-410 site was affected by downstream congestion and the U.S. 183 site is not a typical bottleneck, the relationship between speed and flow at these sites was not evaluated. The speed-flow plots for the I-35/US 67 study site are shown in Figures 25 through 29. These figures show composite plots of all times using 5-minute moving averages for each lane and for all lanes averaged in free-flow conditions. Also shown in the figures are the estimated speed-flow curves suggested by the data. The scattered points in the center of the curve (low flow and higher speed) represent recovery to uncongested conditions.

Figure 30 shows the speed-flow relationships for the I-35/US 67 site. As can be seen, lanes 1 and 2 reach their peak flow rates during free flow conditions, while lanes 3 and 4 are prematurely transitioned into queue discharge. One very interesting aspect of this site is the relation of lanes 2 and 3. Both have approximately the same free-flow speeds and transition into nearly identical queue discharge flow rates, yet lane 2 reaches its peak in free flow conditions and lane 3 peaks in queue discharge. These two lanes illustrate the effects of lane interaction. Because lanes 1 and 2 broke down, the turbulence transitioned lane 3 into queue discharge before it reached its maximum flow rate. Because the speed drop occurred at nearly the same time in all lanes, it is difficult to determine whether lane 1 or

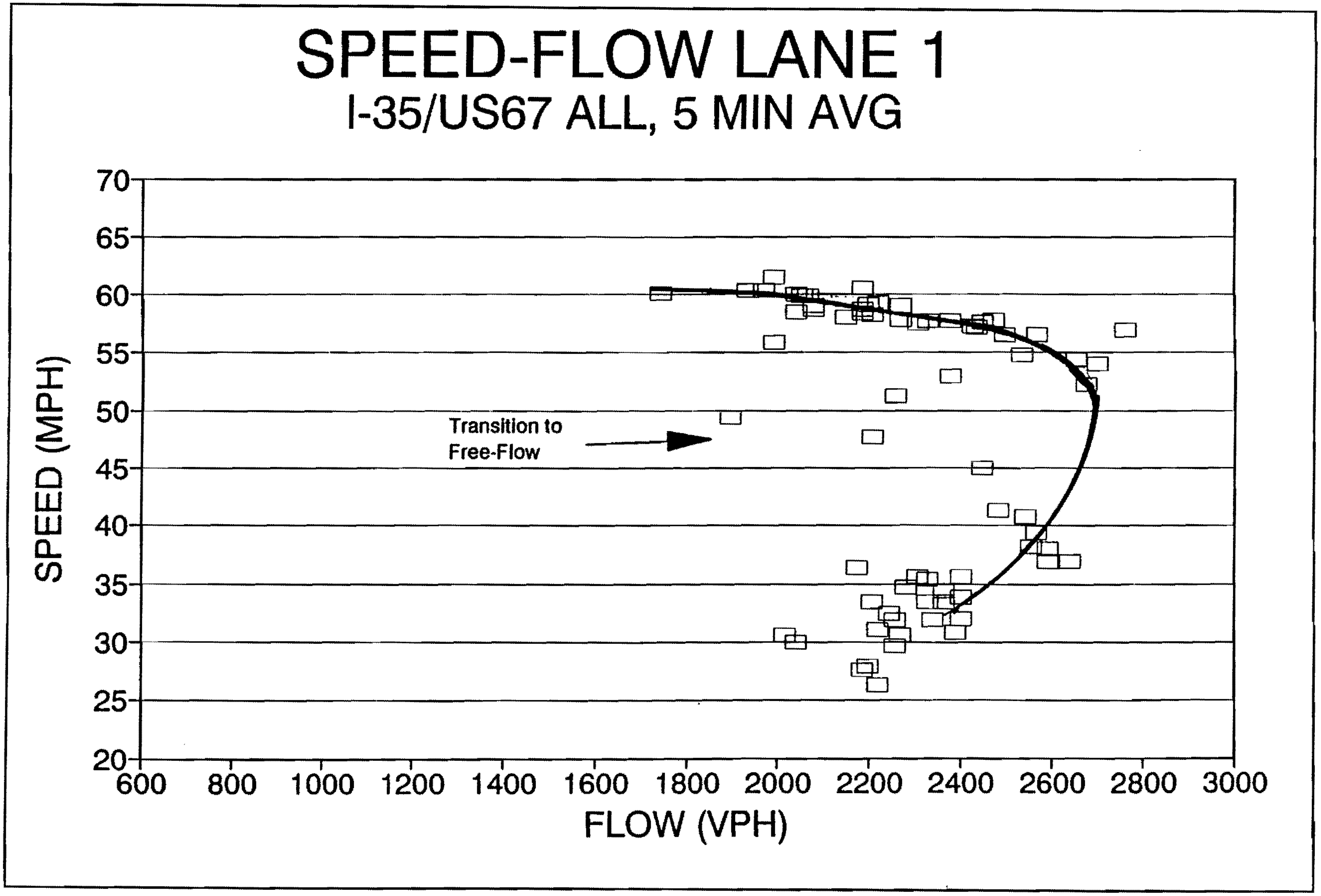


FIGURE 25 **Speed-flow for lane 1 at I-35/US 67 using 5-minute averages.**

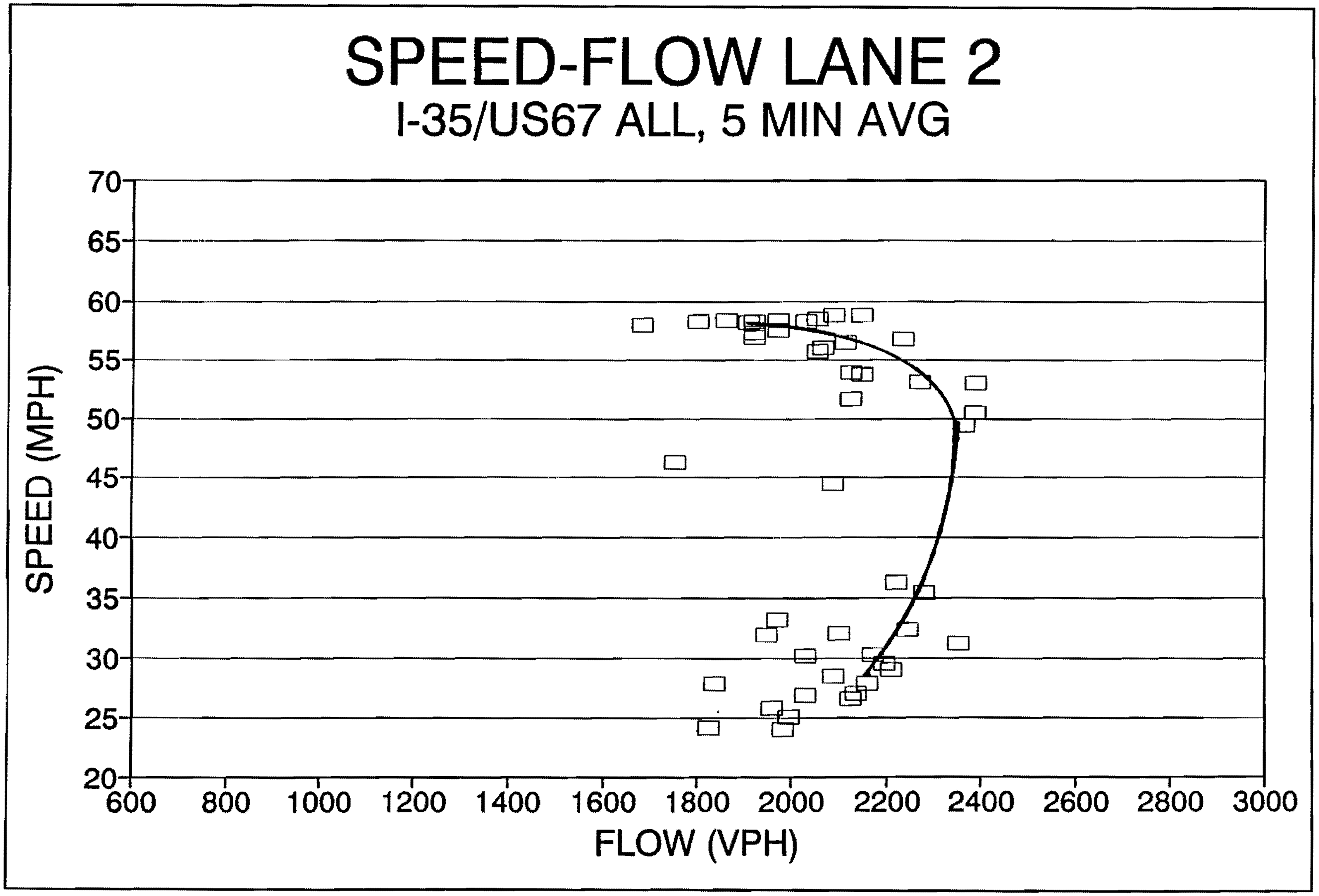


FIGURE 26 Speed-flow for lane 2 at I-35/US 67 using 5-minute averages.

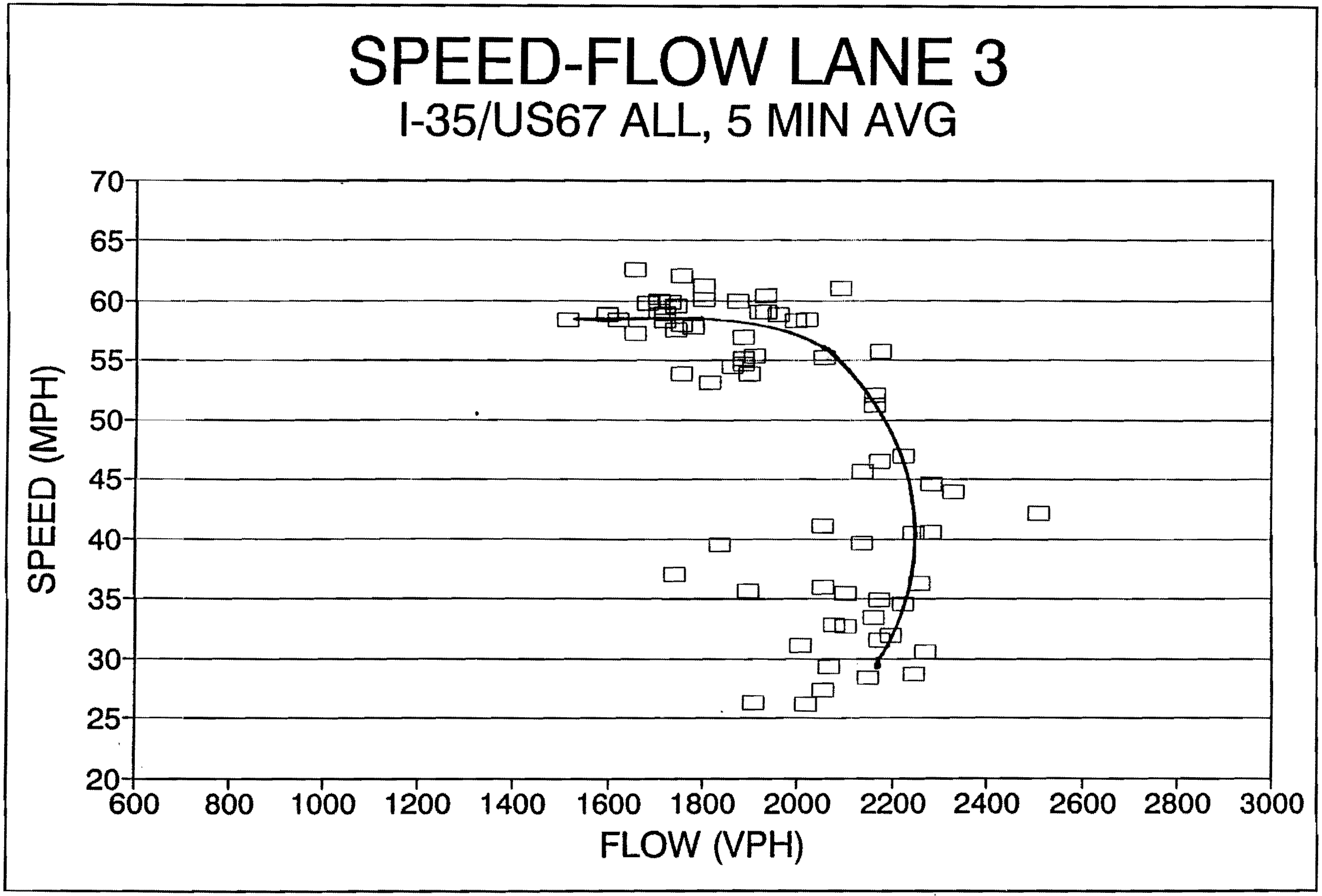


FIGURE 27 Speed-flow for lane 3 at I-35/US 67 using 5-minute averages.

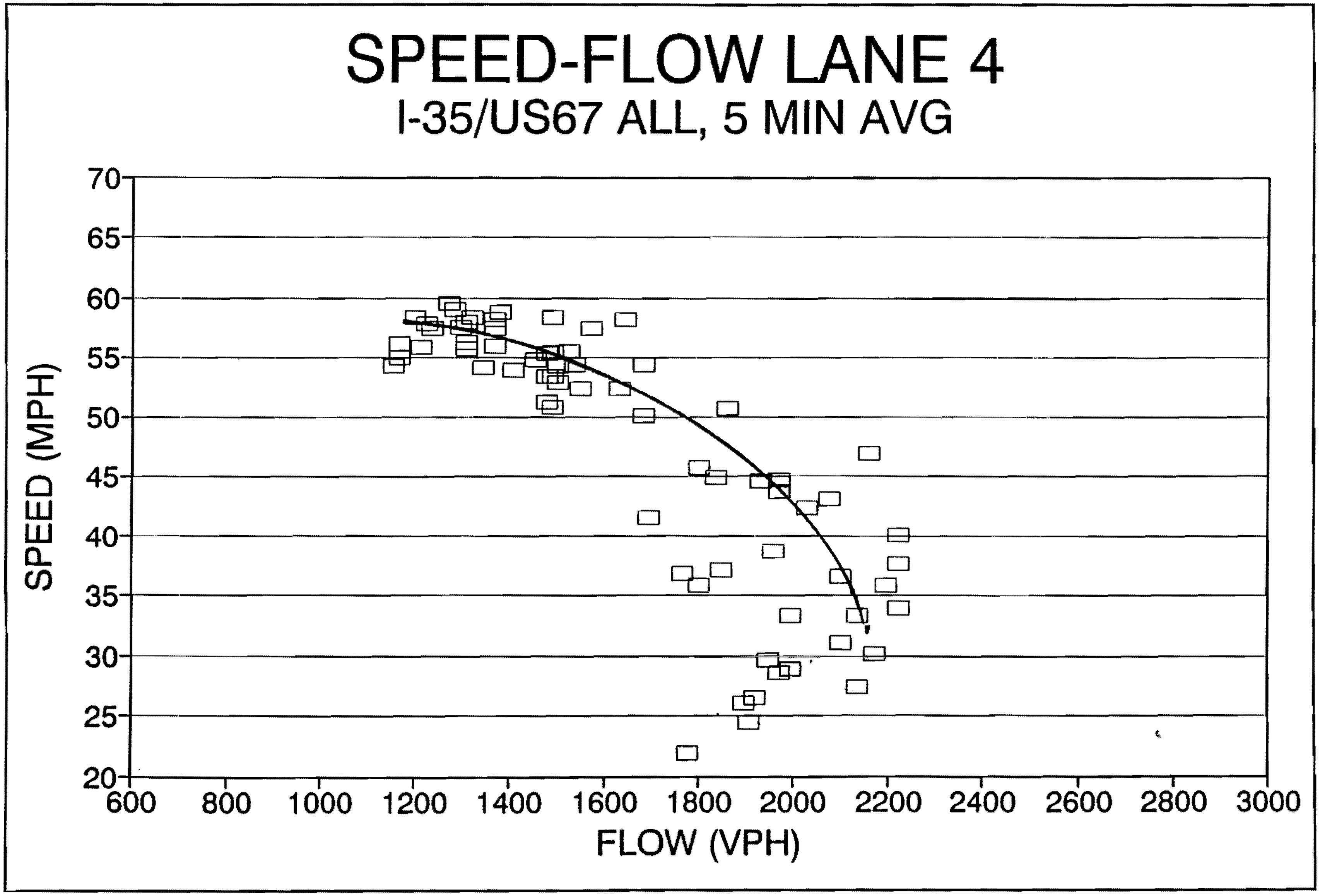


FIGURE 28 Speed-flow for lane 4 at I-35/US 67 using 5-minute averages.

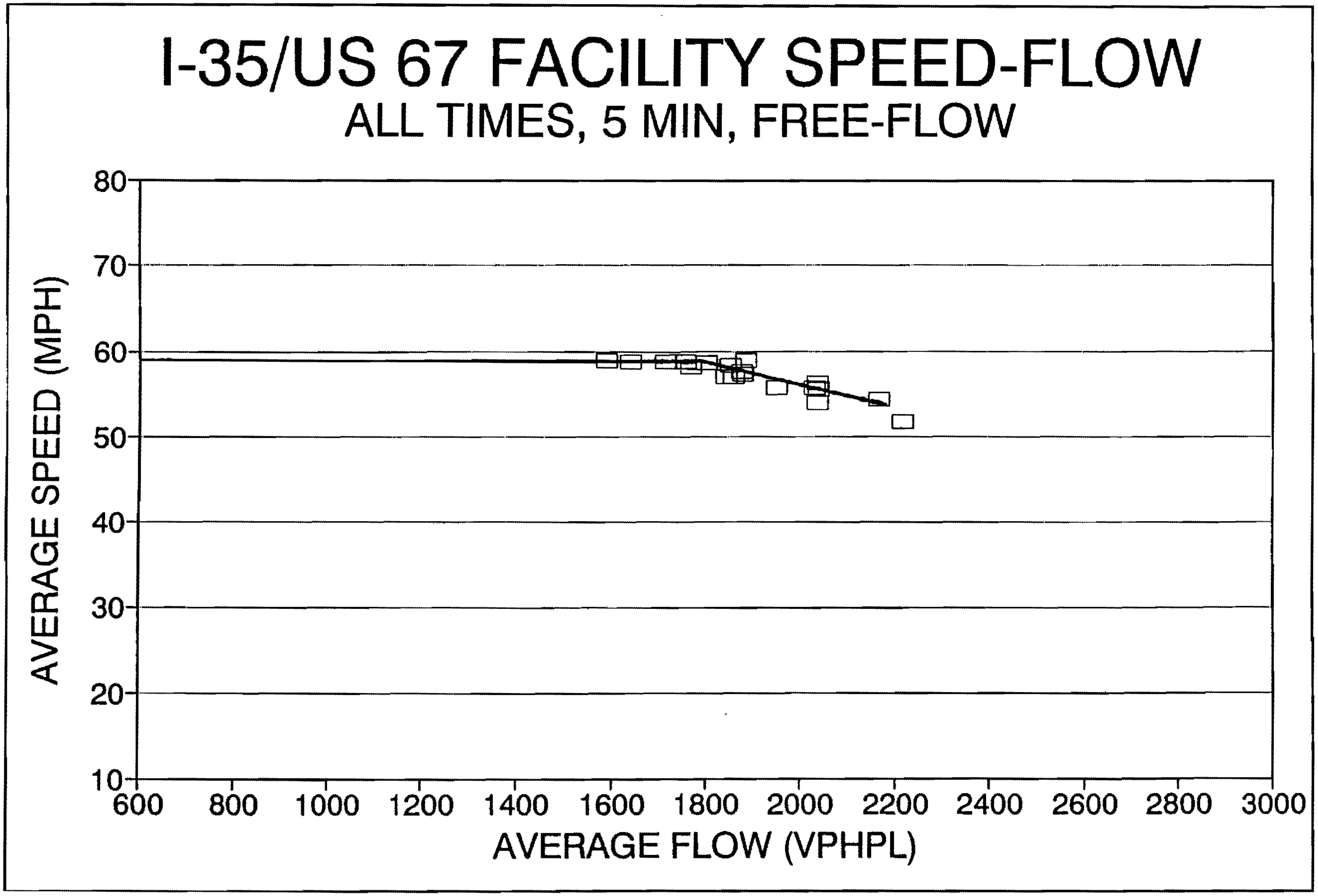
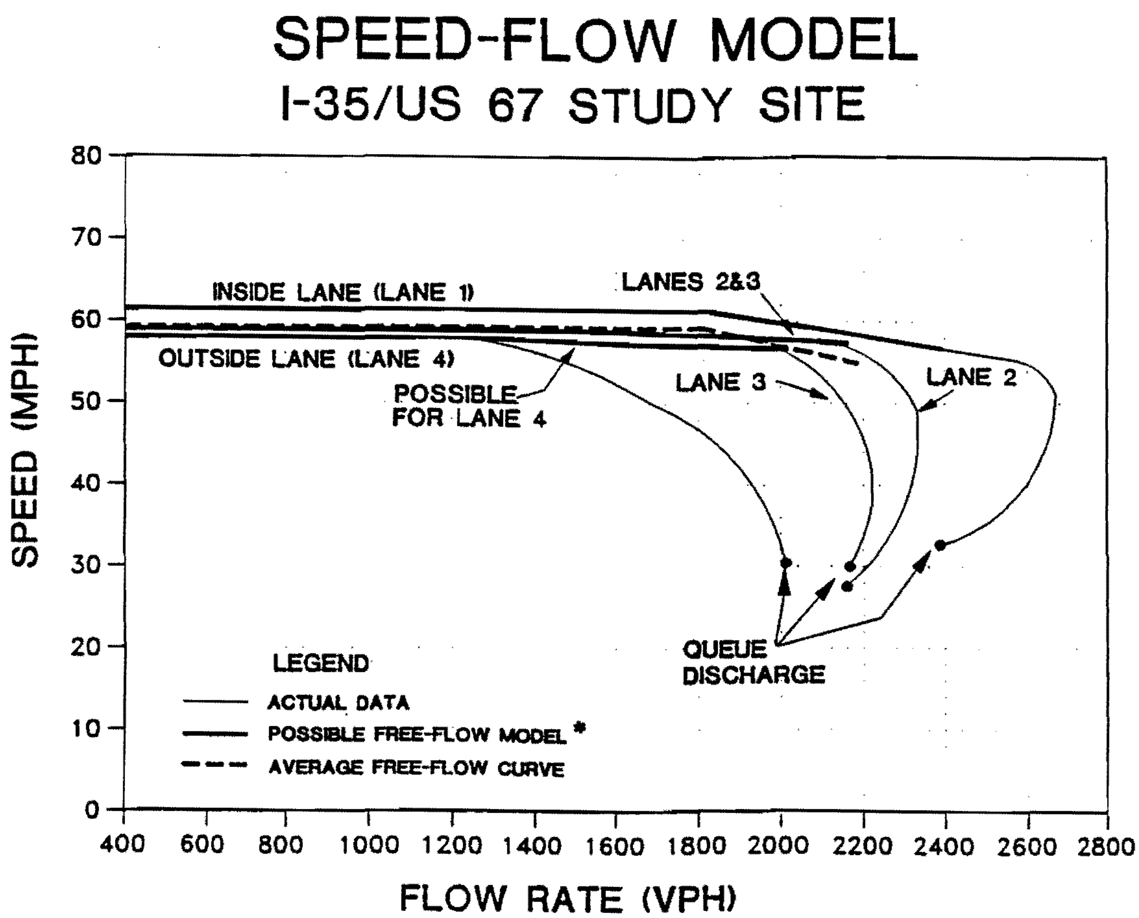


FIGURE 29 Average speed-flow curve for I-35/US 67 during free-flow using 5-minute averages.



* Assumes that lane is not prematurely transitioned to queue discharge.

FIGURE 30 Speed-flow model based on I-35/US 67 site.

2 broke down first.

These results support the hypothesis that once one or more lanes break down, the other lanes are prematurely transitioned into queue discharge conditions. The data from the I-35/US 67 site showed that extremely high flows are possible in free-flow conditions and relatively high flows also occur in queue discharge. Therefore, the measured flows are a function of the interactions between the lanes. This has the greatest effect in free-flow conditions because depending on the interactions, a lane can begin transitioning into queue discharge from almost any point in free flow conditions. The operation of lane 4 at I-35/US 67 indicates that it began transitioning into queue discharge when free flow flow rates were as low as 1,200 vph.

Also shown in Figure 30 are the possible speed-flow models for free-flow conditions in each lane and the overall average across all lanes. The projected free-flow curves assume queue discharge to be the maximum sustainable flow. The free-flow curves for the I-35/US 67 site have much greater variation than the curves for the U.S. 290 site. The queue discharge flow rates ranged from 2,017 vph to 2,396 vph. Although the U.S. 290 flow rates were within these ranges they were much closer.

Adjustments for Heavy Vehicles

Shown in Table 21 are the queue discharge flow rates for each lane, the average for the facility, and the approximate truck percentages. Applying a truck equivalency factor (E_t=2.0) given in the 1985 HCM, the resulting flows were calculated which are also shown in Table 21.

The adjusted flows were approximately 2,400 pcphpl for lane 1, 2,200 pcphpl for lane 2, 2,210 pcphpl for lane 3, 2,065 pcphpl for lane 4, and 2,220 pcphpl for the average across all lanes. Adjusting for trucks lowered the differential between lanes bringing the queue discharge flows much closer, although the variation is still greater than at the U.S. 290 site.

TABLE 21 Adjusted Queue Discharge Flow Rates for I-35/US 67			
Lane Number	Max Queue Discharge Flow	Approximate % Heavy Vehicles	Adjustment 1985 HCM
Lane 1	2395 vph	.3 %	2400 pcphpl
Lane 2	2155 vph	2.2%	2200 pcphpl
Lane 3	2170 vph	1.9%	2210 pcphpl
Lane 4	2015 vph	2.4%	2065 pcphpl
Average	2185 vphpl	1.7%	2220 pcphpl

As was discussed earlier, the overall average flow was very similar to the U.S. 290 site. Figure 31 shows the average queue discharge flow rates for the U.S. 290 and I-35/US 67 sites adjusted for truck percentages. The inside lane has the greatest range of queue discharge flow, while other lanes carry much lower flow for both sites. The overall average queue discharge flows were 2,220 pcphpl for the I-35/US 67 site and 2,230 pcphpl for the U.S. 290 site. Therefore the average flow rate during queue discharge is approximately 2,225 pcphpl but ranged from 2,065 pcphpl up to 2,400 pcphpl for individual lanes, depending on the geometry and traffic characteristics of the bottleneck. A flow rate of approximately 2,200 pcphpl is considered to be the maximum flow that can be sustained in individual lanes as well as over an entire facility.

Measured Flow Rates From Other Sites in Texas

In the process of selecting the primary study sites, data was collected at many other sites throughout Texas. Tables 22 and 23 show the peak 15-minute and peak hour flow rates at other sites in Texas. The U.S. 290, I-410, and U.S. 183 sites are not the same sites used for the previous analysis.

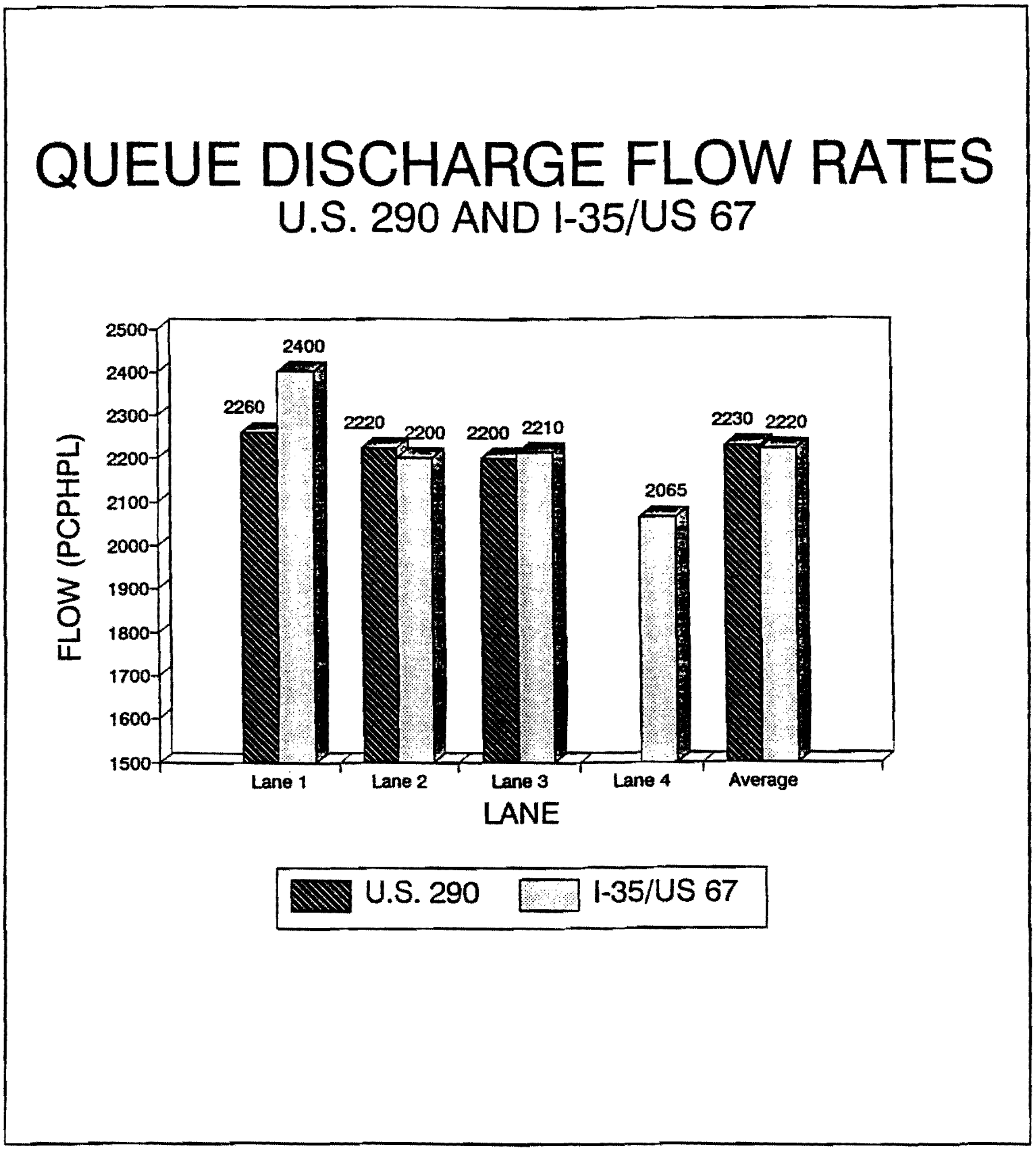


FIGURE 31 Average queue discharge flow rates.

TABLE 22
Peak 15-minute Flow Rate Summaries for Other Sites in Texas

Highway	Observation	Peak 15-Minute Flow Rate by Lanes					% Trucks in Peak 15 Minutes
		1	2	3	4	Average	
I-35/US 67	1	2472	2528	2084	1768	2213	1.2
	2	2392	2532	2180	2088	2298	1.1
	3	2332	2456	2164	2160	2278	2.0
I-635 @ Coit	1	2380	2244	2220	2152	2249	1.9
North Tollway	1	2468	2344	2308	---	2373	.7
I-820	1	2176	1788	---	---	1982	.8
	2	2272	1808	---	---	2040	3.0
US 183 West of Central	1	2548	2244	2044	---	2279	3.2
	2	2608	2248	2388	---	2415	2.5
	3	2468	2116	2200	---	2261	2.6
US 290 @ Pinemont	1	2644	2388	2016	---	2349	2.8
I-410 @ McCullough	1	2348	2076	2120	---	2181	.9

TABLE 23
Summary Volumes from Other Sites in Texas

City	Highway	Number of Lanes	Observation	Peak Hour Volume Average per lane (vehicles/hour)	%Trucks in peak Hour
Dallas	I-35/US 67	4	1 2 3	2125 2211 2194	1.7 1.4 1.9
	I-635 @ Coit	4	1	2117	2.1
	North Tollway	3	1	2026	.5
Ft. Worth	I-820	2	1 2	1887 1882	1.7 3.5
	US 183 West of Central	3	1 2 3	2222 2308 2197	2.5 2.4 2.4
Houston	U.S. 290 @ Pinemont	3	1	2143	3.0
San Antonio	I-410 @ McCullough	3	1	2122	1.4

The peak 15-minute flow rates in Table 22 are very similar to the peak 15-minute flow rates from the primary study sites. The average peak 15-minute flow rates at the US 290 at Tidwell, I-35/US 67, I-410 at West and US 183 at Central were 2,253 vph, 2,155 vph, 2,255 vph and 2,086 vph respectively. The averages at the other sites were very similar except for samples on the North Tollway and on US 183, neither of which reached queue discharge conditions. In addition, I-820 is a four-lane freeway which makes effects in the right lane more significant to the overall flow than occurs on six or eight lane freeways.

Since these sites were not studied in detail, factors affecting the flows were not evaluated. Although there are some differences between all of the sites, the averages generally range between 2,100 vph and 2,300 vph for the peak 15 minutes as well as the peak hour.

Impact of Adverse Weather Conditions

During the data collection a sample was taken at the I-410 study site on a drizzly rainy day. The pavement conditions were wet but visibility and vehicle control was not likely a problem since the rain was light. In most other cases, the loops detectors did not work well in the rain, but they performed adequately during this day. The purpose of excluding this sample from the other samples was the lower flow rates measured. Table 24 shows the peak 15 minute flow rates during this sample.

TABLE 24
Peak 15 Minute Flow Rates During A Rainy Day

Sample	Lane 1	Lane 2	Lane 3	Average
I-410 Time 4 During Rain	1960	1664	1552	1725
I-410 Average	2496	2168	1800	2155

As can be seen in Table 24 the peak flow rates are much lower than the peak flow rates during the other data collection times. Although this is only from one sample, it does indicate that during adverse weather conditions, the maximum flow rates obtainable are likely lower than during ideal weather conditions. It should not be concluded that these values are representative of the impacts of adverse weather, only that adverse weather does reduce capacity, which has been demonstrated in several other studies.

Results of Validation

Based on the analysis of the I-410 and I-35/US 67 study sites some conclusions can be made which are described below.

1. Variance in flow rates decreases after the speed drop to queue discharge.
2. The hypothesis that once one or more lanes break down the other lanes are prematurely transitioned into queue discharge was confirmed.
3. The operational characteristics shown in the speed-flow model in Figure 18 were confirmed, although higher flow rates are possible in non-merge lanes than measured at the U.S. 290 study site.
4. Free-flow speed and type of bottleneck may influence the maximum possible flow obtainable during free-flow and possibly during queue discharge conditions.

Finally, queue discharge flow is considered the best for use as maximum sustainable flow because of the high variation in free-flow flow rates and the complexity of transition between free-flow conditions and queue discharge conditions. Although much higher flow rates can occur, they do not occur in all lanes at the same time and have high variability. The breakdown of one or two lanes causes the premature breakdown of other lanes, making the maximum flow of the other lanes queue discharge flow. Queue discharge is also a clear situation where demand exceeds capacity. Although flows higher than queue discharge will occur, they are unstable. A facility operating at flows greater than queue discharge is likely to break down reducing the flow to queue discharge.

CONCLUSIONS AND RECOMMENDATIONS

The analysis and validation procedure revealed many notable characteristics of flow and the relationship between speed and flow at freeway bottlenecks. Although the validation supported the basic findings for the U.S. 290 study site, there were clear differences among the sites. The purpose of this section is to bring together the findings from all of the sites to form a generalized speed-flow model and estimate the values that should be used as freeway capacity.

Study Results

The study produced four primary findings:

1. Freeway bottlenecks are the best locations for measurement of freeway capacity. This is because of the ability to determine when the transition to queue discharge occurs.
2. Free-flow flow rates have higher variability than queue discharge flow rates.
3. Peak flows during free-flow conditions do not generally occur in all lanes because of an imbalance of flow rates between individual lanes. This prematurely transitions the flow from free-flow into queue discharge in some lanes.
4. Queue discharge is the best estimate for maximum sustainable flow and therefore is recommended for use as capacity.

In addition to these results, individual site characteristics play an important role in the flow processes at specific sites. Because a bottleneck is a location on a freeway where demand exceeds capacity, the operation is influenced by both the type of bottleneck and the location with respect to other bottlenecks. The type of bottleneck significantly influences the distribution of traffic across lanes which determines the shape of the speed-flow relationship for each lane as well as for the average across the facility. For example, a change in cross section, such as the one shown in Figure 5, does not affect the distribution of traffic the same as a high volume right hand merge, which adds vehicles to a singe lane. Addition of a high volume of traffic to a single lane may cause the lane to break down and transition the entire facility into queue discharge, even if the other lanes have lower flows. This was shown in the analysis of U.S. 290 at Tidwell. The more balanced the flow is across all lanes, the greater the possibility is for higher flows before breakdown. At the I-35/US 67 site an average free-flow flow rate of 2680 vphpl was measured in lane 1 before breakdown, which shows that very high flows are possible under the right conditions. The concept of lane interaction is very important and may explain the variety of results obtained in earlier studies which attempted to find a single speed-flow relationship.

The second major factor is the location of the bottleneck with respect to other bottlenecks. In some major urban areas congestion occurs for miles extending through many interchanges which are not all bottleneck locations. There are two types of effects from adjacent bottlenecks. The first is produced by upstream congestion and the second is produced by downstream congestion. Upstream congestion has the effect of metering the flow and therefore, locations downstream may actually experience a reduction in flow once the upstream bottleneck breaks down. The second is the effect of a downstream bottleneck which was the most common at the study sites. Downstream congestion causes queues to back up through upstream bottlenecks. The effect of this phenomenon is lower service rates at the subject bottleneck location. At the I-410 site, a downstream slowdown caused a drop in flow of over 400 vphpl. At the U.S. 290 site, congestion from a downstream off-ramp reduced speed and flows after the subject bottleneck had independently broken down.

It should be noted that the study sites were selected to be the controlling bottleneck and free from downstream congestion. In reality, this condition is difficult to sustain as indicated by the study results. On most freeways, demand changes at different locations throughout the peak period and causes different bottlenecks to break down at different times. Therefore, sites with no upstream or downstream congestion effects are rare. Although a bottleneck may appear congested it is important to determine if the congestion is a result of downstream effects. It is likely that larger reductions in flow due to downstream bottlenecks occur at other sites. The speed-flow relationship and maximum sustainable flow rates proposed in this study are for independent bottlenecks.

Proposed Flow Model

Based on the speed-flow curves for the U.S. 290 and I-35/US 67 sites, a combined speed-flow model was developed, as shown in Figure 32. These curves are based on the free-flow curves in Figures 18 and 30 and are applicable for the overall bottleneck operation as well as individual lanes. As can be seen in Figure 32, there are three apparent regions of flow:

Region 1: Increasing flow at constant speed

Region 2: Increasing flow and slight reduction in speed

Region 3: Unstable flow

Region 1 can be described as high speed uncongested operation and is therefore characterized by "free flow" operation. In Region 2 higher traffic flows produce a slight reduction in speed which is described as "restricted operation." Speeds typically begin to reduce at approximately 1400 vphpl to 1800 vphpl. The reduction in speed is more prevalent for higher free-flow speeds than for lower free flow speeds. In Region 3 the flow rate is over the maximum sustainable flow rate and considered unstable and therefore may break down into queue discharge.

A generalized speed-flow relationship was developed including adjustments for heavy vehicles which is shown in Figure 33. Based on the analysis contained in this report, the maximum sustainable flow was determined to be 2,200 pcphpl. Although it is possible to sustain flows as high as 2,400 pcphpl in high-speed, non-merge lanes, it is not possible for all bottleneck configurations. A flow rate of 2,200 pcphpl can be achieved in almost any type of lane and as well as over an entire bottleneck facility. Table 25 shows the maximum flow rates associated which each operational level.

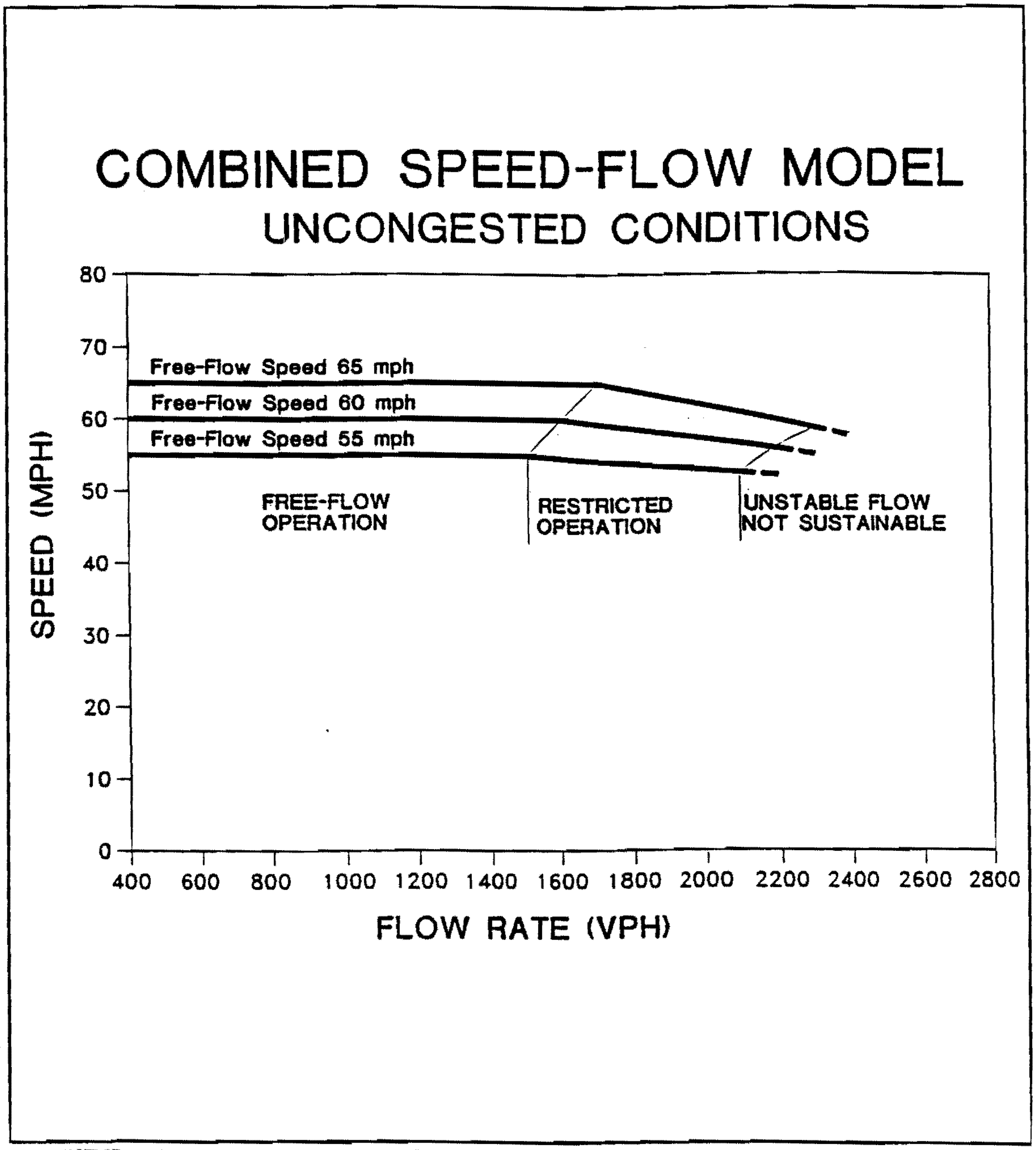


FIGURE 32 Combined speed-flow model for uncongested conditions (w/o adjustments for heavy vehicles).

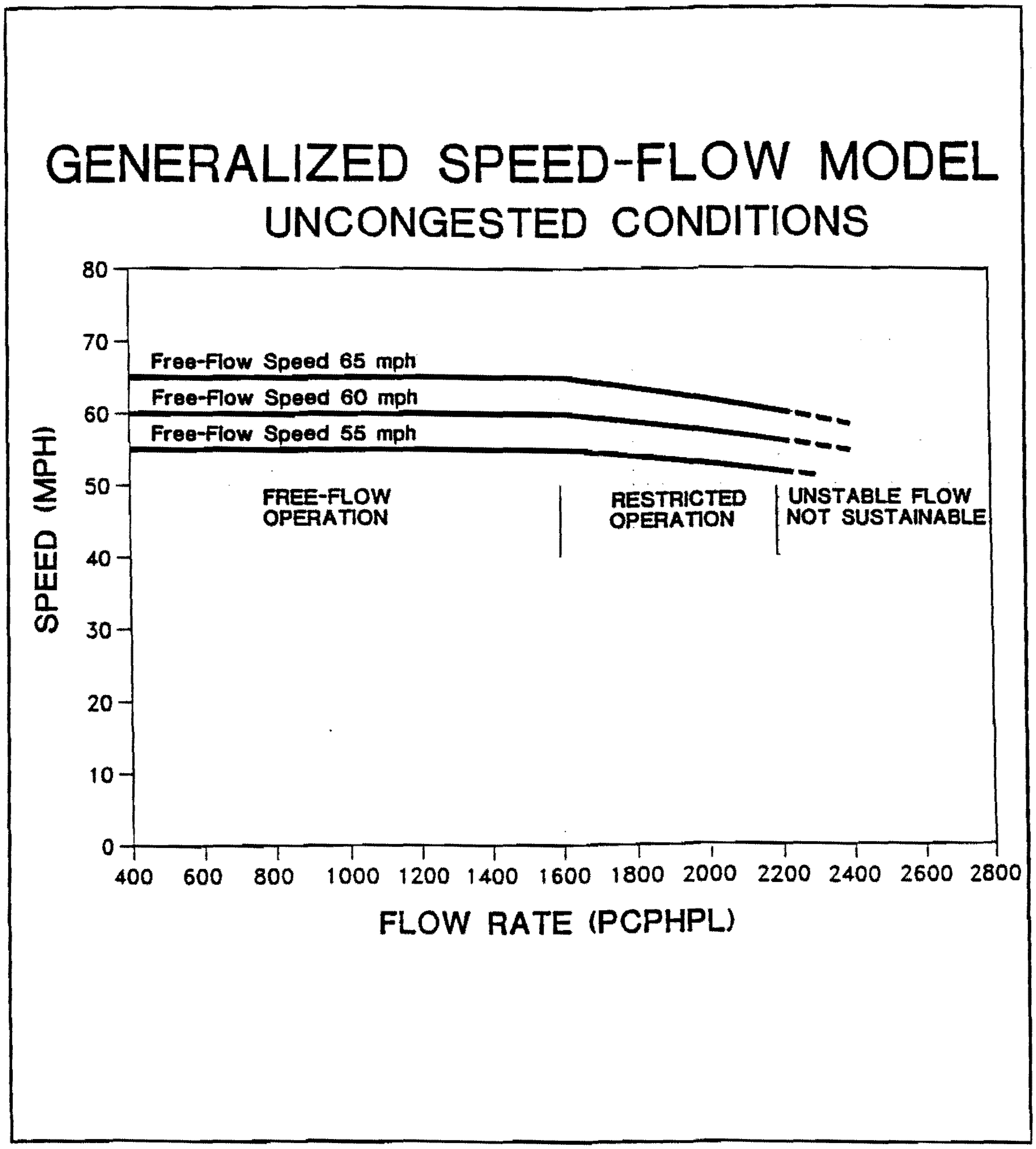


FIGURE 33 Generalized speed-flow model for uncongested conditions.

