

METHODOLOGICAL FRAMEWORK FOR CAPACITY AND LEVEL-OF-SERVICE ANALYSIS OF FREEWAY SYSTEMS



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SUMMARY

This research represents the first phase of a project that is aimed at revising the Highway Capacity Manual (HCM) for Taiwan Area. The focus of this initial research effort is on freeway systems. The project is being conducted jointly by the author of this report and the Planning Division of the Institute of Transportation, Ministry of Transportation and Communications.

The HCM methodologies for capacity and level-of-service analysis have several drawbacks. First, the selected measures of effectiveness and the level-of-service criteria differ from one freeway component to another. As a result, it becomes impossible to compare the performances of different components. This, in turn, makes design consistency difficult to achieve in the planning and design of freeways. Second, the analytical models described in the manual for analysis of toll plazas are unrealistic. Third, the HCM methodologies are based on limited understanding of the flow characteristics on Taiwan's freeways, and they have not been adequately validated. And, finally, congested conditions are assigned a single level of service regardless of the degree of congestion. This makes it impossible to assess the relative severities of congestion for setting priorities for traffic improvement.

Recognizing these drawbacks, the current research effort is focused on developing an adequate database concerning the characteristics of freeway flows and on the development of a methodological framework to guide future research. For analysis, freeways are to be divided into mainline sections, toll plazas, and ramps. Mainline sections include weaving sections, ramp sections, tunnels, and basic sections that are not affected by ramps and weaving sections. Data have been collected at two toll plazas and three freeway sections. Additional data are being collected, or will be collected, at these and other sites.

To promote design consistency, all mainline sections and toll plazas will be evaluated in terms of the same level-of-service criteria. Space-mean speed and occupancy are two major measures of effectiveness being considered for establishing the mainline level-of-service criteria. Toll plazas and ramps, however, have their unique functions and operating characteristics. Therefore, additional measures of effectiveness and level-of-service criteria will be used for evaluating alternative designs and operations of these freeway components.

Toll plaza operations are very complex. Analytical models are often inadequate in describing the actual operations. Therefore, a computer simulation model, referred to as Toll Plaza Simulation Model (TPS), is developed. This model is to be used for operational

analysis of toll plazas; it may also be used for planning analysis if a detailed assessment of a toll plaza design is desired. A set of analytical models has also been developed for planning analysis and preliminary operational analysis. The average queue length and the average time vehicles spent in a toll collection system are suggested for use as the measures of effectiveness. The level-of-service criteria based on these two measures have been established. The procedures for planning analysis and operational analysis are illustrated in this report.

The development of methodologies for analyzing mainline sections and ramps has not been completed. The major task at the present time is to identify the flow characteristics on various mainline sections and ramps. The findings of this task will serve as the basis for establishing level-of-service criteria and for formulating detailed analysis procedures. The flow characteristics of several mainline sections and their implications on the development of analysis methodologies are discussed in this report.

1.0 INTRODUCTION

In 1990 the Institute of Transportation (IOT), Ministry of Transportation and Communications published the Highway Capacity Manual (HCM) for Taiwan Area [1]. This manual contains a variety of methodologies to assist in the planning, design, and operation of highway facilities. These methodologies reflect the state of the art of highway capacity analysis at the time. As the needs for highway capacity analysis evolve, it is felt that the existing methodologies will have difficulties addressing complex problems in highway capacity analysis. Therefore, the Planning Division of IOT initiated a research project in 1992 to revise the manual.

The first phase of the project is devoted to the development of improved methodologies for the analysis of freeways. The research tasks include: data collection and reduction; modeling traffic operations; and the development of methodological frameworks to guide future efforts in revising the HCM. The level-of-service concept being considered under this project has significant departures from existing ones.

The traffic operations on freeways are commonly classified into six levels of service (LOS): A, B, C, D, E, and F. LOS A represents a light flow condition under which there is little or no interaction among vehicles. At the other extreme, LOS F represents highly constrained traffic movement under congested conditions. Current LOS classification schemes, however, lack uniformity in several respects. For example, in Taiwan's HCM, LOS C represents an average travel speed of at least 53 kph for basic sections with a design speed of 100 kph and at least 60 kph for those sections with a design speed of 120 kph. In the U.S. HCM [2], LOS C implies an average travel speed of at least 69 kph for basic sections with a design speed of 80 kph and at least 86 kph for those with a design speed of 120 kph. At a given level of service, the traffic conditions specified in these manuals for basic sections also differ from those for ramp junctions. This use of different criteria to denote the same level of service makes it difficult to assess design consistency that is an important consideration in highway design.

Recognizing this drawback, the methodologies being developed will use a common set of criteria to facilitate system-wide comparisons of the levels of service of different components of freeway mainlines. Since freeways are intended for high-speed, uninterrupted movement, a logical measure of effectiveness to use is average vehicle speed.

On the other hand, the operating characteristics of the various freeway components have differences and cannot be adequately reflected by a single measure of effectiveness. Therefore, there is

also a need to establish criteria pertinent to each component. To satisfy this need, it is suggested that freeway systems be divided into the following components for level-of-service analysis:

- Mainline sections
 1. Basic sections beyond the influence of ramps or weaving sections
 2. Sections under the influence of ramps (ramp sections)
 3. Weaving sections
 4. Tunnels
- Toll plazas
- On-ramps
- Off-ramps

The function of the mainline sections is to carry vehicles at high speeds. Their levels of service may be defined by the same measures of effectiveness, such as average speed. At toll plazas, the vehicular traffic is interruptible. Therefore, the major level-of-service concern is not speed but other measures of effectiveness such as delay and queue length. Similarly, queue length and delay may also be more important than other measures of effectiveness for the evaluation of ramps.

Due to resources constraints, the current research effort is not intended to be all-inclusive. For example, weaving sections and tunnels are excluded from consideration. So far, a methodology for analyzing the operations of toll plazas has been developed. This methodology can address the toll collection methods being employed on Chung-San Freeway; it should be enhanced to deal with other types of gate operations. The development of methodologies for the analysis of other freeway components is an ongoing effort. Data are still being gathered for modeling the traffic flow characteristics on mainline sections. It is expected that the data collection will be completed by the end of 1993.

The objective of this report is to describe the characteristics of freeway flows and the methodological framework being considered. Chapter 2.0 presents a methodology for the analysis of toll plazas. Chapter 3.0 describes the characteristics of the flows on mainline sections and how such characteristics may be incorporated into an analysis framework. Chapter 4.0 discusses the conceptual framework being considered for analysis of ramps. Chapter 5.0 summarizes the findings.

2.0 TOLL PLAZAS

Toll plazas on freeway mainlines constitute potential bottlenecks, and the congestion brought about by them can be serious enough to warrant the consideration of alternative plaza designs and operating strategies. In Taiwan, the operations of the toll plazas on Chung-San Freeway have been the focus of public debate. Currently, there are ten mainline toll plazas on this 400-km long freeway. On national holidays, the traffic volumes on this freeway are so huge that the additional congestion created by toll collection has brought outcries from drivers and media. This, in turn, has exerted tremendous pressure on the government to search for remedies. Among the remedies that have been considered are waiving tolls on national holidays and installing electronic systems for automatic toll collection to minimize delays.

To assist in the planning and design of toll plazas, Taiwan's HCM has devoted a chapter to the level-of-service analysis of toll plazas. The methodology described in that chapter is based on classic queuing theory. It assumes random arrival of vehicles, random service time at toll gates, and first-come, first-served operation. This methodology leads to the formulation of several models that can be conveniently used to estimate average waiting time and average queue length. It, however, has a number of weaknesses.

First, although it is reasonable to assume that vehicles arrive at the toll plaza at random, it is generally erroneous to assume that the service times are random. For example, a toll gate serving small vehicles with exact changes has an average service time of about 4.6 sec per queuing vehicle. Based on this average service time, the assumption of random service time implies that approximately 35 percent of the service times would be less than 2 sec while, in fact, service times less than 2 sec are rare for vehicles in a queue. Second, the queuing models suggested in the manual are steady-state models; they assume that the same arrival rate persists indefinitely. This assumption makes it impossible for the resulting models to deal with variable arrival rates. It also leads to overestimates of delay and queue length when the arrival rate approaches the service rate. Third, the queuing models are mostly applicable only when the arrival rate is smaller than the service rate. When the arrival rate approaches or exceeds the service rate, the delay and queue length can become time-dependent. Queuing models are not suitable for such situations. Furthermore, queuing models are difficult to develop and to use for analyzing a toll plaza that has multiple gates with different operating characteristics.

Because of the dynamic operating characteristics of toll plazas, analytical models have difficulties satisfying the varying needs of

capacity analysis. Therefore, a simulation model, referred to as Toll Plaza Simulation (TPS) model, is developed to serve as the primary tool for analysis of toll plazas. For planning analysis, a high degree of accuracy in estimating the performance of a toll plaza may not be necessary. Therefore, a set of empirical models is also developed for planning applications.

2.1 Site Characteristics And Service Capacities of Toll Plazas

Fig.1 shows the general layout of the toll plazas on Chung-San Freeway. The number of gates at these plazas ranges from 5 to 10 for each direction of travel. In principle, the number of gates is equal to 2.5 times the number of the downstream freeway lanes. Each gate has a manned booth for toll collection. Immediately downstream of the booth in the travel lane is an inductive loop detector, which is linked to a computer in the adjacent administrative building to tally the number of departing vehicles. A weigh station is usually located downstream of the gates on a side road next to the right lane. Some weigh stations, however, are upstream of toll gates.

Currently, tolls on Chung-San Freeway are manually collected. Toll gates are classified into four types in accordance with vehicle type and method of payment. For convenience, these four types are referred to as Type 1, Type 2, Type 3, and Type 4, respectively. Type 1 gates serve small vehicles with either exact changes or prepaid toll tickets. Small vehicles include passenger cars, pick-up trucks, and other small commercial and recreational vehicles. Type 2 gates serve small vehicles with change transactions. Type 3 gates are for heavy single-unit trucks. Type 4 gates serve tractor-trailers and large buses. Type 4 gates are always located on the far right of a plaza in the direction of travel (e.g., Gate 1 in Fig. 1). Type 3 gates are next to the Type 4 gates. Type 1 gates are the inside gates (e.g., Gate 4 in Fig.1) and Type 2 gates lie between Type 1 and Type 3 gates. This arrangement minimizes the interference of traffic movement by heavy trucks that have to be weighed.

The service capacity of a toll gate may be defined as the maximum number of vehicles that can be expected to go through the gate in one hour under prevailing roadway and traffic conditions. Under light traffic conditions, a vehicle may arrive at a gate without being interrupted by a vehicle ahead. In such a case, the driver will decelerate, pull the vehicle alongside the toll booth to pay toll, and then accelerate to leave. The service time required to process such a vehicle through a gate may be defined as the elapsed time from the moment a driver is in a position to pay toll until the rear end of the vehicle crosses a reference line drawn from the booth. Fig.2 shows the cumulative frequency distributions of the service times of non-queuing vehicles observed at two toll plazas.