Although, theoretically, if all the flow rates in all lanes are kept below queue discharge, the facility will remain uncongested; this may not always be easily achieved. Merging and weaving activity may cause premature transitions to queue discharge. In addition, lane preference alone causes lane imbalances which may cause the facility to prematurely break down. Nevertheless, these capacities are sustainable under queue discharge conditions. Therefore, although the capacities are applicable in both uncongested and congested conditions, the curves shown in Figure 33 illustrate the operation assuming the facility remains uncongested.

TABLE 25
Maximum Flow Rates For Operational Conditions

Flow Condition	Flow
Free-Flow Operation	0 - 1600 pcphpl
Restricted Operation	1600 - 2200 pcphpl
Unstable	> 2200 pcphpl

Recommendations for Use

To effectively utilize the proposed model, the conditions on which the model was based should be understood. The speed-flow model shown in Figure 33 represents the following conditions:

1. The conditions directly downstream of the bottleneck, which is usually some type of merge location.
2. No downstream congestion is present.
3. The traffic conditions remain uncongested.
4. The terrain is nearly level.
5. The heavy vehicle percentages are low.

If a site does not match the conditions listed above, the results may vary from those predicted by the model.

The traffic conditions assume that the flows remain uncongested and that no downstream congestion is present. Because congestion can be caused by the breakdown of the subject bottleneck or by downstream congestion, the cause must be ascertained. Congestion caused by downstream congestion commonly results in a substantial reduction in flow and can usually be identified by slow speeds and lack of acceleration after the bottleneck. Congestion caused by demand exceeding the capacity of the bottleneck can usually be identified by slowing or stop and go conditions at the bottleneck location with traffic accelerating back toward free-flow speed beyond the bottleneck.

The analysis of freeway systems is obviously complicated by the effects of queuing upstream of the bottleneck. If conditions exceed a capacity of 2,200 pcphpl, the only practical analysis requires the use of a computer simulation model to accurately assess conditions. A section upstream of a section operating with demand in excess of capacity can not be assumed to operate at flows up to 2,200 pcphpl.

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APPENDIX A

U.S. 290 SPEED AND FLOW PROFILES

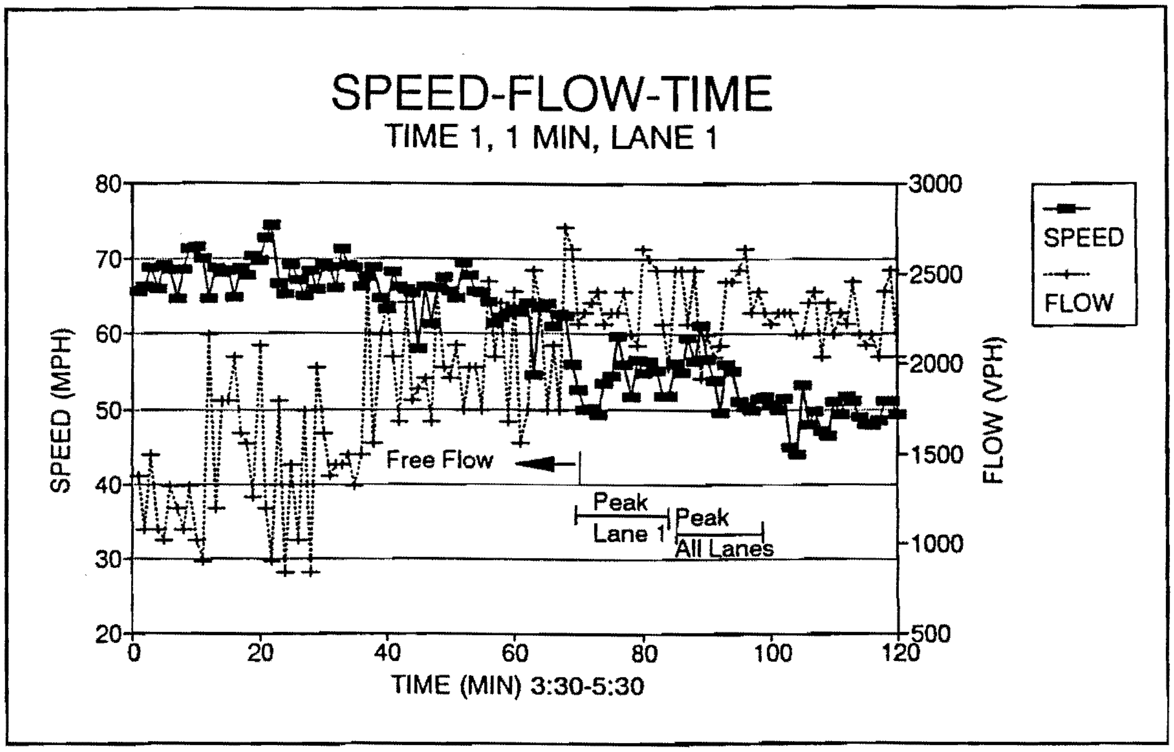

FIGURE A-1 Speed and flow vs time at US 290 for time 1 lane 1 using 1-minute intervals.

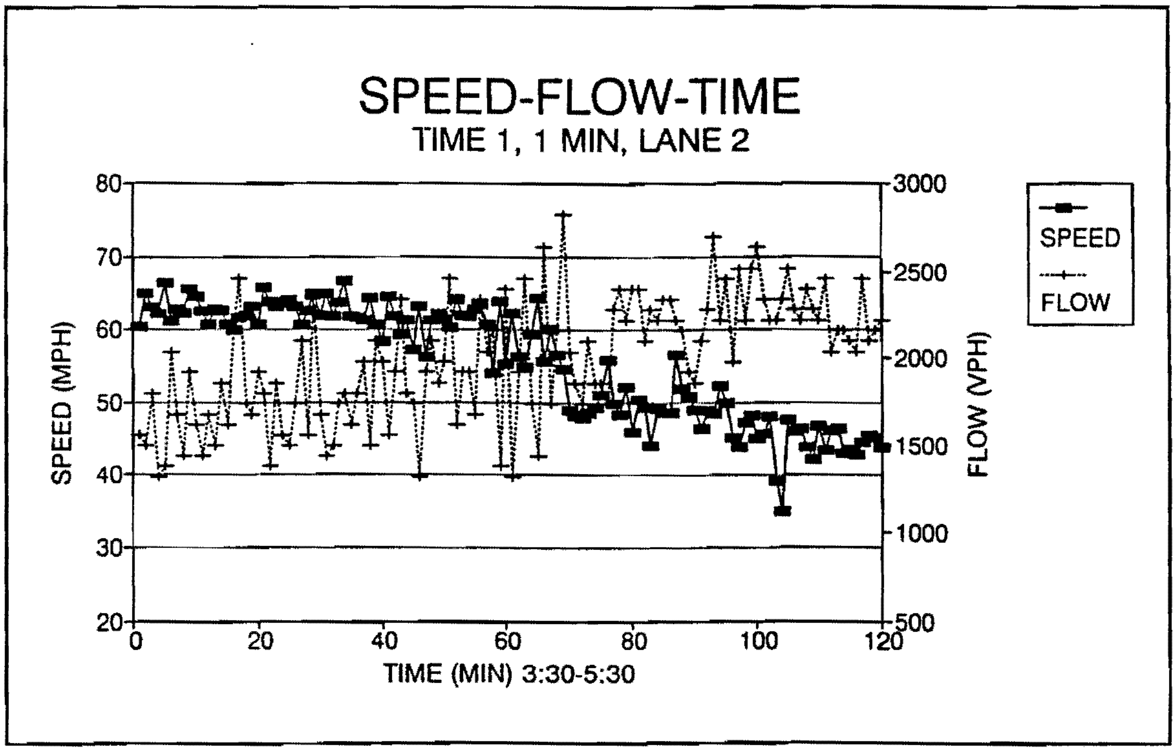

FIGURE A-2 Speed and flow vs time at US 290 for time 1 lane 2 using 1-minute intervals.

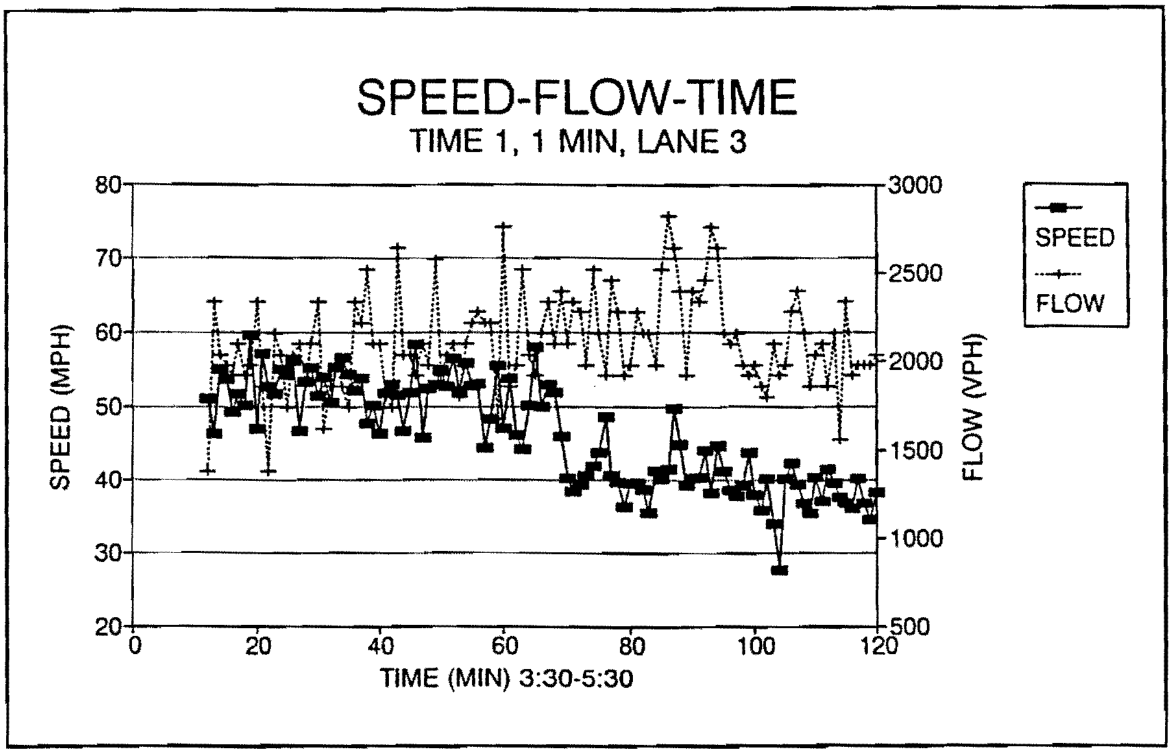

FIGURE A-3 Speed and flow vs time at US 290 for time 1 lane 3 using 1-minute intervals.

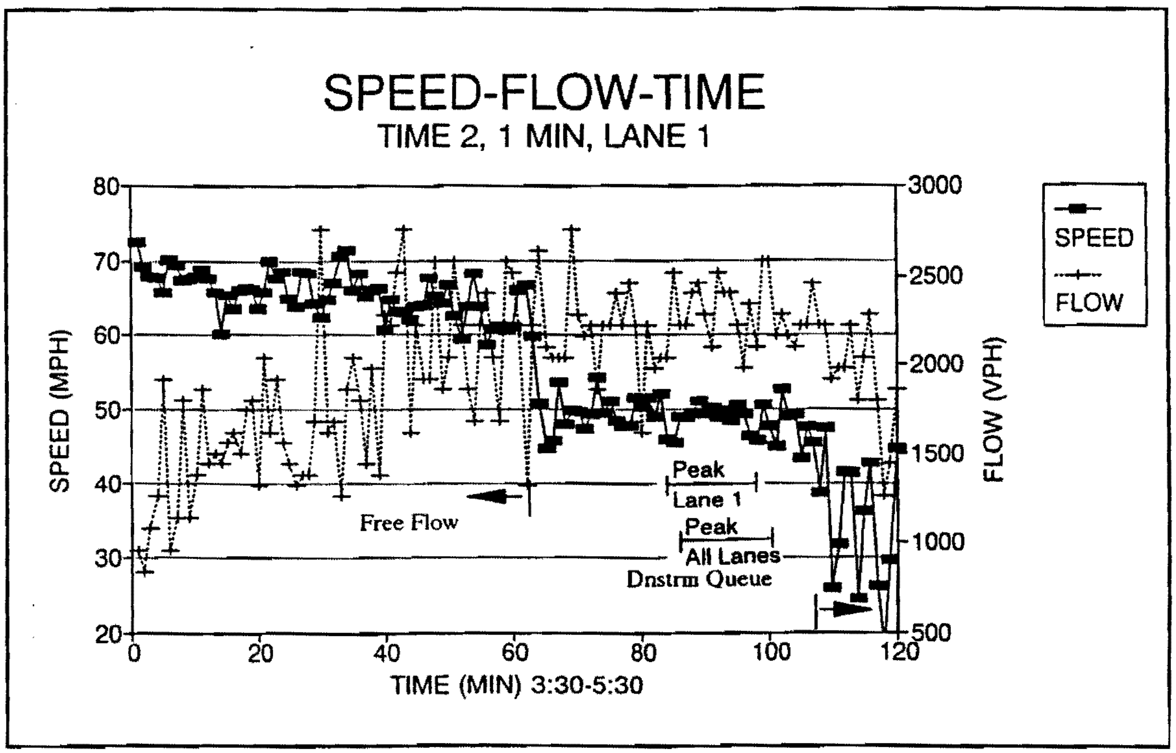

FIGURE A-4 Speed and flow vs time at US 290 for time 2 lane 1 using 1-minute intervals.

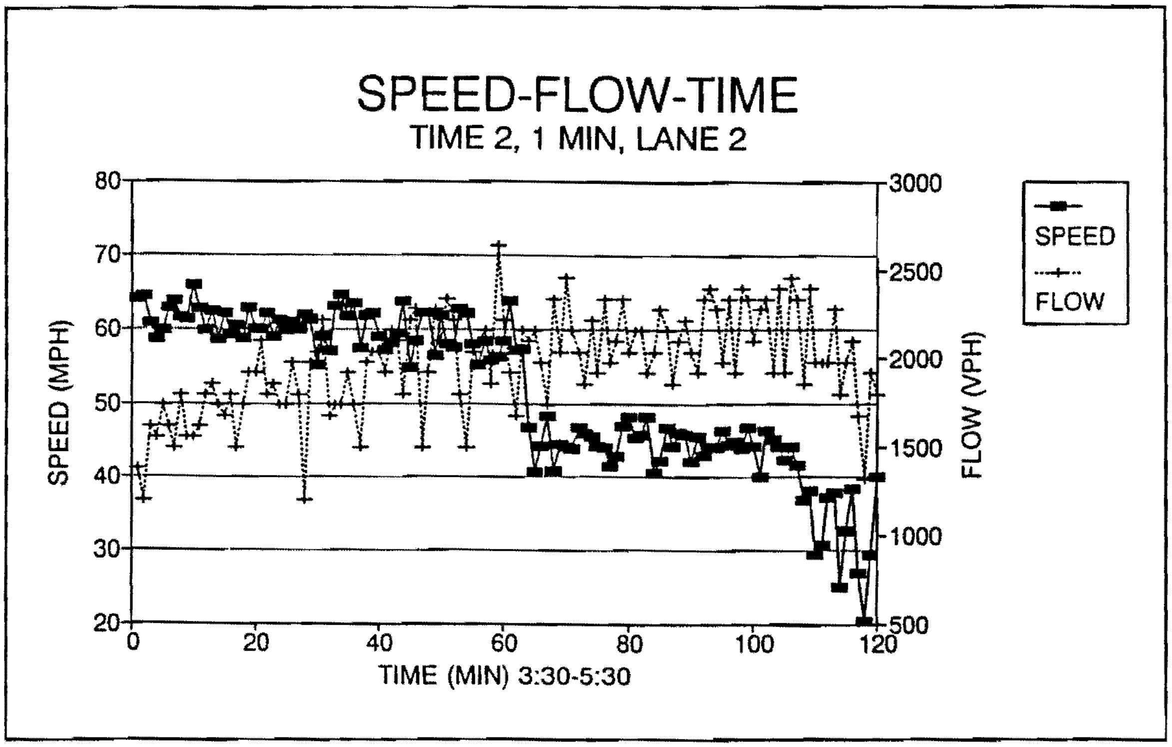

FIGURE A-5 Speed and flow vs time at US 290 for time 2 lane 2 using 1-minute intervals.

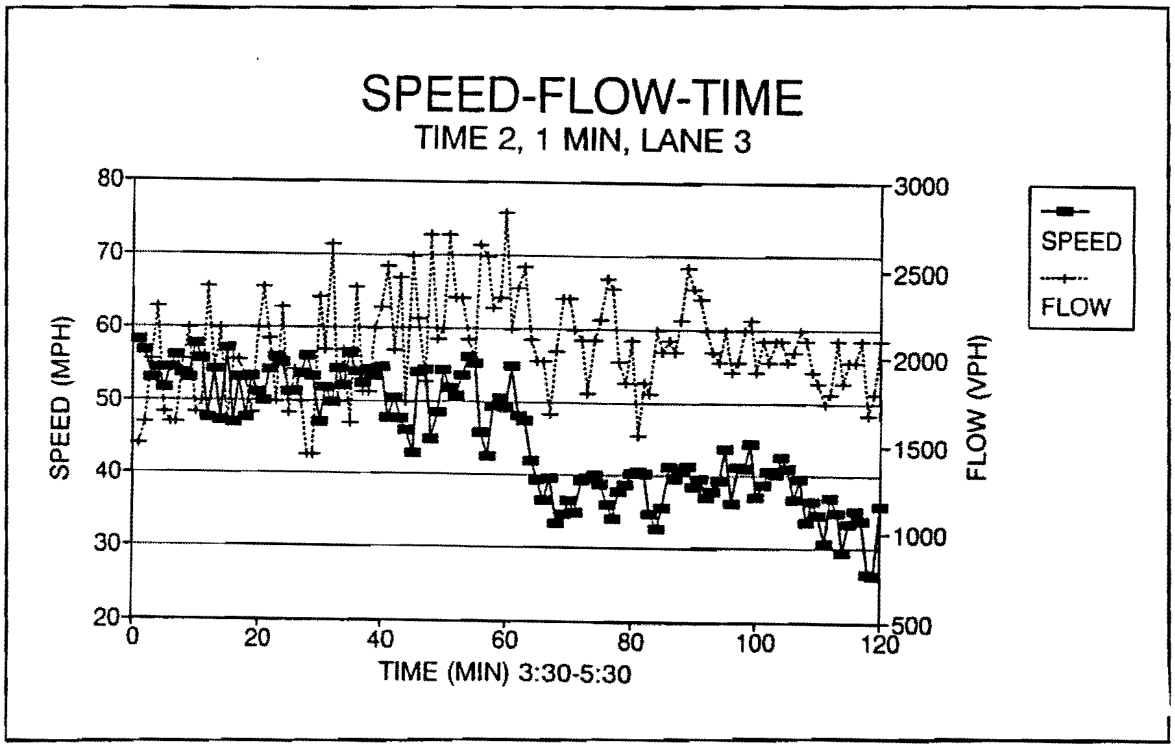

FIGURE A-4 Speed and flow vs time at US 290 for time 2 lane 3 using 1-minute intervals.

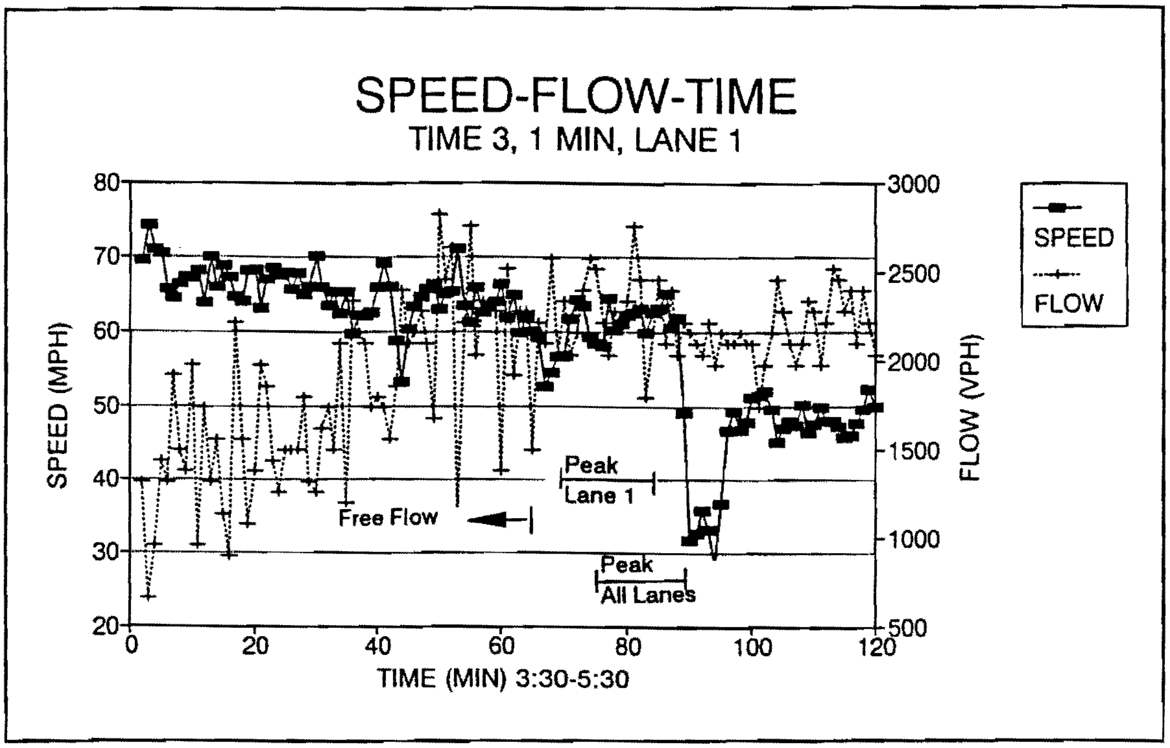

FIGURE A-7 Speed and flow vs time at US 290 for time 3 lane 1 using 1-minute intervals.

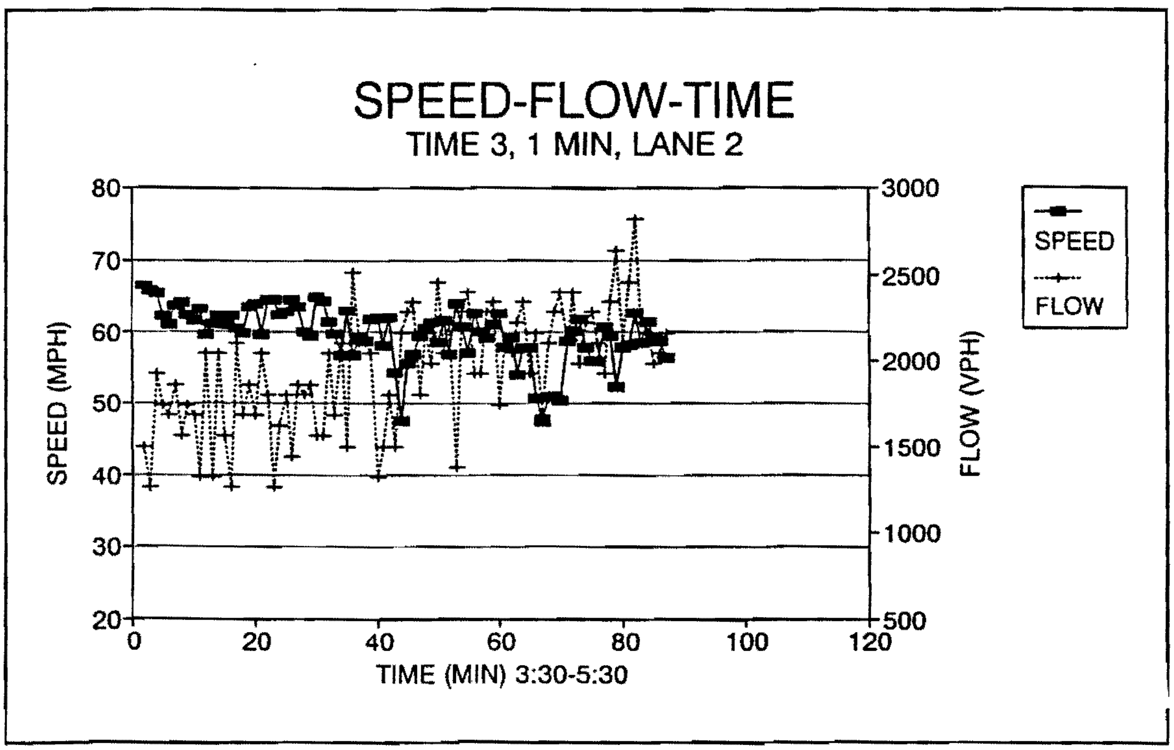

FIGURE A-8 Speed and flow vs time at US 290 for time 3 lane 2 using 1-minute intervals.

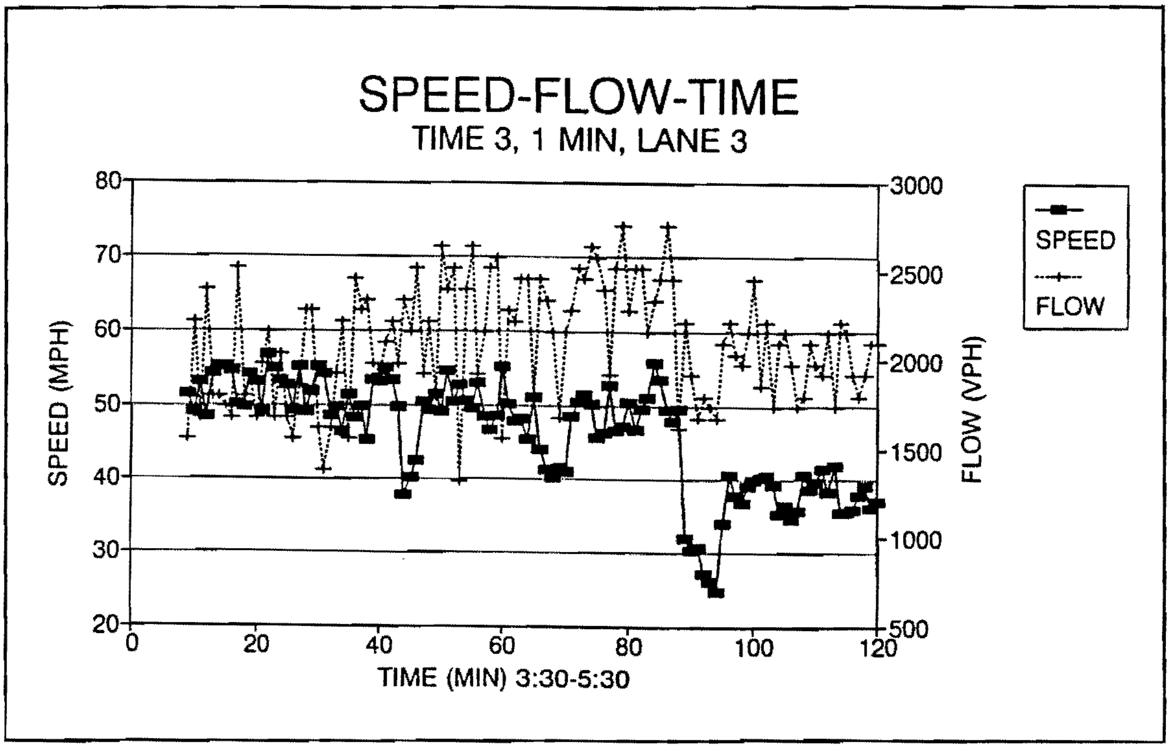

FIGURE A-9 Speed and flow vs time at US 290 for time 3 lane 3 using 1-minute intervals.

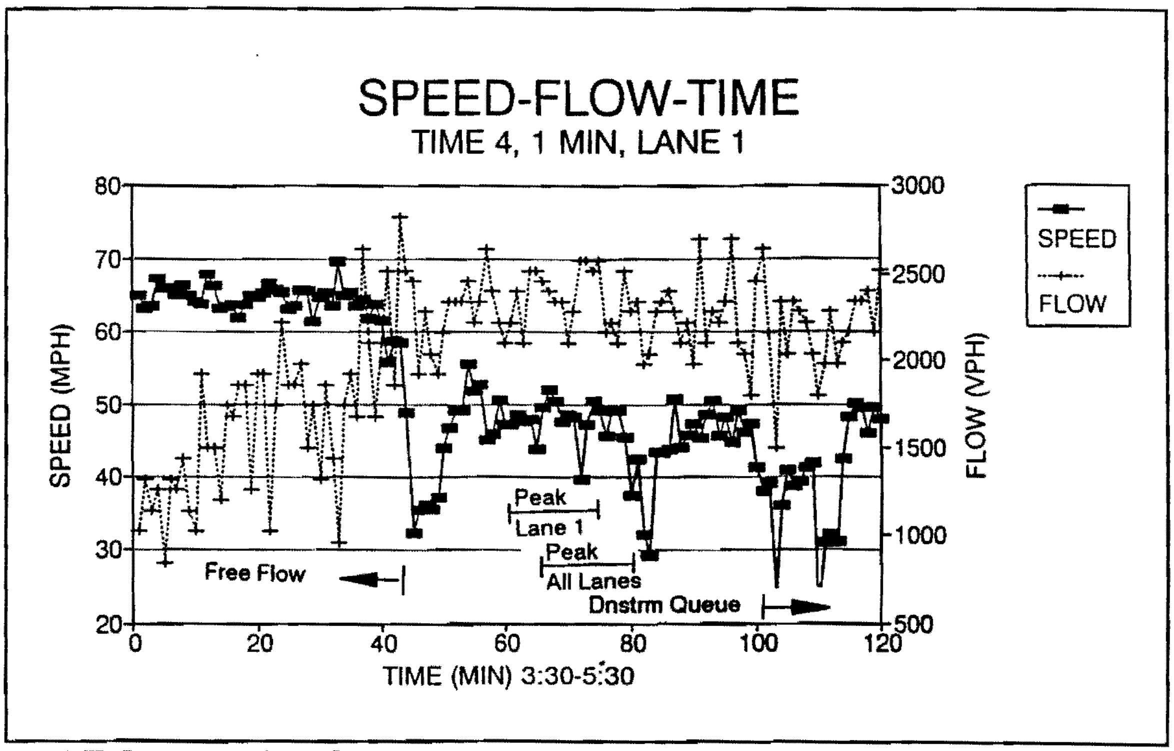

FIGURE A-10 Speed and flow vs time at US 290 for time 4 lane 1 using 1-minute intervals.

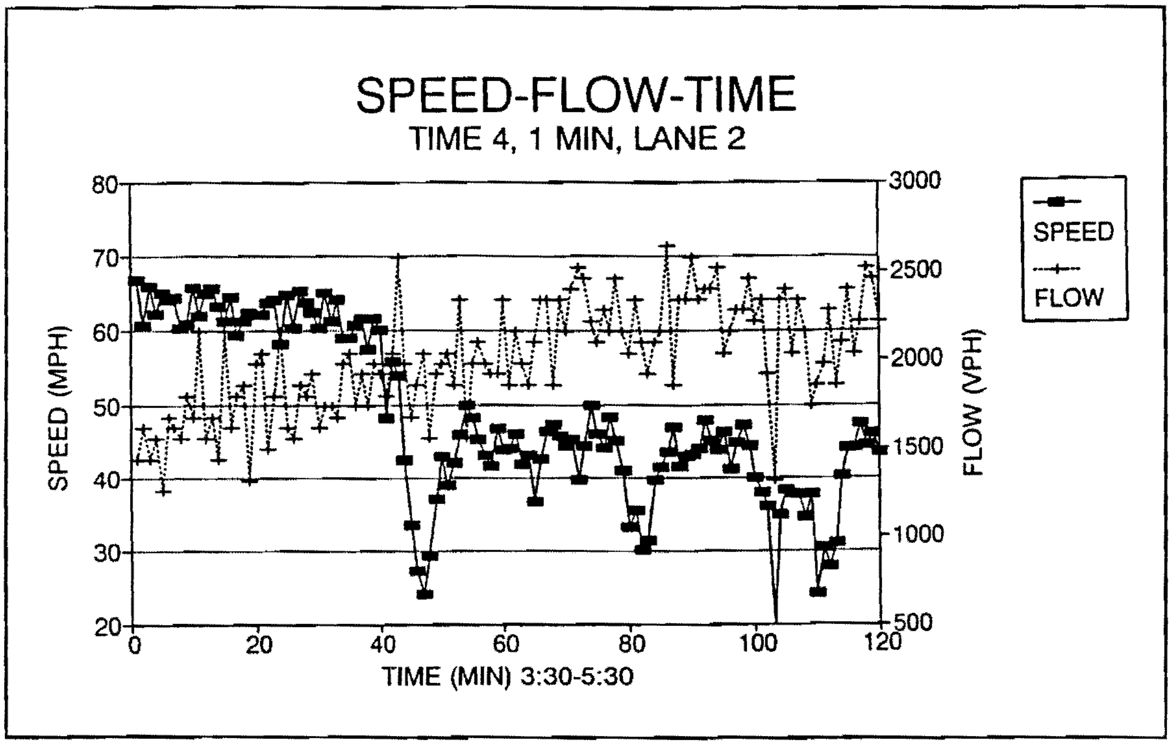

FIGURE A-11 Speed and flow vs time at US 290 for time 4 lane 2 using 1-minute intervals.

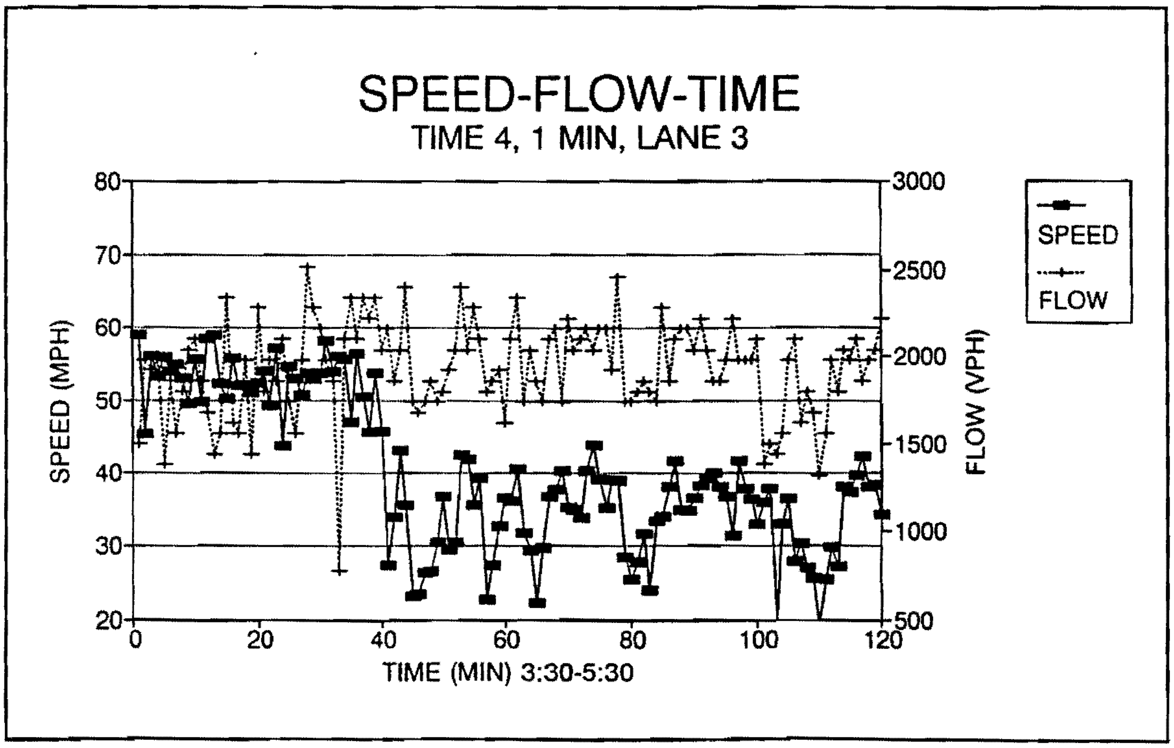

FIGURE A-12 Speed and flow vs time at US 290 for time 4 lane 3 using 1-minute intervals.

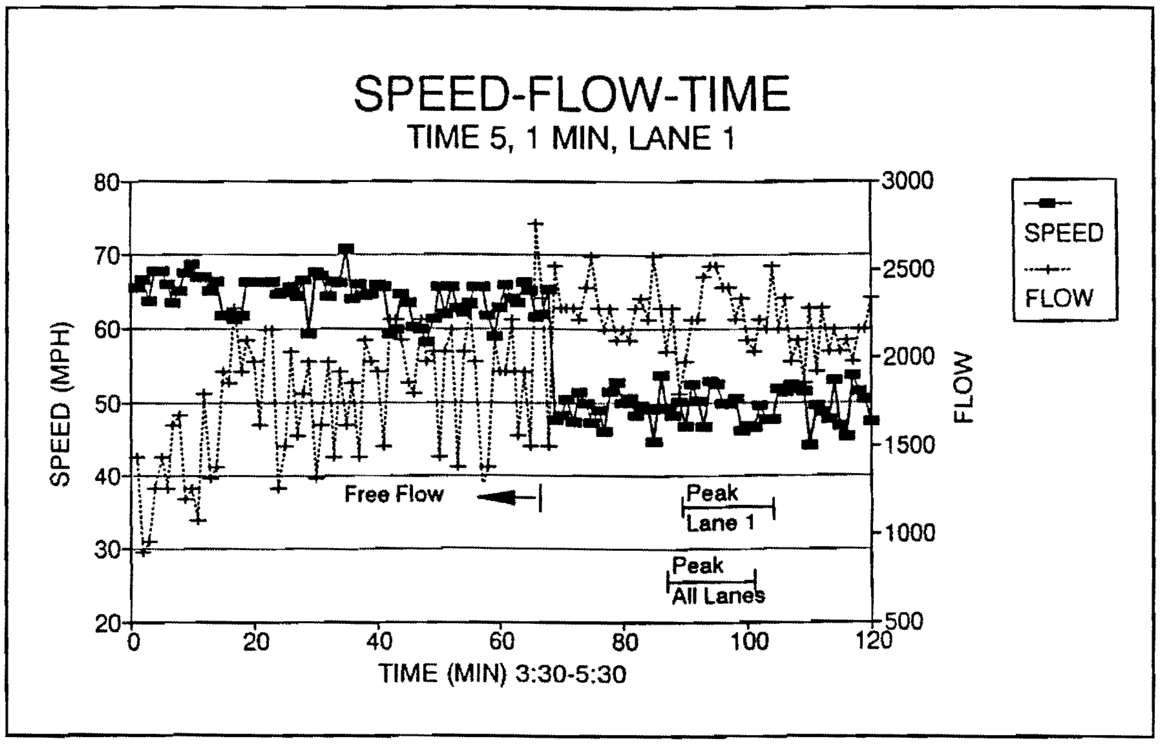

FIGURE A-13 Speed and flow vs time at US 290 for time 5 lane 1 using 1-minute intervals.

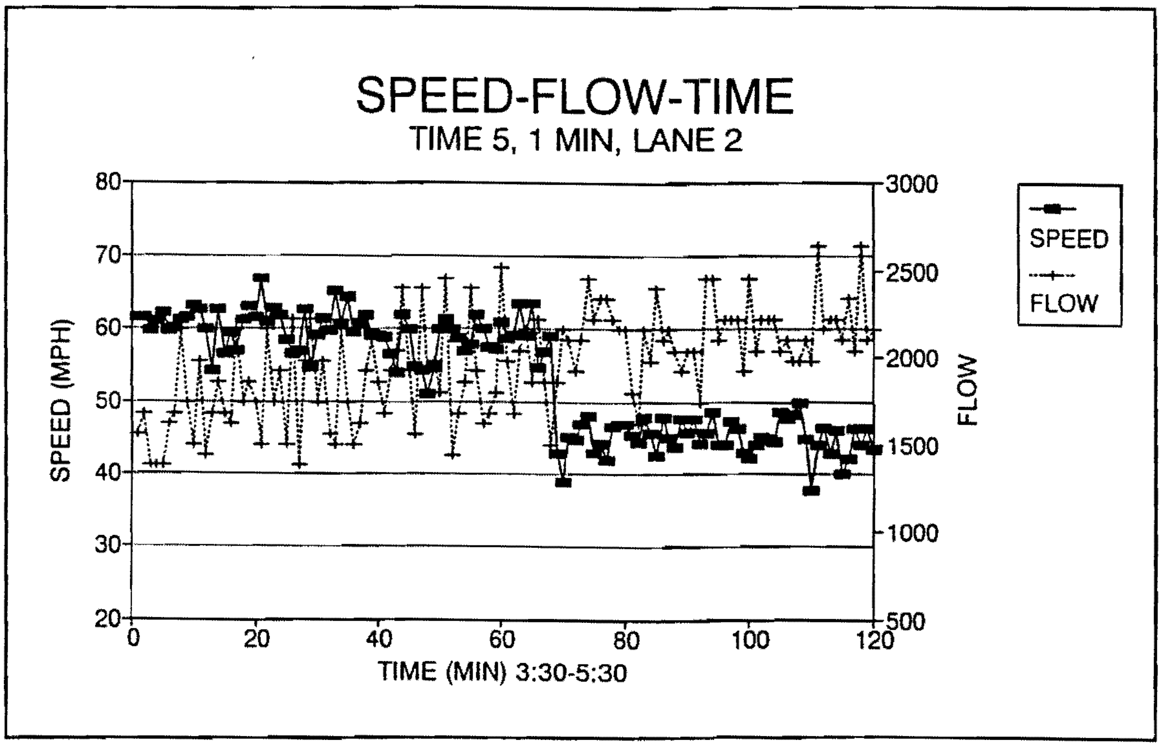

FIGURE A-14 Speed and flow vs time at US 290 for time 5 lane 2 using 1-minute intervals.

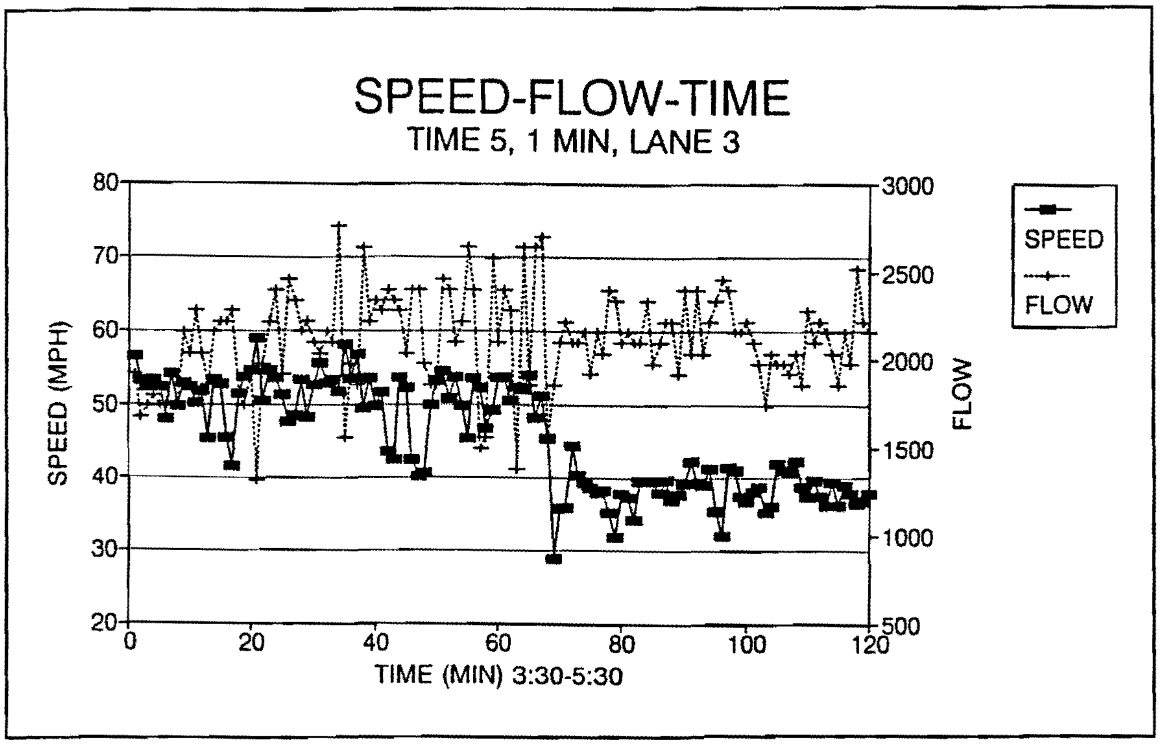

FIGURE A-15 Speed and flow vs time at US 290 for time 5 lane 3 using 1-minute intervals.

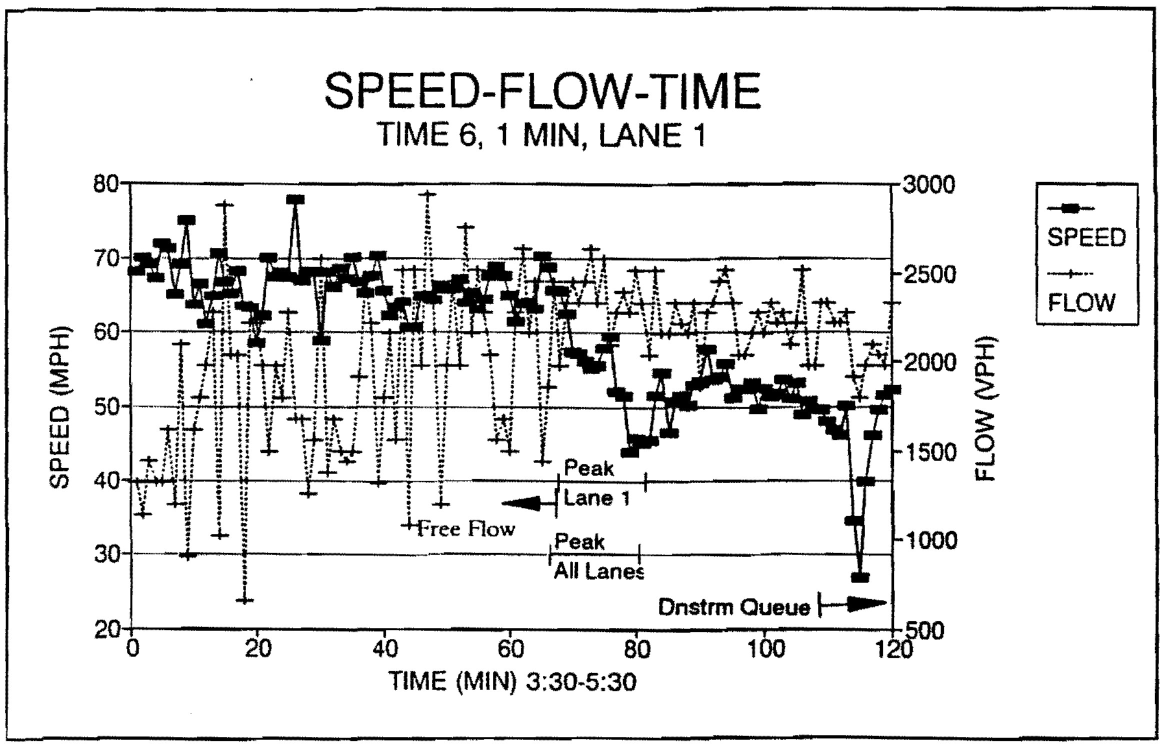

FIGURE A-16 Speed and flow vs time at US 290 for time 6 lane 1 using 1-minute intervals.

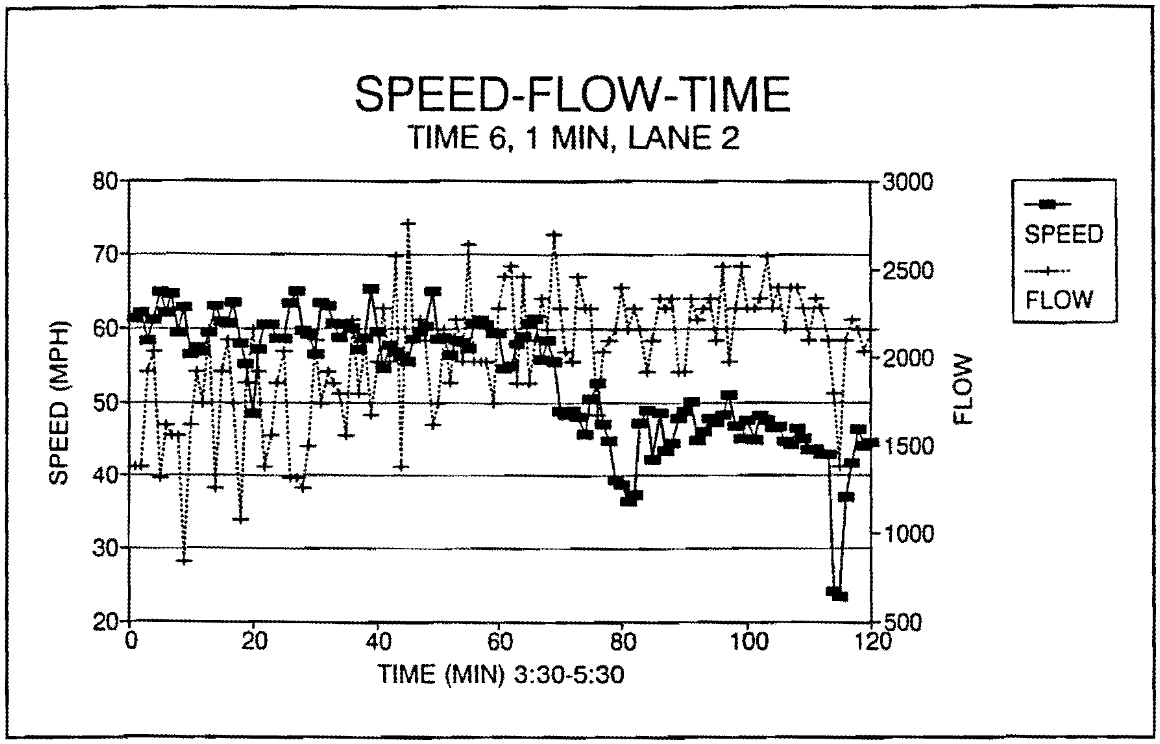

FIGURE A-17 Speed and flow vs time at US 290 for time 6 lane 2 using 1-minute intervals.

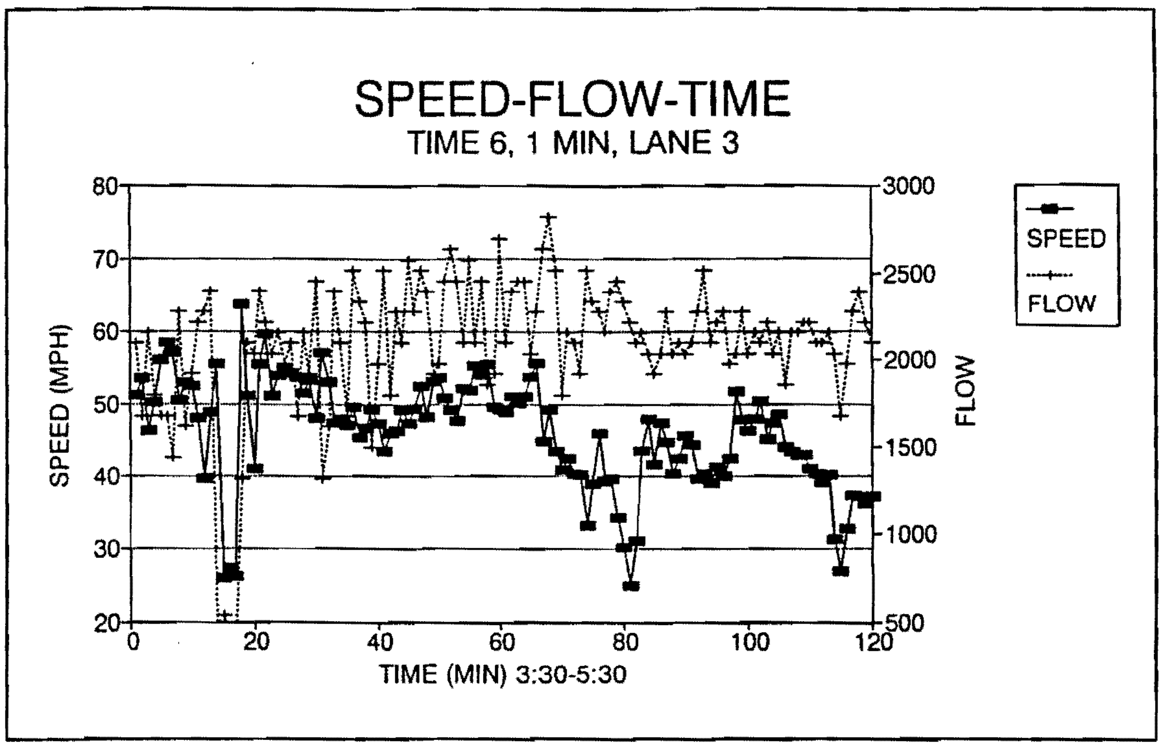

FIGURE A-18 Speed and flow vs time at US 290 for time 6 lane 3 using 1-minute intervals.

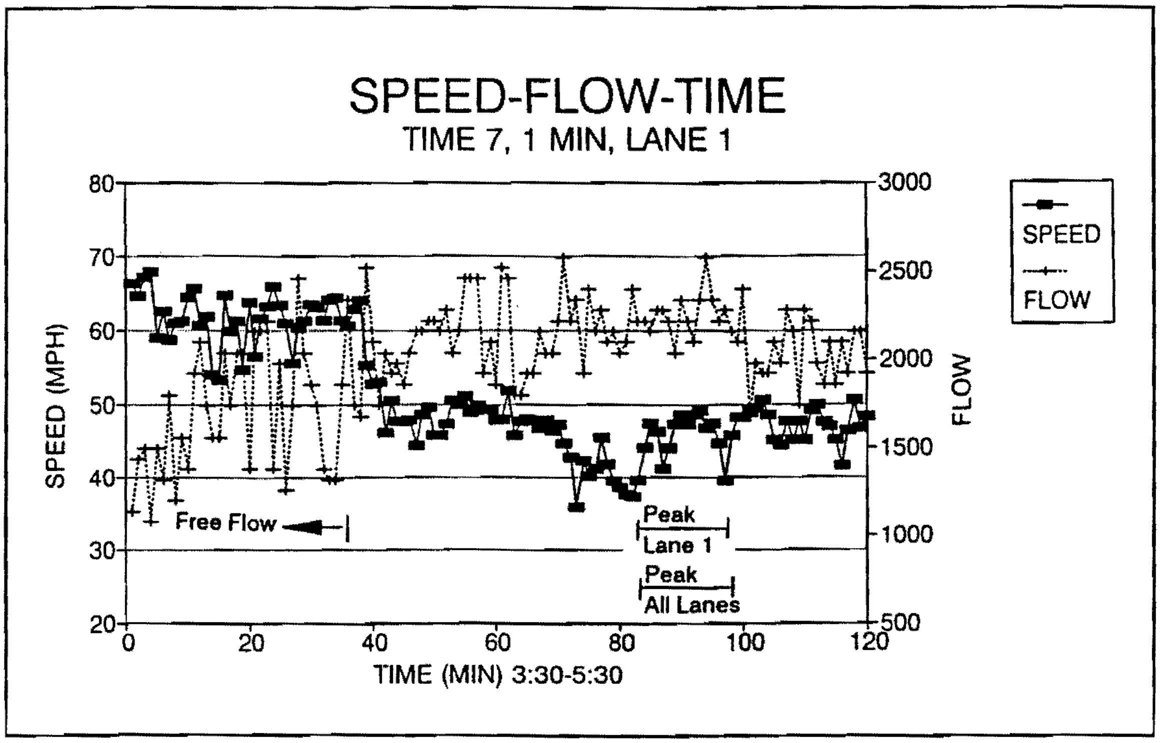

FIGURE A-19 Speed and flow vs time at US 290 for time 7 lane 1 using 1-minute intervals.

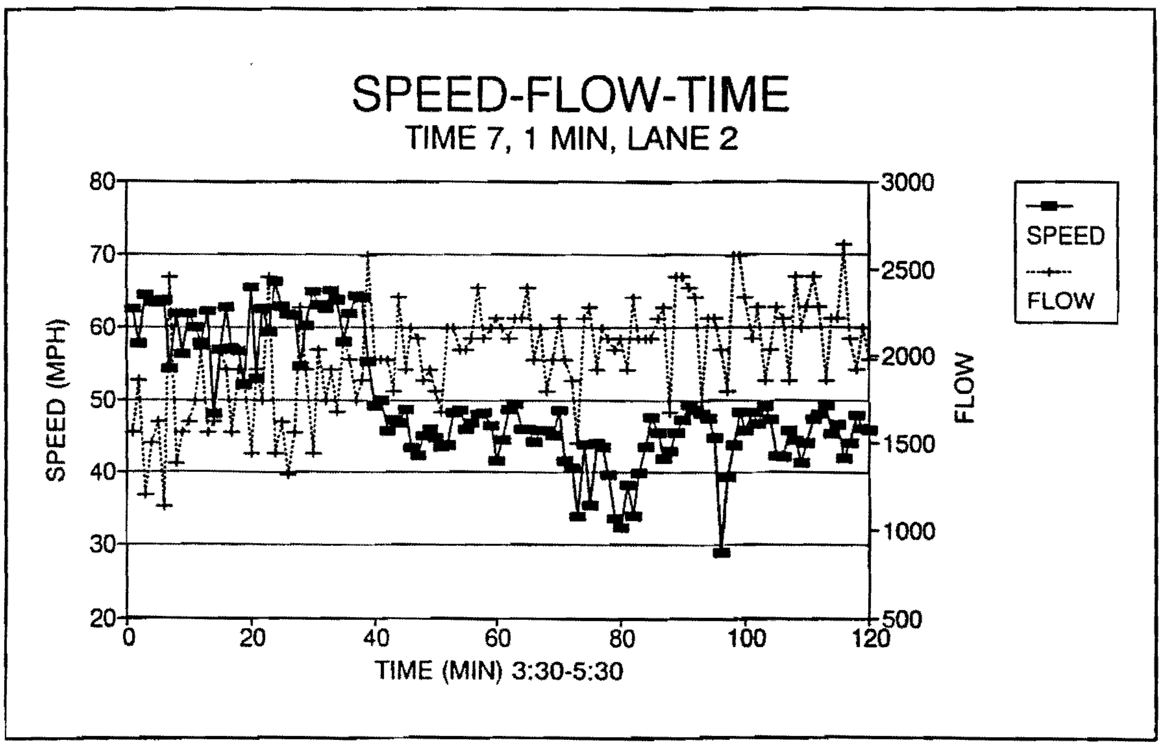

FIGURE A-20 Speed and flow vs time at US 290 for time 7 lane 2 using 1-minute intervals.

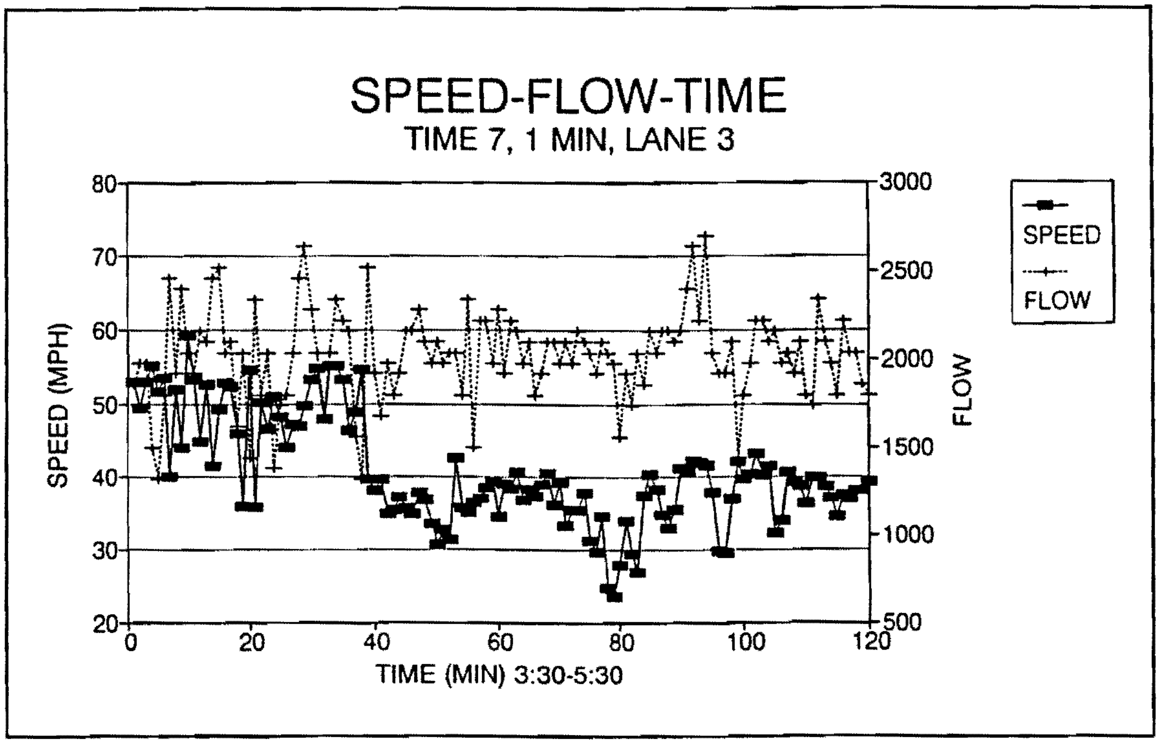

FIGURE A-21 Speed and flow vs time at US 290 for time 7 lane 3 using 1-minute intervals.

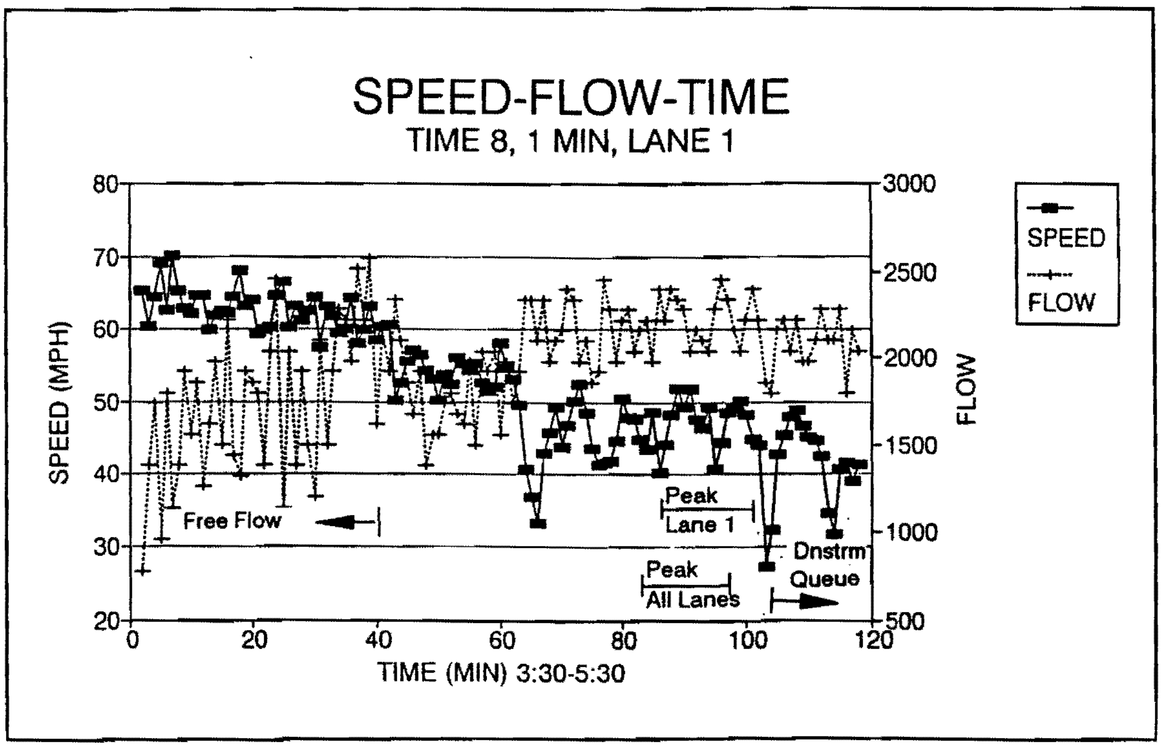

FIGURE A-22 Speed and flow vs time at US 290 for time 8 lane 1 using 1-minute intervals.

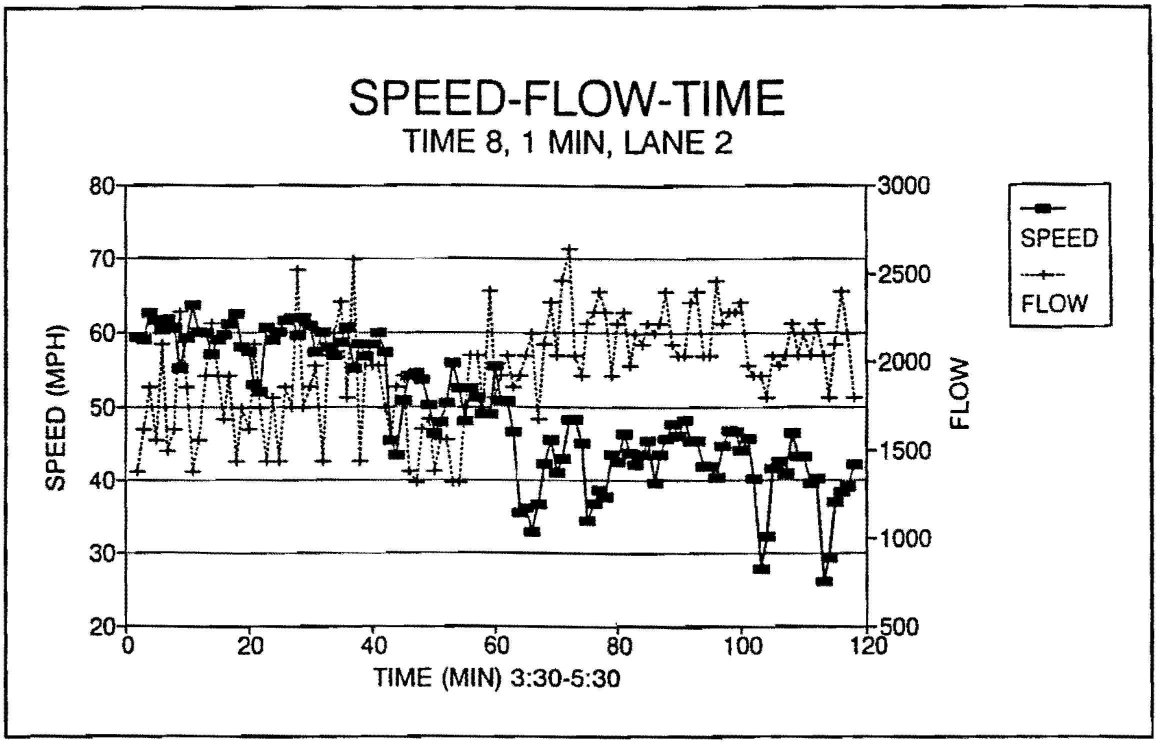

FIGURE A-23 Speed and flow vs time at US 290 for time 8 lane 2 using 1-minute intervals.

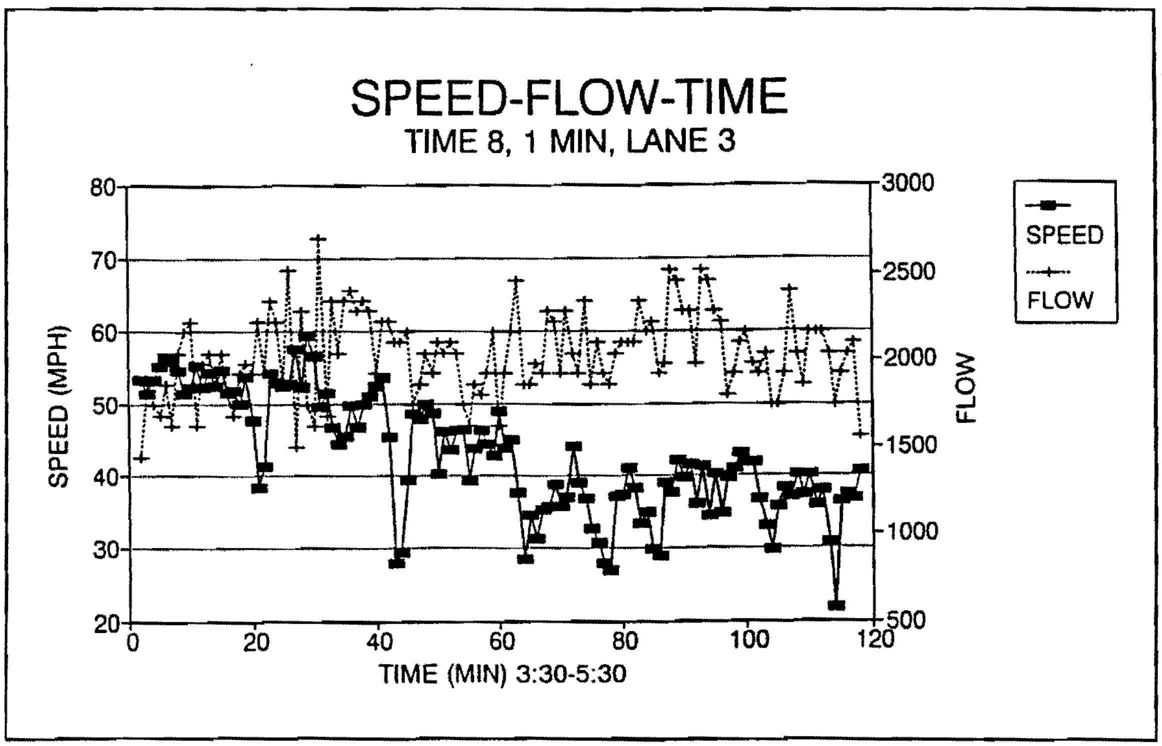

FIGURE A-24 Speed and flow vs time at US 290 for time 8 lane 3 using 1-minute intervals.

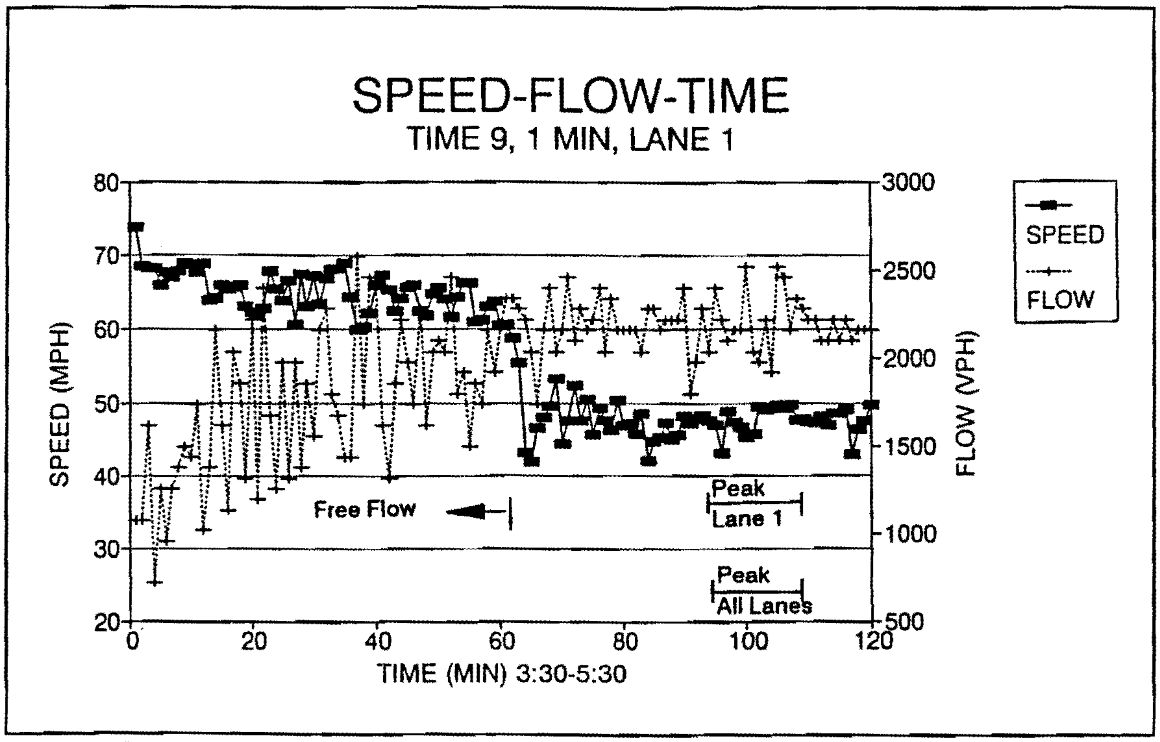

FIGURE A-25 Speed and flow vs time at US 290 for time 9 lane 1 using 1-minute intervals.

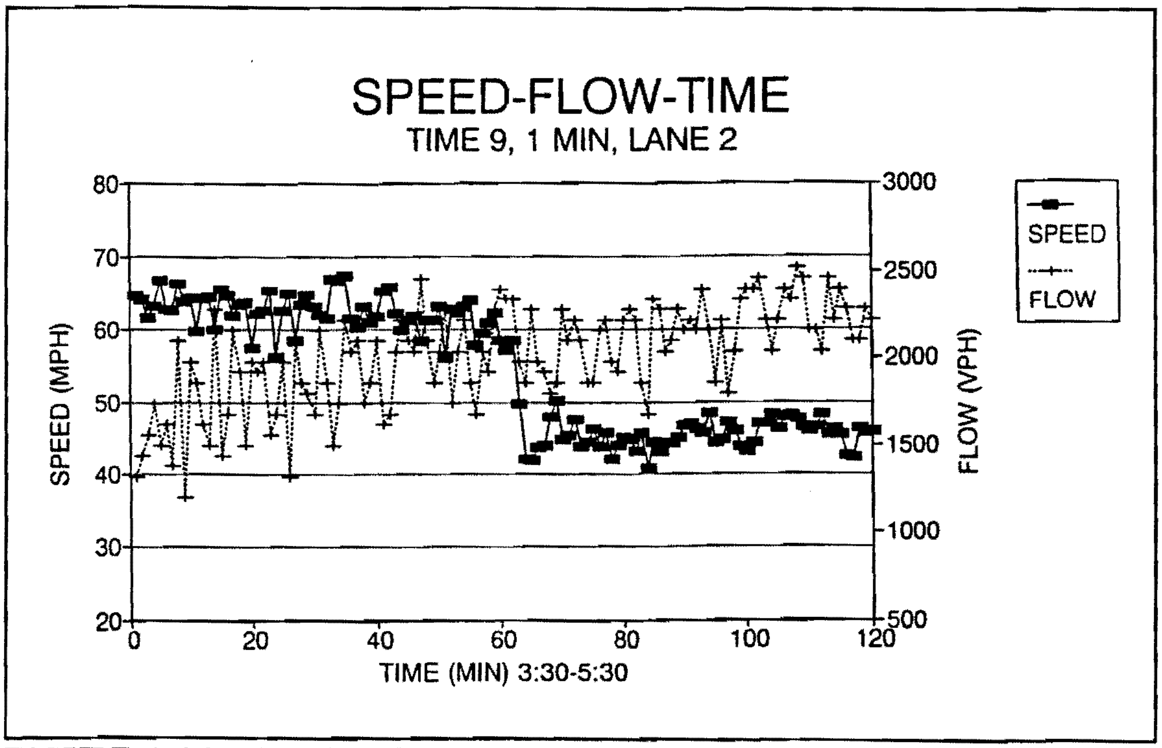

FIGURE A-26 Speed and flow vs time at US 290 for time 9 lane 2 using 1-minute intervals.

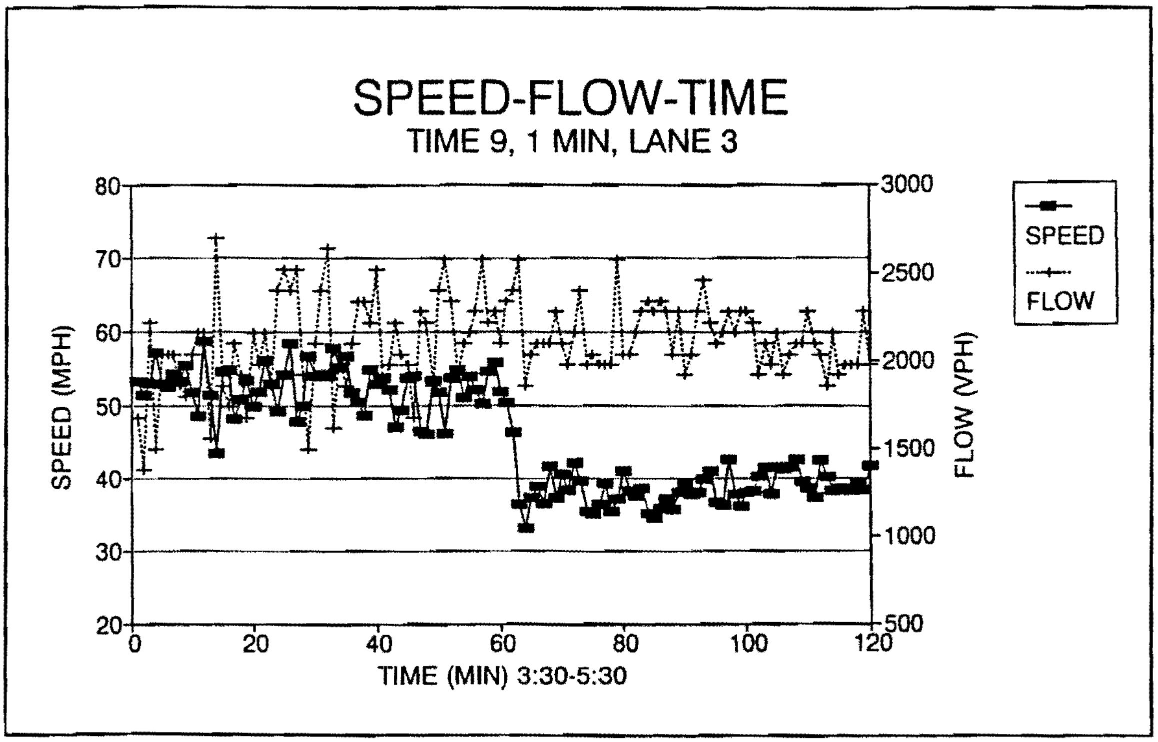

FIGURE A-27 Speed and flow vs time at US 290 for time 9 lane 3 using 1-minute intervals.

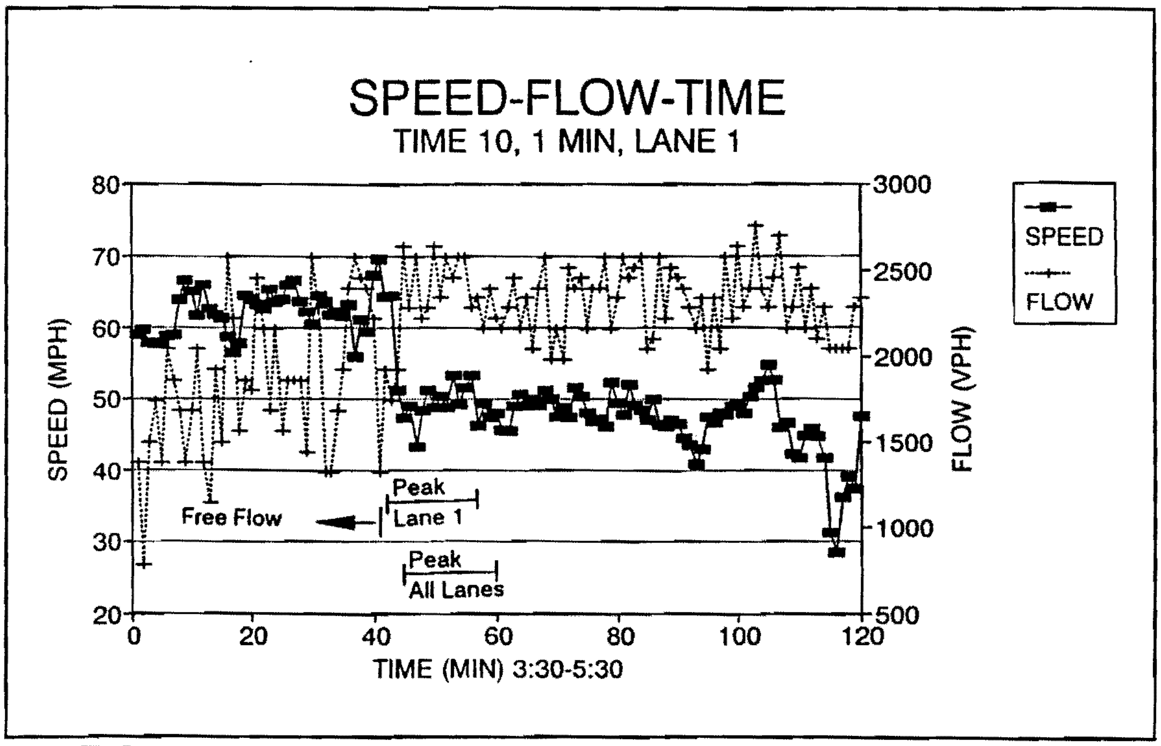

FIGURE A-28 Speed and flow vs time at US 290 for time 10 lane 1 using 1-minute intervals.

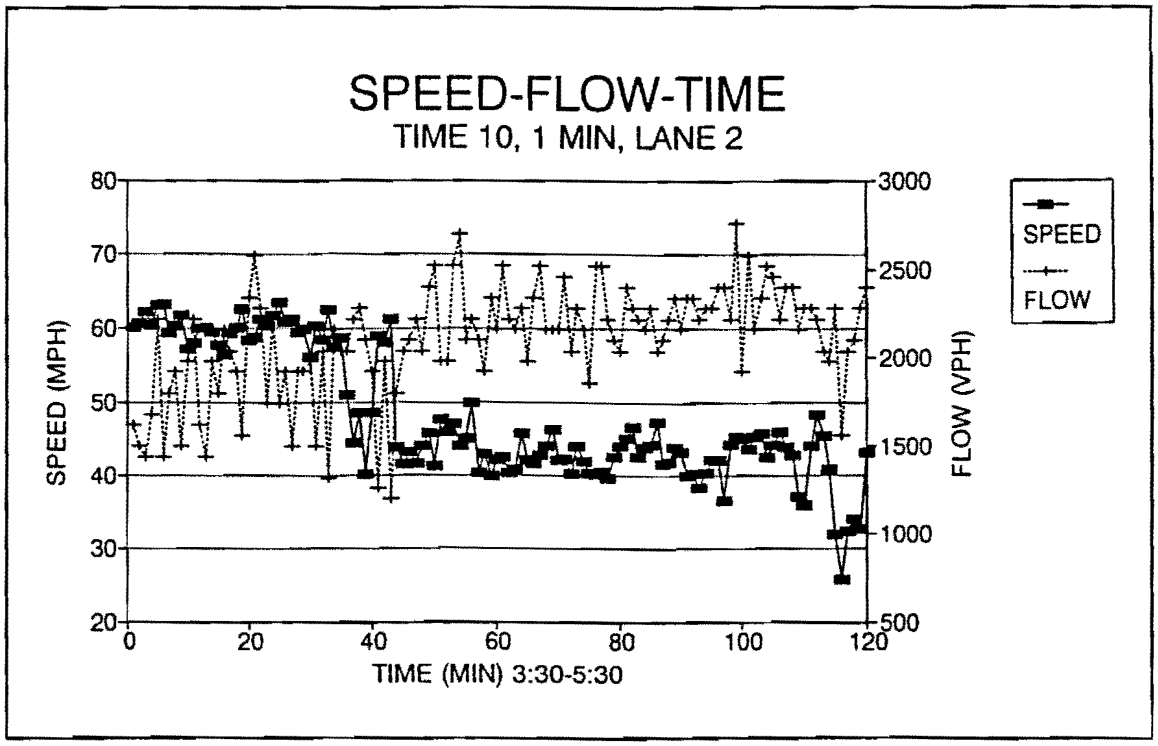

FIGURE A-29 Speed and flow vs time at US 290 for time 10 lane 2 using 1-minute intervals.

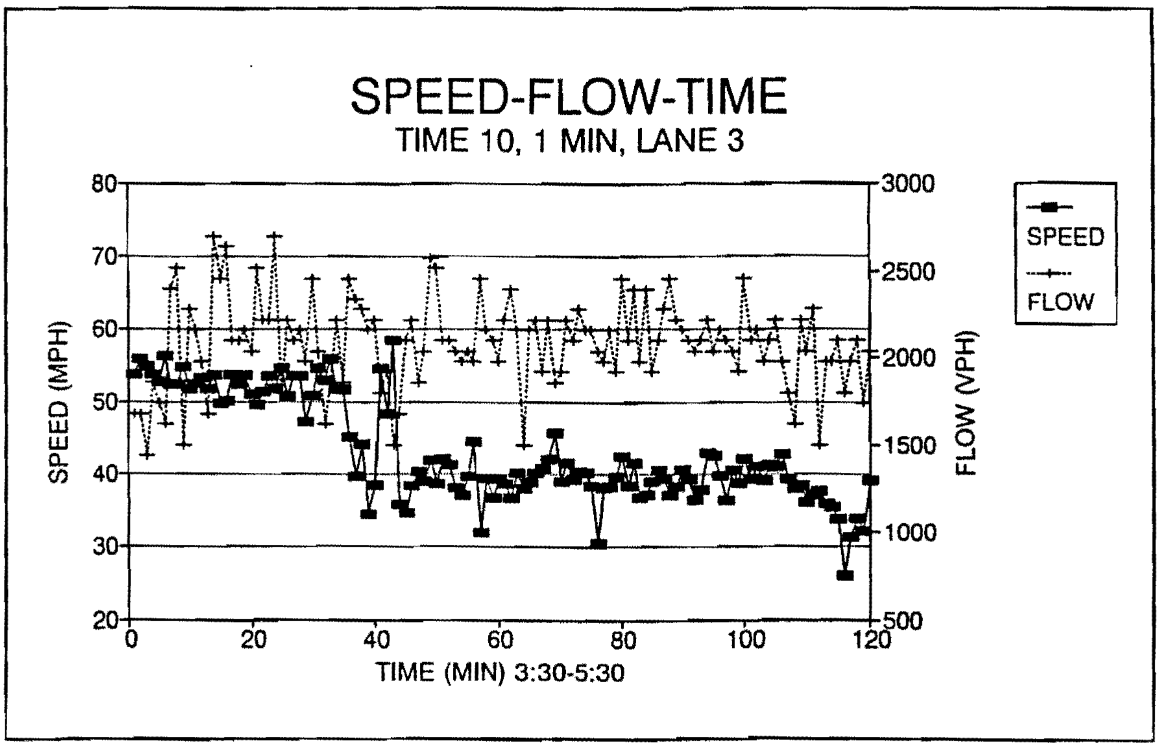

FIGURE A-30 Speed and flow vs time at US 290 for time 10 lane 3 using 1-minute intervals.

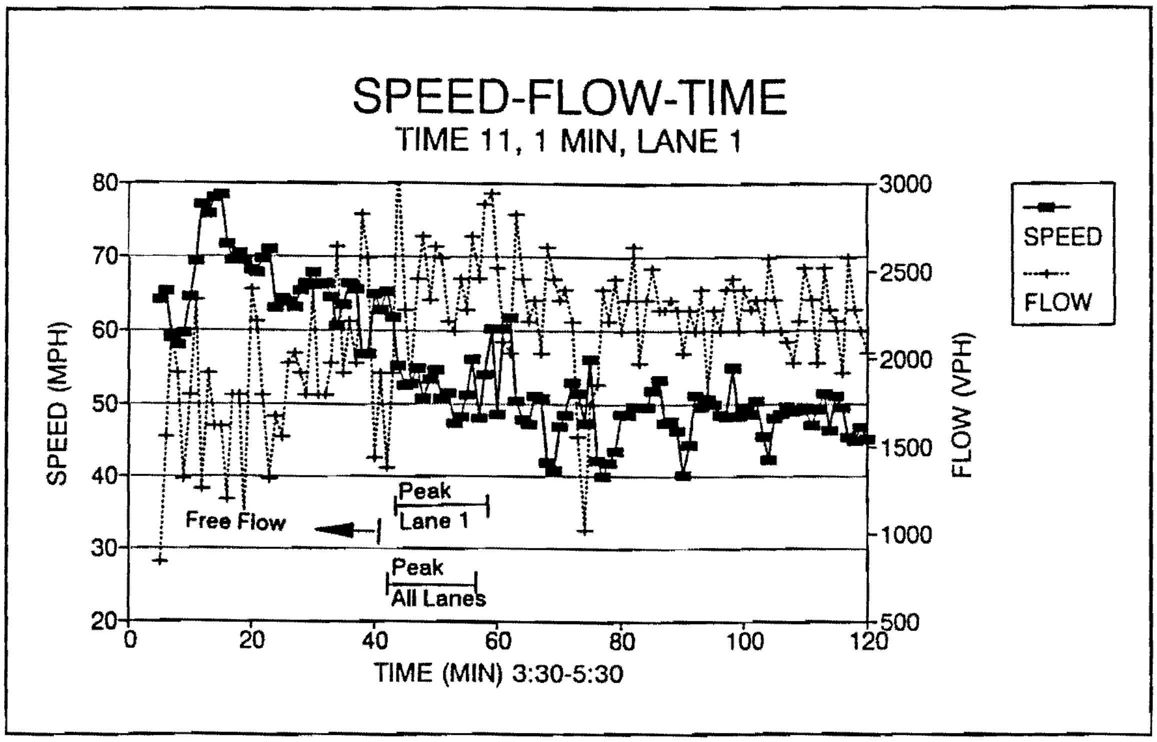

FIGURE A-31 Speed and flow vs time at US 290 for time 11 lane 1 using 1-minute intervals.