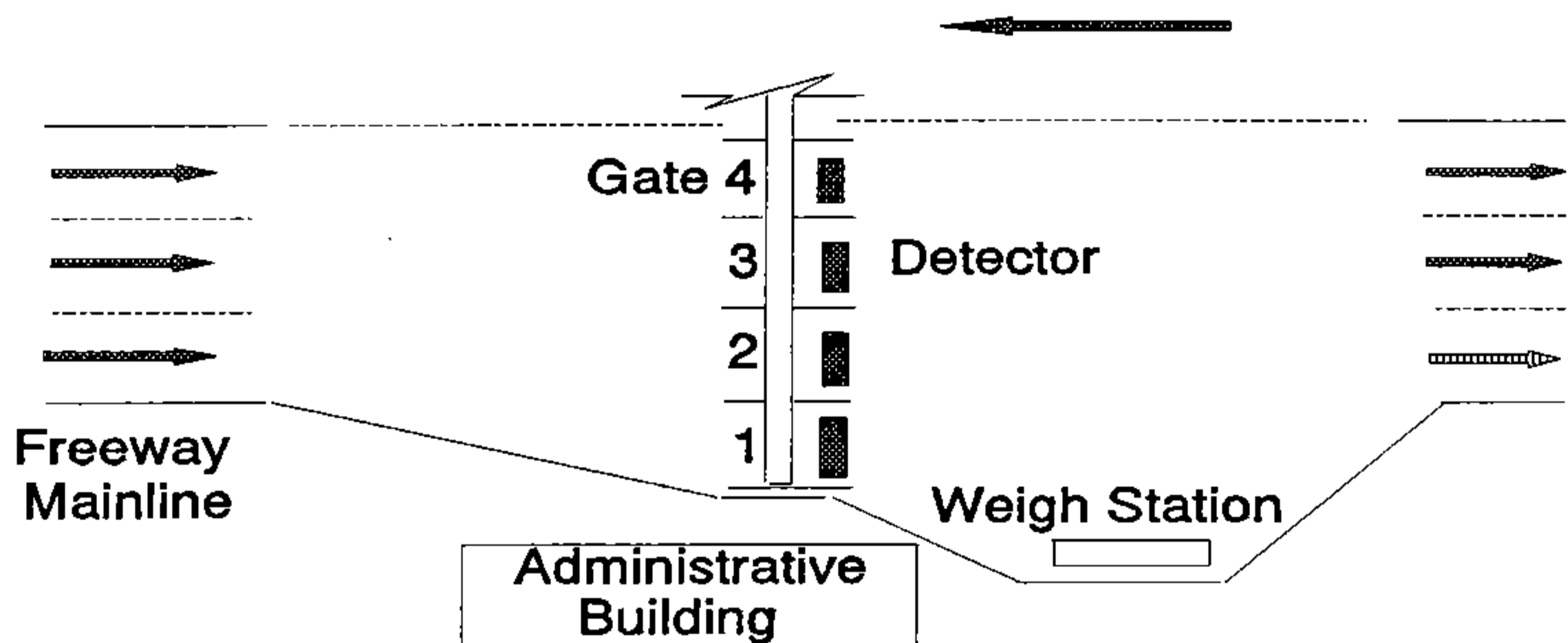


Fig. 1 Schematic of Toll Plaza Layout

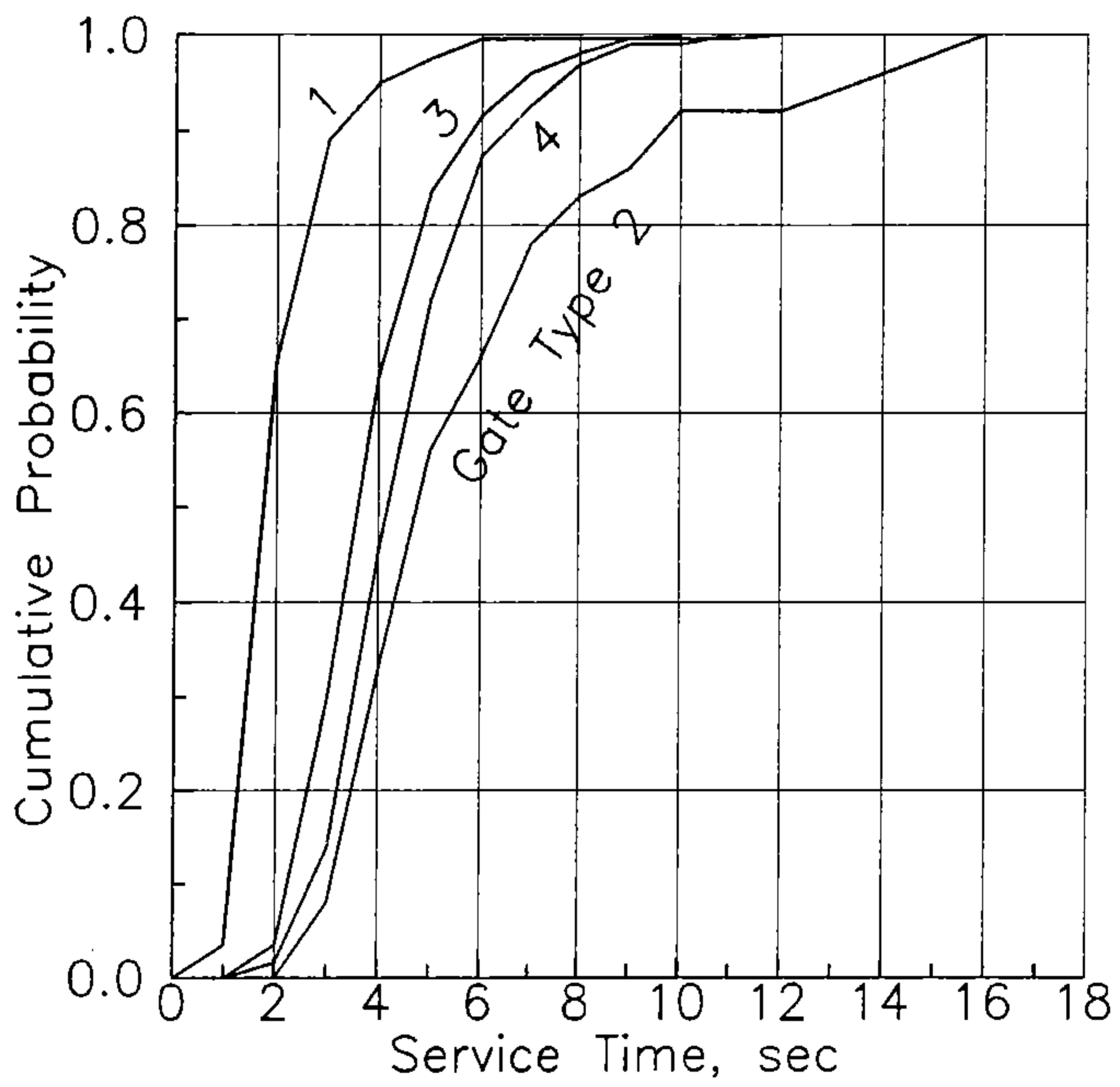


Fig. 2 Service Time Distributions of Vehicles not in Queue

These distributions have the following mean service times for the four types of gates mentioned above: 2.05 sec for Type 1, 4.86 sec for Type 2, 3.87 sec for Type 3, and 4.42 sec for Type 4.

The service times of non-queuing vehicles are not critical to the operation of a toll plaza because they are associated only with light traffic flow conditions. The service capacity of a toll gate is dictated by the time required to process queuing vehicles through the gate. Such service time of a queuing vehicle can be measured from the moment the rear end of the vehicle ahead crosses a reference line drawn from the toll booth until the moment the rear end of the subject vehicle crosses the same line. The cumulative frequency distributions of the service times of queuing vehicles observed at two plazas are shown in Fig.3. These distributions are constructed from data recorded on video tapes under day-time, fair weather conditions. The mean service times of these distributions are about 4.64 sec for Type 1 gates, 9.99 sec for Type 2 gates, 7.16 sec for Type 3 gates, and 7.34 sec for Type 4 gates. These service times are equivalent to a capacity of about 775 vph for Type 1 gates, 360 vph for Type 2 gates, 505 vph for Type 3 gates, and 490 vph for Type 4 gates. To provide a contrast, the capacities reported in a 1987 study [3] are 802 vph for Type 1 gates, 553 vph for Type 2 gates, 513 vph for Type 3 gates, and 455 vph for Type 4 gates. It is not clear why there is a difference of nearly 200 vph in the reported capacities of Type 2 gates. The sources of this discrepancy warrant investigation.

Time of day and weather conditions can also affect the capacities of toll gates. No field data have been collected under the current project to assess their effects. Based on the findings of the 1987 study [3] mentioned above, the capacity reduction factors given in Table 1 may be applied to the capacities observed under day-time, fair weather conditions.

For toll-free operation or automatic toll collection, vehicles are not required to stop at a gate. The presence of a gate, however, can impede traffic movement. Therefore, the gate capacity may be substantially lower than the capacity of a freeway lane upstream. With automatic toll collection, the experiences in Texas, California, and Virginia indicate that the capacity of a toll gate may be on the order of 1,200 to 1,400 vph [4]. The gate capacity under toll-freeway operation is unknown.

2.2 Toll Plaza Simulation (TPS) Model

The Toll Plaza Simulation Model, or TPS Model for short, is intended for personal computer applications. It is written in FORTRAN 77. To provide for maximum accuracy in duplicating the performance of a real-life system, the processing of vehicles in a simulation model should be time-advanced in that the movements of

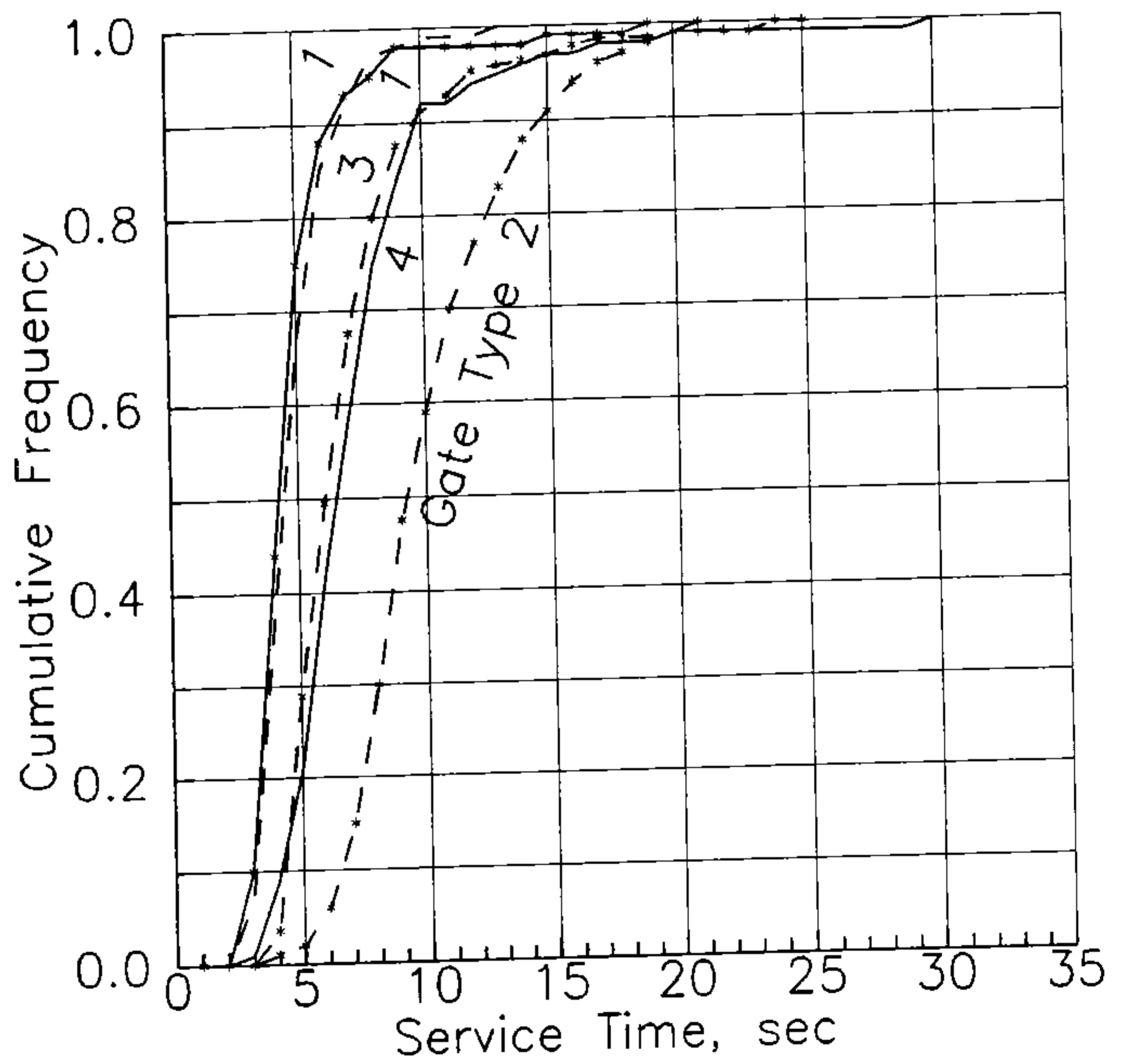


Fig. 3 Service Time Distributions of Vehicles in Queue

Table 1 Capacity Reductions under Nighttime
and Rainy Conditions

Type 1 Gates: Small Vehicles with Exact Changes
or Prepaid Toll Tickets

Fair Weather, Nighttime : 4 %
Rainy Weather, Daytime : 13 %
Rainy Weather, Nighttime : 23 %

Type 2 Gates: Small Vehicles without Exact Changes
or Prepaid Toll Tickets

Fair Weather, Nighttime : 4 %
Rainy Weather, Daytime : 6 %
Rainy Weather, Nighttime : 6 %

Type 3 Gates: Heavy Single-Unit Trucks

Fair Weather, Nighttime : 21 %
Rainy Weather, Daytime : 11 %

Type 4 Gates: Tractor-Trailers and Buses

Fair Weather, Nighttime : 7 %
Rainy Weather, Daytime : 4 %

individual vehicles are tracked and updated at short intervals (e.g., 1 sec). Time-advanced simulation, however, requires tremendous computational efforts that may limit the use of a model. Out of this concern, the processing of vehicles in the TPS Model is event-driven. Event-driven processing eliminates the need to update the movements of individual vehicles at short intervals. For the simulation of toll plaza operations, this gain in computational efficiency is obtained at the expense of a modest reduction in model flexibility and accuracy.

2.2.1 Model Structure

The basic structure of the TPS Model is shown in Fig.4. This model structure has eight major components. The key features of these components are described below.