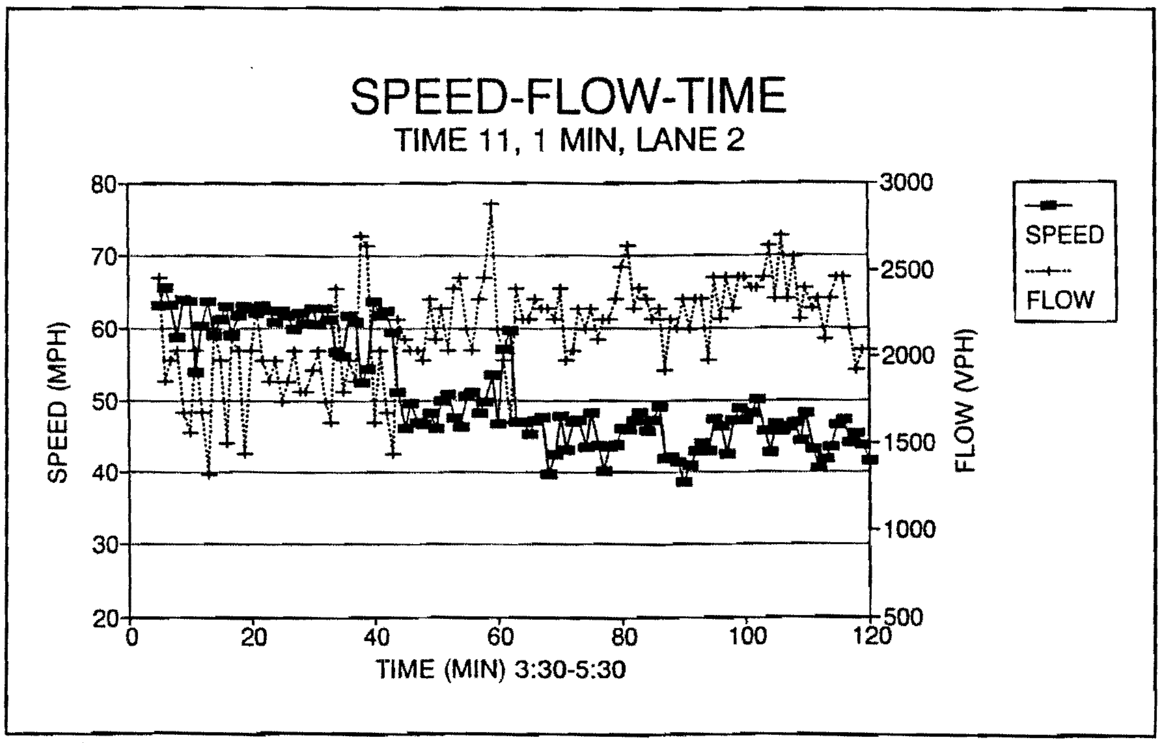

FIGURE A-32 Speed and flow vs time at US 290 for time 11 lane 2 using 1-minute intervals.

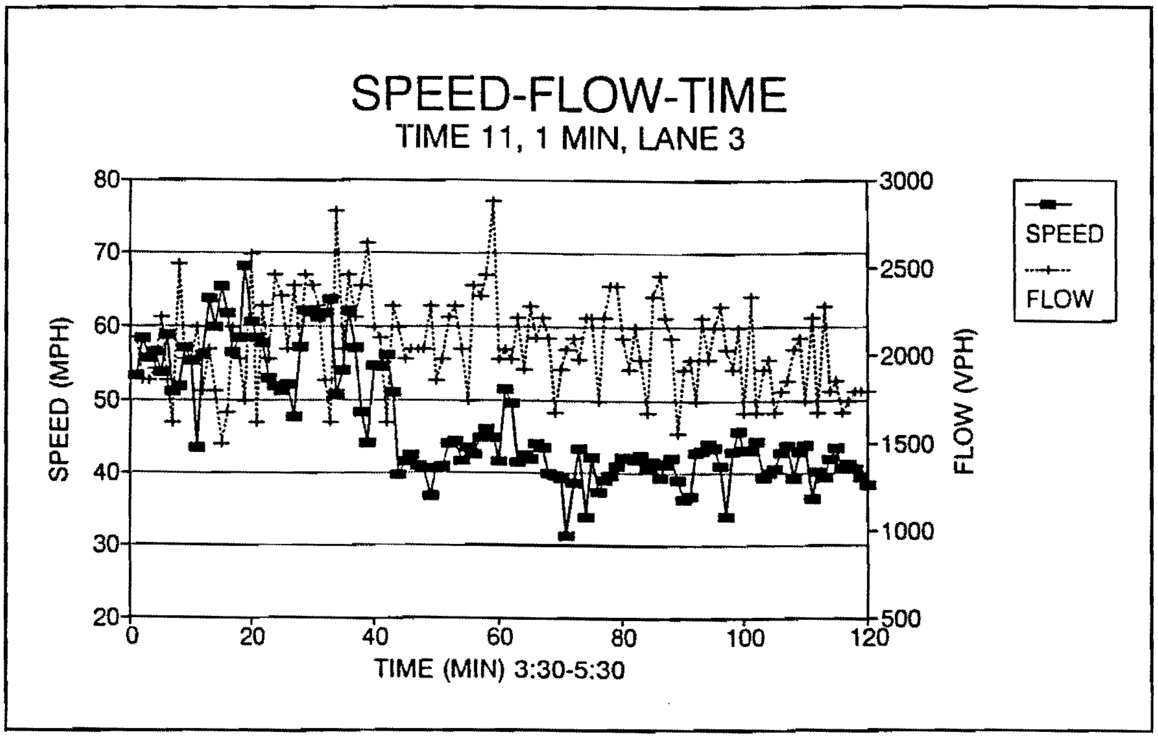

FIGURE A-33 Speed and flow vs time at US 290 for time 11 lane 3 using 1-minute intervals.

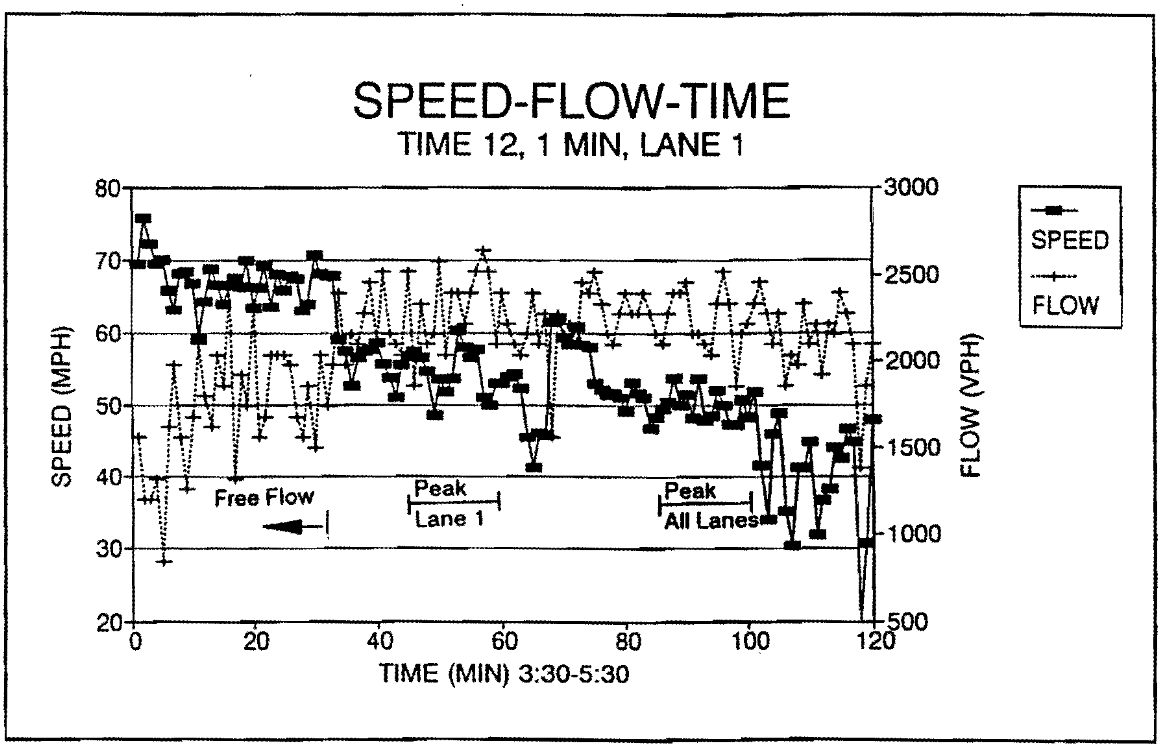

FIGURE A-34 Speed and flow vs time at US 290 for time 12 lane 1 using 1-minute intervals.

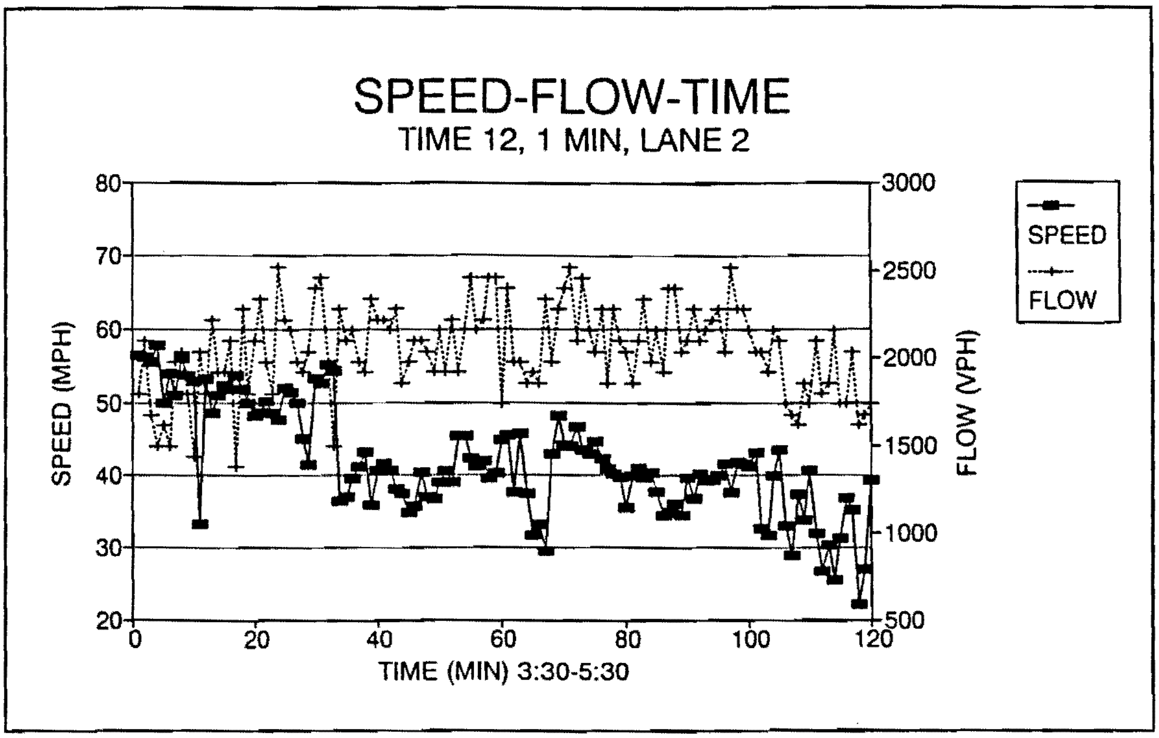

FIGURE A-35 Speed and flow vs time at US 290 for time 12 lane 2 using 1-minute intervals.

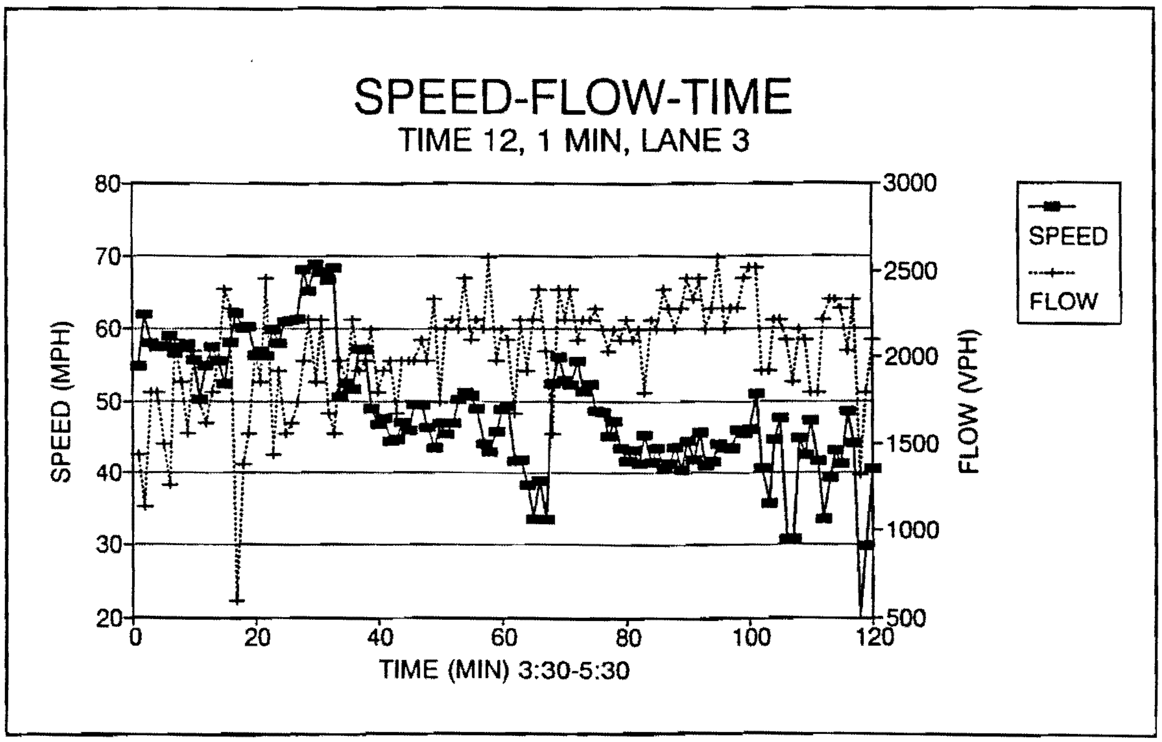

FIGURE A-36 Speed and flow vs time at US 290 for time 12 lane 3 using 1-minute intervals.

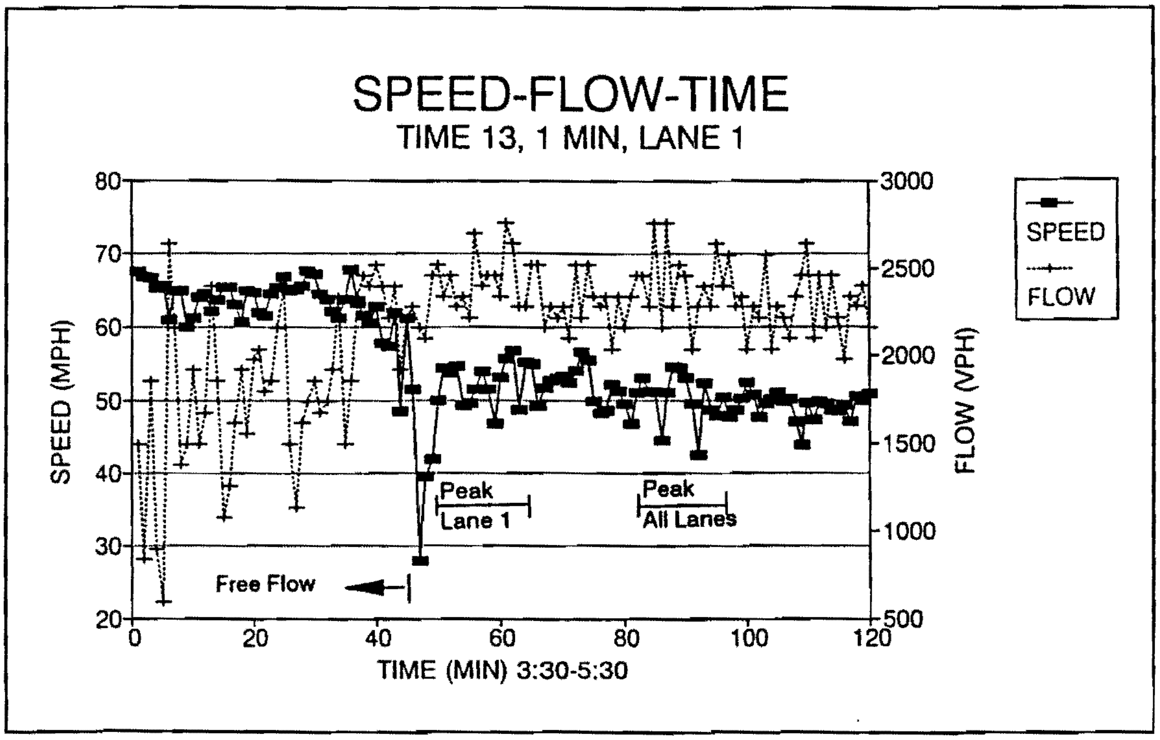

FIGURE A-37 Speed and flow vs time at US 290 for time 13 lane 1 using 1-minute intervals.

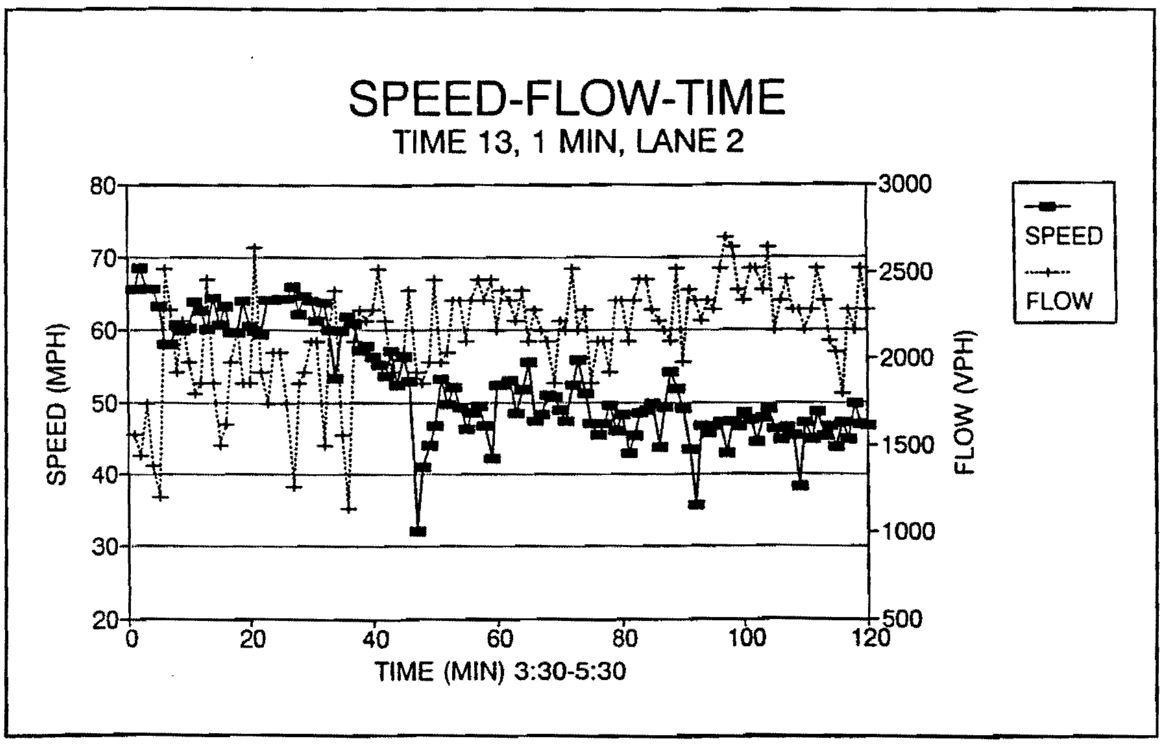

FIGURE A-38 Speed and flow vs time at US 290 for time 13 lane 2 using 1-minute intervals.

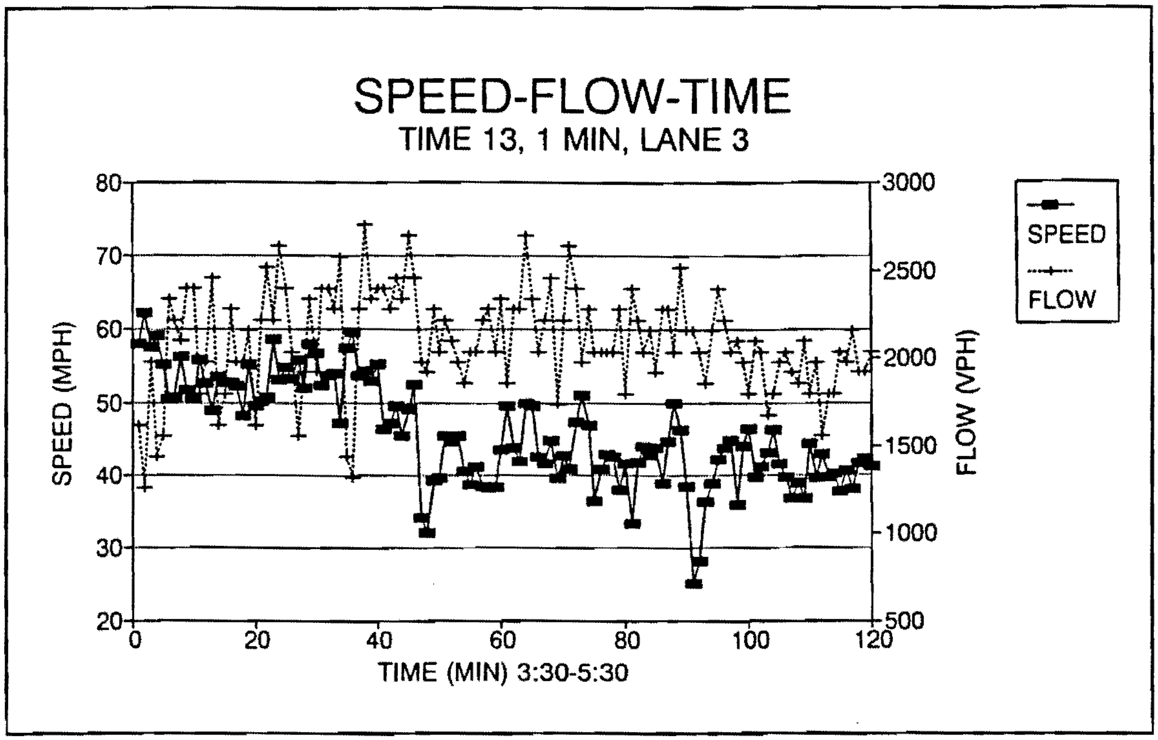

FIGURE A-39 Speed and flow vs time at US 290 for time 13 lane 3 using 1-minute intervals.

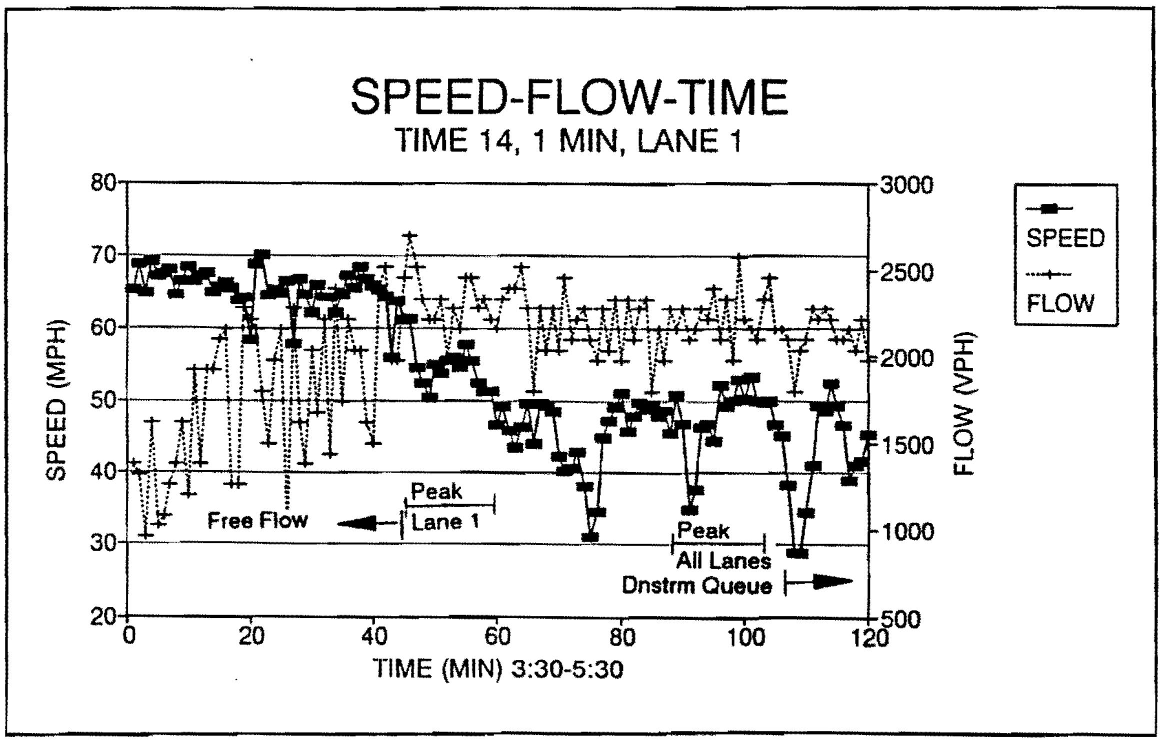

FIGURE A-40 Speed and flow vs time at US 290 for time 14 lane 1 using 1-minute intervals.

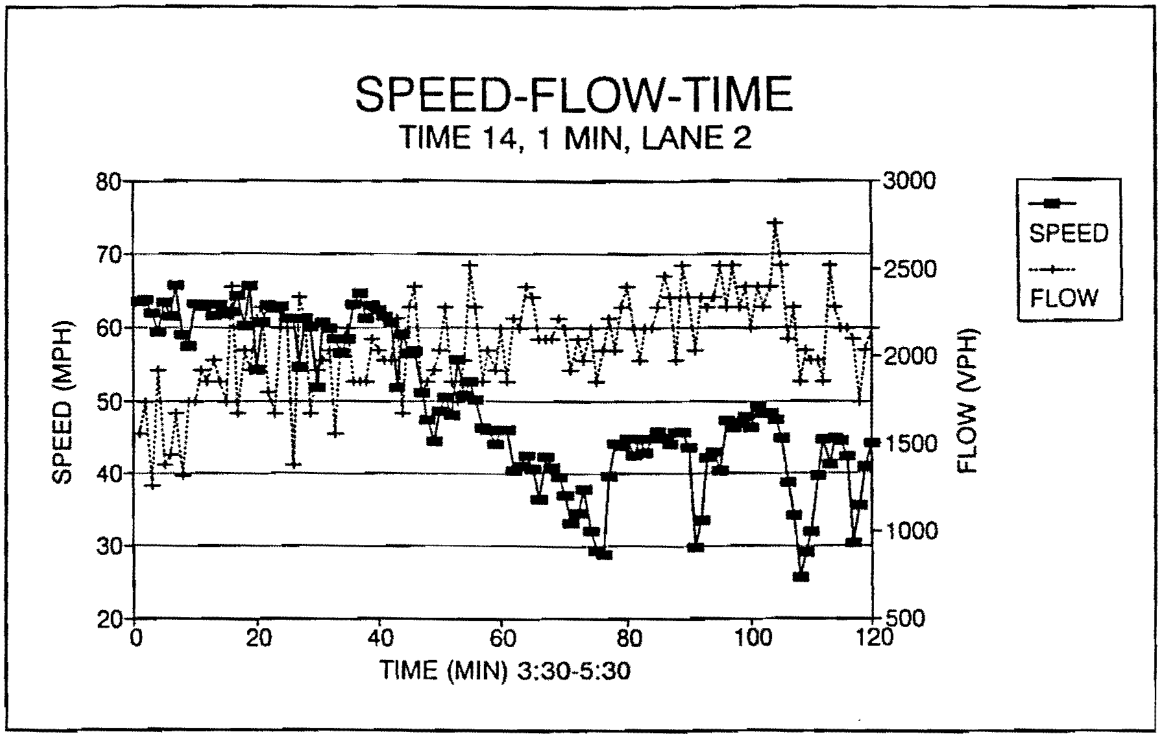

FIGURE A-41 Speed and flow vs time at US 290 for time 14 lane 2 using 1-minute intervals.

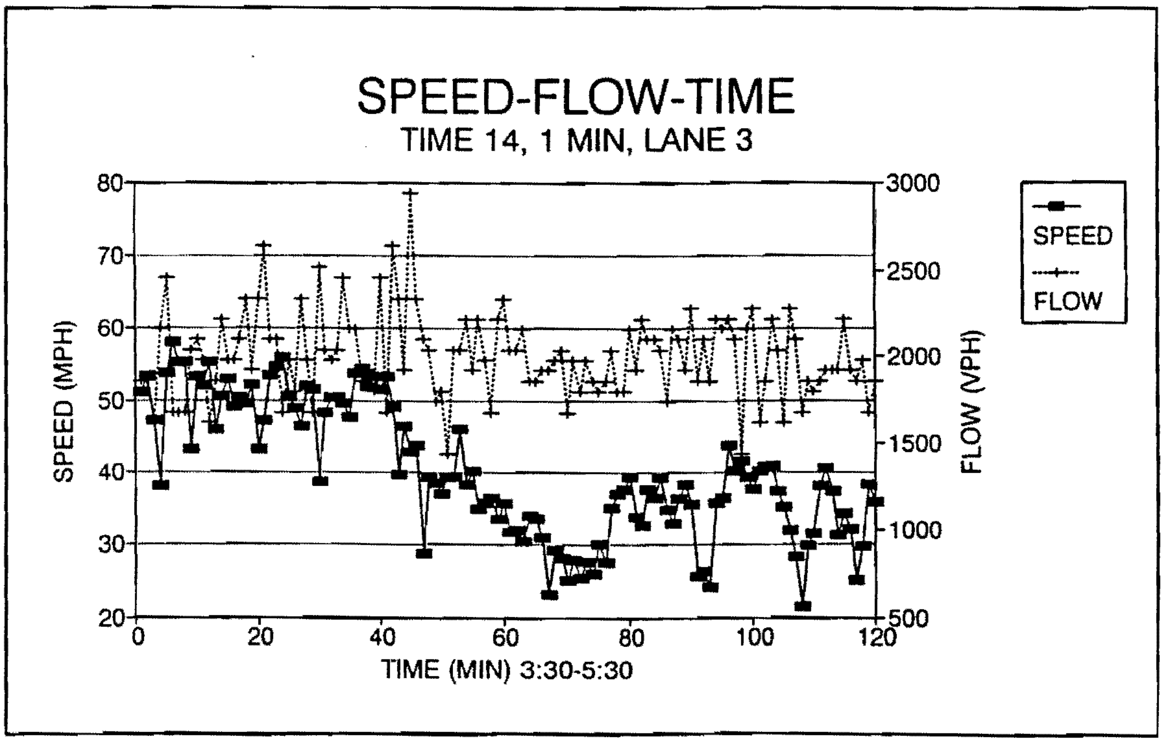

FIGURE A-42 Speed and flow vs time at US 290 for time 14 lane 3 using 1-minute intervals.

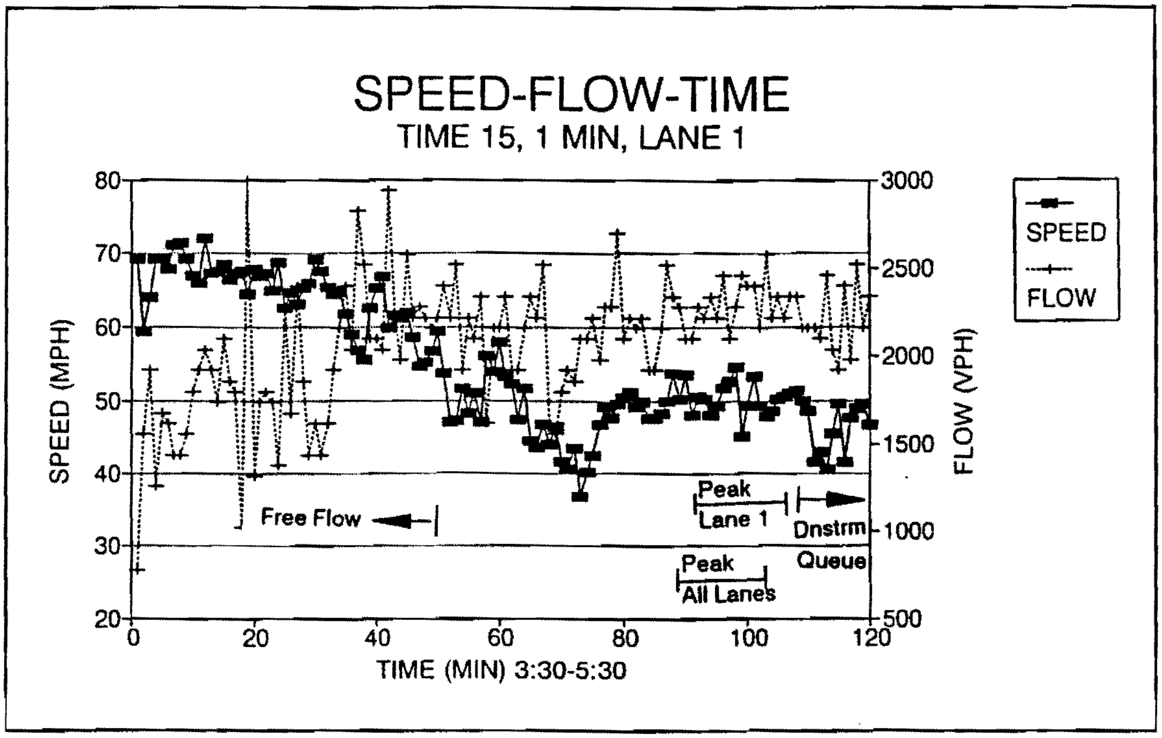

FIGURE A-43 Speed and flow vs time at US 290 for time 15 lane 1 using 1-minute intervals.

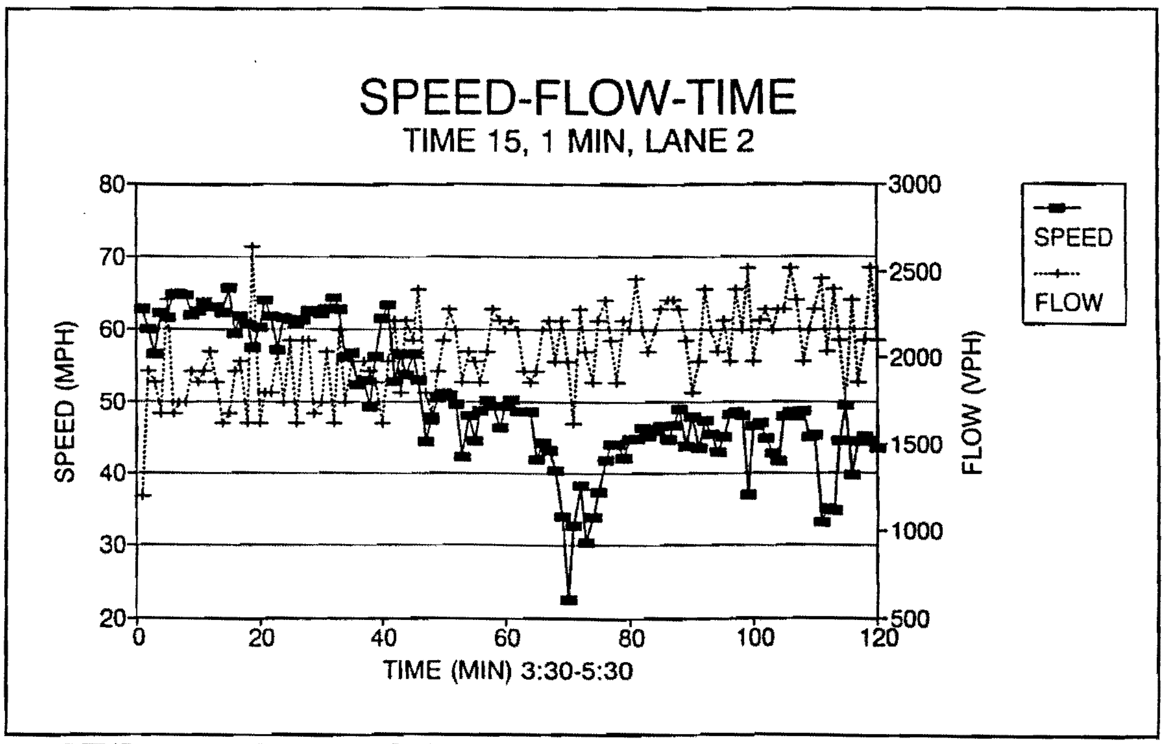

FIGURE A-44 Speed and flow vs time at US 290 for time 15 lane 2 using 1-minute intervals.

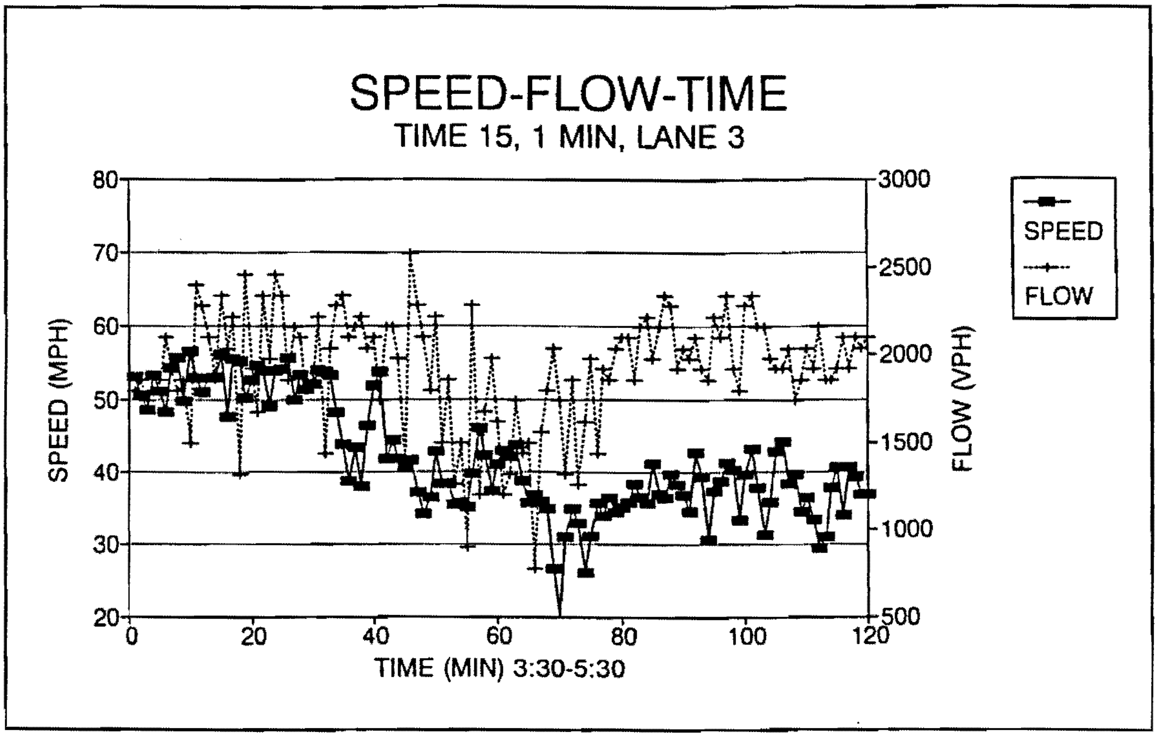

FIGURE A-45 Speed and flow vs time at US 290 for time 15 lane 3 using 1-minute intervals.

APPENDIX B

U.S. 290 SPEED-FLOW PLOTS

- Lane 1 Time Traced Plots
- All Lanes

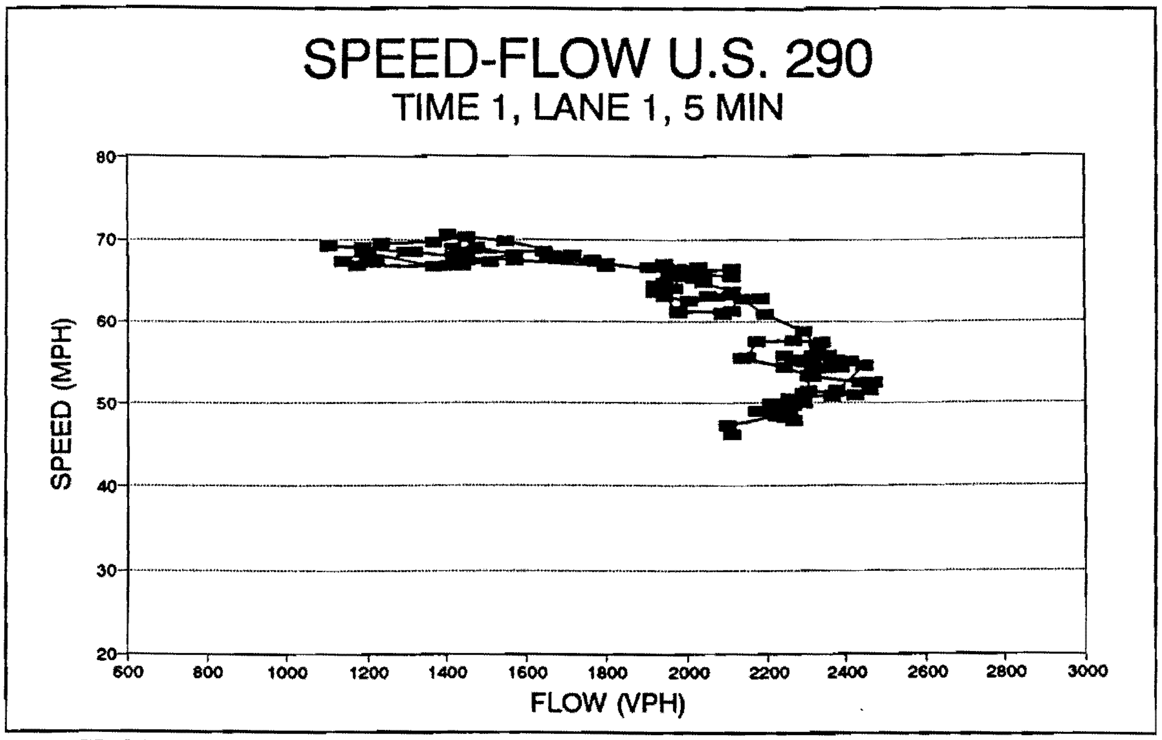

FIGURE B-1 Speed vs flow for lane 1, time 1, at US 290 using 5-minute moving averages.

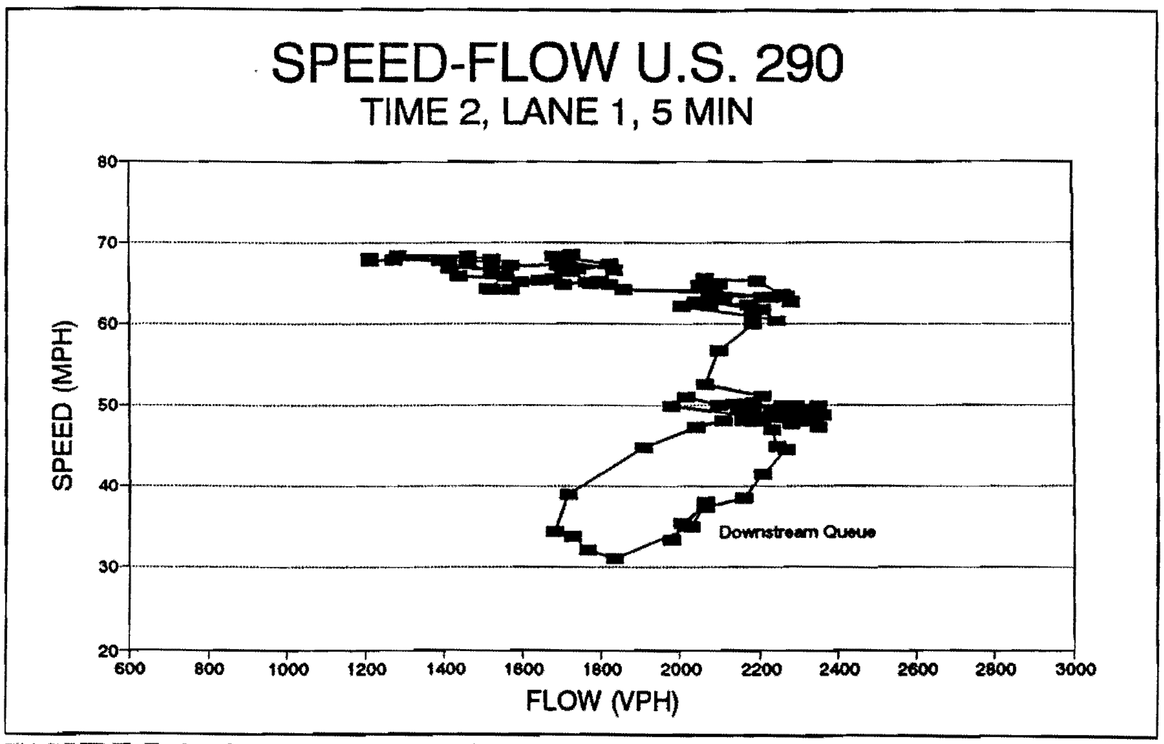

FIGURE B-2 Speed vs flow for lane 1, time 2, at US 290 using 5-minute moving averages.