Input Module

The Input Module allows one to define the characteristics of the

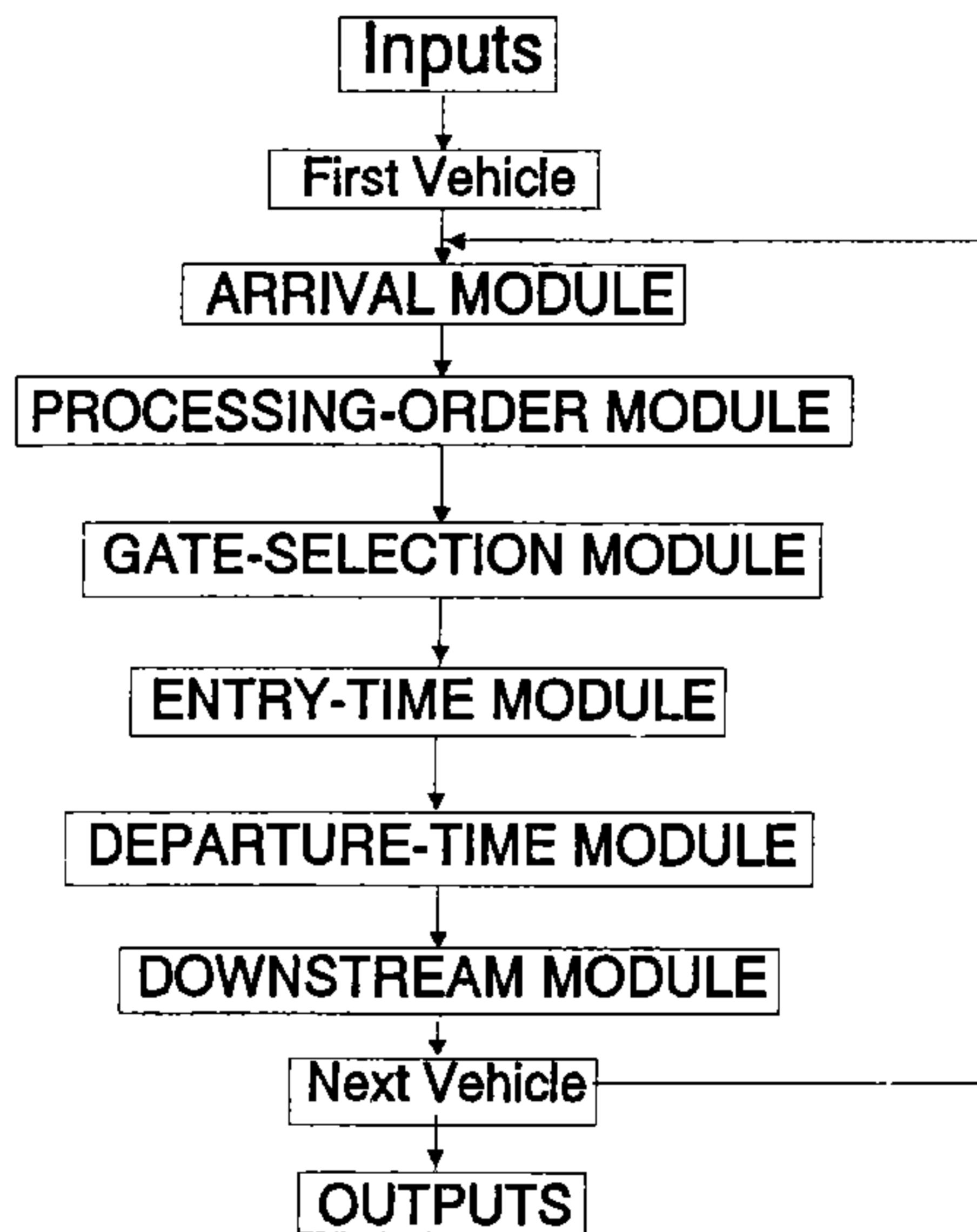


Fig. 4 Basic Structure of TPS Model

toll plaza being analyzed and to control the simulation process. The primary inputs are as follows:

1. Types of vehicle arrival patterns and service time distributions to be simulated

The model can be used to simulate toll-free operations, electronic automatic toll collection, and manual toll collection. Several theoretical and observed arrival patterns and service time distributions are embedded in the model. Available options include random arrival, uniform arrival, random service time, uniform service time, and the actual service time distributions shown in Figs. 2 and 3.

2. Simulation time and initialization period

Simulation time defines a duration of time in which toll plaza operations are to be simulated. At the start of the simulation, the toll plaza is empty. The initialization period is a time interval in which no data collection is made because the toll plaza may not have reached a representative state of operation.

3. Arrival flow rates

The model allows the simulation time be divided into small intervals, each of which may have a different flow rate in the freeway lanes upstream of the toll plaza.

4. Number of freeway lanes upstream of toll plaza

5. Number of gates and type of gates

6. Types of vehicles classified by gate type for each upstream freeway lane

7. Proportion and average length of each type of vehicles

8. Alignment of gates with respect to freeway mainline

9. Average vehicle speed in each freeway lane at the upstream end of the toll plaza

10. Capacity of each type of gates

11. Lengths of the upstream and the downstream sections of the plaza

12. Average free-flow travel time of each type of vehicles from gate to the downstream end of the plaza

13. Number of freeway lanes downstream of the plaza

14. Total capacity of the downstream freeway lanes at the junction between the plaza and the freeway mainline

15. Seed numbers for generating probabilistic events

Arrival Module

Given the inputs, the Arrival Module generates one arrival for each upstream freeway lane. The arrival times of the vehicles at the upstream end of the toll plaza in a given freeway lane are determined from arrival headways. For random arrivals, the headways can be assumed to conform to the following shifted negative exponential function:

$$f(h \geq z) = e^{-\frac{z - \tau}{H - \tau}} \quad (1)$$

where $f(h \geq z)$ = probability that a headway h is greater than or equal to z ; H = average headway which can vary from one interval to another within the specified simulation time; and τ = minimum headway between vehicles. This embedded arrival pattern can be easily changed should one decides to use other arrival patterns.

Each generated vehicle is identified with a vehicle type which is also determined probabilistically. To avoid unnecessary complications, the lengths and the approach speeds of the vehicles that belong to the same vehicle type are assumed to be the same.

Processing-Order Module

In the Processing-Order module, the arrival times of the vehicles generated for the upstream freeway lanes are compared. The vehicle that has the earliest arrival time is processed first. After this vehicle is processed, another vehicle is generated in its place and a comparison of the arrival times of the generated vehicles that have not been processed are again compared.

Gate-Selection Module

Once a vehicle is chosen for processing, the Gate-Selection Module begins to identify the gate to be used by this vehicle. To provide for realistic simulation, video cameras were used to record the gate-selection behavior of drivers. The video data reveal that the drivers in a given freeway lane prefer to use the gate directly downstream of that lane so that lane changes both upstream and downstream of the gate can be avoided. It was often observed that the queue lengths at two adjacent gates have to differ by four or more vehicles before an arriving driver would change lane and go to the gate with a shorter queue. Based on the observed behavior, the gate selection behavior is simulated by comparing the equivalent

queue lengths at available gates. An arriving driver is assigned to the available gate that has the shortest equivalent queue length. The equivalent queue lengths are determined as follows:

1. A gate that is directly downstream of the freeway lane in which an arriving vehicle is traveling is referred to as preferred gate.
2. For a preferred gate, the equivalent queue length is the actual queue length at that gate.
3. For other gates, the equivalent queue length is determined as

$$Q_e = Q_a + (2 + 1.5 \kappa) (0.8 + 0.4 R) \epsilon \quad (2)$$

where Q_e = equivalent queue length, in vehicles; Q_a = actual queue length, in vehicles; κ = distance between the gate being considered and the preferred gate, in gates; and R = a uniformly distributed random number that characterizes the arriving driver; and $\epsilon = 1.0$ for Type 1 and Type 2 gates and 0.4 for Type 3 and Type 4 gates.

The above equation implies that a deviation from the preferred gate by one gate will increase the equivalent queue length by an average of 3.5 vehicles for Type 1 and Type 2 gates. It also implies that a deviation by another gate will increase the equivalent queue length by an additional 1.5 vehicles for the same two types of gates.

Entry-Time Module

This module is for the purpose of determining when a vehicle enters the toll collection system. For vehicles not interfered with by a queue of vehicles, their entry times are considered to be the times at which they arrive at the toll booths. For vehicles that have to join a queue, their entry times are the times at which they become part of the queue.

For a given arrival, the first task of this module is to analyze the downstream conditions and determines whether or not the arriving vehicle has to join a queue. The estimation of the entry time of an arriving vehicle, that is not interfered with, can be readily estimated from its approach speed and deceleration rate. The entry times of queuing vehicles in an event-driven simulation process are difficult to estimate. In the TPS model, such entry times are estimated according to an approximation procedure illustrated in Fig. 5. In this figure, T_1 is the time when a vehicle reaches the upstream end of the plaza, T_2 is the departure time of

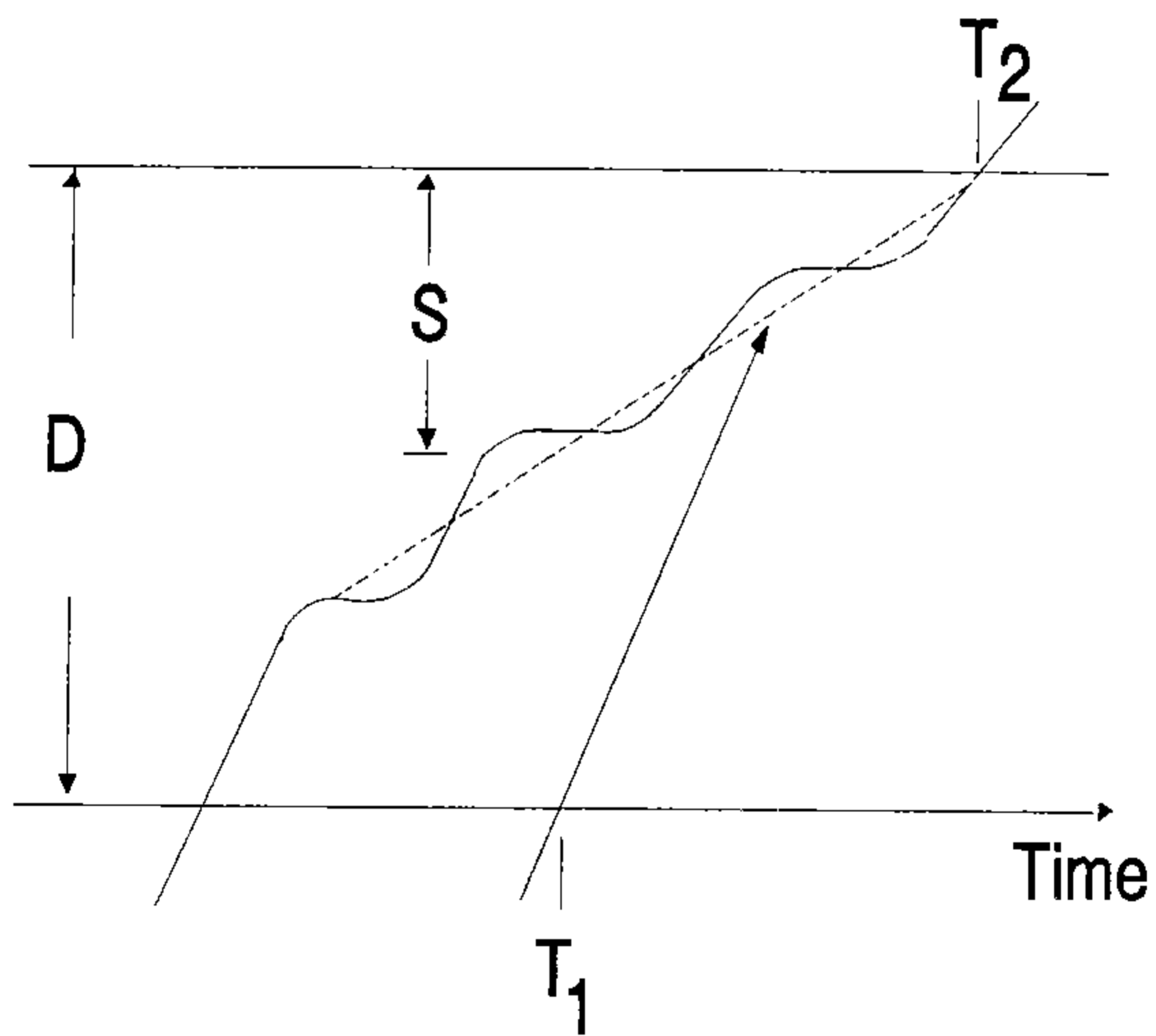


Fig. 5 Determination of Entry Time

the last queuing vehicle, and D is the distance between the gate and the upstream end of the plaza. To estimate the entry time of the arriving vehicle, the module estimates the distance, S , between the gate and the last queuing vehicle at time T_1 first. Over this distance, the trajectory of the last queuing vehicle is expected to follow a zigzag pattern. This zigzag trajectory is approximated by a straight-line trajectory. The intersection between the trajectory of the arriving vehicle and that of the last queuing vehicle is used to define the entry time of the arriving vehicle.

Departure-Time Module

This module is for the purpose of providing a preliminary estimate of the departure time of a vehicle from a gate. The estimation of departure times in this module is based on the assumption that the downstream section of the plaza does not have queuing vehicles that will hinder the departure of a vehicle from a gate. The departure times estimated in this manner are subject to change if the presence of queues downstream of the gate interferes with the departures from a gate. Modification of departure times is made in the Downstream Module which will be described later.