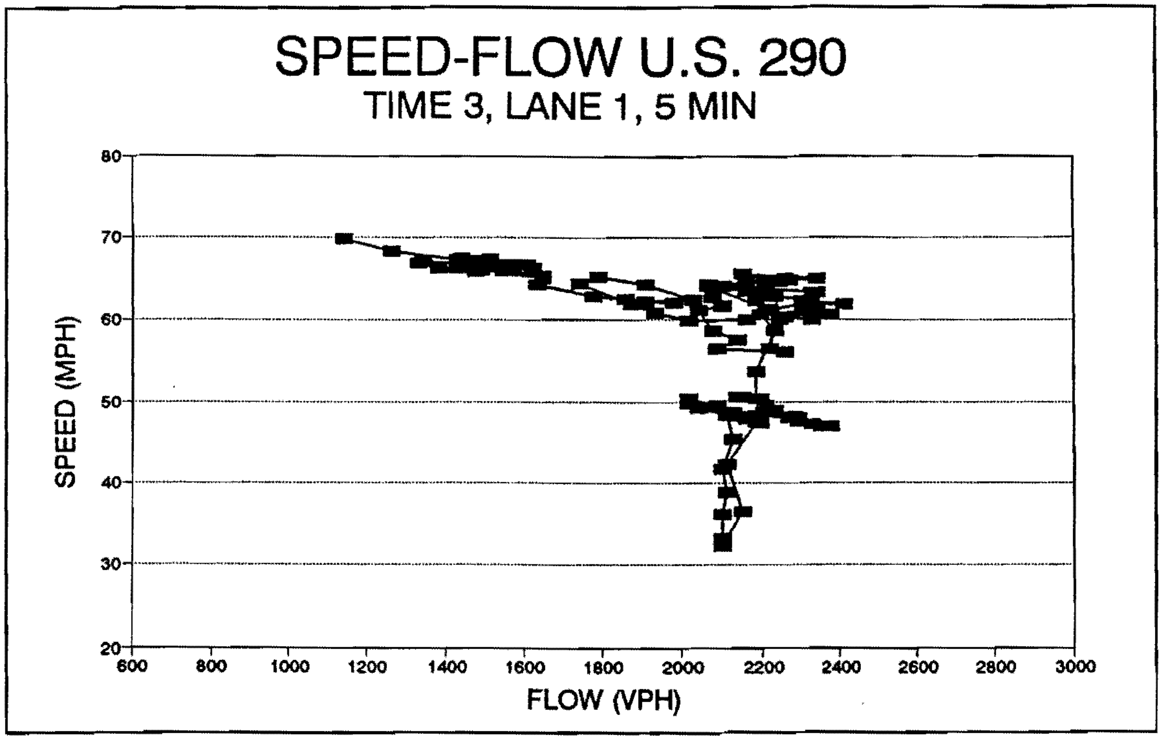

FIGURE B-3 Speed vs flow for lane 1, time 3, at US 290 using 5-minute moving averages.

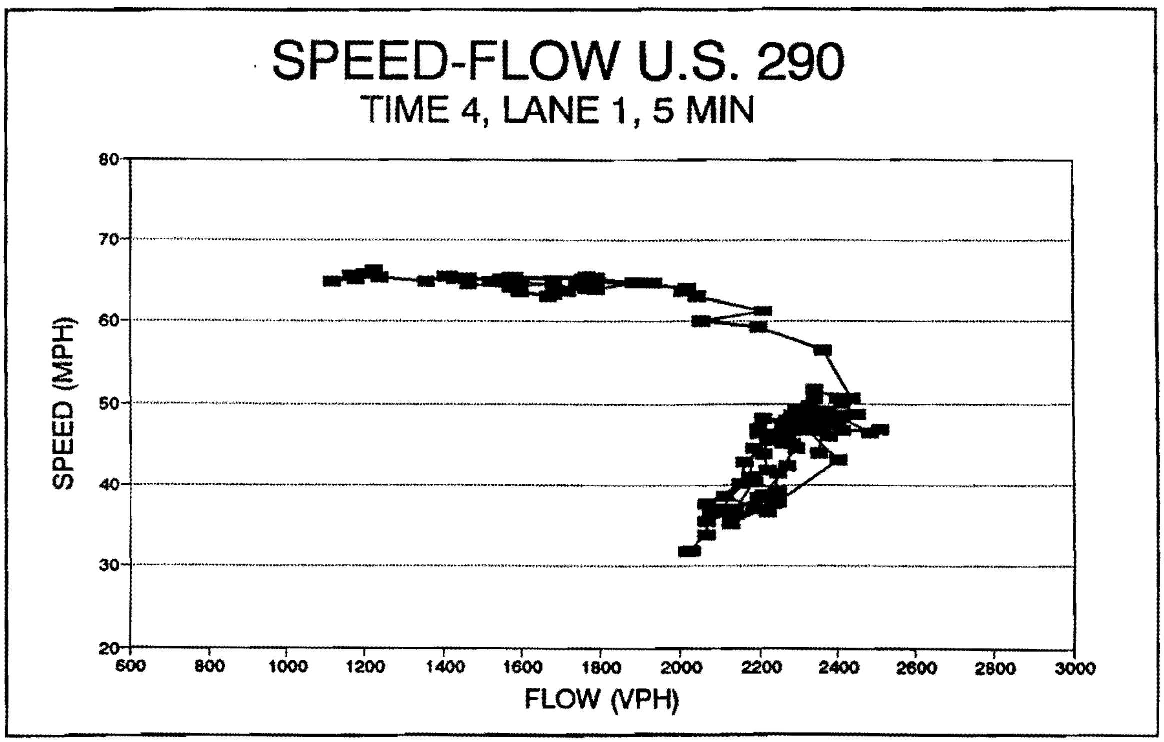

FIGURE B-4 Speed vs flow for lane 1, time 4, at US 290 using 5-minute moving averages.

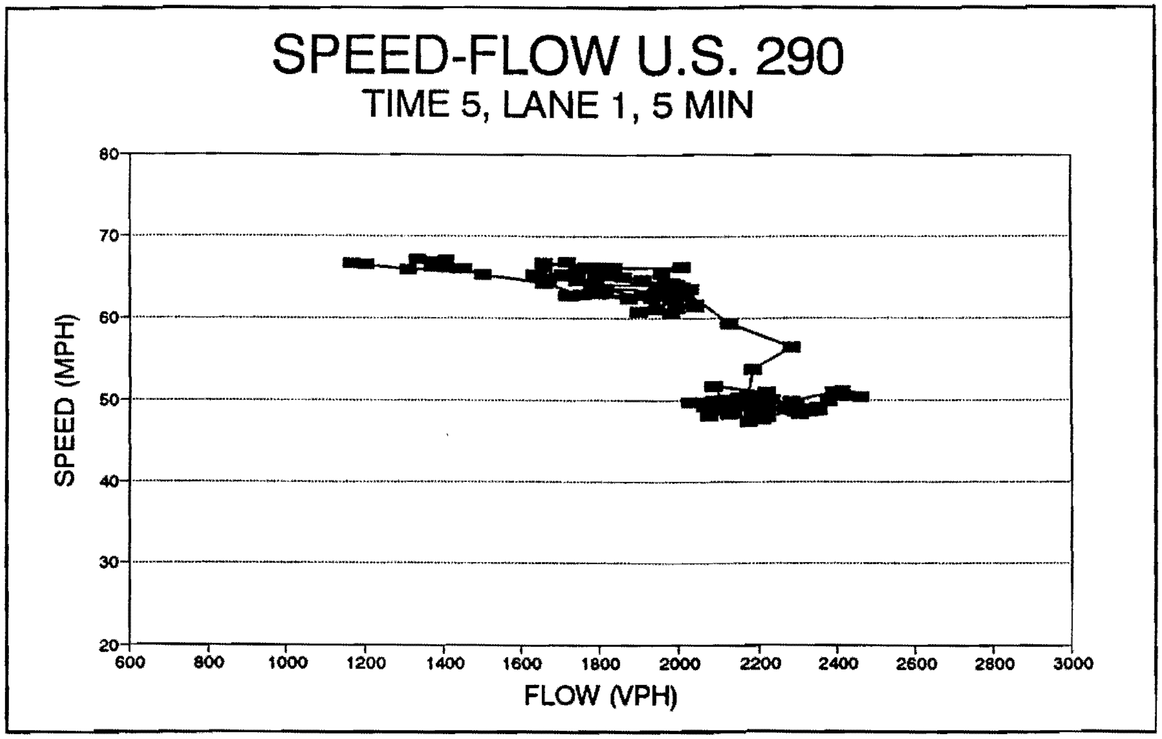

FIGURE B-5 **Speed vs flow for lane 1, time 5, at US 290 using 5-minute moving averages.**

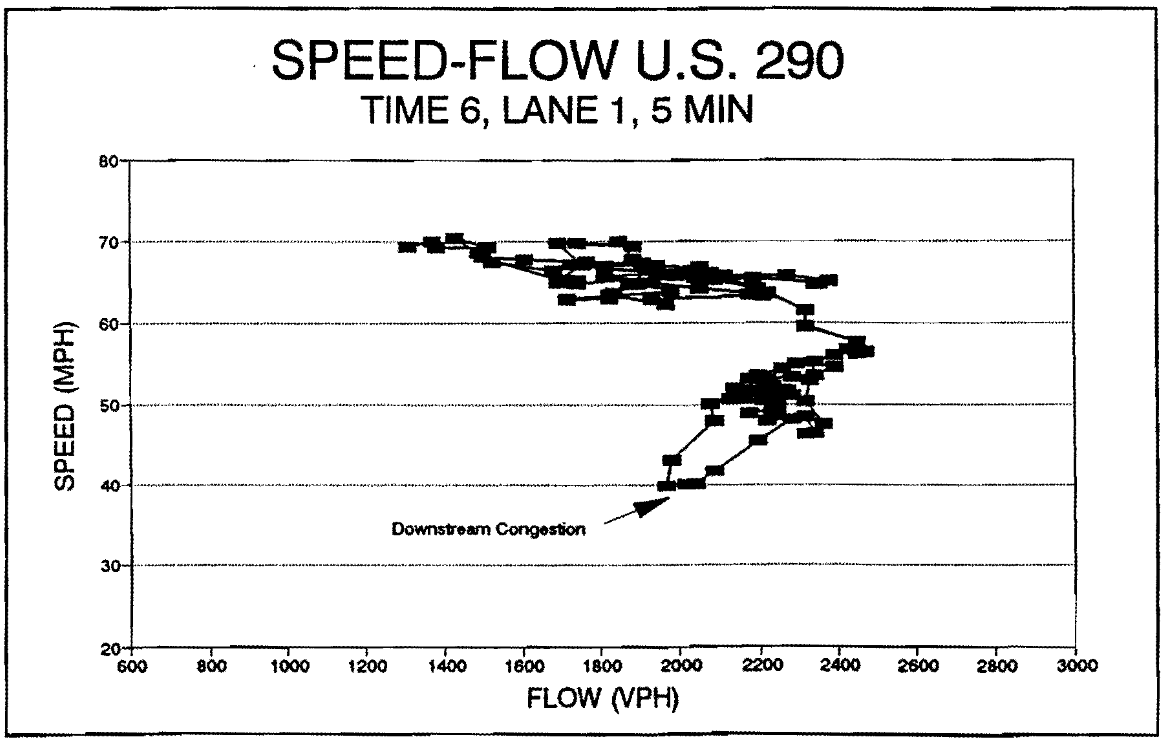

FIGURE B-6 **Speed vs flow for lane 1, time 6, at US 290 using 5-minute moving averages.**

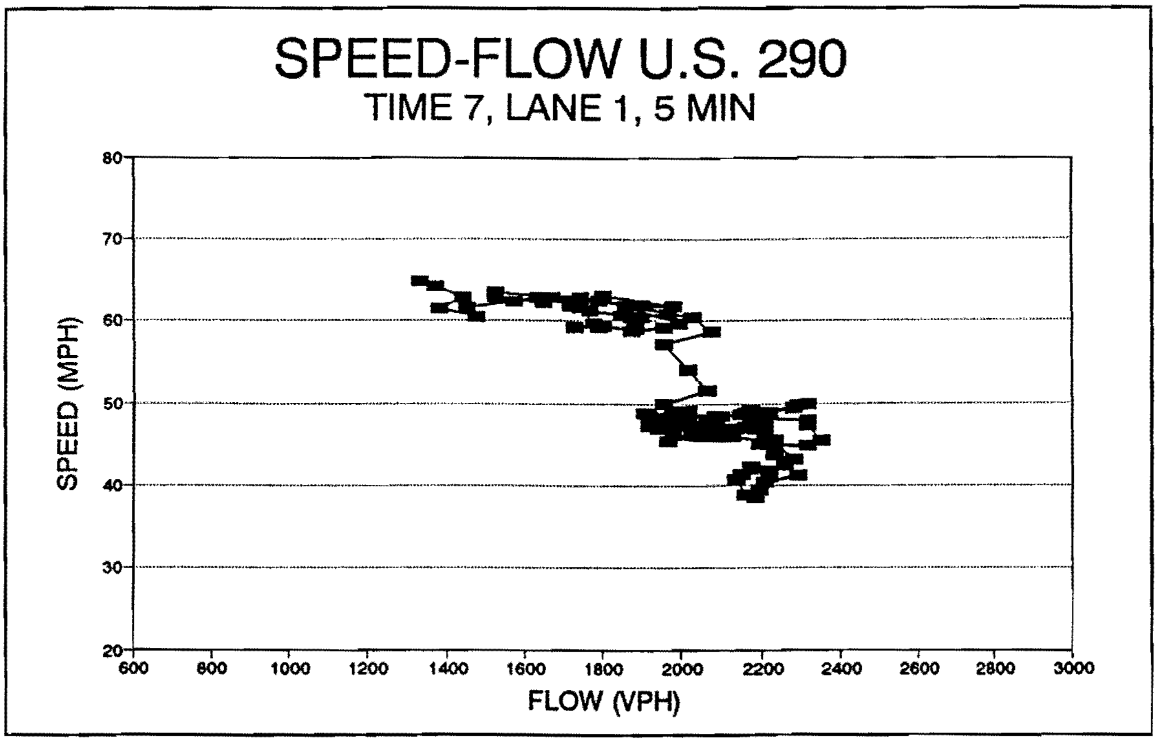

FIGURE B-7 Speed vs flow for lane 1, time 7, at US 290 using 5-minute moving averages.

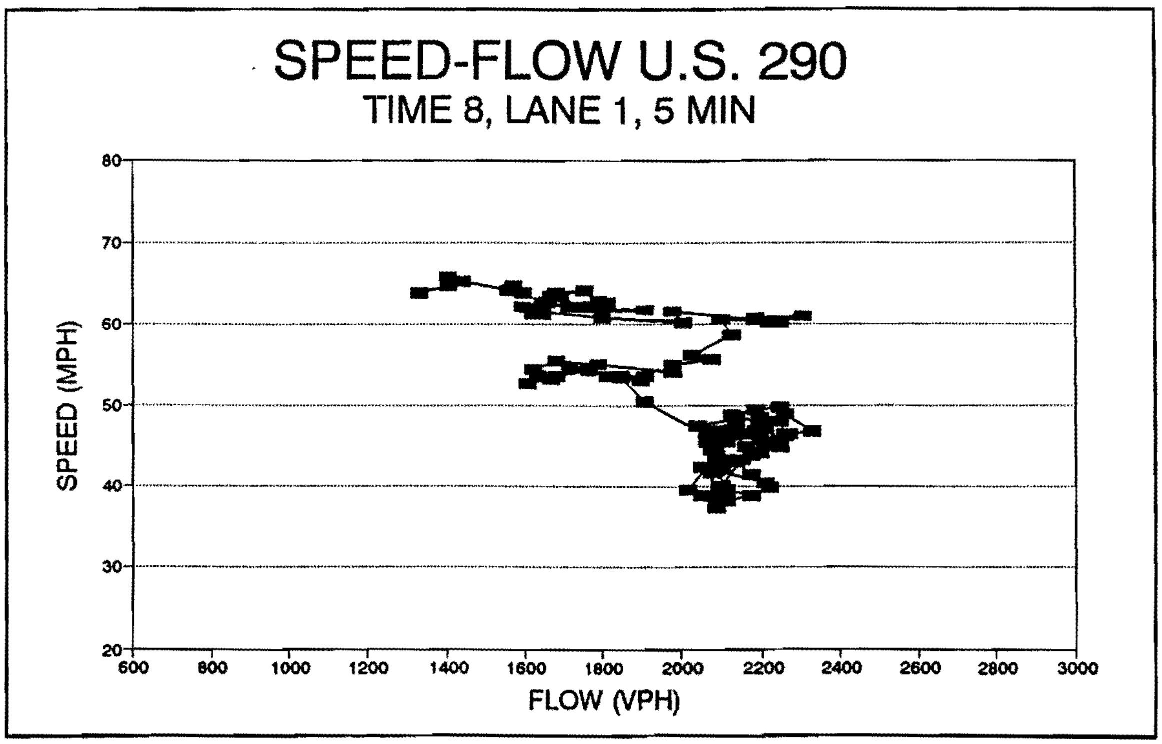

FIGURE B-8 Speed vs flow for lane 1, time 8, at US 290 using 5-minute moving averages.

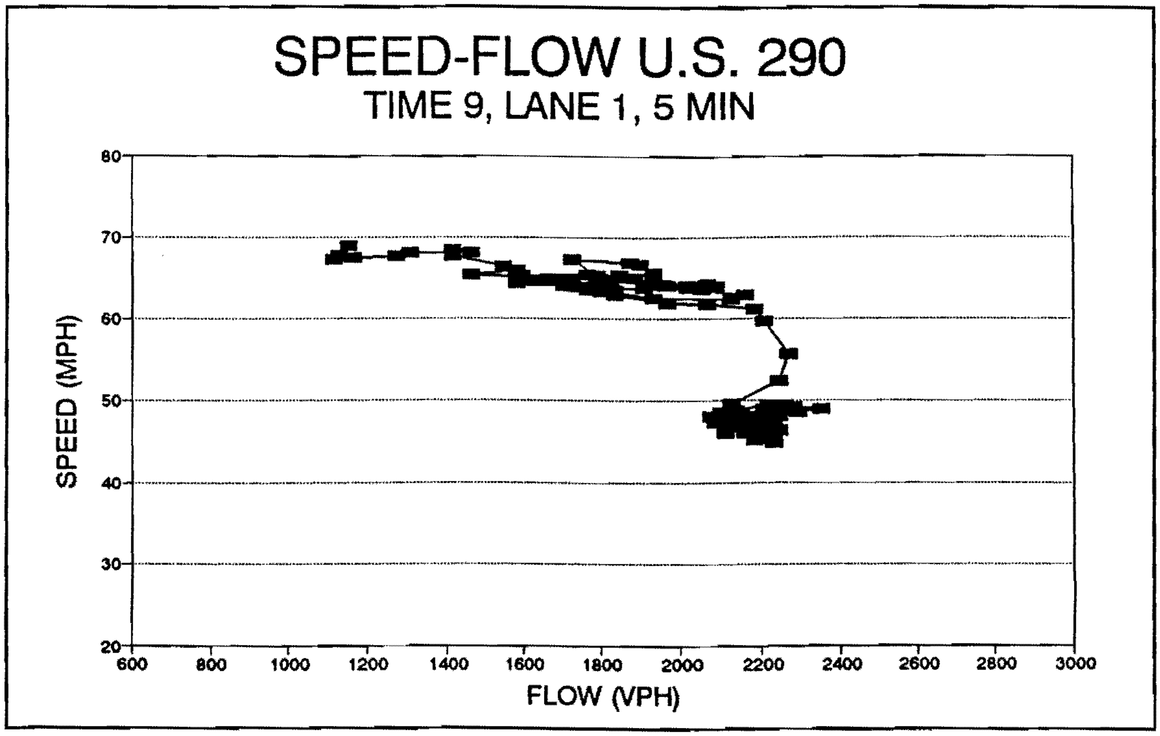

FIGURE B-9 Speed vs flow for lane 1, time 9, at US 290 using 5-minute moving averages.

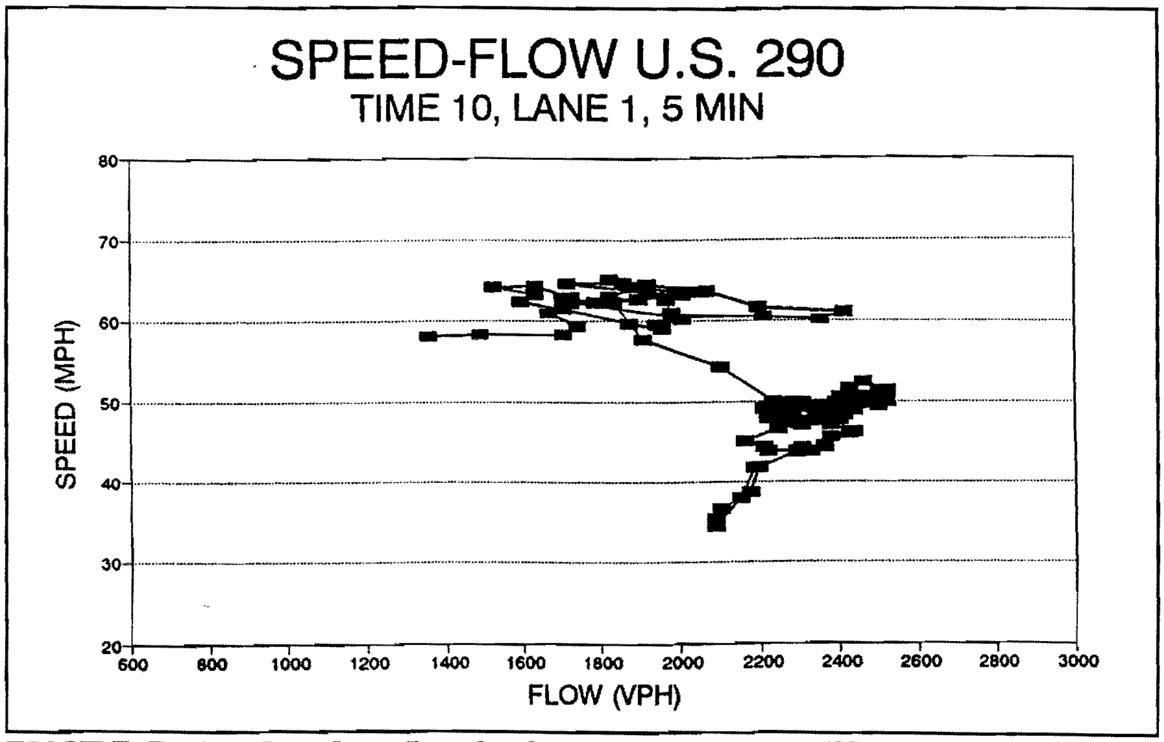

FIGURE B-10 Speed vs flow for lane 1, time 10, at US 290 using 5-minute moving averages.

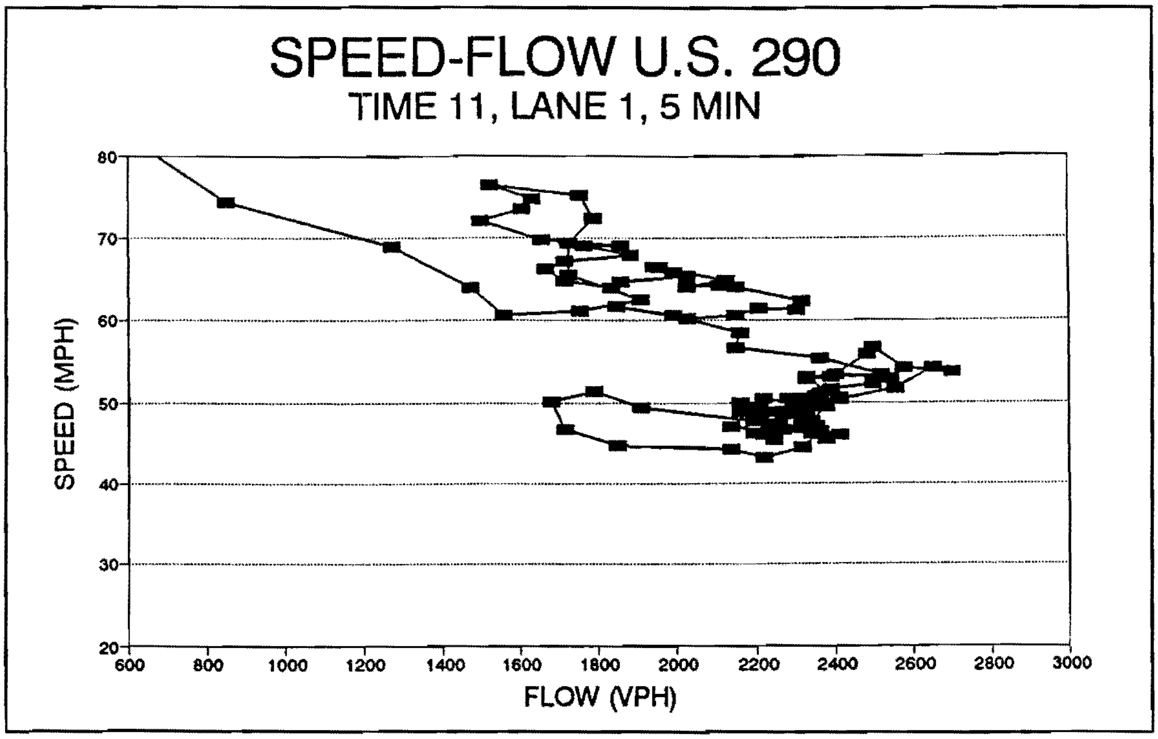

FIGURE B-11 Speed vs flow for lane 1, time 11, at US 290 using 5-minute moving averages.

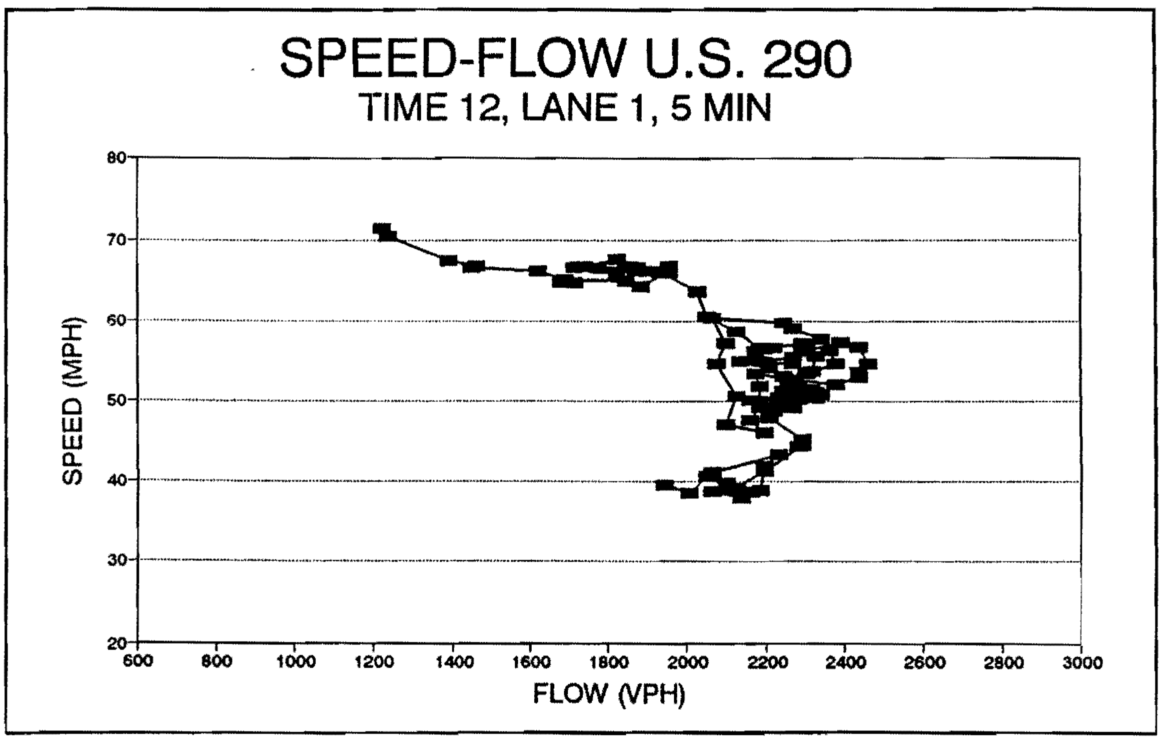

FIGURE B-12 Speed vs flow for lane 1, time 12, at US 290 using 5-minute moving averages.

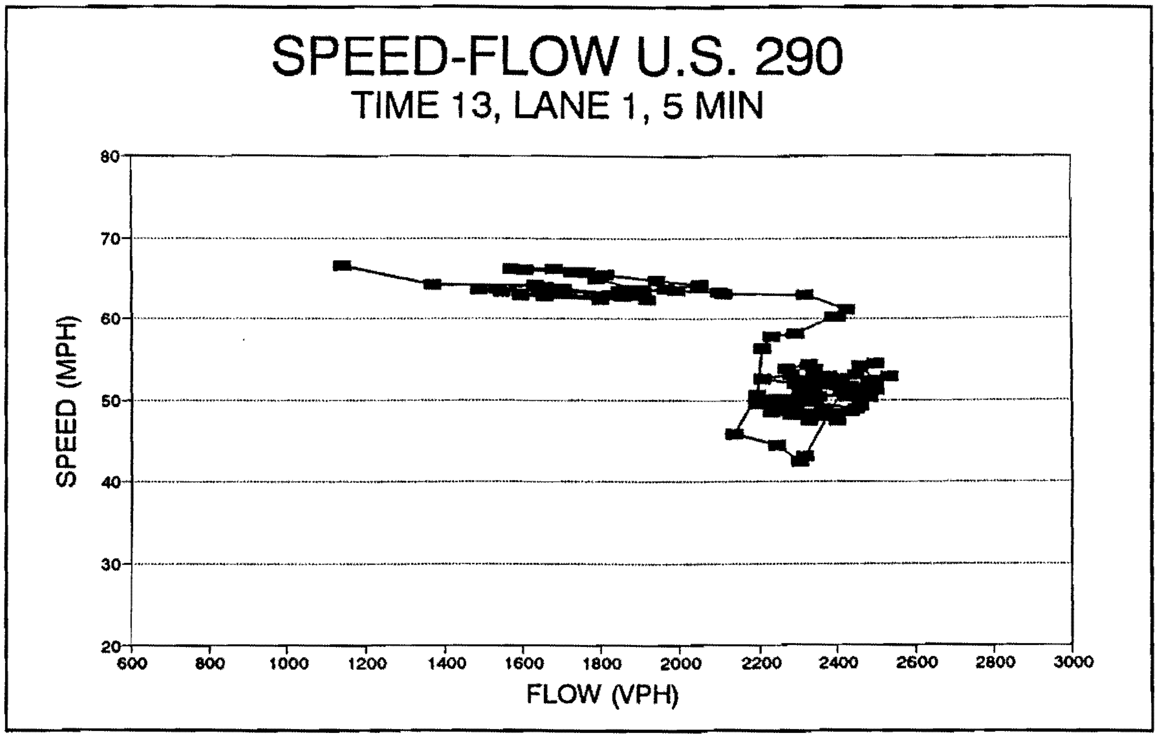

FIGURE B-13 Speed vs flow for lane 1, time 13, at US 290 using 5-minute moving averages.

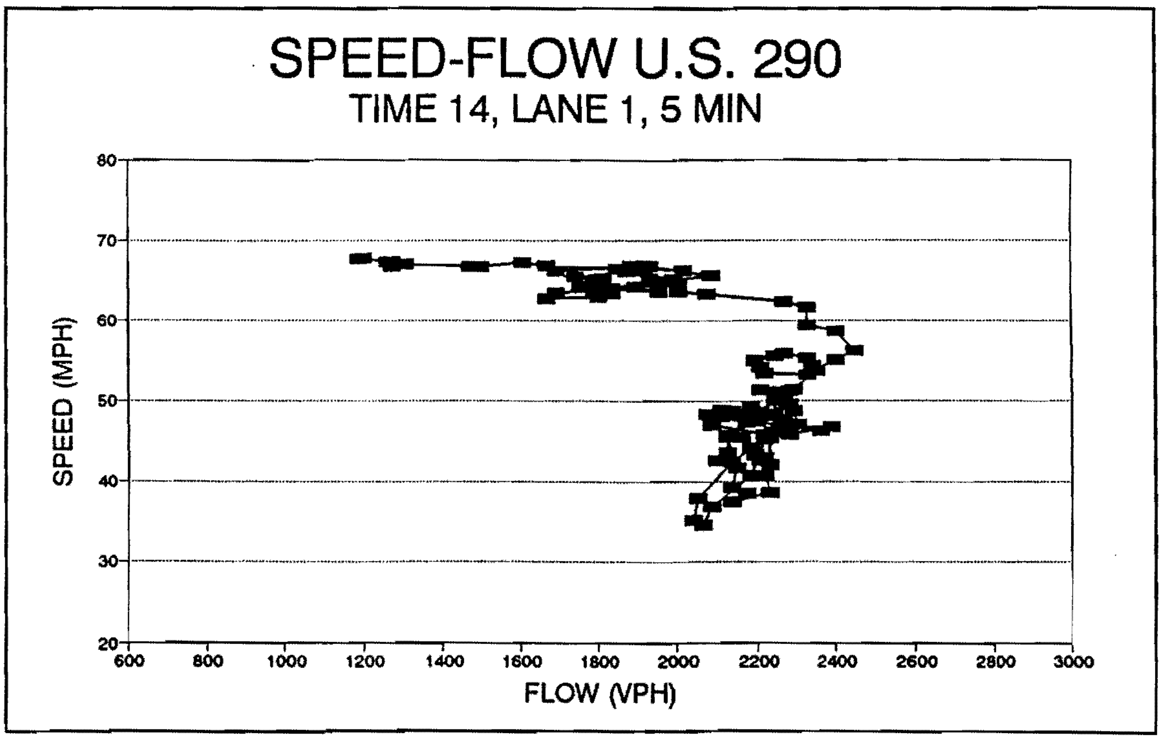

FIGURE B-14 Speed vs flow for lane 1, time 14, at US 290 using 5-minute moving averages.

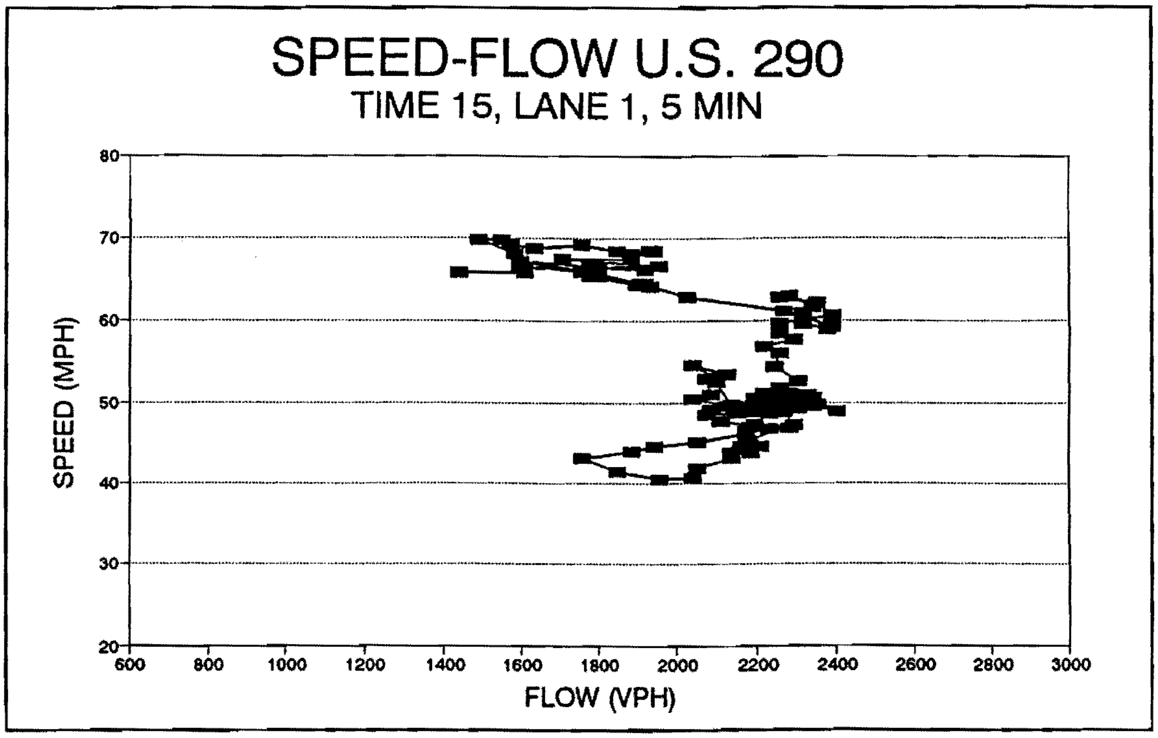

FIGURE B-15 Speed vs flow for lane 1, time 15, at US 290 using 5-minute moving averages.

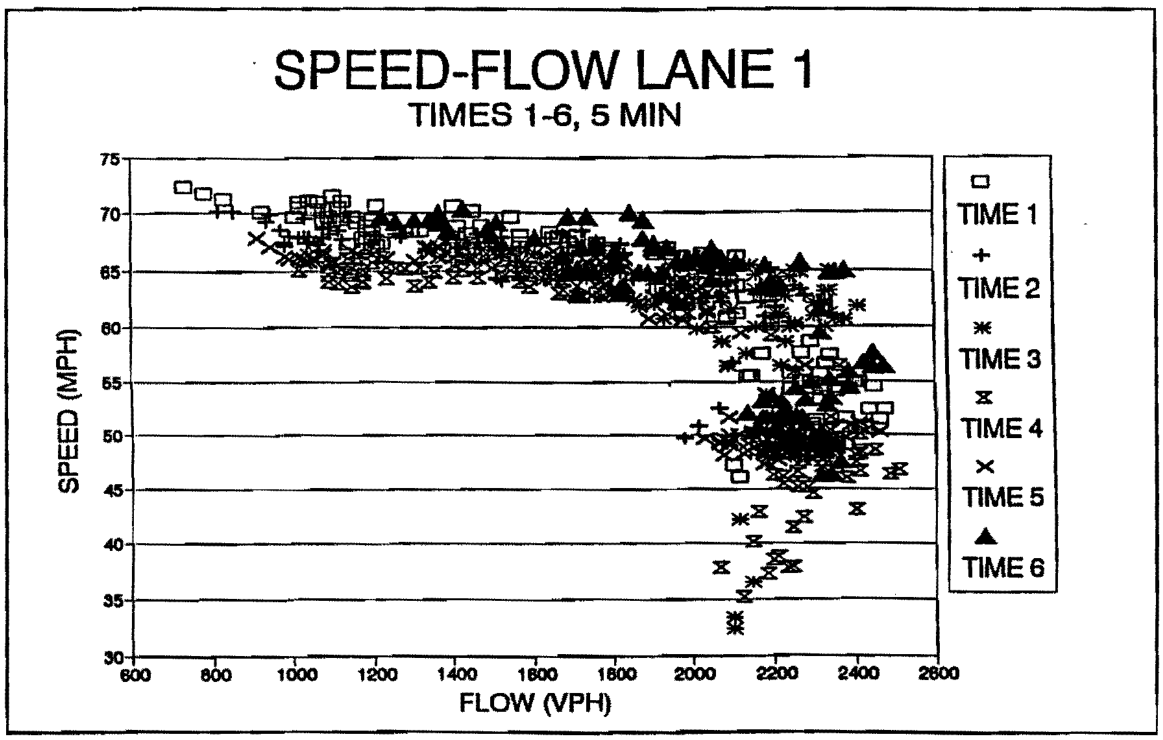

FIGURE B-16 Speed vs flow for lane 1 for times 1-6 at US 290 using 5-minute moving averages.

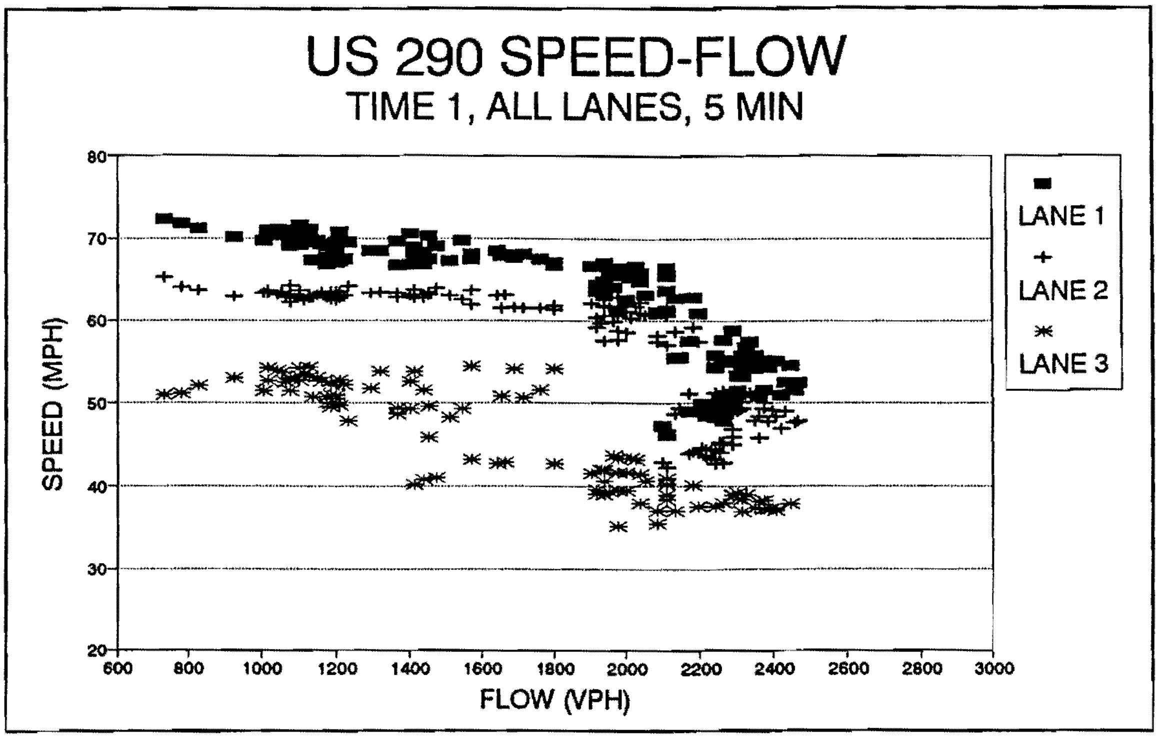

FIGURE B-17 Speed vs flow for all lanes, time 1, at US 290 using 5-minute moving averages.

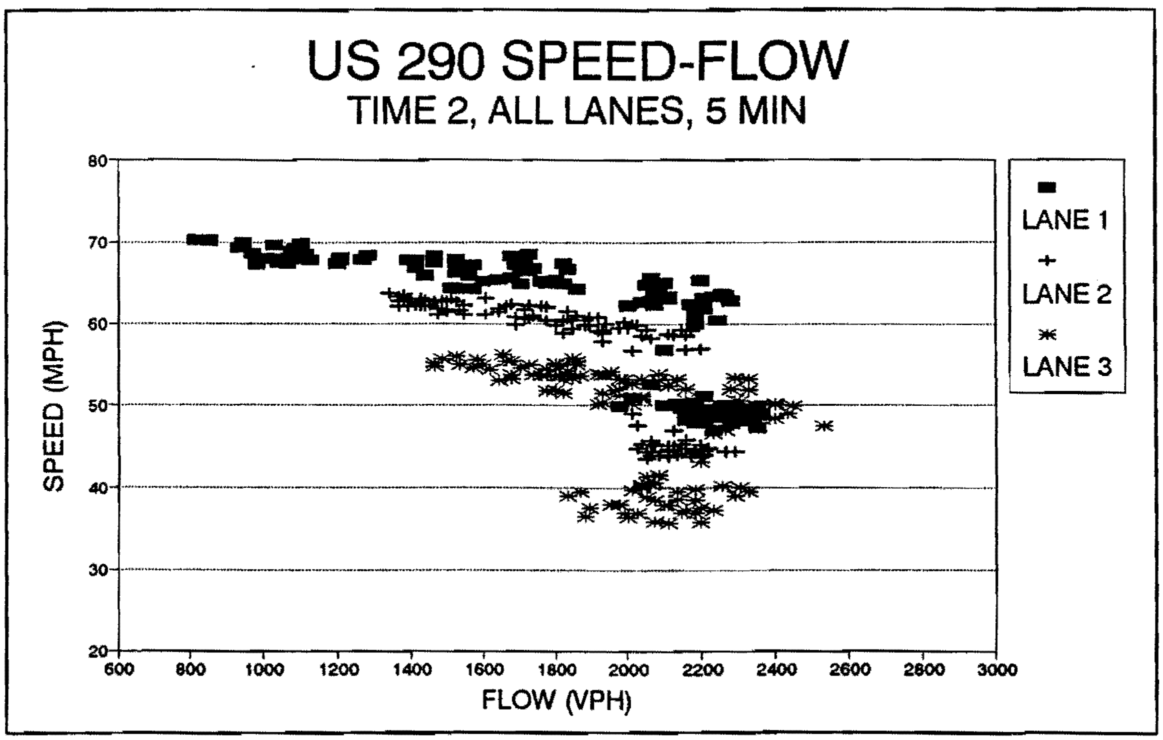

FIGURE B-18 Speed vs flow for all lanes, time 2, at US 290 using 5-minute moving averages.

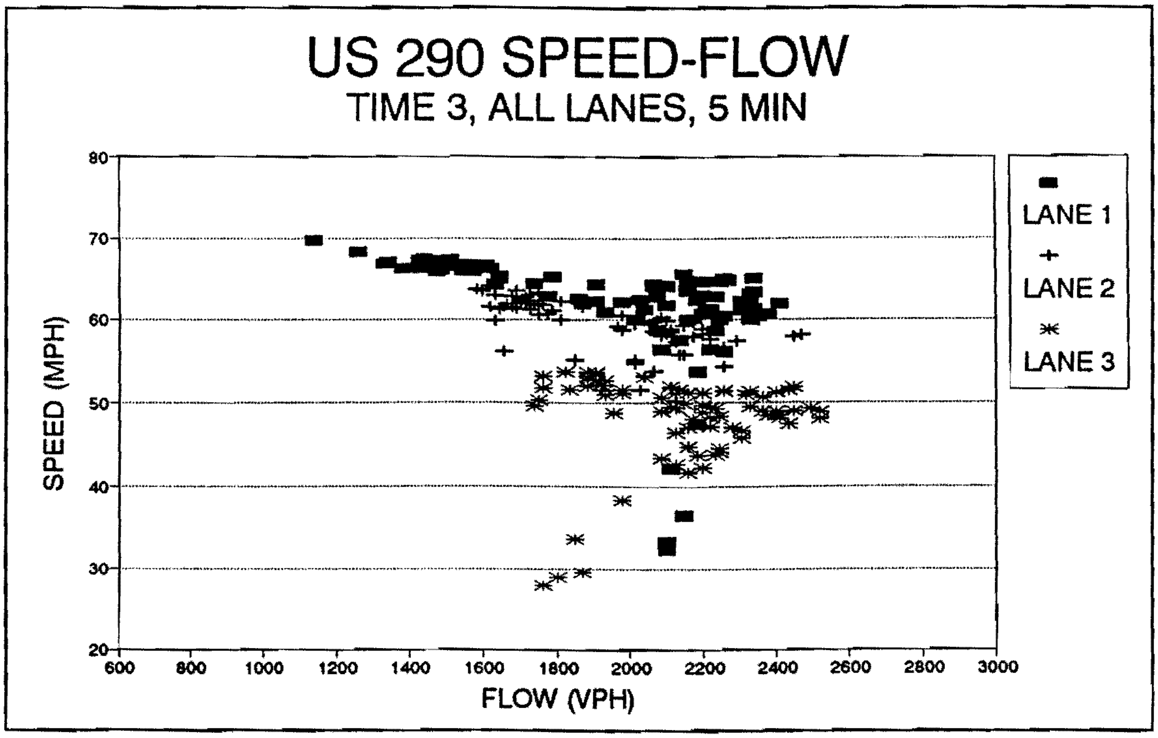

FIGURE B-19 Speed vs flow for all lanes, time 3, at US 290 using 5-minute moving averages.