The departure time of a vehicle not interfered with by queuing vehicles is determined as the sum of its entry time (i.e., time of arrival at the booth) and service time. For actual manual toll collection, the service time of an uninterrupted vehicle is generated probabilistically according to the cumulative distributions shown in Fig.2.

For a vehicle that is in a queue, its departure time is determined as the sum of its service time and the departure time of the vehicle in front. For actual manual toll collection, the service time of a queuing vehicle is also generated probabilistically according to the distributions shown in Fig. 3.

Downstream Module

The traffic operations downstream of the gates depend on such factors as the rate at which vehicles are released from the gates, vehicle mix, the alignment of gates with respect to the freeway mainline downstream, and the capacity of the freeway lanes at the downstream end of the plaza. The formation of queues downstream of the gates may affect service time. To reflect this possibility, the TPS model checks the downstream conditions immediately after the preliminary departure time of a vehicle has been determined. If a long queue exists downstream and the queue interferes with the departure of a vehicle from a gate, the departure time of the affected vehicle is revised.

Output Module

The primary outputs of the TPS model include the following items:

1. Generated flow rates for each upstream freeway lane
2. Flow rate accommodated by each gate and its related average approach delay, average time in system, average queue length, and maximum queue length.

Approach delay is measured as the difference between the actual departure time of a vehicle and the corresponding expected departure time. The expected departure time is the time of departure from a gate if an arriving vehicle can proceed through a toll plaza without changing its approach speed. Time in system is the difference between entry time and departure time. Since the movement of a vehicle after entering a toll collection system is characterized by low speed stop-and-go maneuvers, time in system can be treated as the equivalent of stopped delay. Queue length refers to the number of vehicles in a queue, which includes the vehicle being served at a toll gate and the vehicles waiting behind. Average queue length is the average number of queuing vehicles at any moment in time.

3. Average upstream travel time and speed.

Upstream travel time is measured from the time a vehicle reaches the upstream end of the plaza until the vehicle leaves the gate. The average upstream speed is the length of the section of the plaza upstream of the gates divided by the average travel time.

4. Holding capacity of the downstream section of the plaza in term of the number of queuing vehicles that can be stored in the downstream section of the plaza.
5. Average ratio of the number of queuing vehicles in the downstream section to the holding capacity.
6. Time series of the ratio of the number of queuing vehicles to holding capacity.
7. Average speed through plaza.

2.2.2 Testing of Model

The TPS model was tested in several respects against field observations and theoretical expectations. The field data used for the testing were collected at a toll plaza for a Type 2 gate. The data were recorded on a video tape and then reduced to show the

number of arrivals, the number of departures, and the queue lengths at 10-sec intervals. In testing the model, the arrivals were generated according to the observed number of arrivals in each 10-sec interval. These arrivals were then processed through the simulated gate in accordance with the internal logics of the model. Fig.6 gives a comparison of the simulated and the observed queue lengths at 10-sec intervals. The simulated queue lengths differ from the observed values by at most 3 vehicles; the average deviation from the observed values is 1.26 vehicles. The corresponding average time in system is 149.5 sec/veh for the observed operation and 144.5 sec/veh for the simulated one.

As shown in Fig. 6, the simulated queue lengths track the observed values reasonably well over time. The discrepancies between the observed and the simulated queue lengths are largely attributable to the fact that no attempt has been made in the test to match the simulated service times of individual vehicles with the observed values. As a result, the accumulated number of simulated departures at a given point in time may differ from the observed value. The largest difference is 3 vehicles and the average difference is 1.12 vehicles per 10-sec interval. Nevertheless, Fig. 7 shows that the accumulated number of simulated departures follow closely the observed values over time. If the observed service times were used in the test, much of the deviation of the simulated Fig.6 queue lengths from the observed values would have disappeared.

Because of the difficulties in collecting and reducing field data for comprehensive testing of the model, analytical queuing models were also relied upon to test the TPS model. For a single-gate operation with random arrivals, random service time, and first-come, first-served operation, it can be shown [5] that the average time in system and the average number of vehicles in system (i.e., average sum of the number of vehicles in queue and the number of vehicles being served) can respectively determined as

$$T = \frac{1}{C - \lambda} \quad (3)$$

and

$$L = \frac{\lambda}{C - \lambda} \quad (4)$$

where T = average time in system for each arriving vehicle; L = average number of vehicles in system, i.e., average queue length as defined herein; λ = arrival flow rate; and C = service capacity of toll gate.

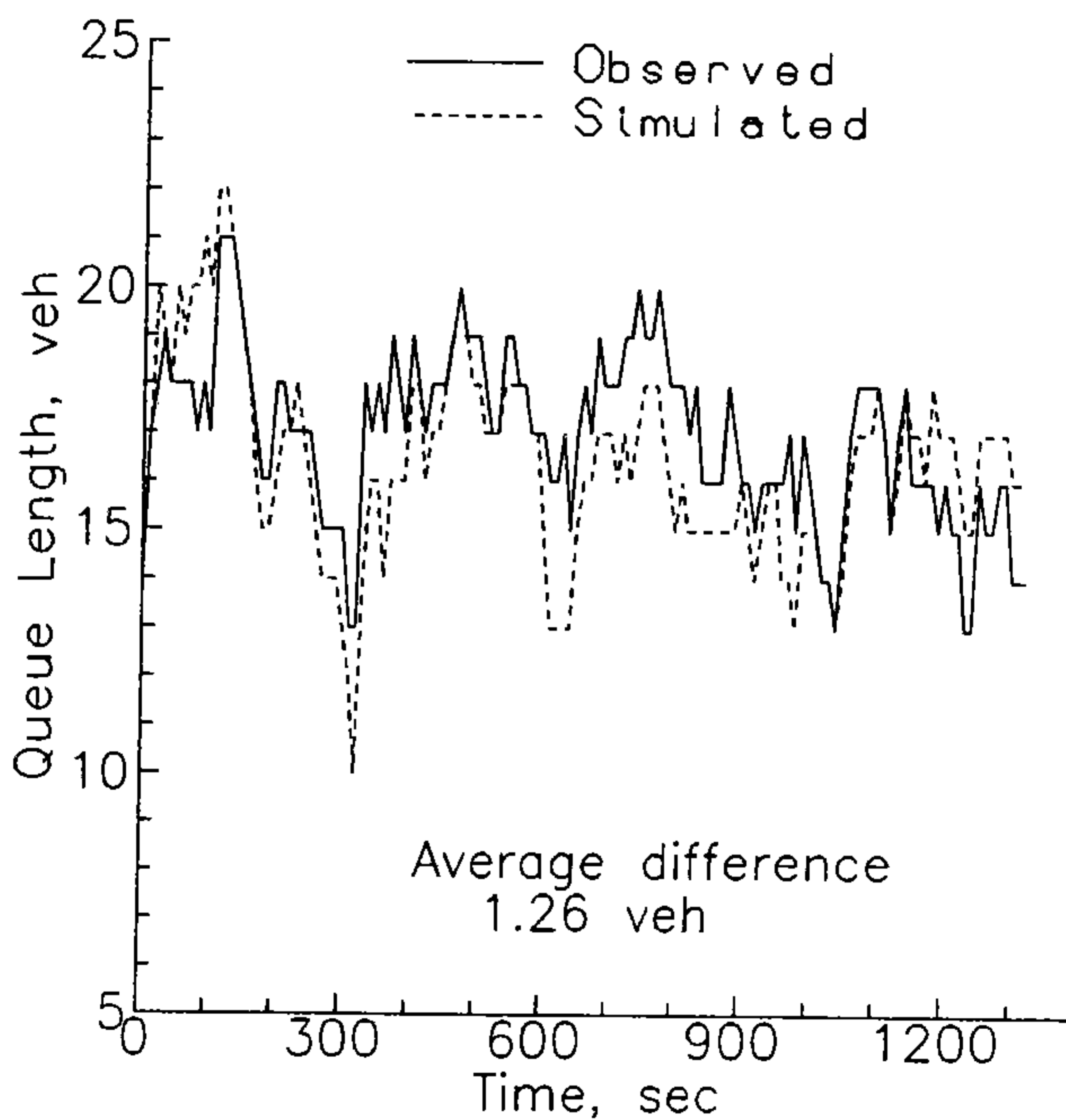


Fig. 6 Comparison of Simulated and Observed Queue Length

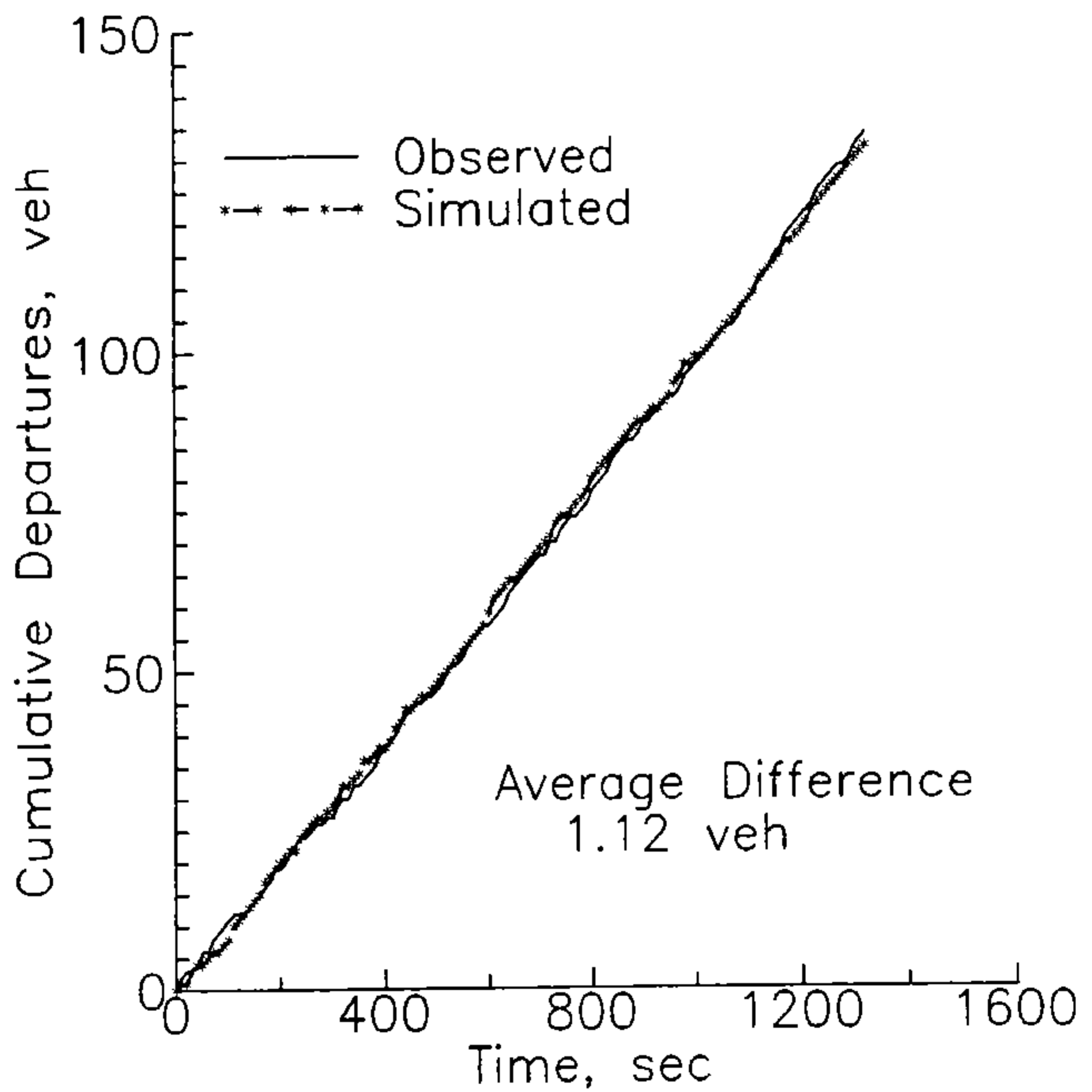


Fig. 7 Simulated and Observed Cumulative Departures from Gate

To test against the analytical model, the TPS model was used to simulate the toll collection operation for 45 minutes. Figs. 8 and 9 show respectively the simulated times and numbers in system in comparison with the theoretical values at two levels of gate capacity. These figures show that the simulated values match very well with the observed values until the arrival rate approaches the service capacity of a gate. When the arrival flow rate approaches the capacity of a gate, the time and number in system as estimated from the analytical models approach infinity. By comparison, the simulated values are much smaller. These discrepancies at the high end of arrival flow rate are due to the fact that the analytical models assume that the arrival rate persists indefinitely while the simulated operation lasts only for 45 minutes. If a longer simulation period is used, the simulated time and number in system will become closer to the analytical values.

2.3 Performance Characteristics of Gate Operations

The TPS model is used to analyze the performance characteristics of the four types of gates currently in use on Chung-San Freeway. The purpose of this analysis is to generate a knowledge base for selecting the level-of-service criteria. The analysis conducted in this study is based primarily on flow patterns with constant flow rate and random arrivals.

2.3.1 Stability of Performance

The traffic operation at a toll gate can be classified into the following three states: stable, metastable, and unstable. These three states are illustrated in Fig.10 in terms of 6 simulated time series of the average queue length at a Type 2 gate that has a capacity of 360 vph. The simulated arrival flow rates are 300 vph for series 1 and 2, 345 vph for series 3 and 4, and 400 vph for series 5 and 6.