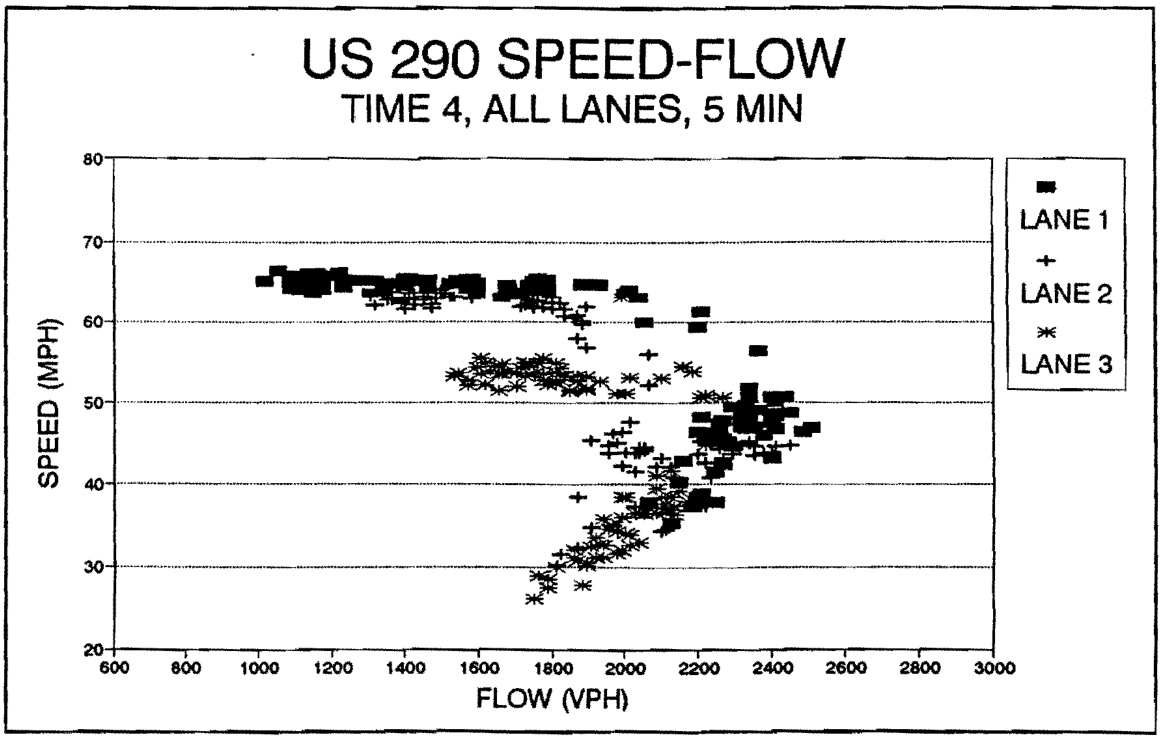

FIGURE B-20 Speed vs flow for all lanes, time 4, at US 290 using 5-minute moving averages.

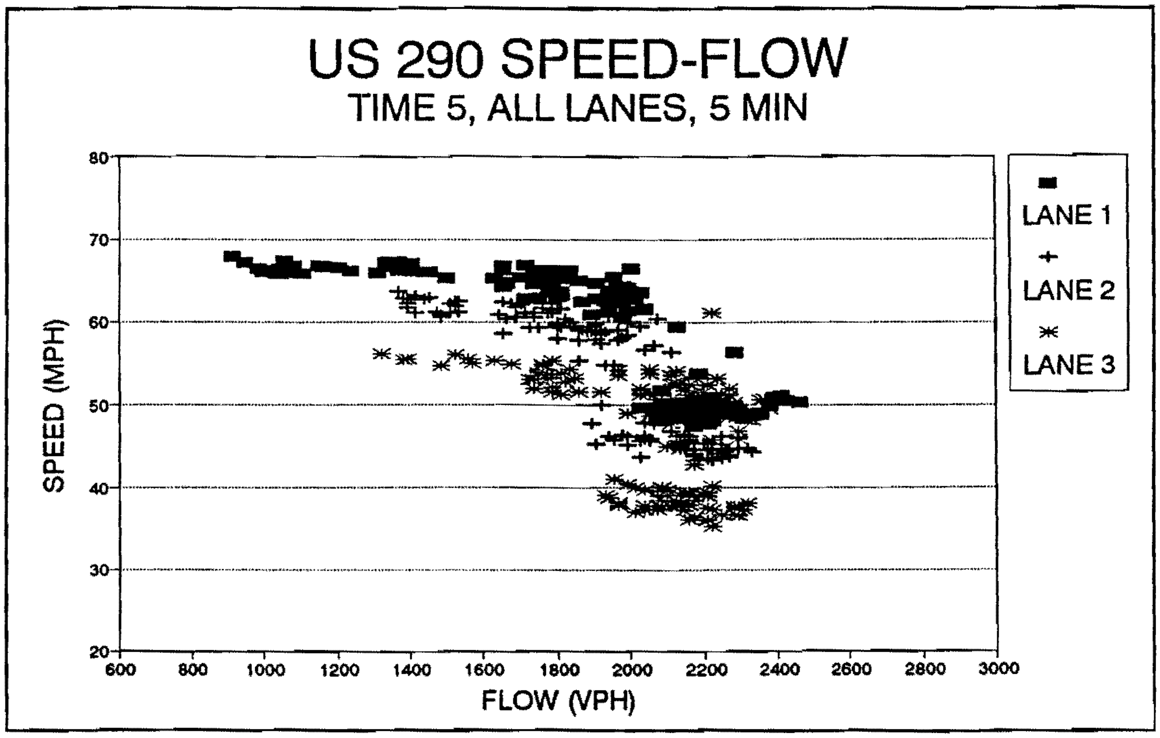

FIGURE B-21 Speed vs flow for all lanes, time 5, at US 290 using 5-minute moving averages.

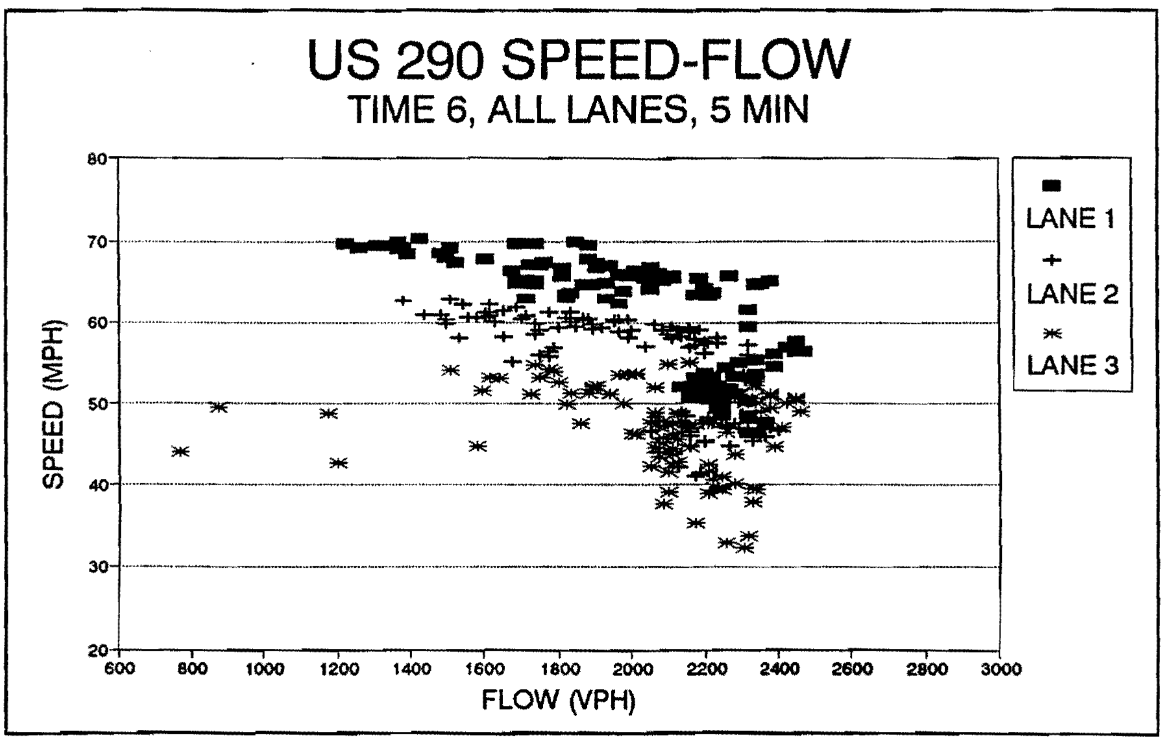

FIGURE B-22 Speed vs flow for all lanes, time 6, at US 290 using 5-minute moving averages.

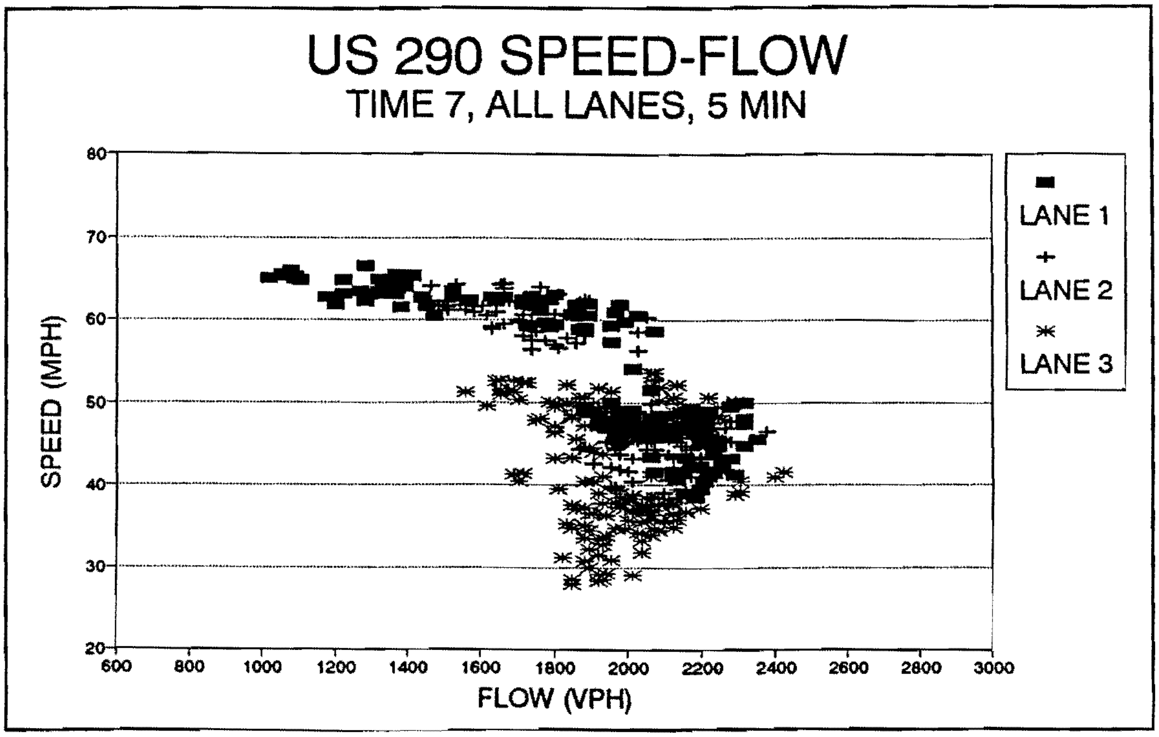

FIGURE B-23 Speed vs flow for all lanes, time 7, at US 290 using 5-minute moving averages.

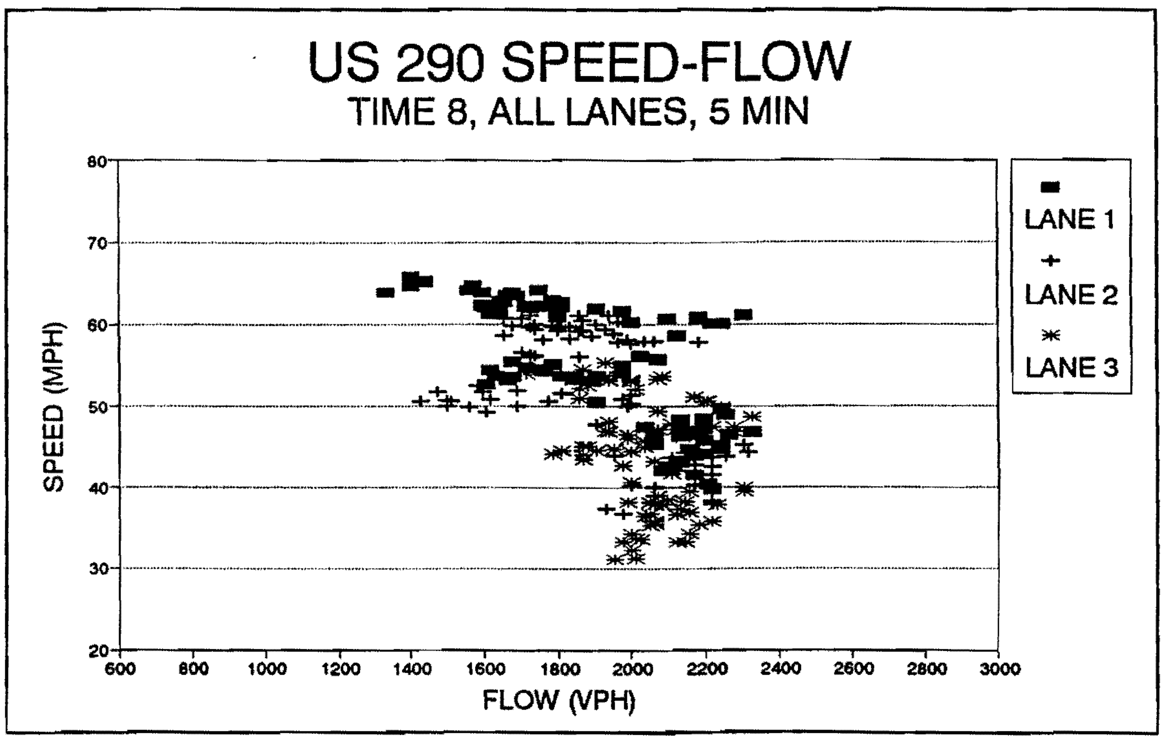

FIGURE B-24 Speed vs flow for all lanes, time 8, at US 290 using 5-minute moving averages.

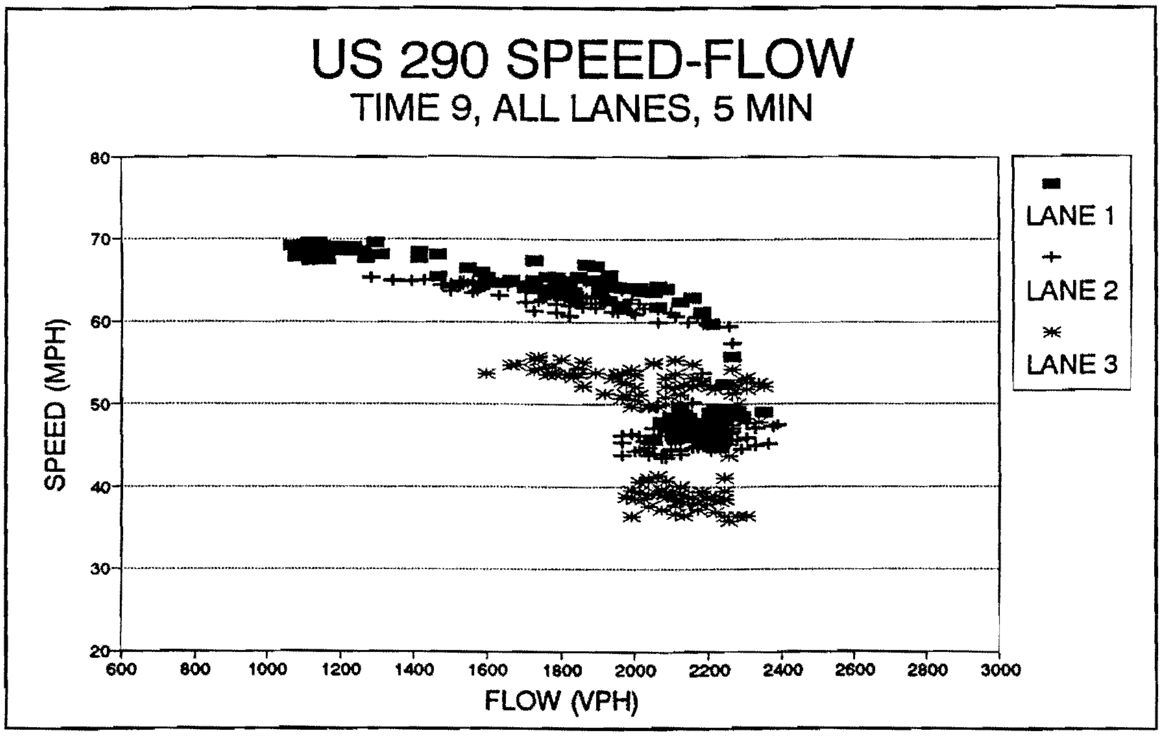

FIGURE B-25 Speed vs flow for all lanes, time 9, at US 290 using 5-minute moving averages.

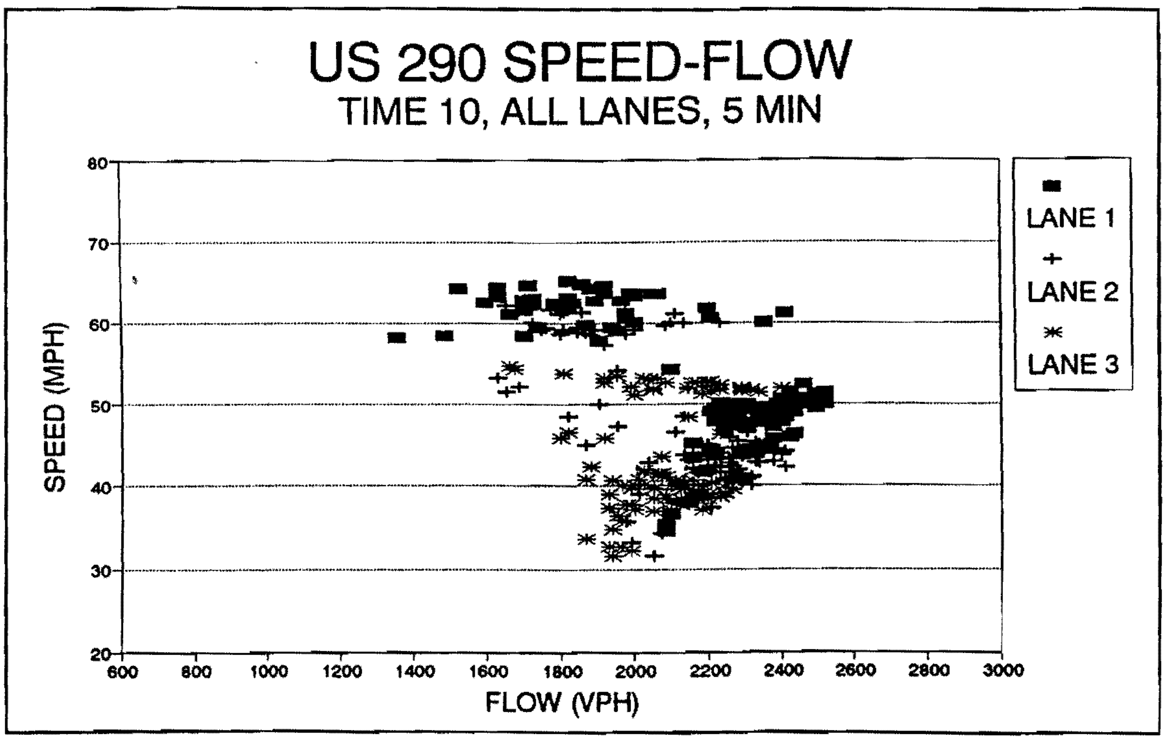

FIGURE B-26 Speed vs flow for all lanes, time 10, at US 290 using 5-minute moving averages.

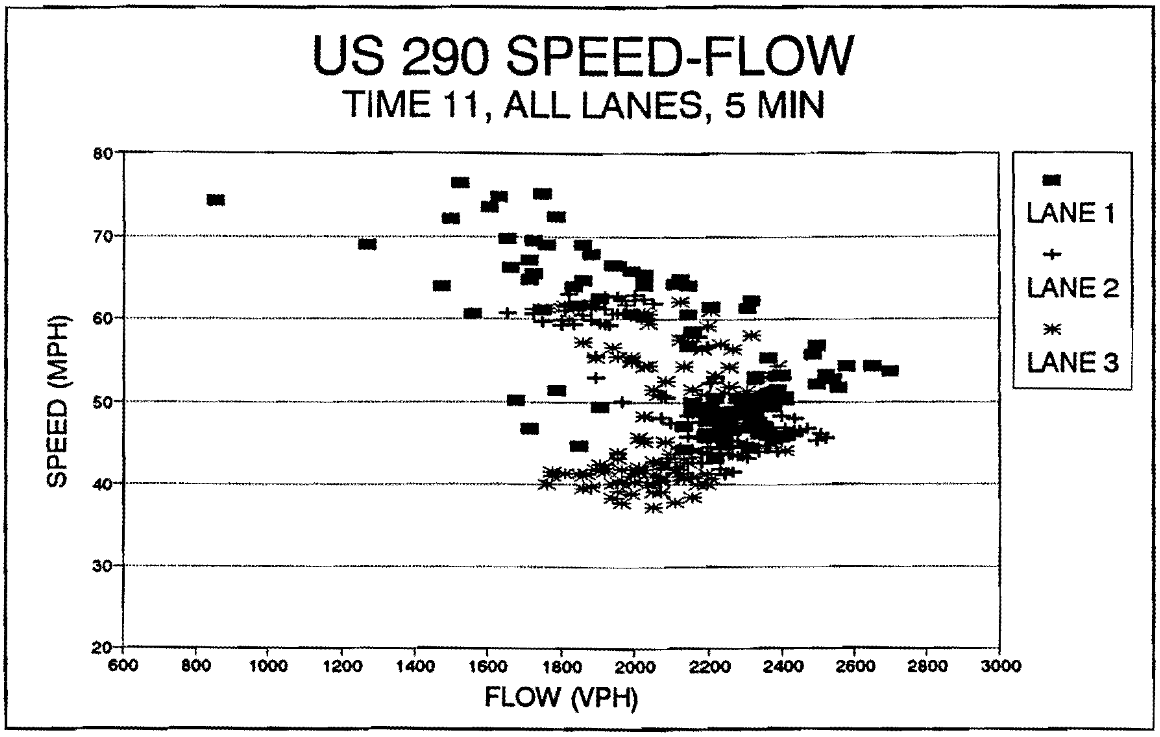

FIGURE B-27 Speed vs flow for all lanes, time 11, at US 290 using 5-minute moving averages.

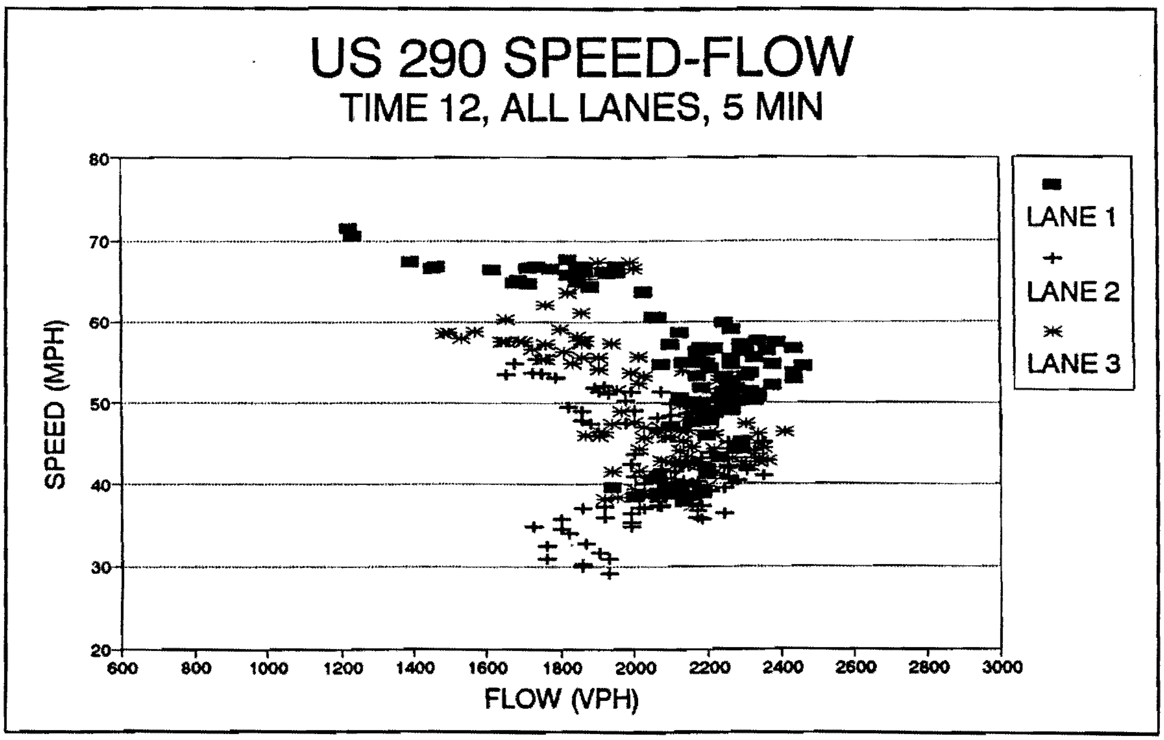

FIGURE B-28 Speed vs flow for all lanes, time 12, at US 290 using 5-minute moving averages.

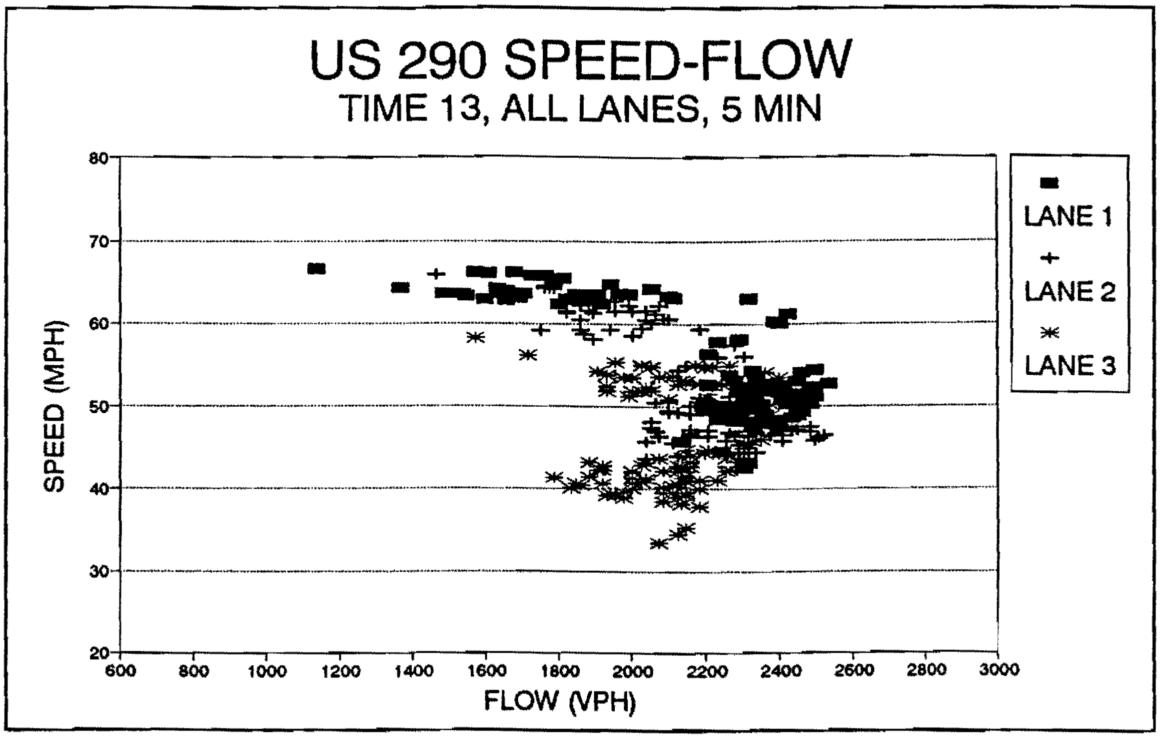

FIGURE B-29 Speed vs flow for all lanes, time 13, at US 290 using 5-minute moving averages.

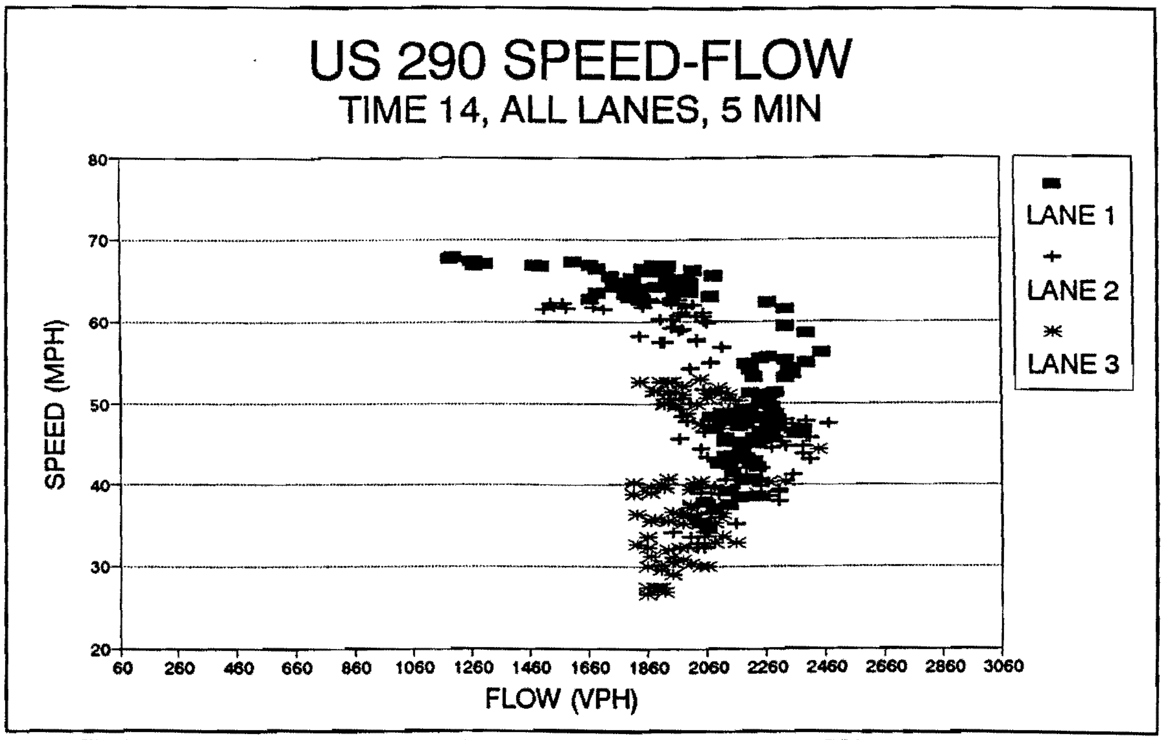

FIGURE B-30 Speed vs flow for all lanes, time 14, at US 290 using 5-minute moving averages.

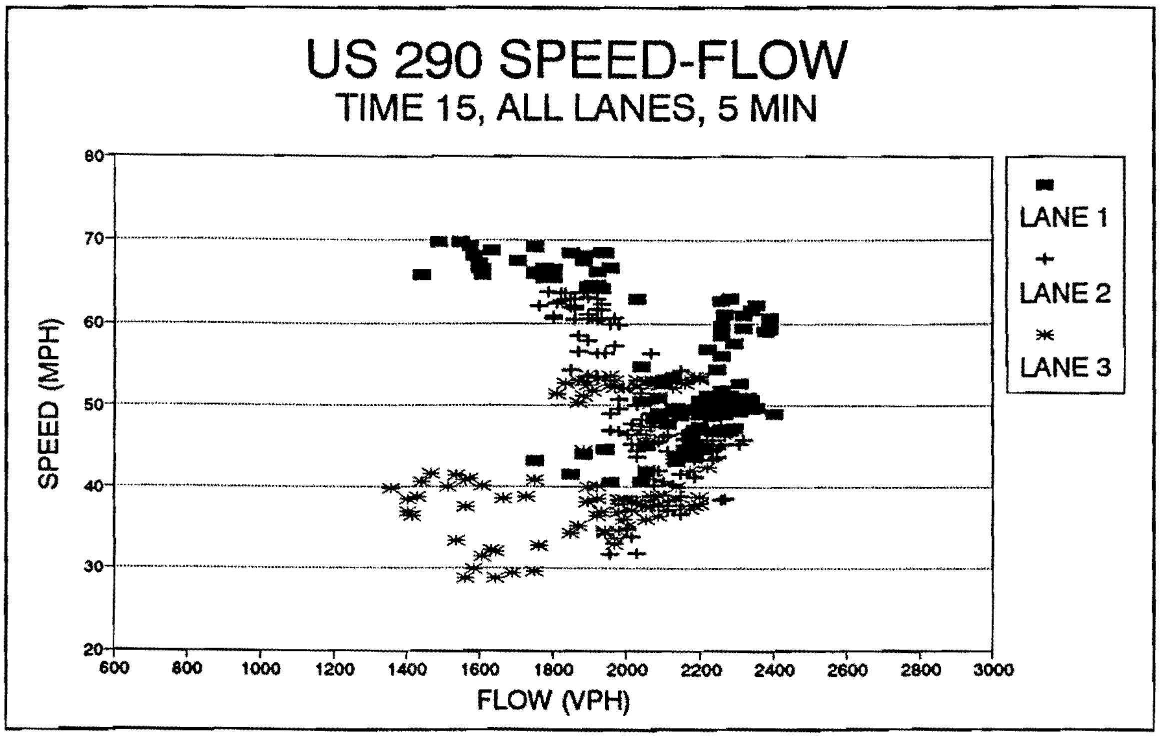

FIGURE B-31 Speed vs flow for all lanes, time 15, at US 290 using 5-minute moving averages.

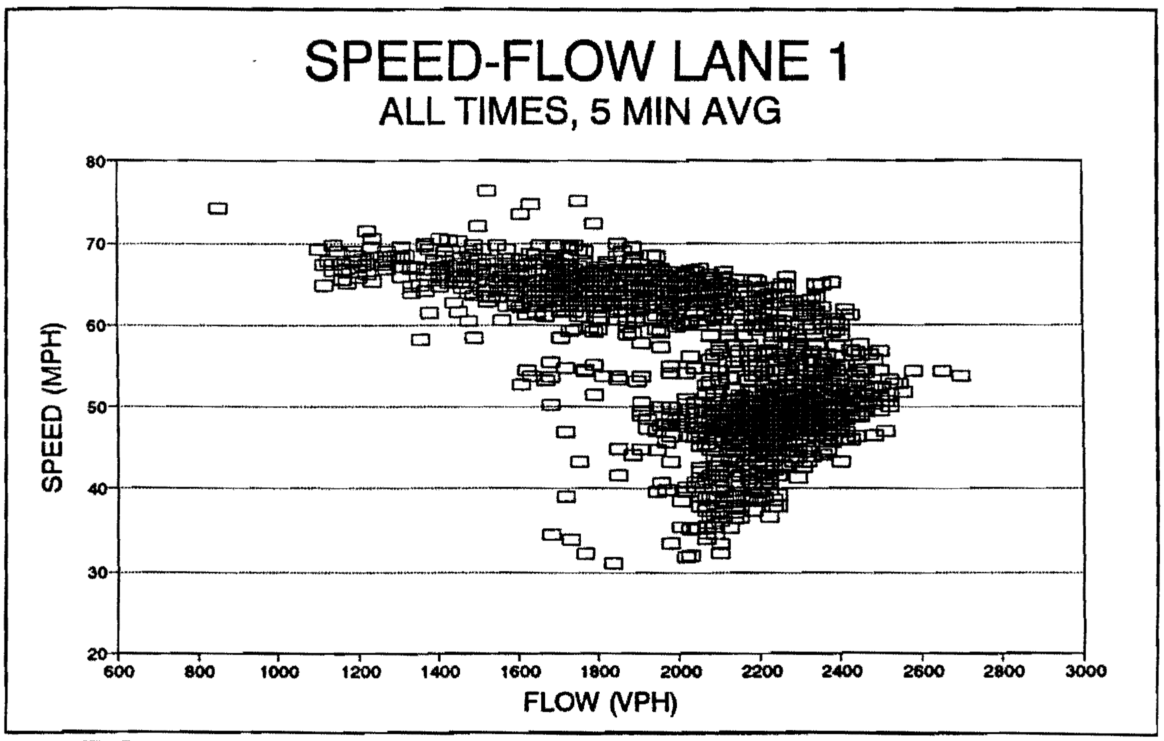

FIGURE B-32 Speed vs flow for lane 1, all times together, at US 290 using 5-minute moving averages.

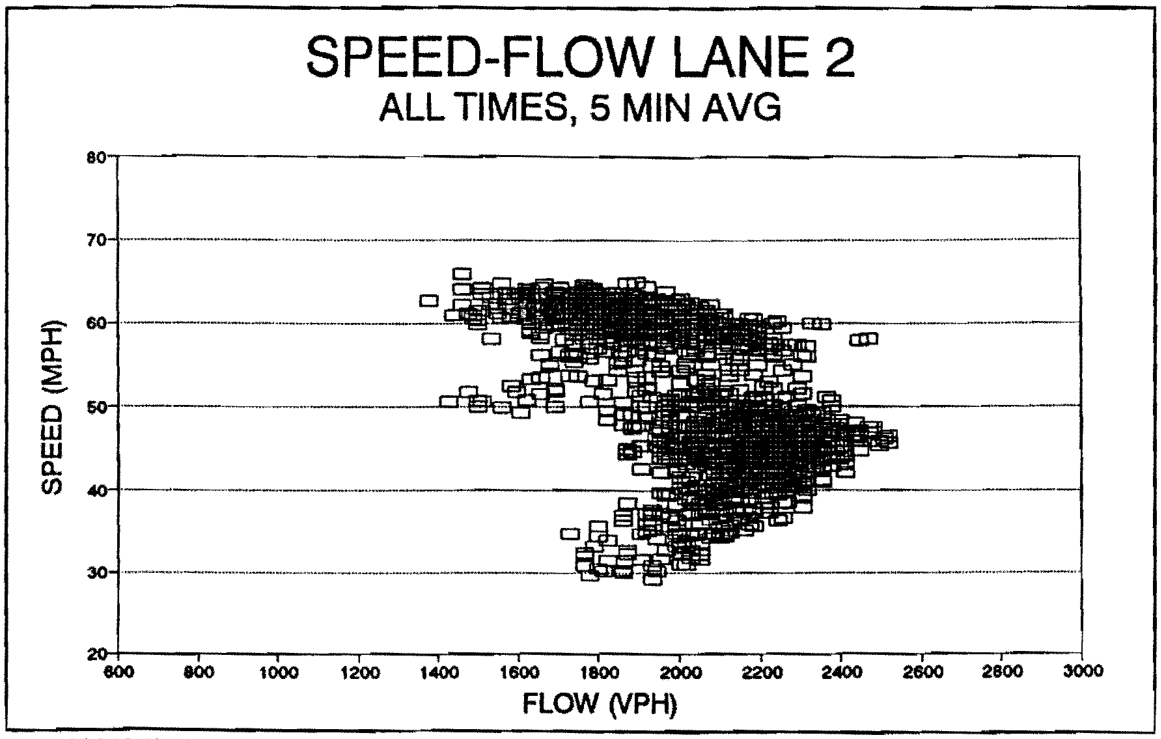

FIGURE B-33 Speed vs flow for lane 2, all times together, at US 290 using 5-minute moving averages.

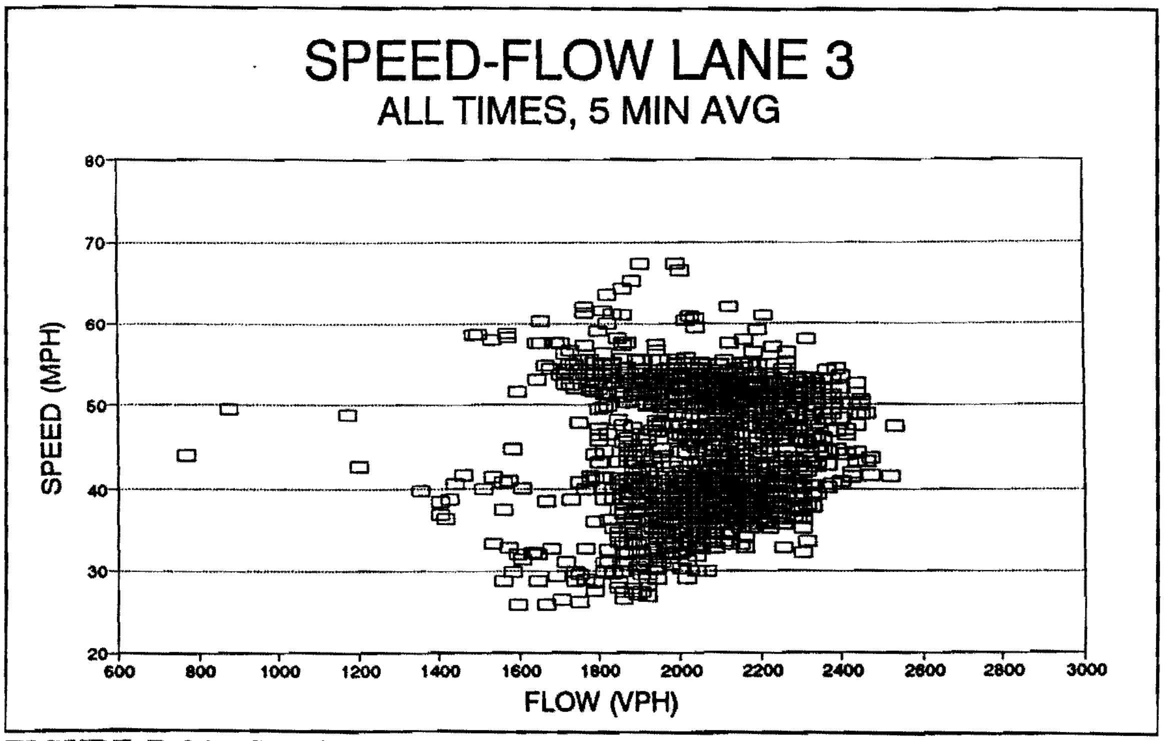

FIGURE B-34 Speed vs flow for lane 3, all times together, at US 290 using 5-minute moving averages.

APPENDIX C

SPEED AND FLOW PROFILES FOR VALIDATION SITES

- I-410 Site
- I-35/US 67 Site
- U.S. 183 Site

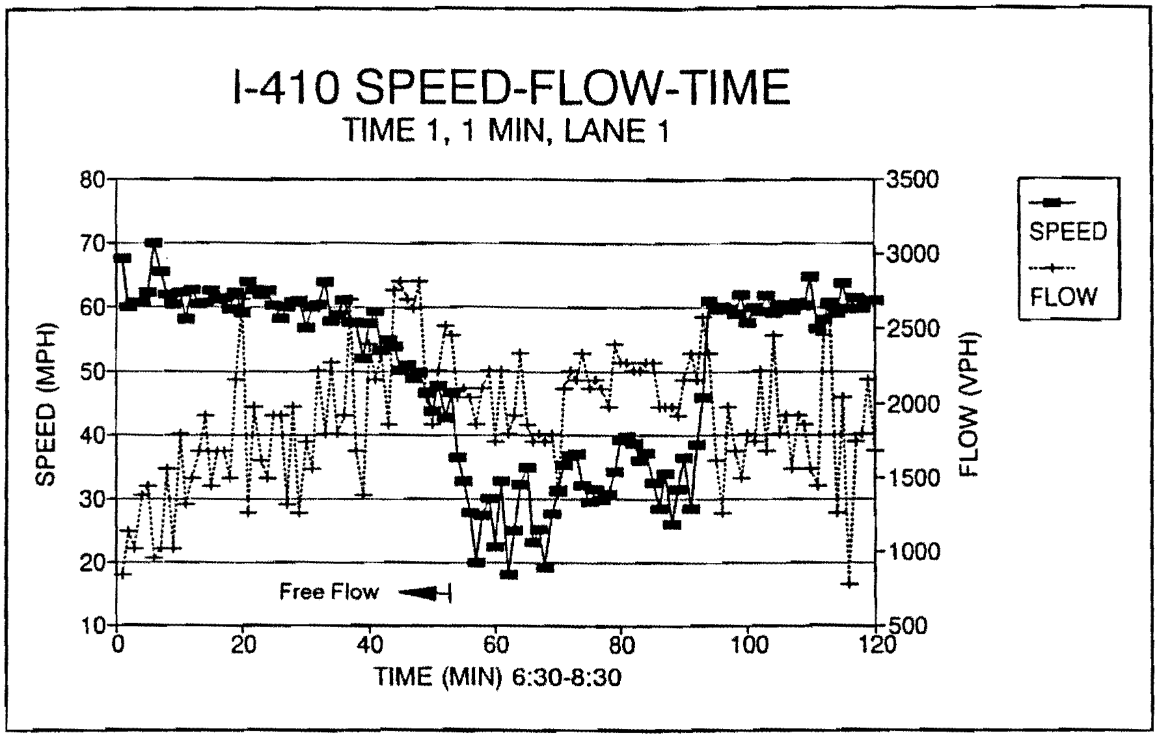

FIGURE C-1 Speed and flow vs time at I-410 for time 1, lane 1, using 1-minute intervals.