In a stable state, the average queue length and other measures of effectiveness are primarily a function of the arrival flow rate; they are independent of the sequence of arrival headways and the duration in which a given flow rate persists. Under such a condition the measures of effectiveness estimated from two simulation runs, that involve the same flow rate but have different sequences of arrival headways, tend to converge and remain stable over time as long as the flow rate remains unchanged. As a result, flow rate alone can be used to estimate accurately such measures of effectiveness as delay and queue length for a given toll gate. This characteristic is illustrated by time series 1 and 2 in Fig. 10. With multiple gates available, a stable state can usually be maintained when the ratio of arrival volume to gate capacity (V/C ratio) is about 0.93 or lower. With only one gate available, a stable state is associated with V/C ratio of under 0.9.

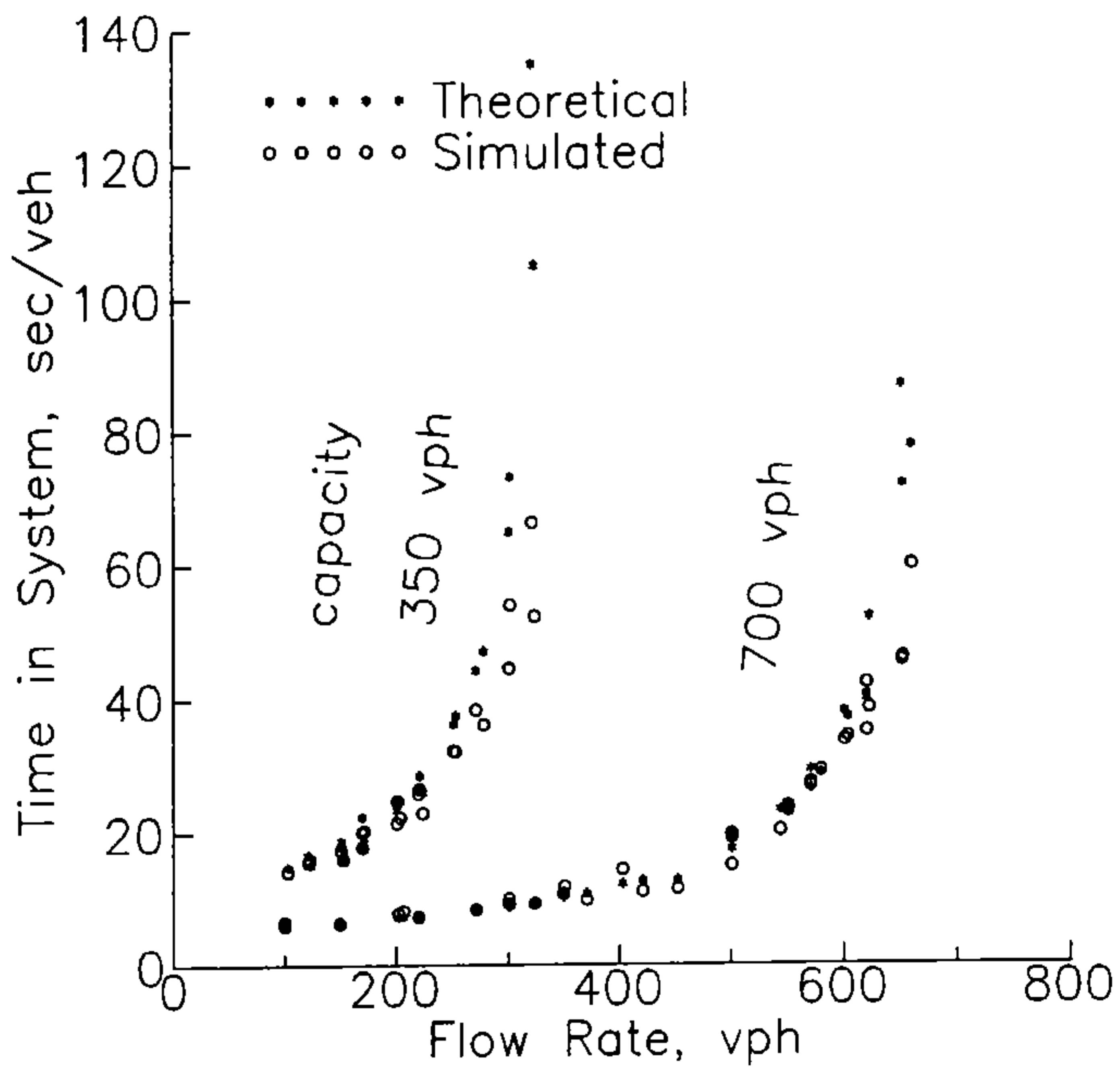


Fig. 8 Comparison of Simulated and Theoretical Times in System

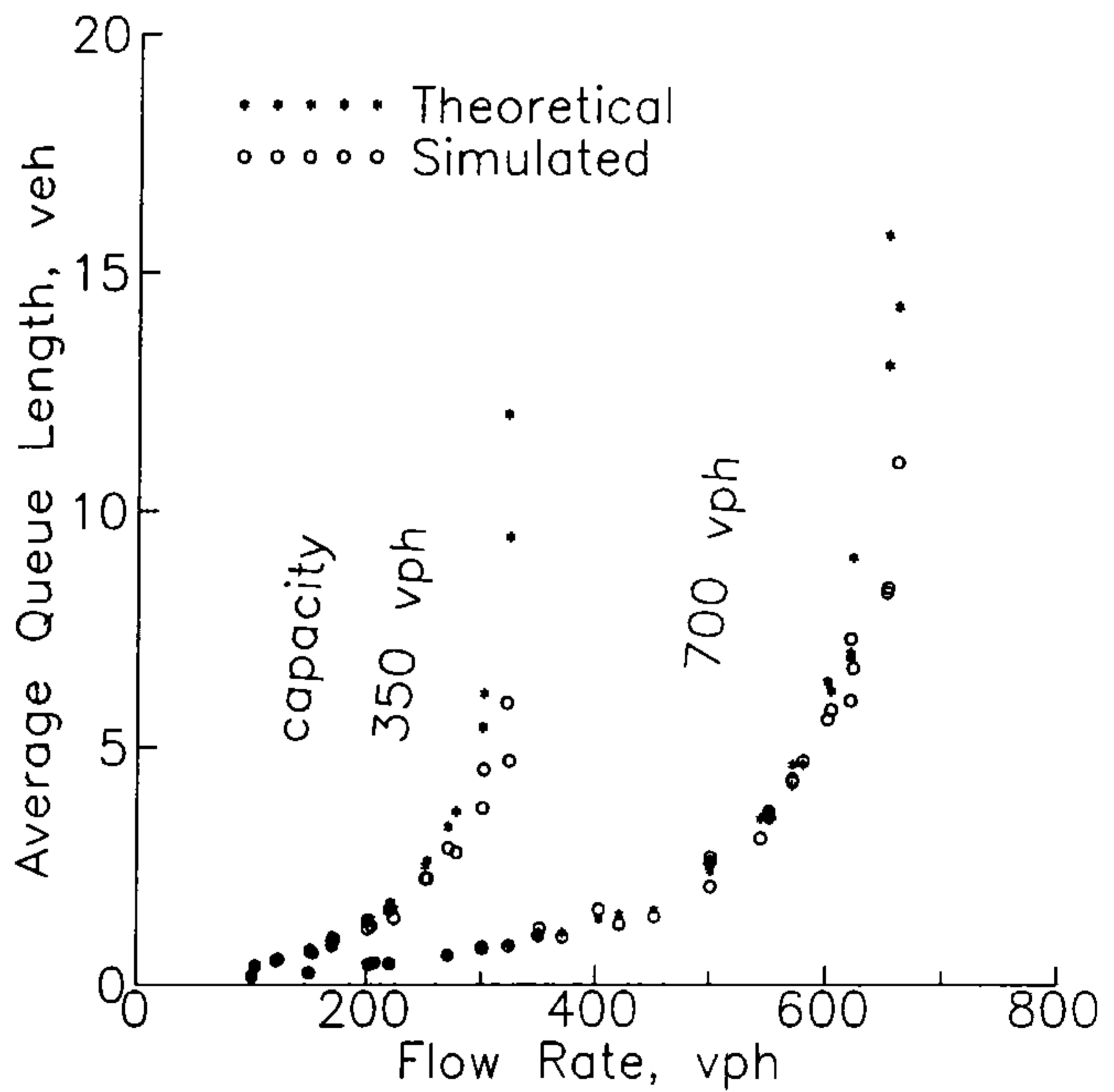


Fig. 9 Comparison of Simulated and Theoretical Queue Lengths

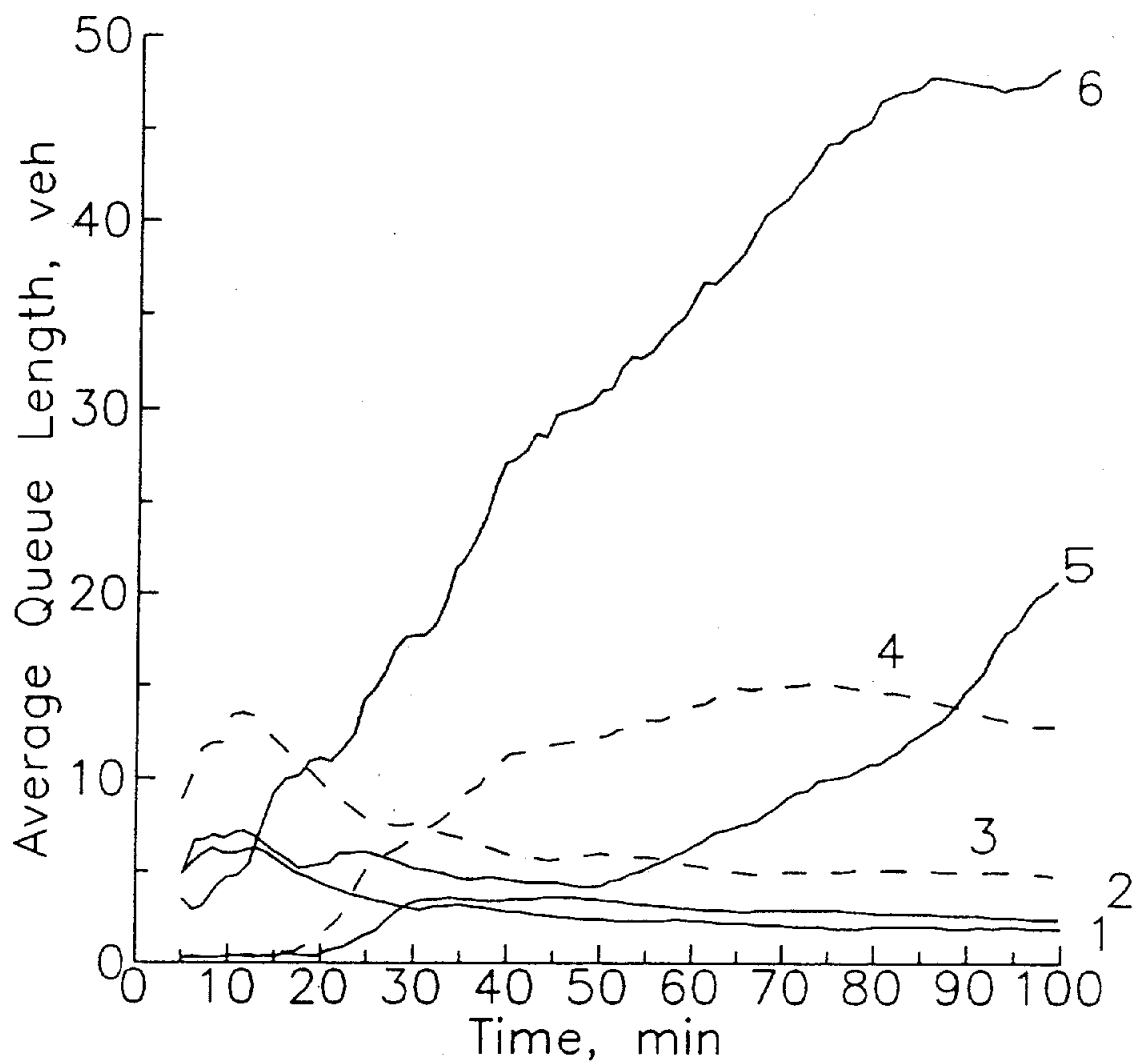


Fig. 10 States of Toll Plaza Operations

As arrival flow rate increases and approaches the capacity of a toll gate, the traffic operation may enter a metastable state. In a metastable state, traffic operation depends not only on arrival flow rate but also on the sequence of the headways of the arriving vehicles. In other words, significantly different performances may exist at the same level of flow rate if the sequences of the arrival headways are different. For a given sequence of arrival headways, however, the traffic operation can still remain stable over time. Series 3 and 4 of Fig. 10 show the nature of this state. A metastable state is often associated with a V/C ratio in the range of 0.93 to 0.97 for multiple-gate operation and between 0.9 and 0.94 for single-gate operation.

At a higher V/C ratio, the performance of a toll gate depends not only on arrival flow rate and the sequence of arrival headways but also on the duration in which the arrival rate persists. In this unstable state, queue length grows over time. Series 5 and 6 of Fig.10 are representative of this nature of gate performance.

Fig.11 shows how the average queue length changes as the arrival flow rate at a Type 1 gate (capacity: 775 vph) increases. Based on simulation data such as those shown in this figure, Figs. 12, 13, 14, and 15 are developed for the four types of toll gates on Chung-San Freeway. These figures are based on flows approaching two available gates at a constant rate, but they also approximate the operations involving more than two gates. The average queue lengths given in these figures can be estimated as

$$L \approx 0 \quad \text{if } \frac{V}{C} \leq 0.5 \quad (5)$$

$$L = 7 \frac{V}{C} - 3.5 \quad \text{if } 0.5 < \frac{V}{C} \leq 0.93 \quad (6)$$

and

$$L = 3 \left[1 + 6.29 \left(\frac{V}{C} - 0.93 \right) \left(\frac{C}{360} - 1 \right) \right] \left[1 + \left(14 \frac{V}{C} - 13 \right)^2 t \right] \quad \text{if } \frac{V}{C} > 0.93 \quad (7)$$

where t is the flow duration, in hours.

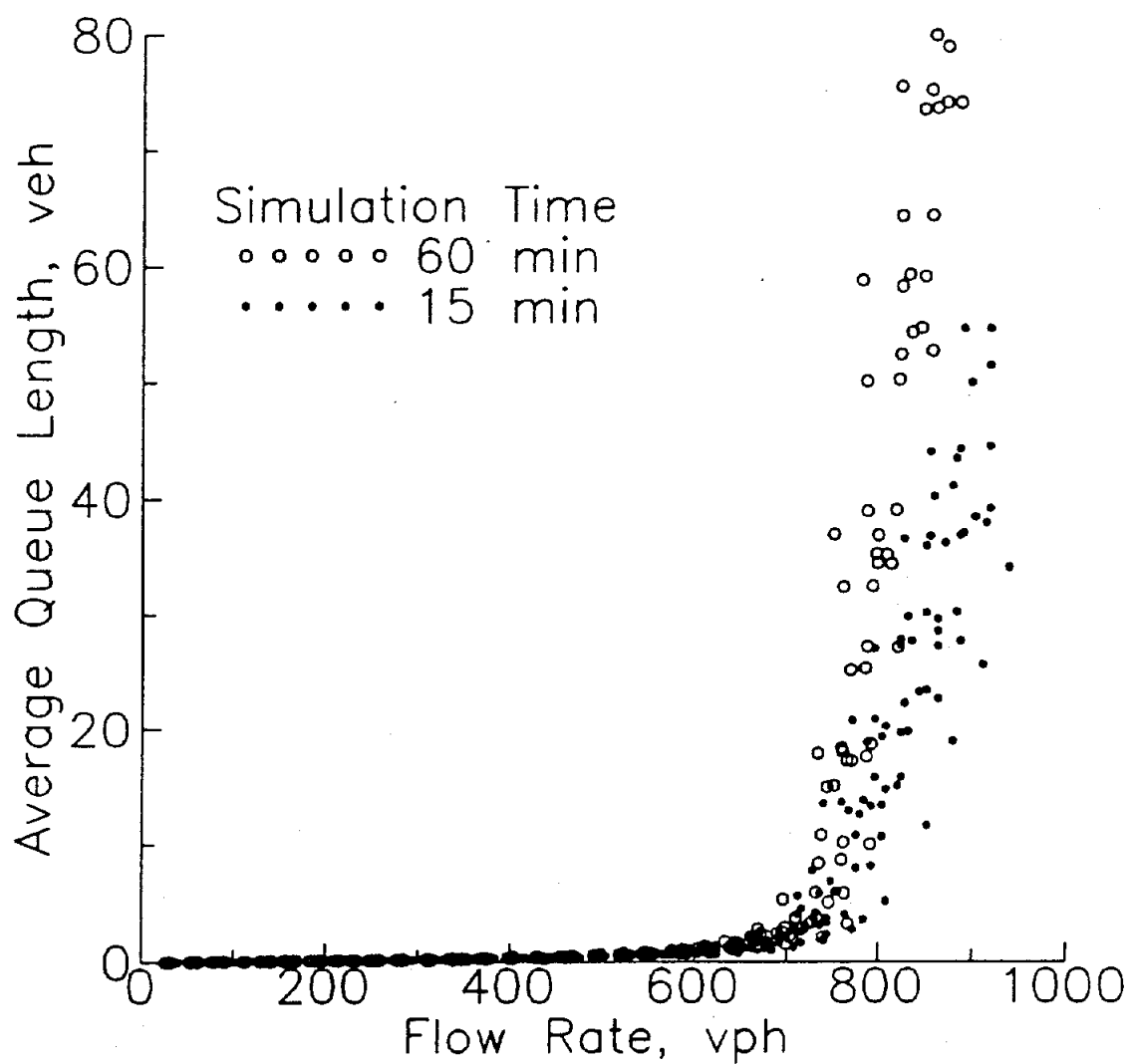


Fig. 11 Variation of Queue Length with Flow Rate and Flow Duration

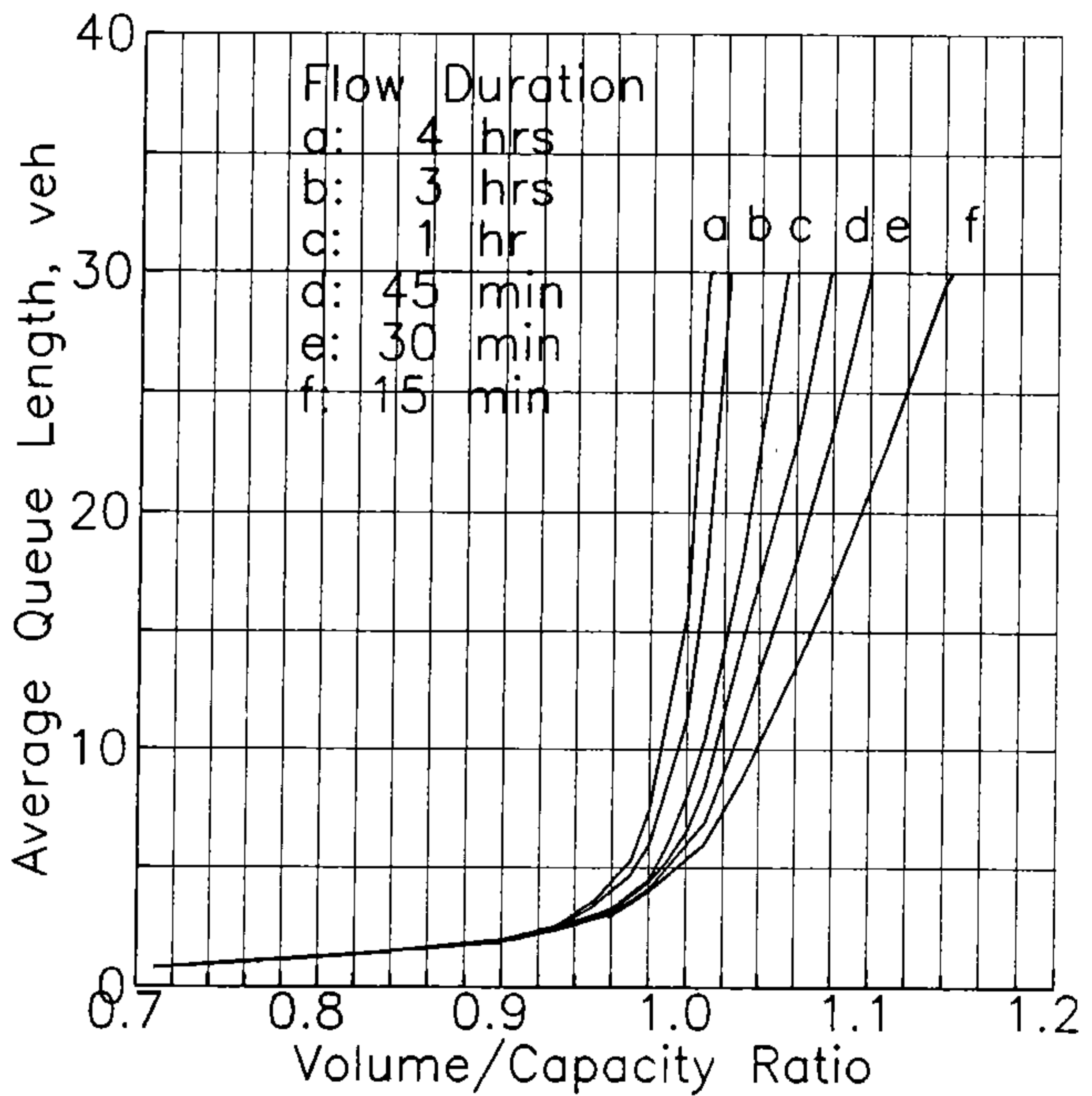


Fig. 12 Average Queue Length as a Function of V/C Ratio
(Multiple Type 1 Gates)

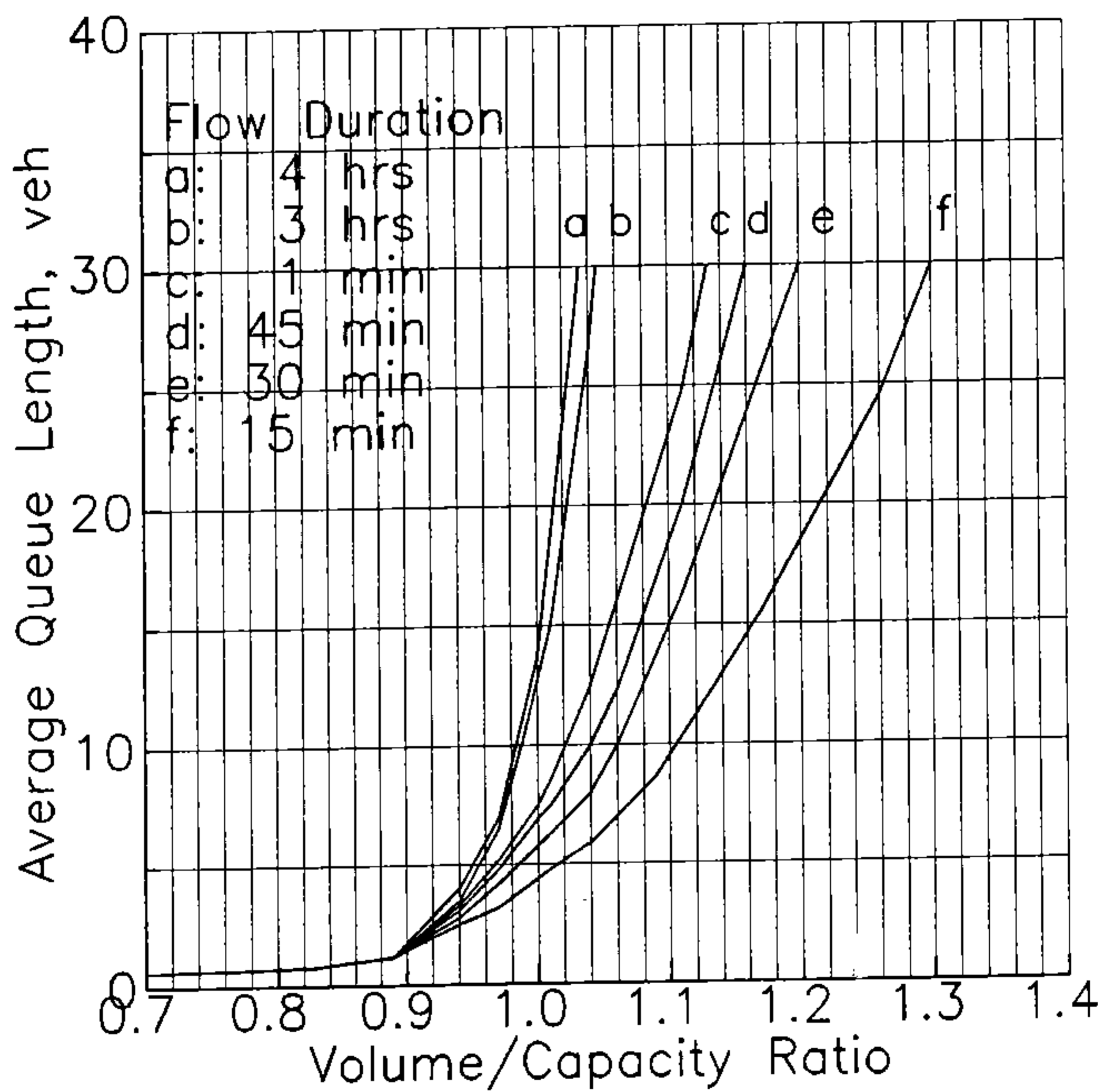


Fig. 13 Average Queue Length as a Function of V/C Ratio
(Multiple Type 2 Gates)