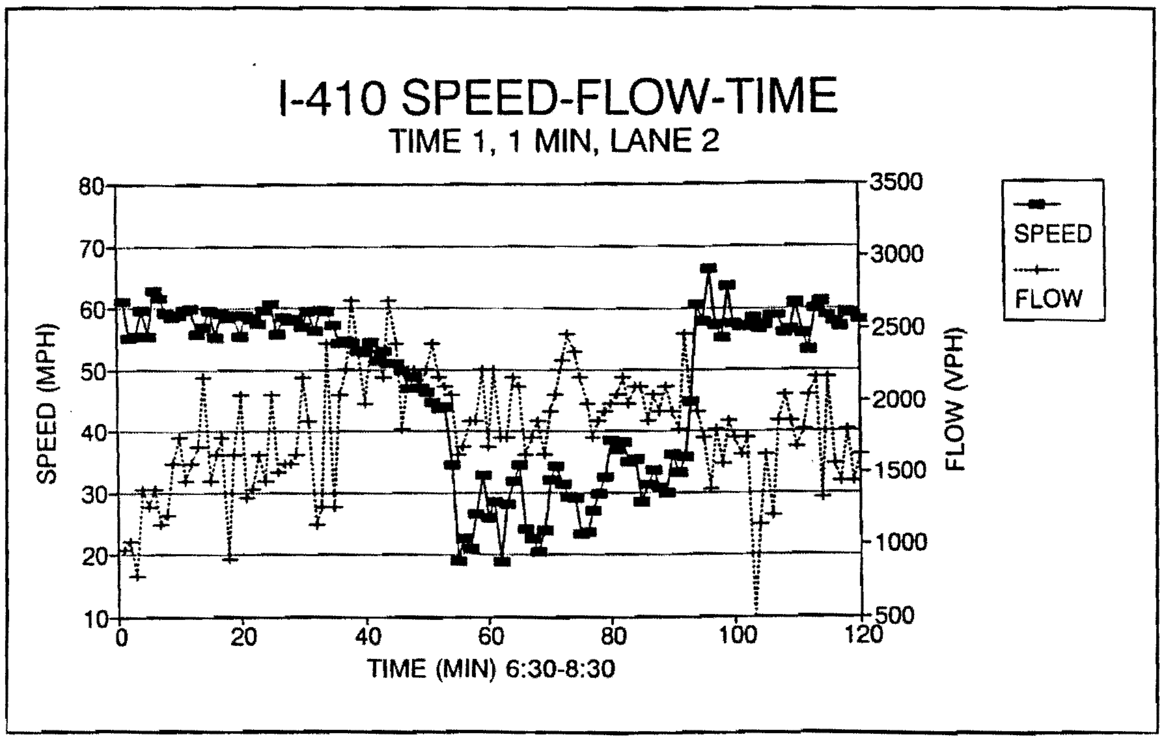

FIGURE C-2 Speed and flow vs time at I-410 for time 1, lane 2, using 1-minute intervals.

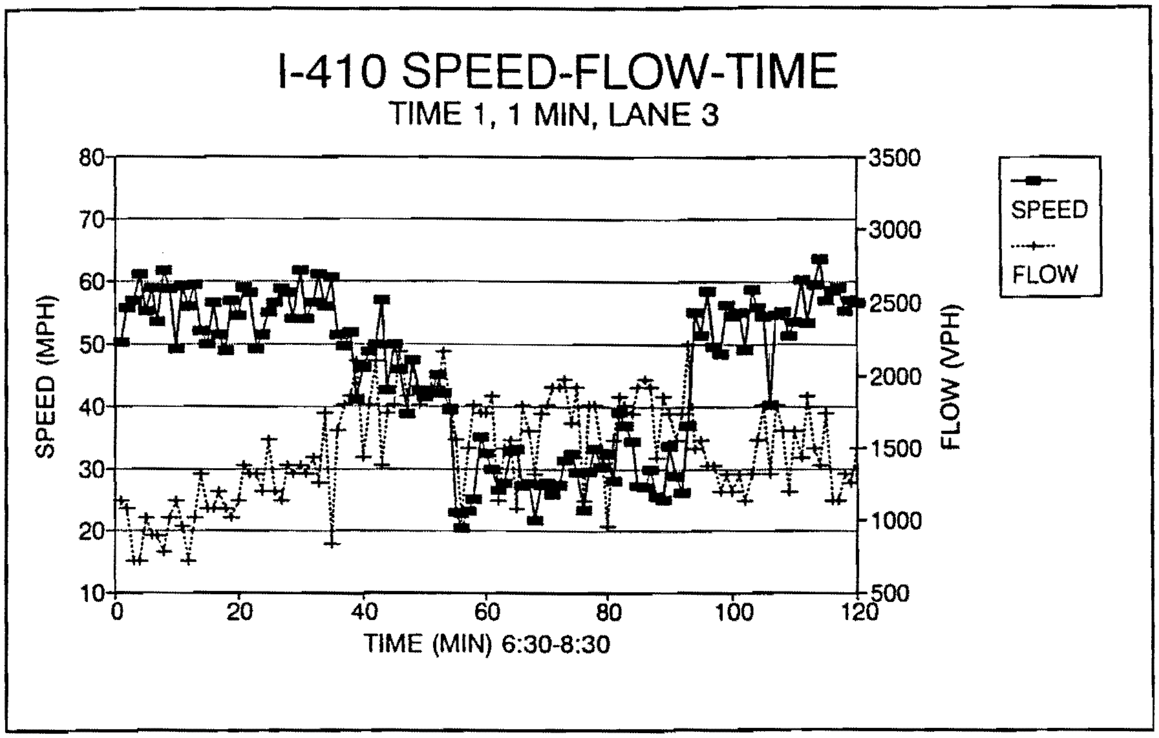

FIGURE C-3 Speed and flow vs time at I-410 for time 1, lane 3, using 1-minute intervals.

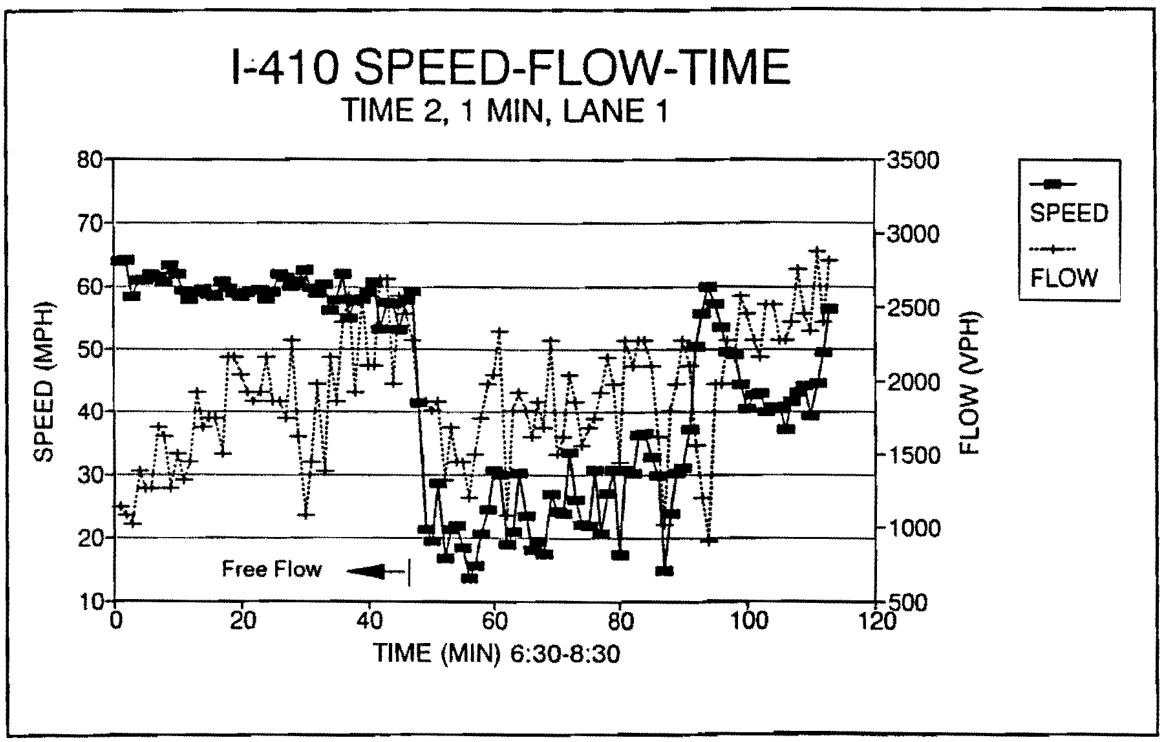

FIGURE C-4 Speed and flow vs time at I-410 for time 2, lane 1, using 1-minute intervals.

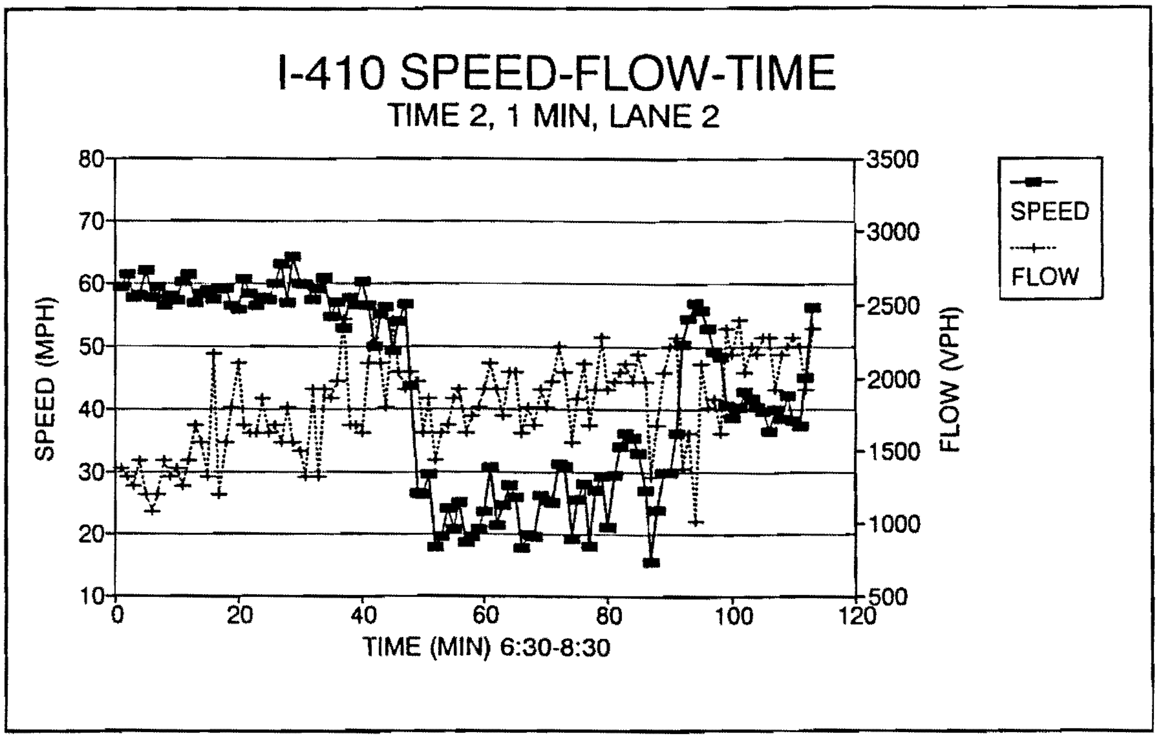

FIGURE C-5 Speed and flow vs time at I-410 for time 2, lane 2, using 1-minute intervals.

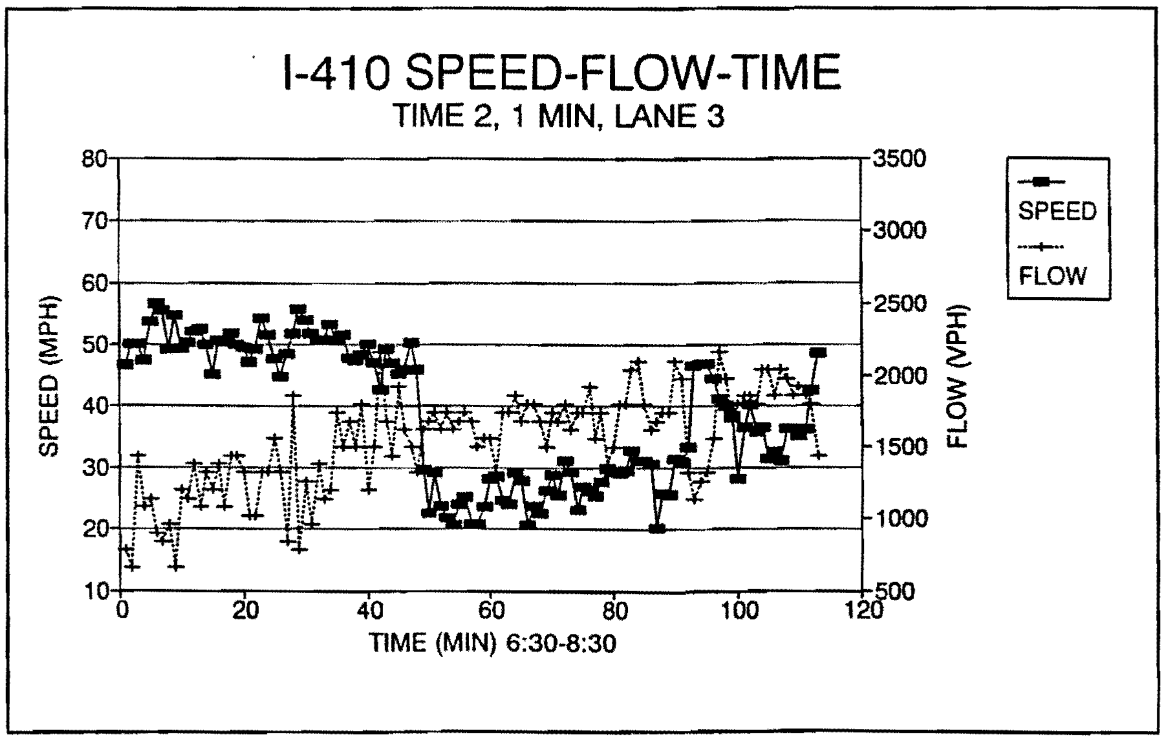

FIGURE C-6 Speed and flow vs time at I-410 for time 2, lane 3, using 1-minute intervals.

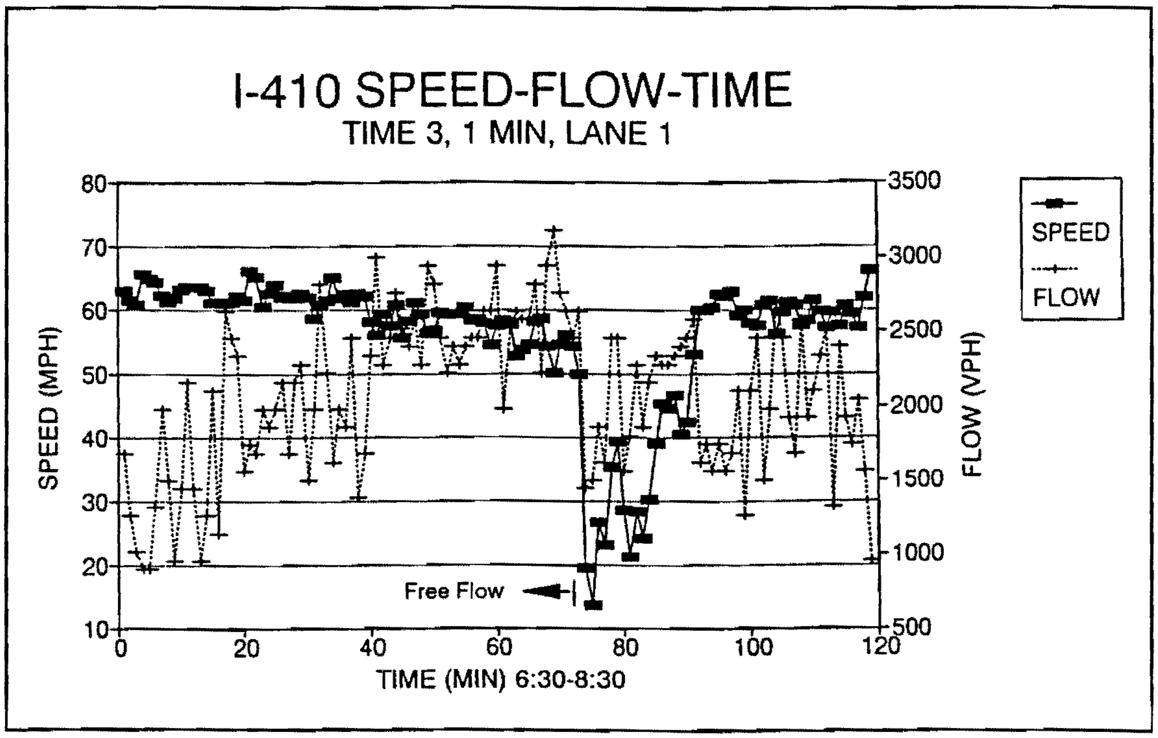

FIGURE C-7 Speed and flow vs time at I-410 for time 3, lane 1, using 1-minute intervals.

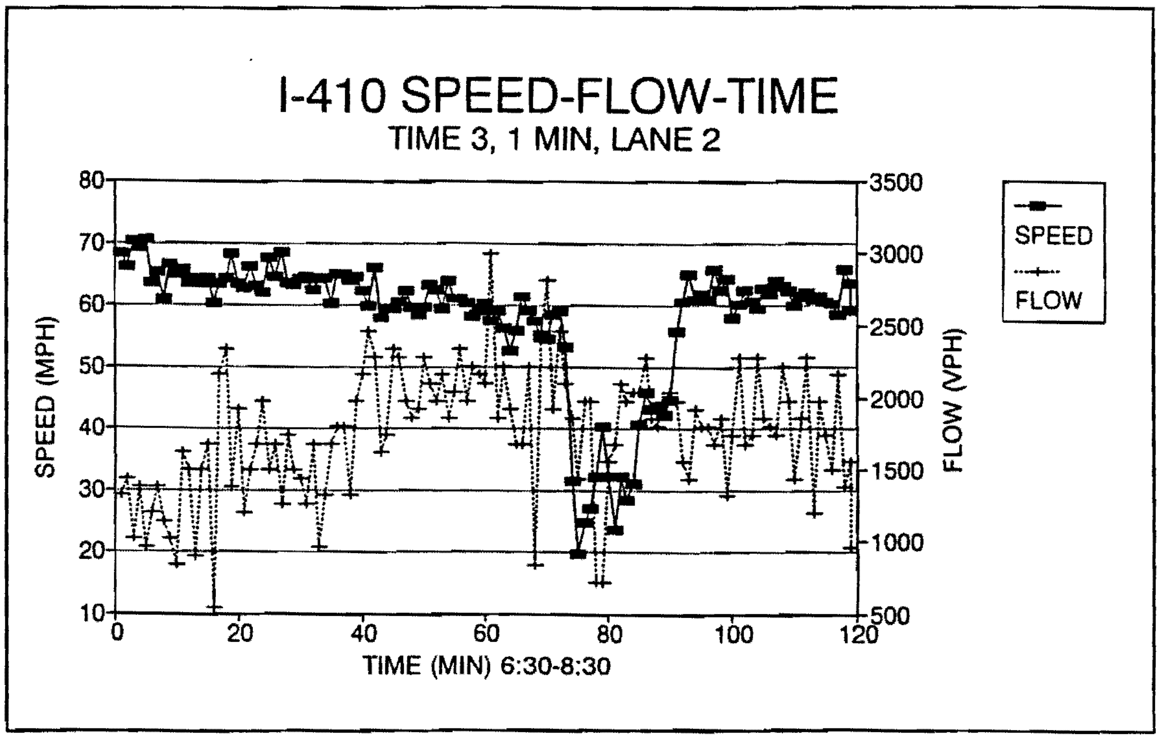

FIGURE C-8 Speed and flow vs time at I-410 for time 3, lane 2, using 1-minute intervals.

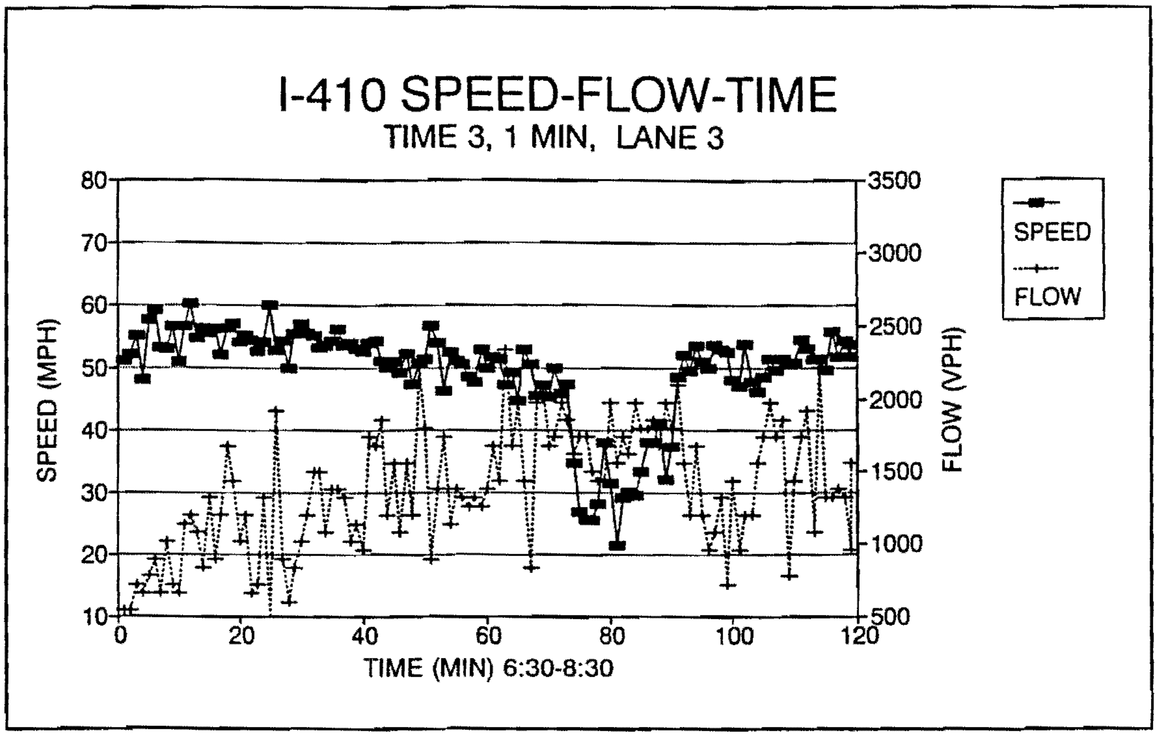

FIGURE C-9 Speed and flow vs time at I-410 for time 3, lane 3, using 1-minute intervals.

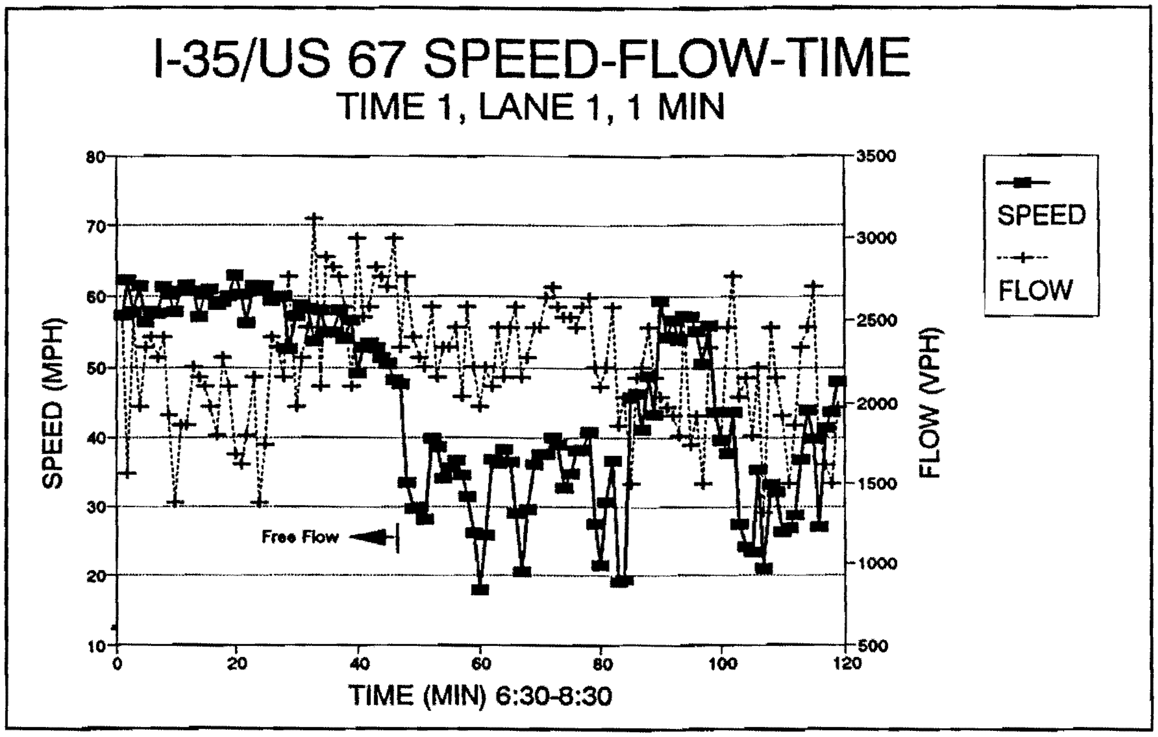

FIGURE C-10 Speed and flow vs time at I-35/US 67 for time 1, lane 1, using 1-minute intervals.

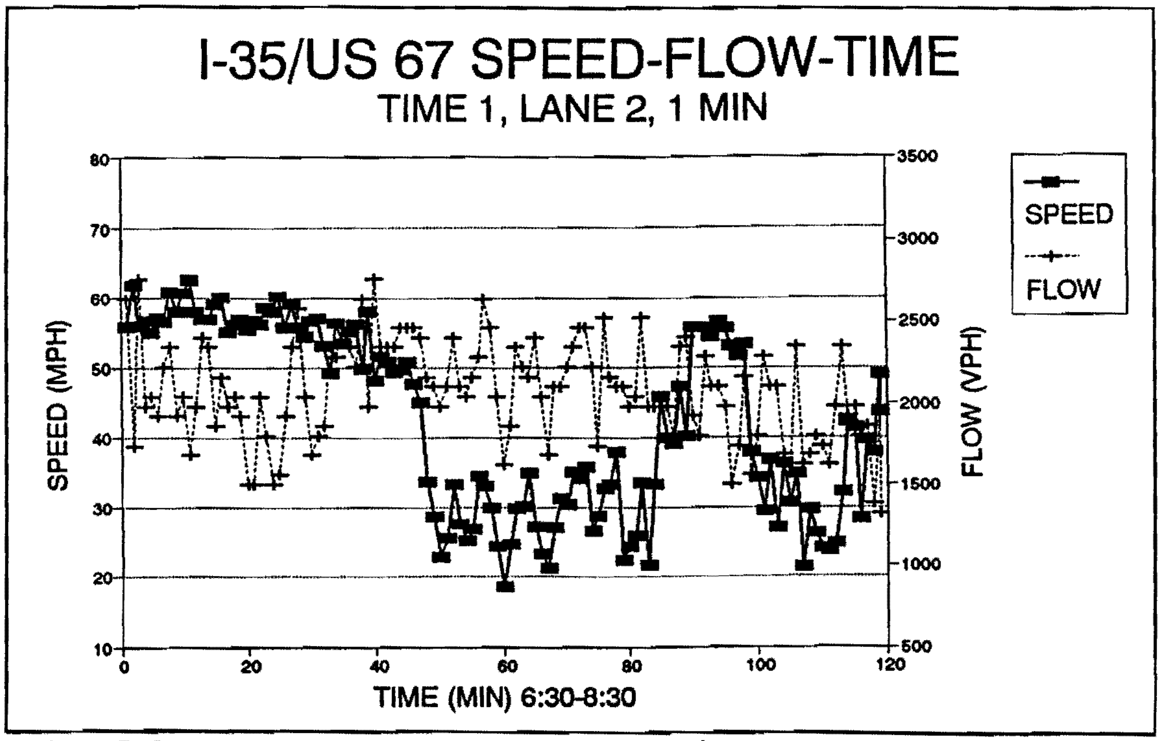

FIGURE C-11 Speed and flow vs time at I-35/US 67 for time 1, lane 2, using 1-minute intervals.

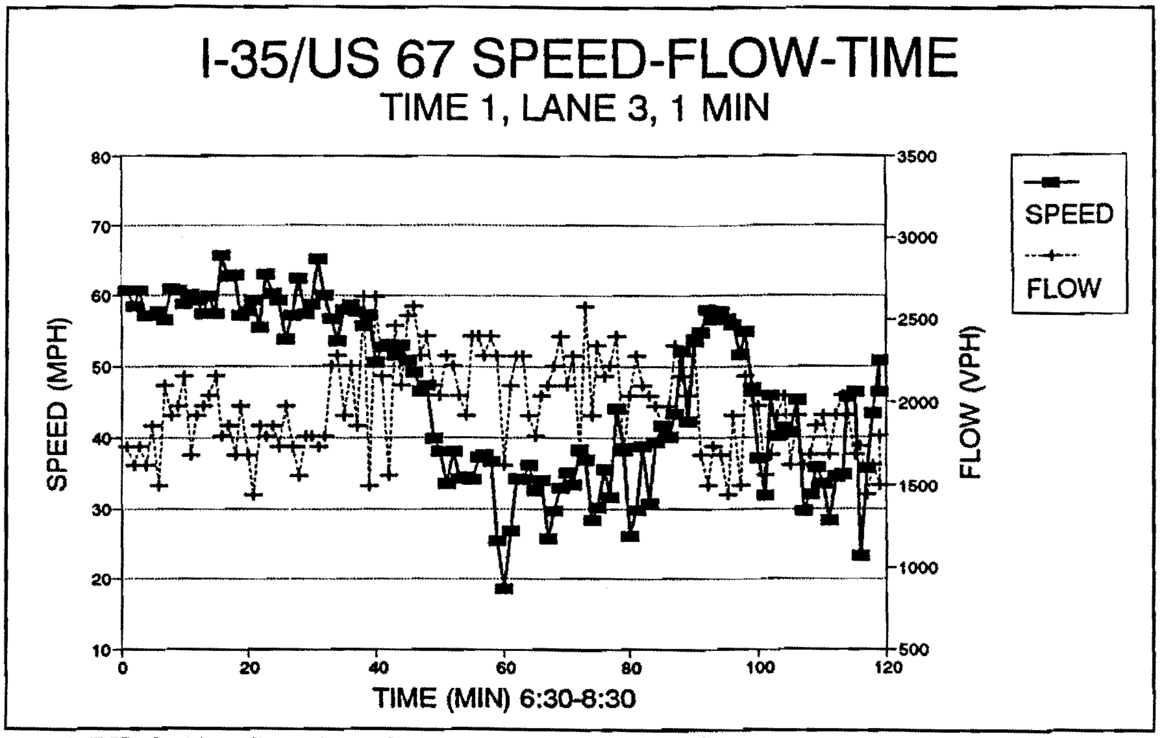

FIGURE C-12 Speed and flow vs time at I-35/US 67 for time 1, lane 3, using 1-minute intervals.

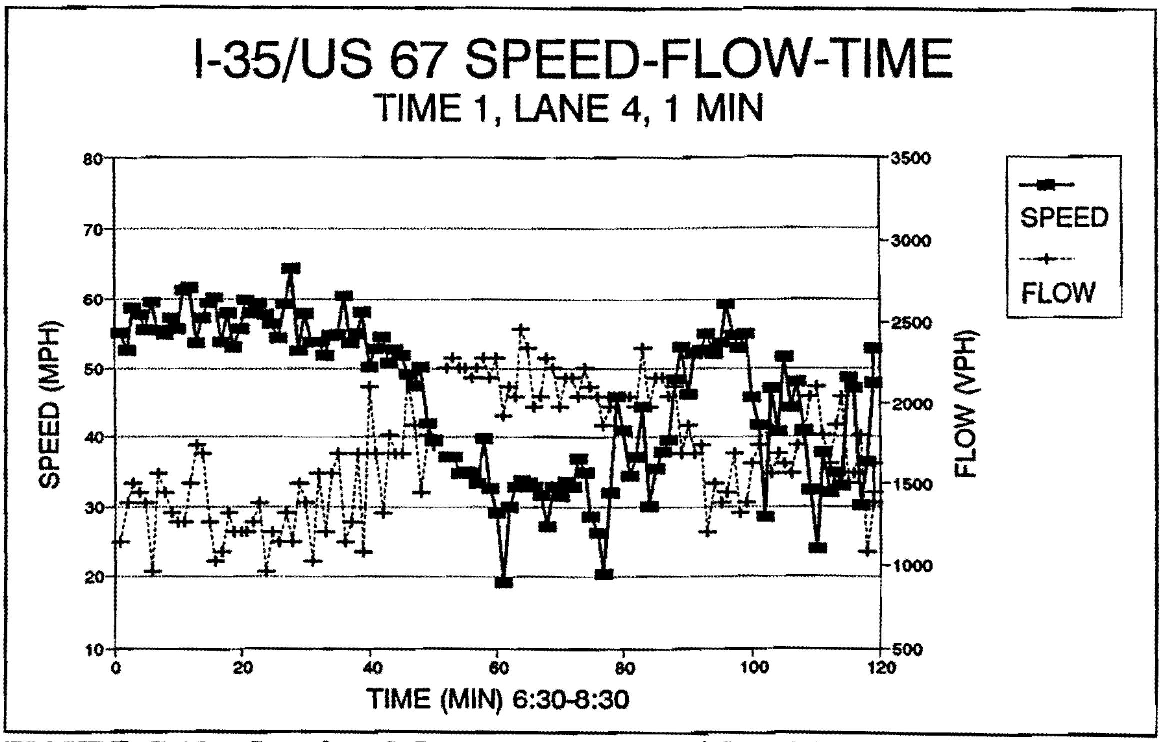

FIGURE C-13 Speed and flow vs time at I-35/US 67 for time 1, lane 4, using 1-minute intervals.

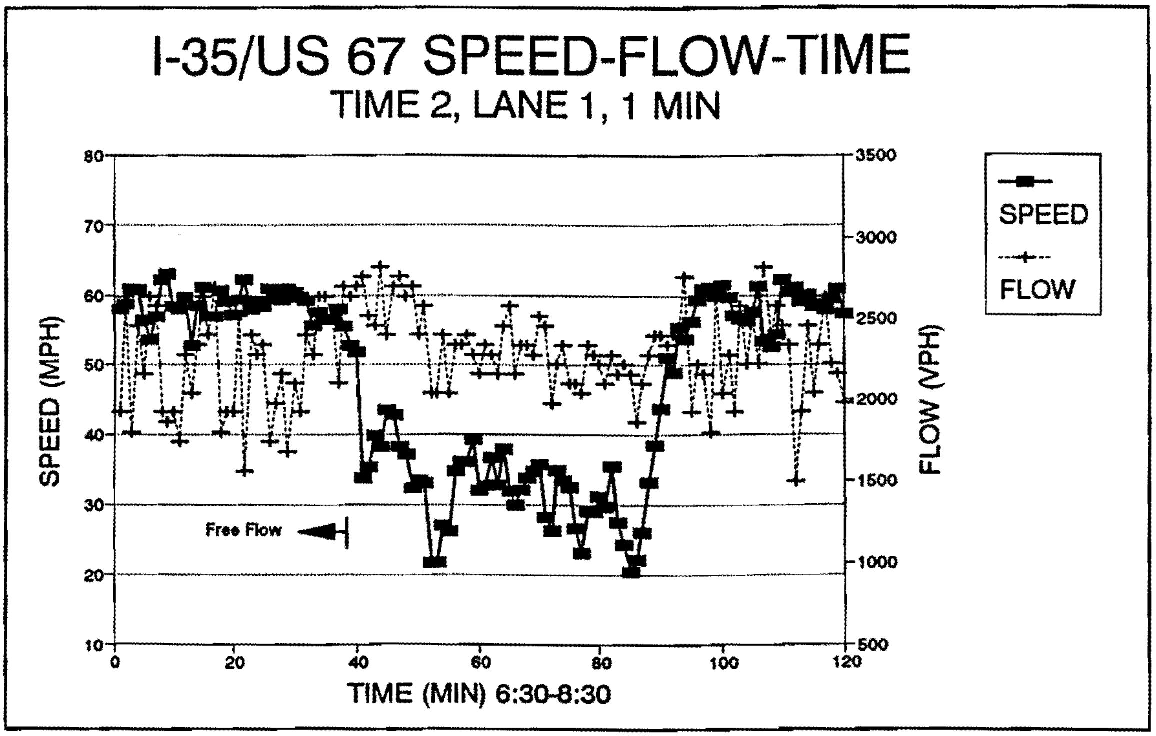

FIGURE C-14 Speed and flow vs time at I-35/US 67 for time 2, lane 1, using 1-minute intervals.

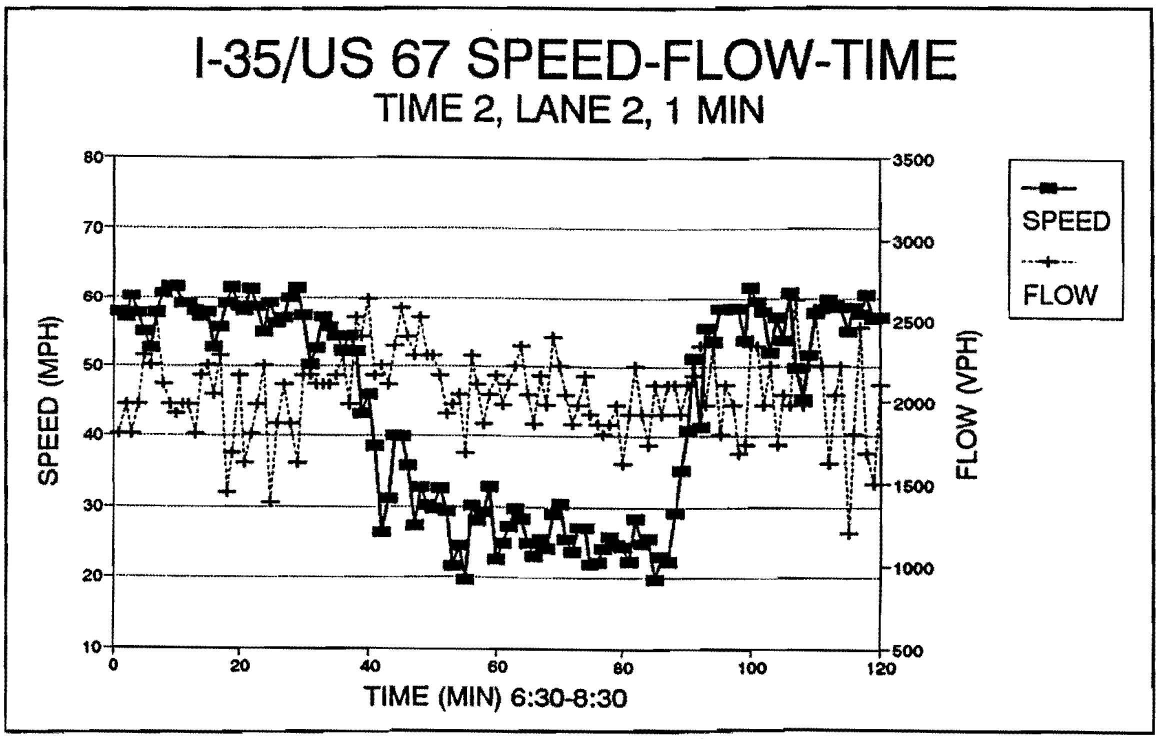

FIGURE C-15 Speed and flow vs time at I-35/US 67 for time 2, lane 2, using 1-minute intervals.

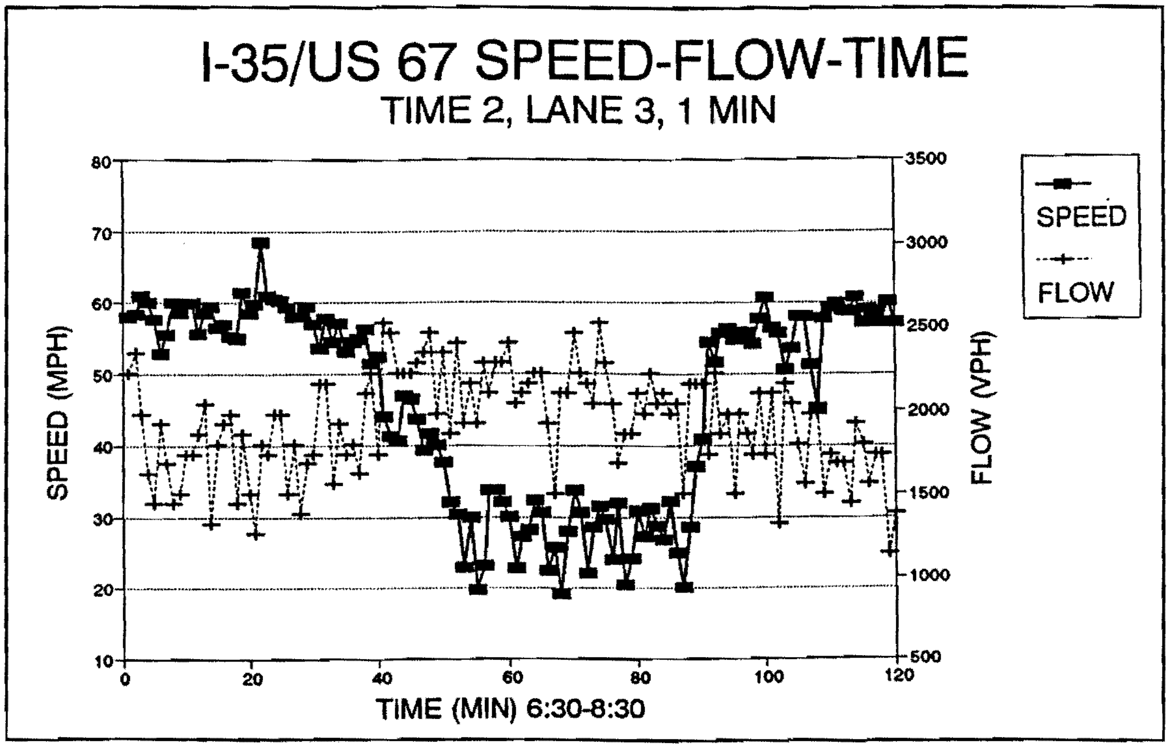

FIGURE C-16 Speed and flow vs time at I-35/US 67 for time 2, lane 3, using 1-minute intervals.

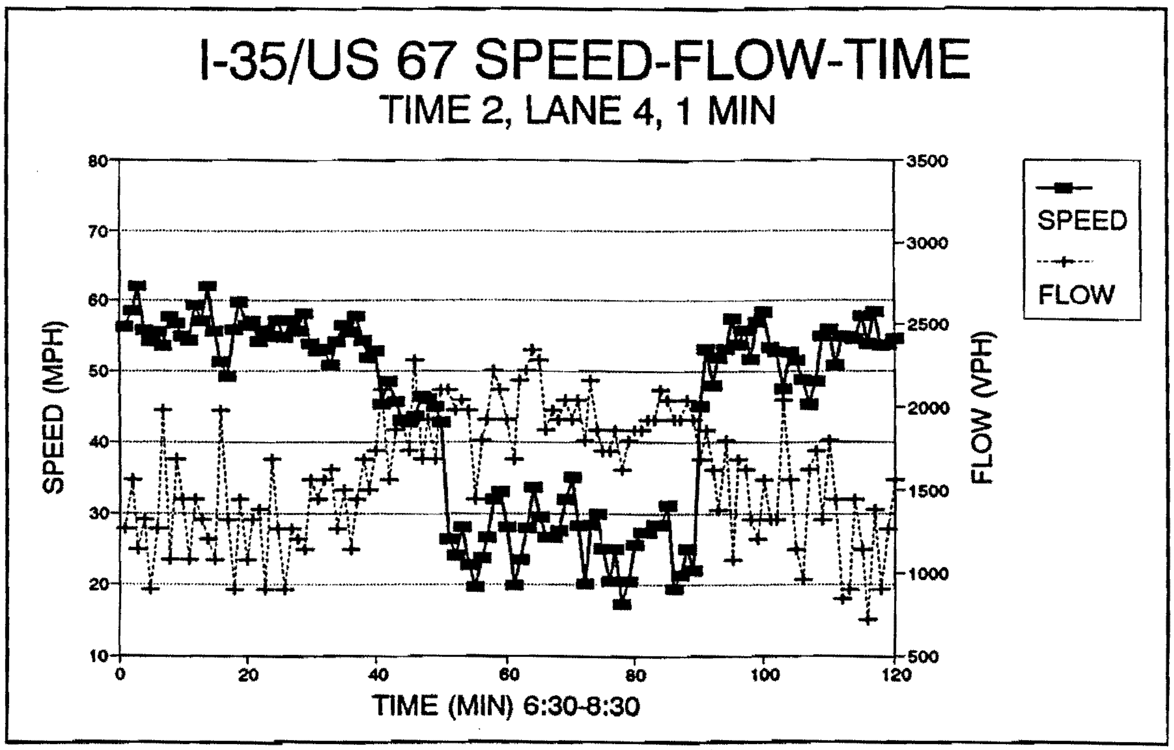

FIGURE C-17 Speed and flow vs time at I-35/US 67 for time 2, lane 4, using 1-minute intervals.