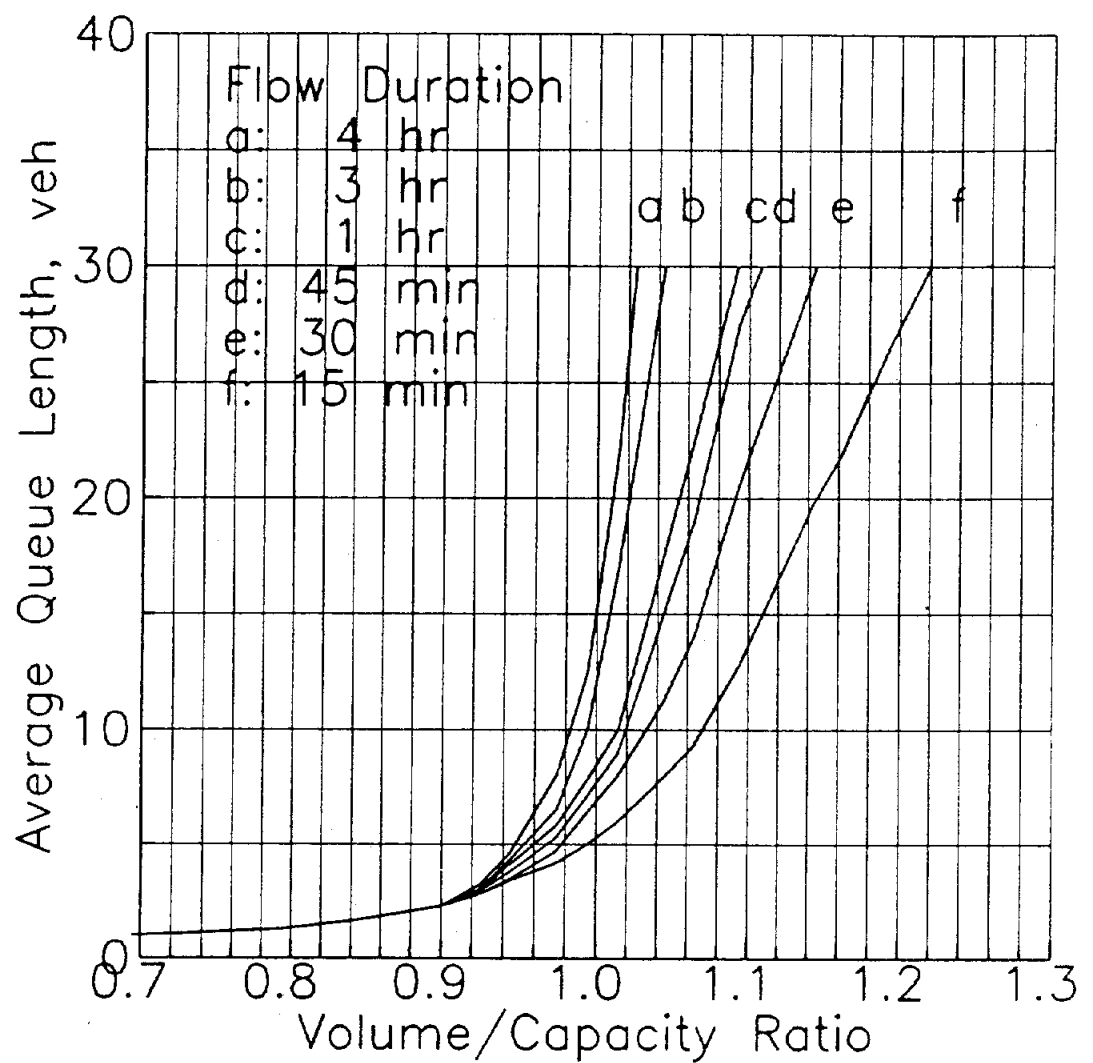


Fig. 14 Average Queue Length as a Function of V/C Ratio
(Multiple Type 3 Gates)

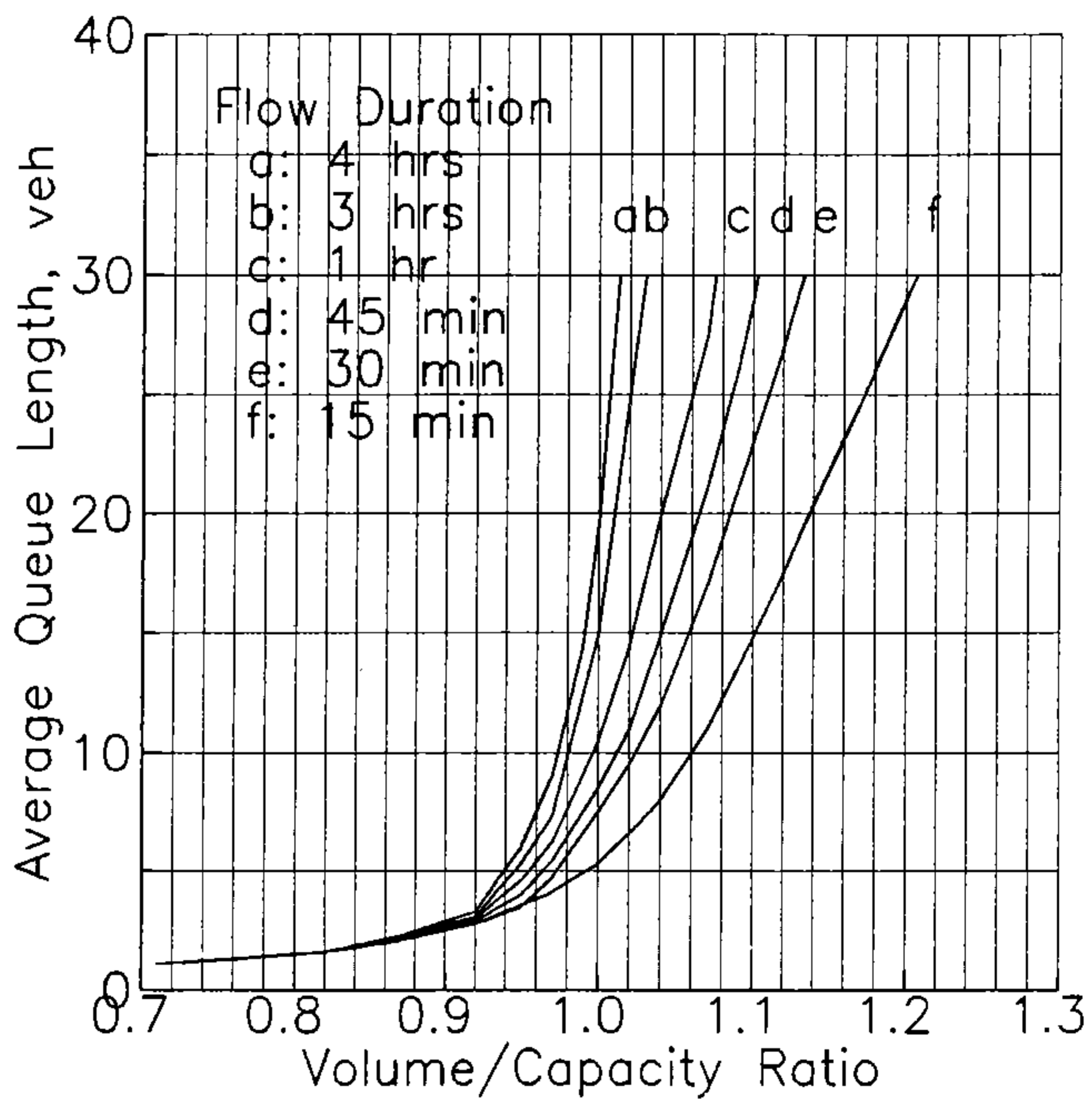


Fig. 15 Average Queue Length as a Function of V/C Ratio
(Multiple Type 4 Gates)

Average queue length is a random variable. The simulation data shows that the average queue length at a toll gate differs from its 85th percentile value by approximately one standard deviation. The relationship between the average queue length and its corresponding 85th percentile length is shown in Fig.16. This relationship can be represented by the following equations:

$$L_{85} = 1.28 L \quad \text{if } L \leq 15 \quad (8)$$

and

$$L_{85} = 3.9 + 1.08 L \quad \text{if } L > 15 \quad (9)$$

where L = average queue length, in veh and L_{85} = 85th percentile value of average queue length, in veh.

Maximum queue length, which is an important consideration in the geometric design of a toll plaza, can be much longer than average queue length. Fig.17 shows the relationship between average queue length and maximum queue length. For planning purposes, the following relationship may be used to estimate the maximum queue length L_{\max} from a given average queue length L :

$$L_{\max} = 7 + 1.7 L \quad \text{if } L \leq 10 \quad (10)$$

and

$$L_{\max} = 11 + 1.3 L \quad \text{if } L > 10 \quad (11)$$

These equations represent the solid lines in Fig. 17 and are intended to give the expected value of the maximum queue length for a given average queue length.

2.3.2 Effects of Number of Gates for a Given Gate Type

When multiple gates are available to a given class of vehicles, the arriving drivers have the option to use the gates that have shorter queues. This flexibility lessens the detrimental impact of random arrivals and allows each gate to be more efficiently utilized than when only one gate is available. Figs. 18 and 19, which are based respectively on Type 1 and Type 2 gates, highlight this phenomenon. For operations involving more than two gates of the same type, the