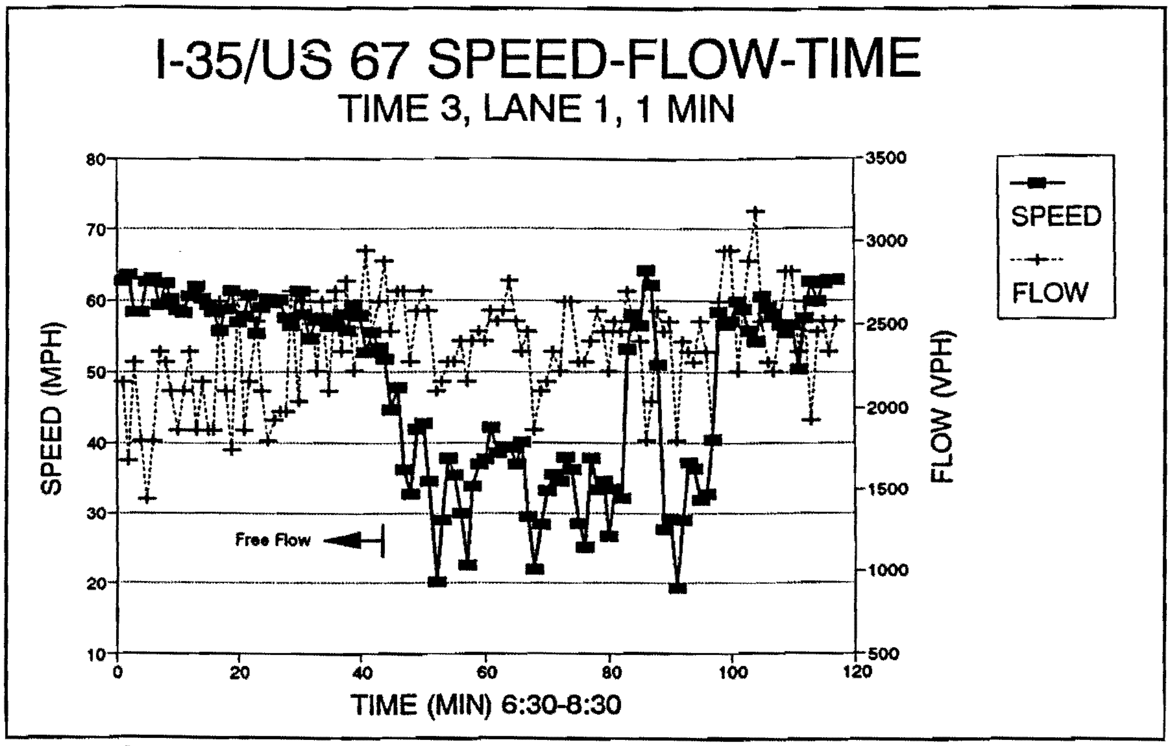

FIGURE C-18 Speed and flow vs time at I-35/US 67 for time 3, lane 1, using 1-minute intervals.

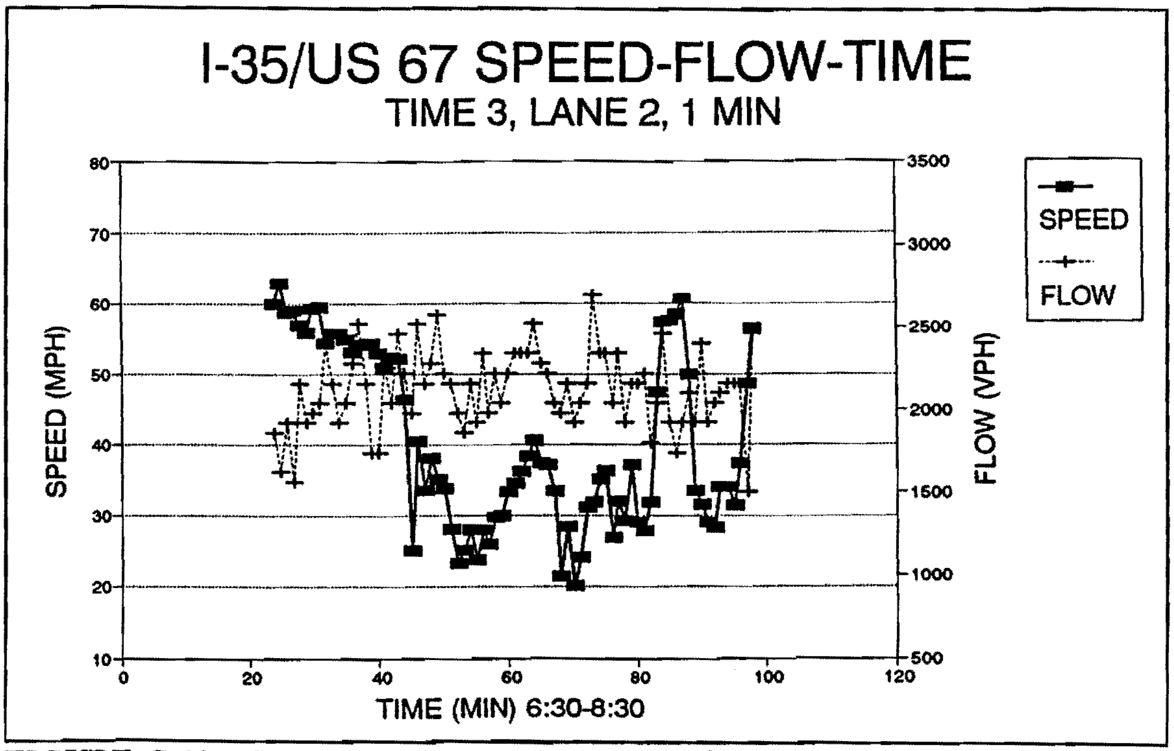

FIGURE C-19 Speed and flow vs time at I-35/US 67 for time 3, lane 2, using 1-minute intervals.

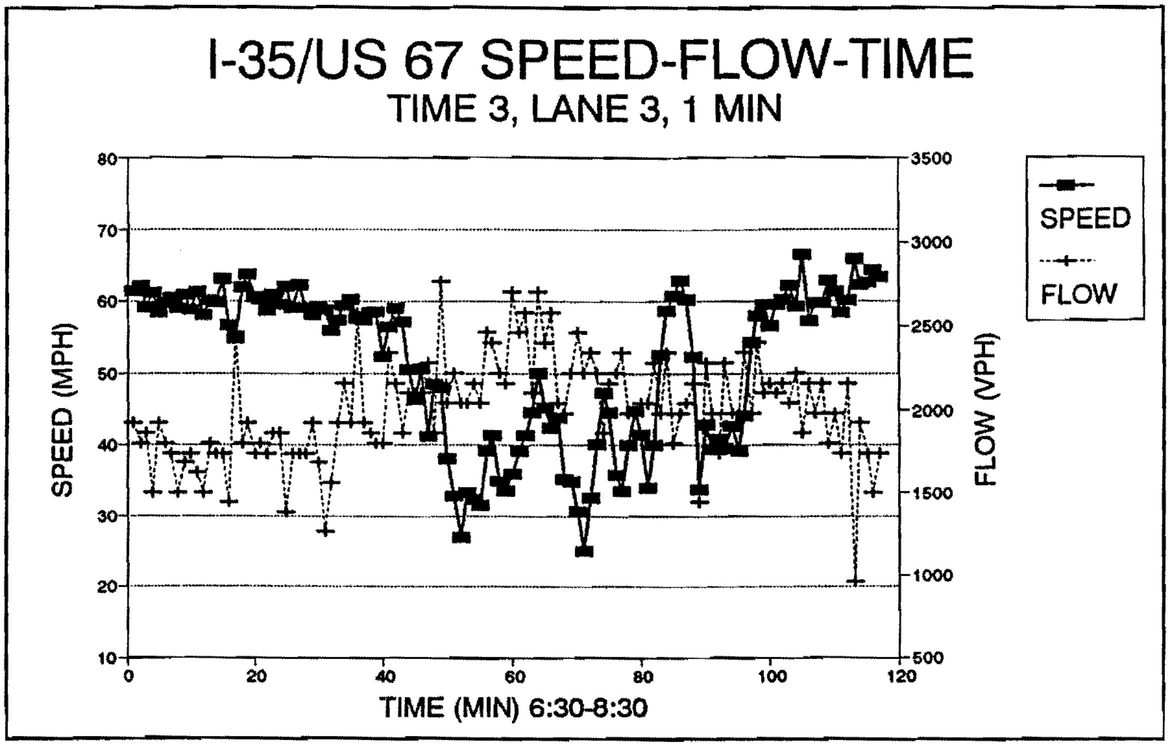

FIGURE C-20 Speed and flow vs time at I-35/US 67 for time 3, lane 3, using 1-minute intervals.

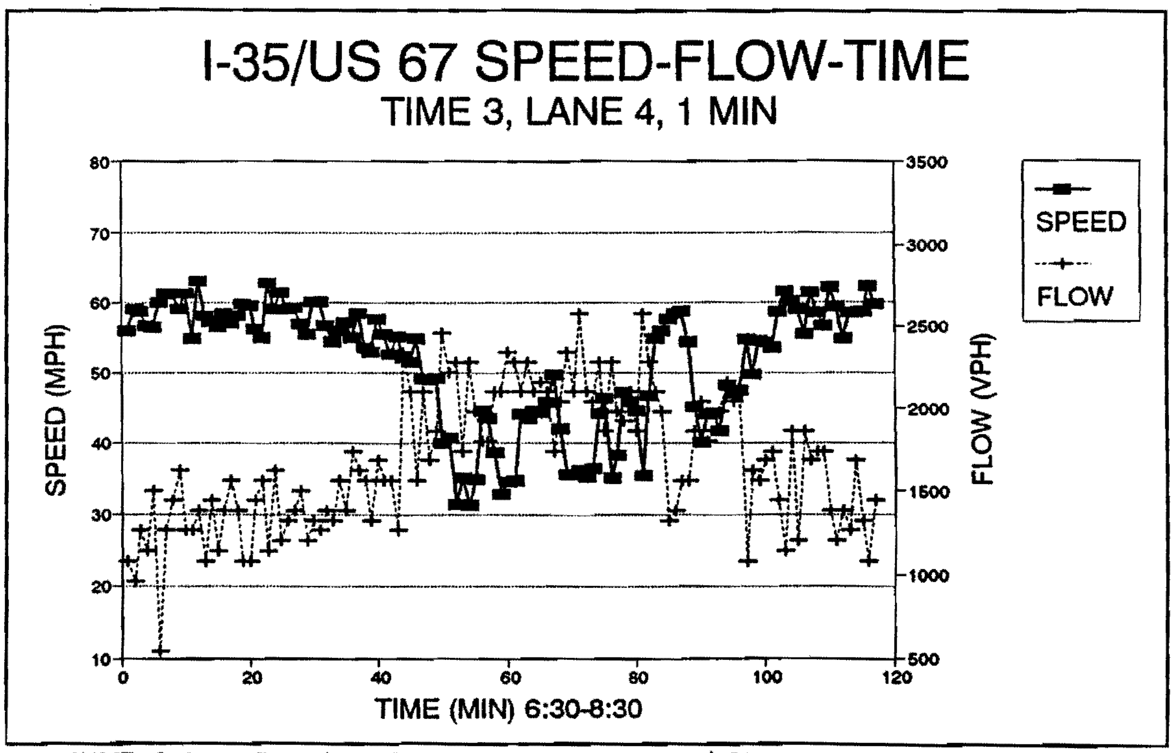

FIGURE C-21 Speed and flow vs time at I-35/US 67 for time 3, lane 4, using 1-minute intervals.

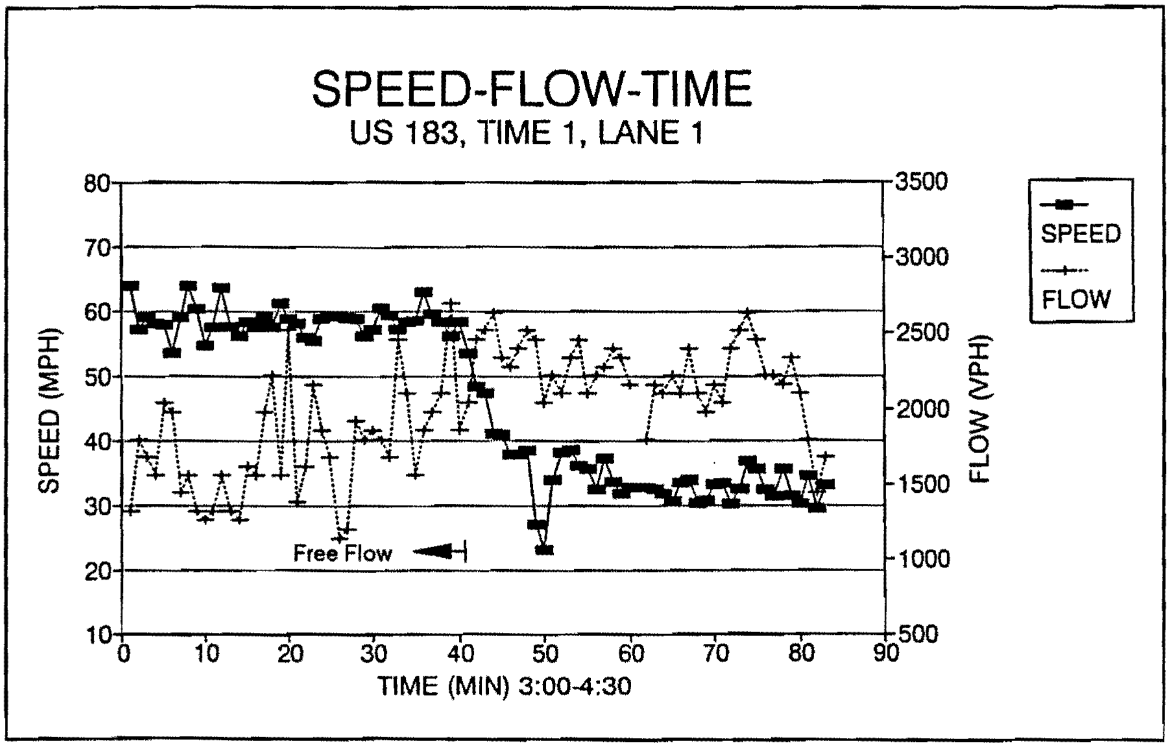

FIGURE C-22 Speed and flow vs time at US 183 for time 1, lane 1, using 1-minute intervals.

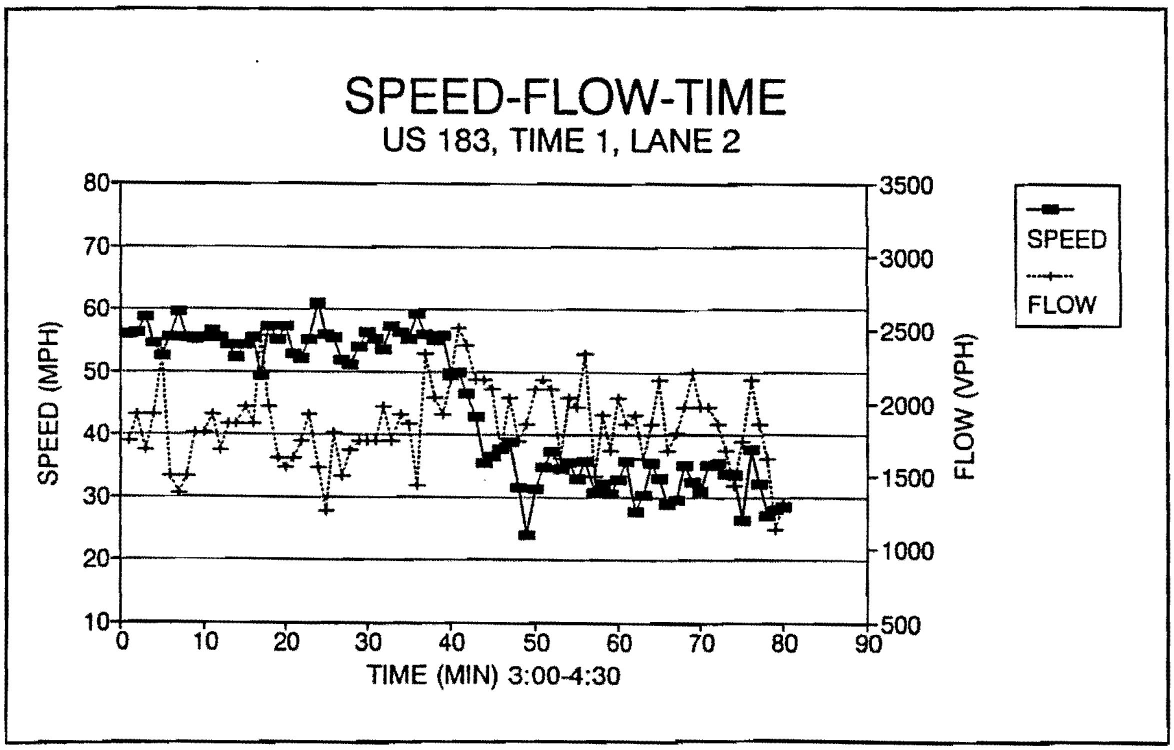

FIGURE C-23 Speed and flow vs time at US 183 for time 1, lane 2, using 1-minute intervals.

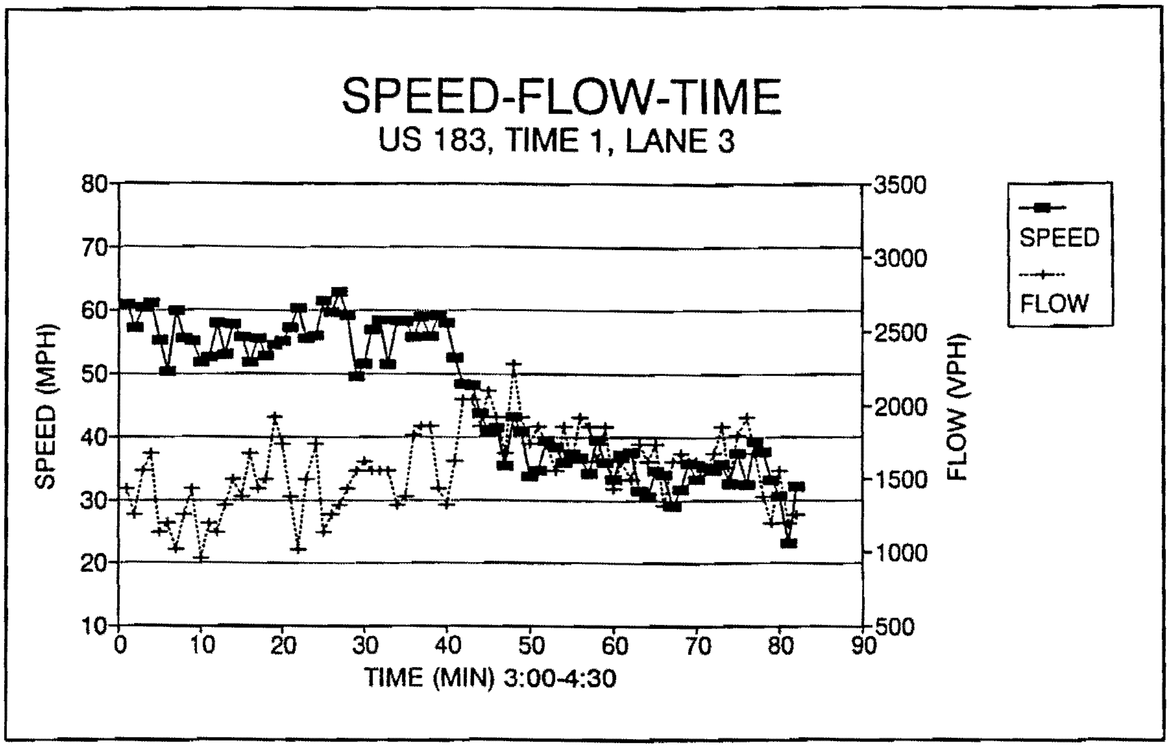

FIGURE C-24 Speed and flow vs time at US 183 for time 1, lane 3, using 1-minute intervals.

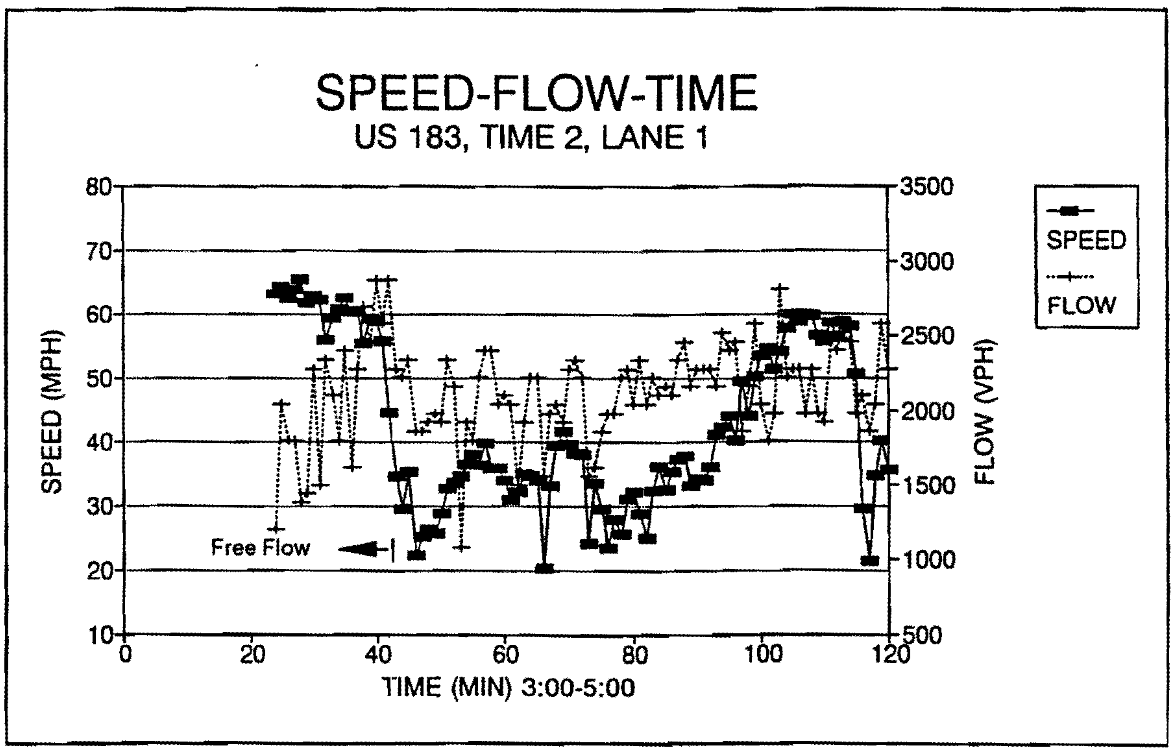

FIGURE C-25 Speed and flow vs time at US 183 for time 2, lane 1, using 1-minute intervals.

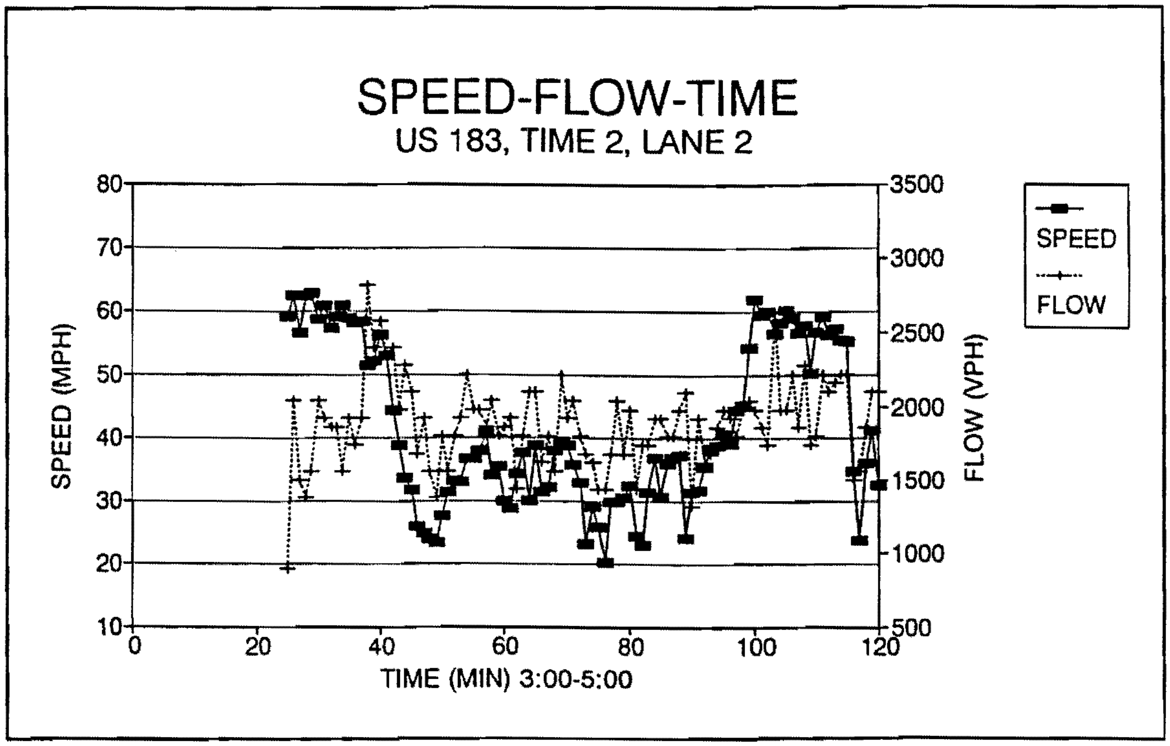

FIGURE C-26 Speed and flow vs time at US 183 for time 2, lane 2, using 1-minute intervals.

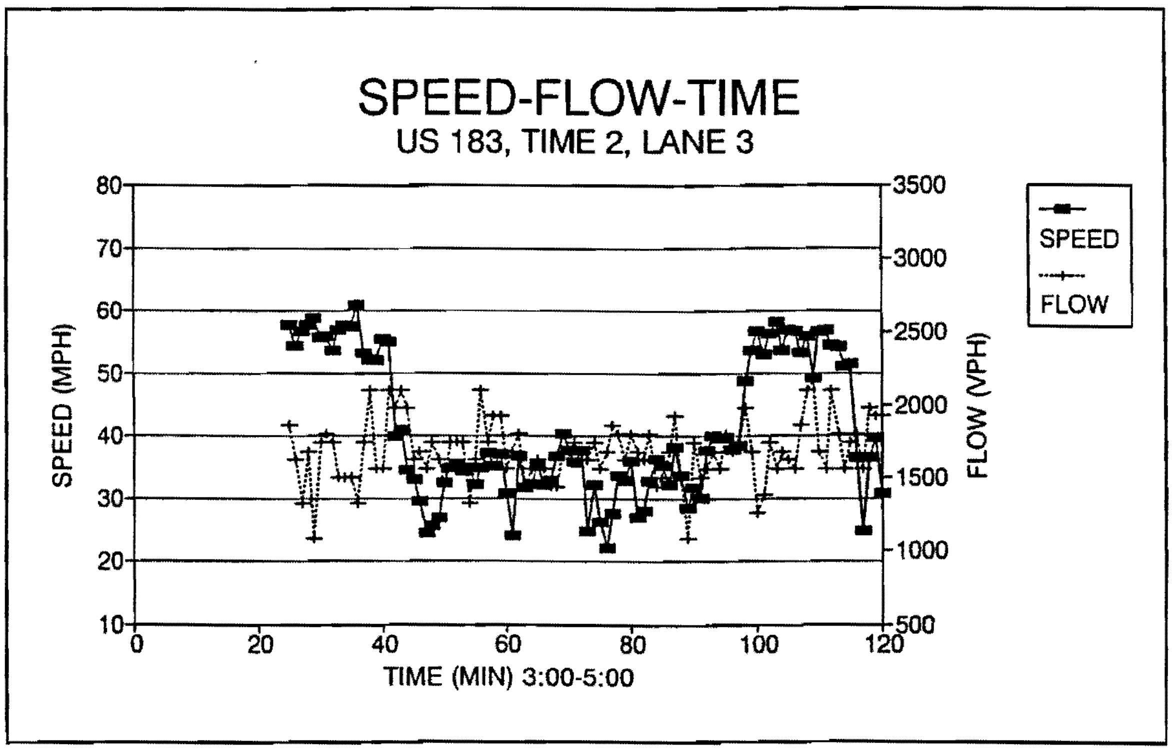

FIGURE C-27 Speed and flow vs time at US 183 for time 2, lane 3, using 1-minute intervals.

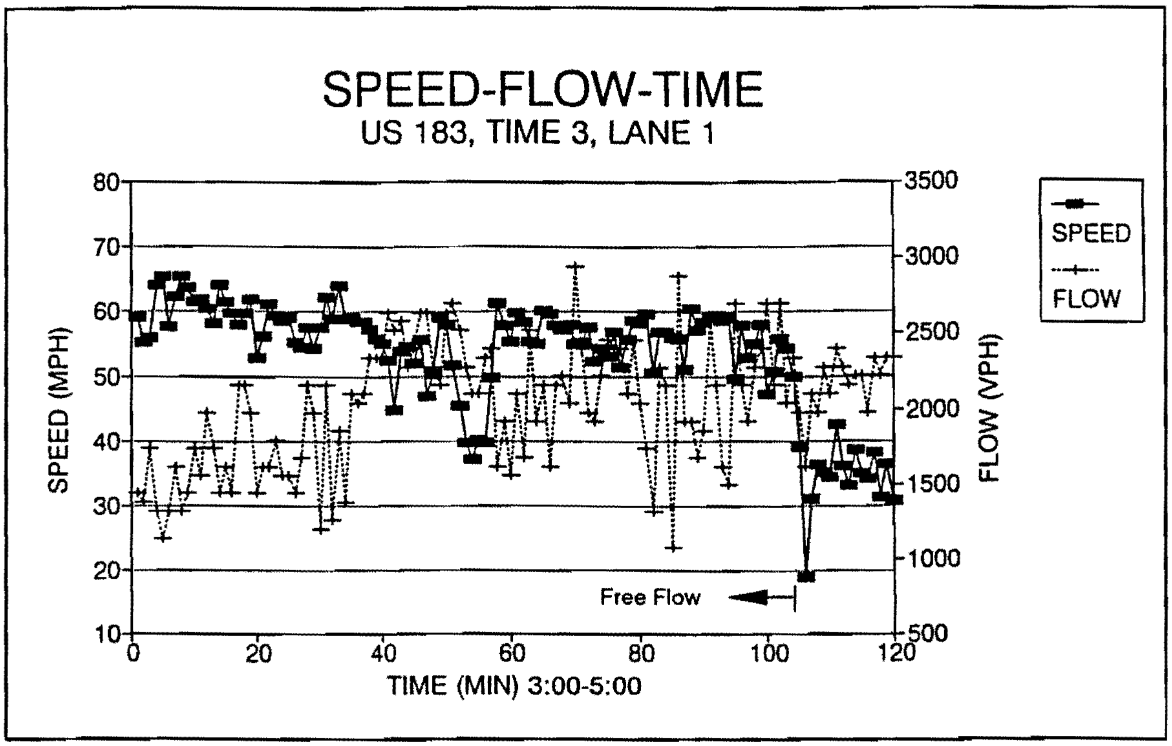

FIGURE C-27 Speed and flow vs time at US 183 for time 3, lane 1, using 1-minute intervals.

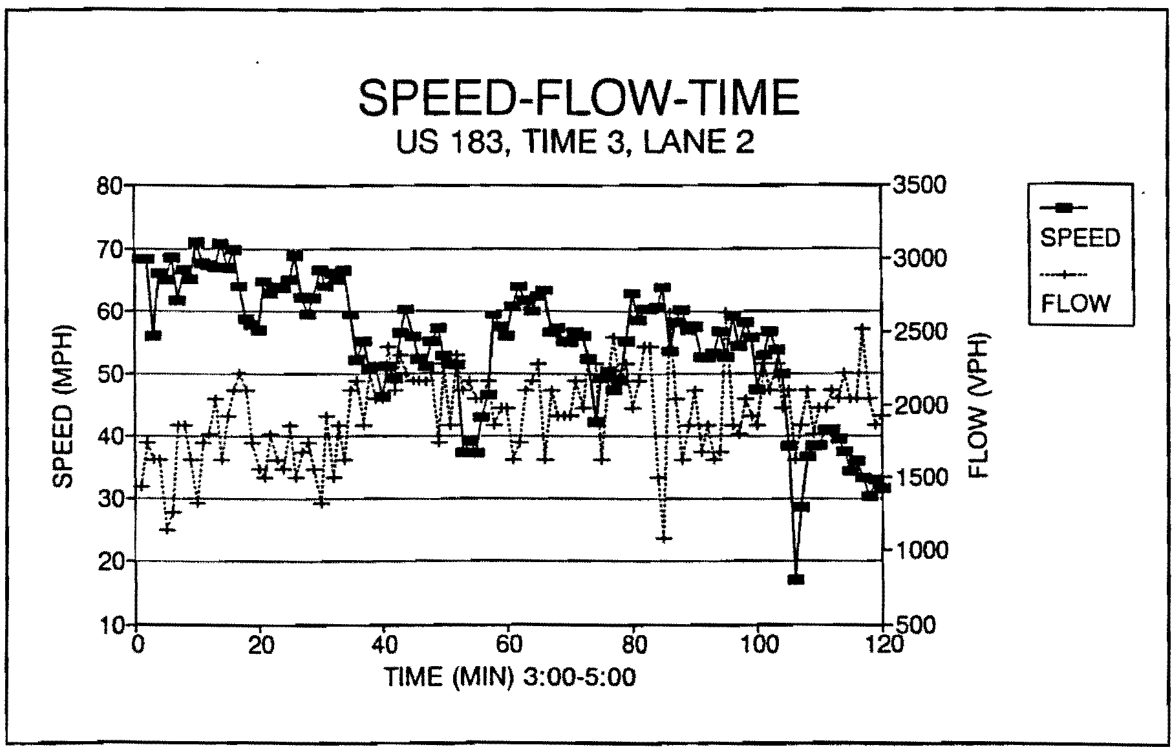

FIGURE C-28 Speed and flow vs time at US 183 for time 3, lane 2, using 1-minute intervals.

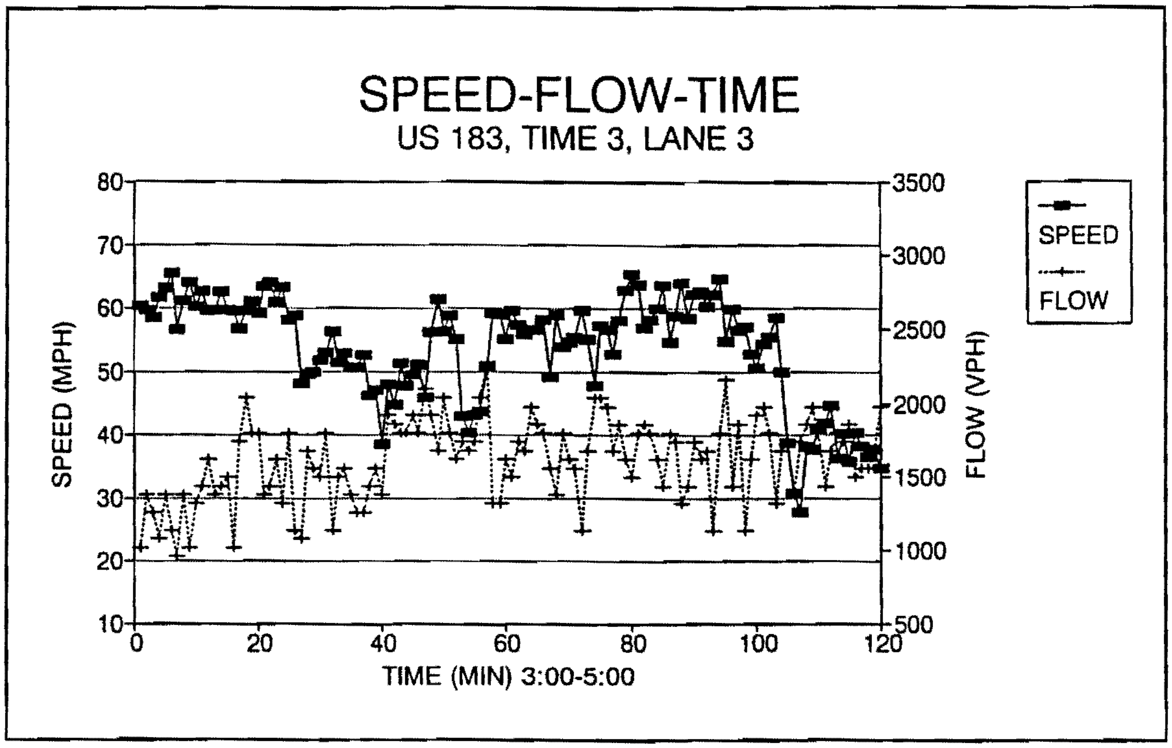

FIGURE C-29 Speed and flow vs time at US 183 for time 3, lane 3, using 1-minute intervals.

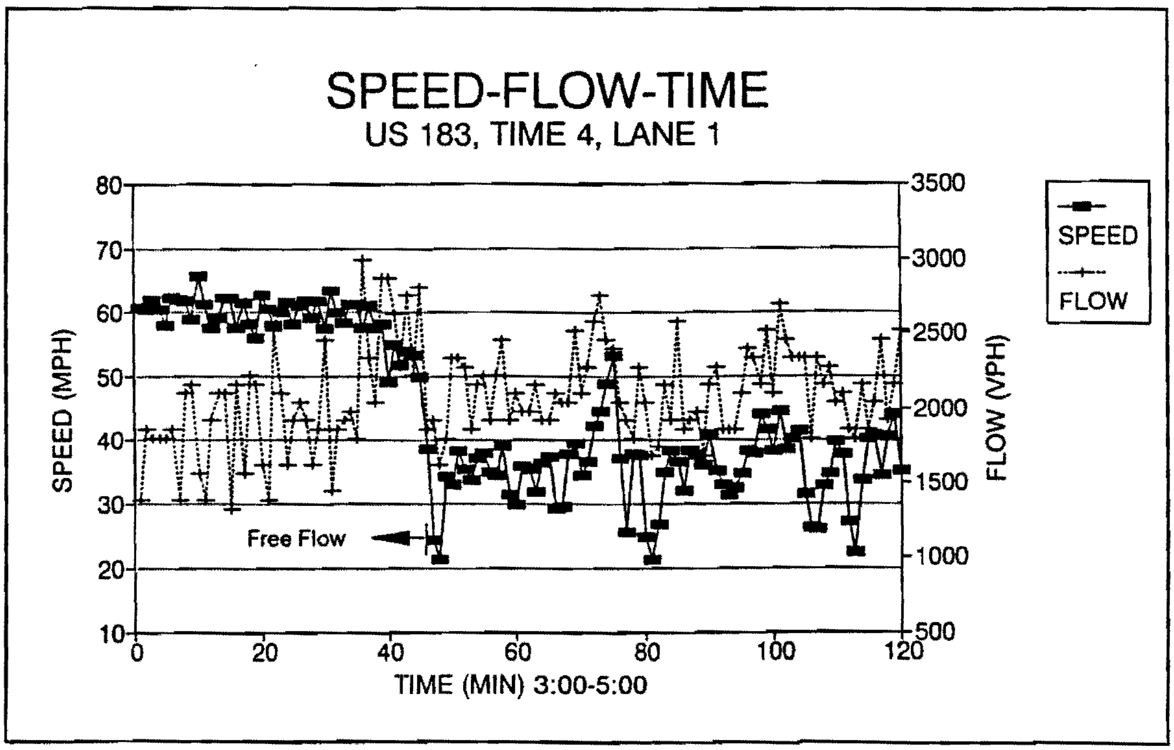

FIGURE C-30 Speed and flow vs time at US 183 for time 4, lane 1, using 1-minute intervals.

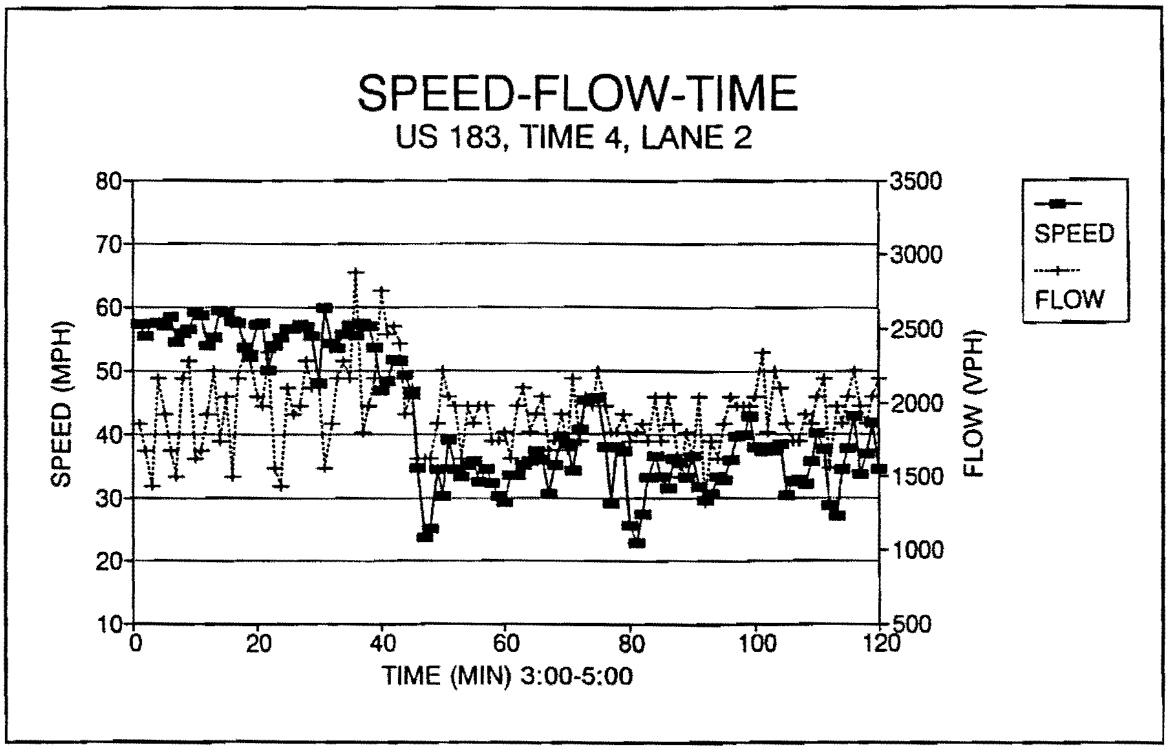

FIGURE C-31 Speed and flow vs time at US 183 for time 4, lane 2, using 1-minute intervals.

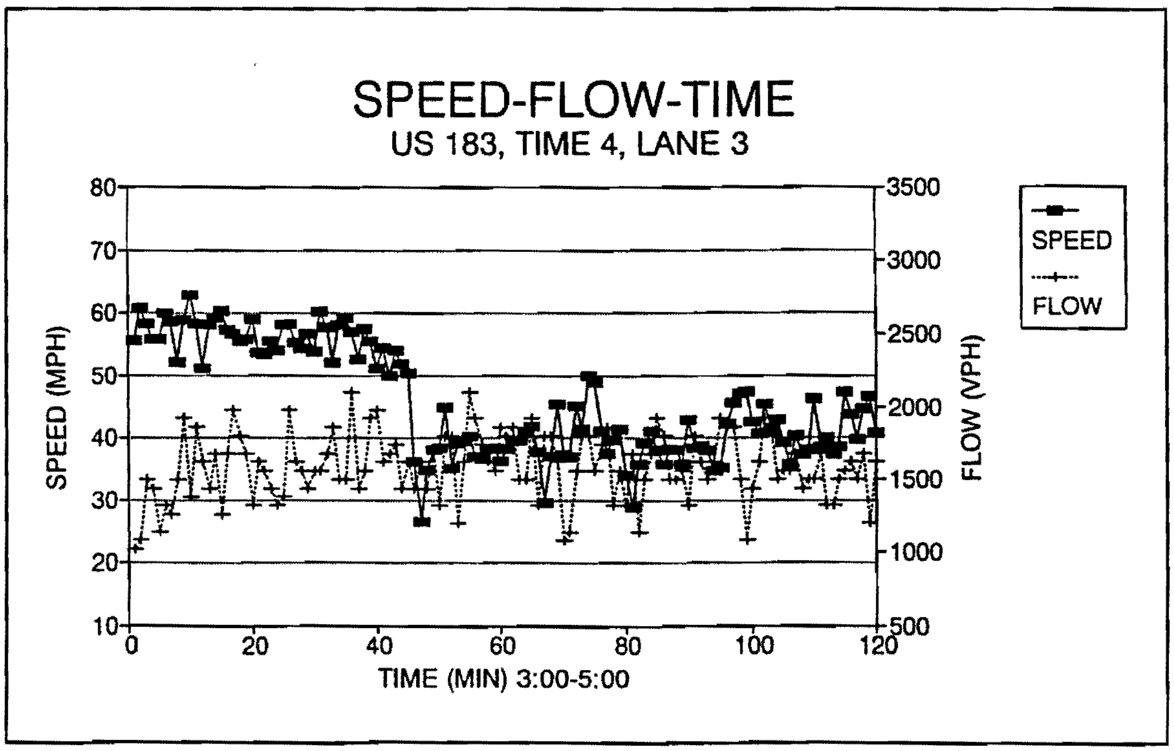

FIGURE C-32 Speed and flow vs time at US 183 for time 4, lane 3, using 1-minute intervals.