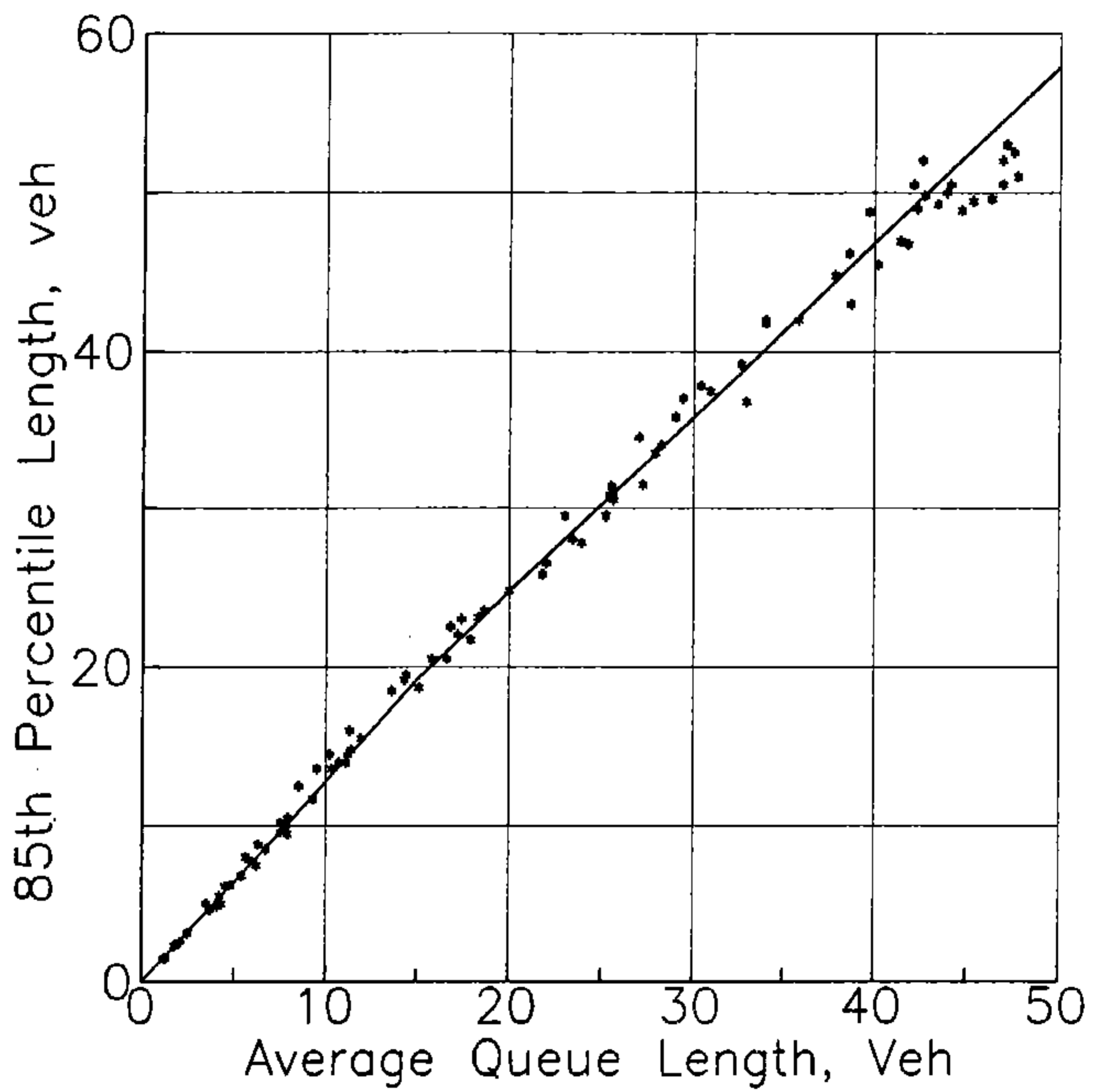


Fig. 16 Relationship between Average Queue Length and Its 85th Percentile Value

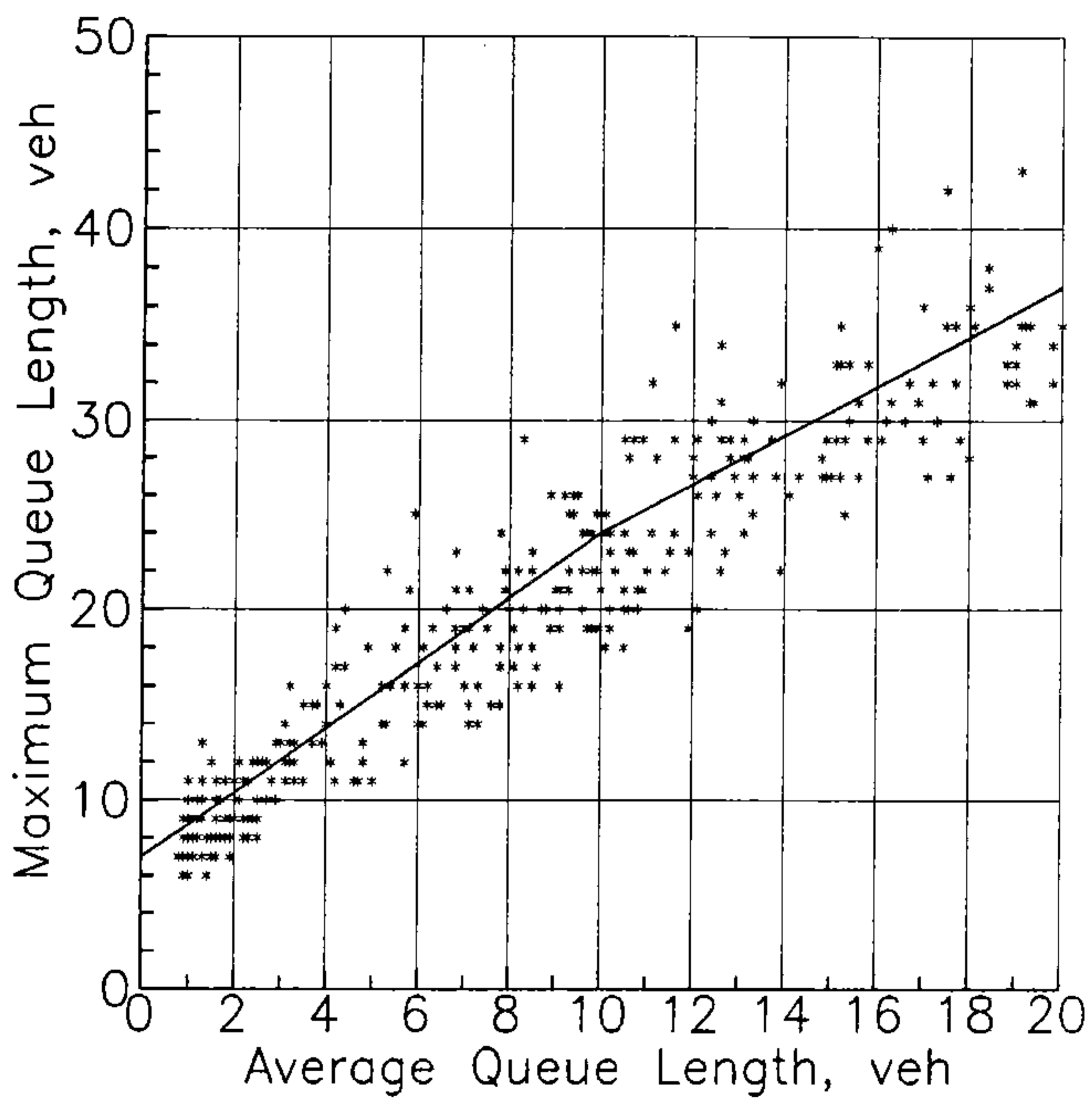


Fig. 17 Relationship between Average Queue Length and Maximum Queue Length

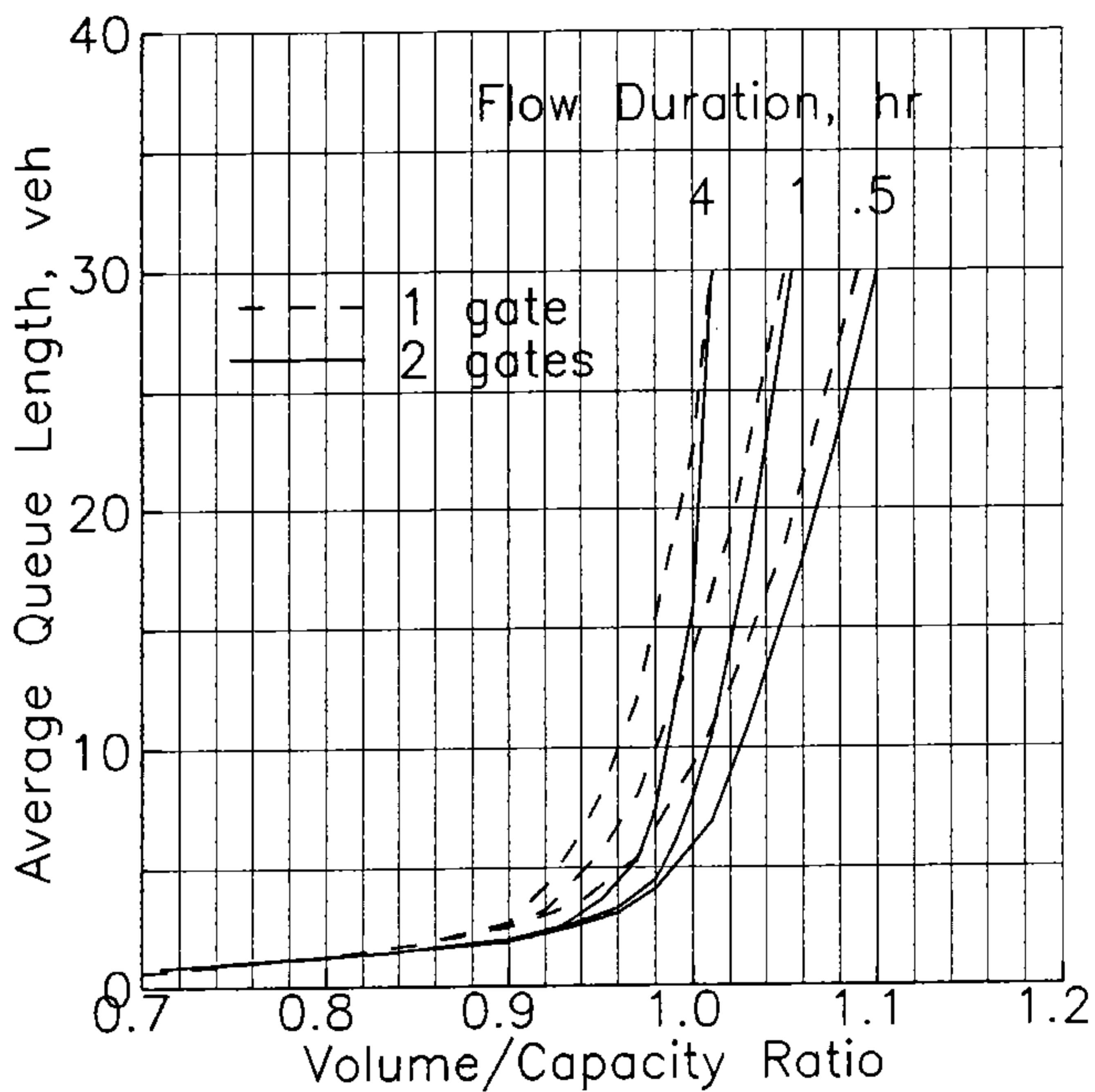


Fig. 18 Single-Gate Average Queue Length vs. Multiple-Gate Operation (Type 1 Gates)

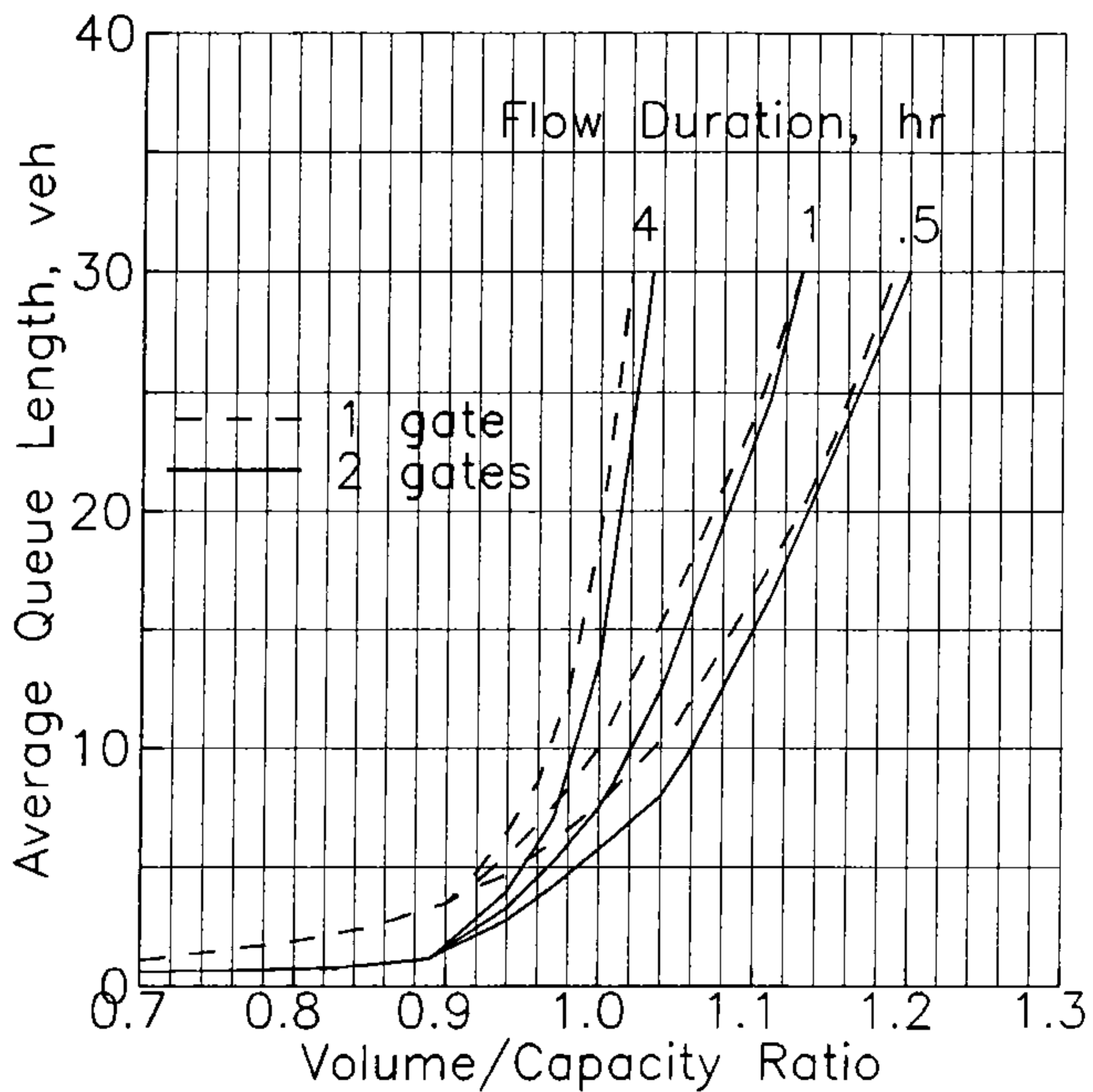


Fig. 19 Single-Gate Average Queue Length vs. Multiple-Gate Operation (Type 2 Gates)

characteristics of their average queue lengths differ very little from those of two-gate operations. The V/C ratio that will effect a given average queue length under a single-gate operation is smaller than that under a multiple-gate operation. For planning purposes, this difference can be considered to be about 0.03.

2.3.3 Effects of Temporal Variation in Flow Rate

For a given volume, an arrival flow pattern with a constant flow rate will result in a better gate performance than one with a variable flow rate. The resulting differences depend on gate type, flow duration, and the extent of the temporal variation of the flow rate. Fig.20 shows a comparison of the average queues for flow patterns with constant flow rates and those for flow patterns with flow rates varying from 80 percent to 120 percent of the respective average flow rates. It is clear from this figure that the temporal variation in flow rate can have significant detrimental impact on the traffic operation at a toll gate. Analytical models are not suitable for estimating such impact.

2.3.4 Relationships among Measures of Effectiveness

As shown in Figs.21, 22, and 23, average queue length, average in-system time, and average approach delay are strongly correlated. Every data point in these figures represents the average value for at least 30 simulation runs for a given combination of gate type and flow rate. Each simulation run has a unique sequence of arrival headways. Based on these figures, the approximate relationships among the three measures of effectiveness can be identified.

Let L = average queue length, in veh, T = average time in system, in sec/veh, and d = average approach delay, in sec/veh. Then, the average approach delay can be estimated from the average time in system as follows:

$$d = 0.92 T \quad (12)$$

Given the average queue length, the average time in system can be estimated as

$$T = \frac{1605 + 3250 L}{C} \quad \text{for } L \leq 15 \quad (13)$$

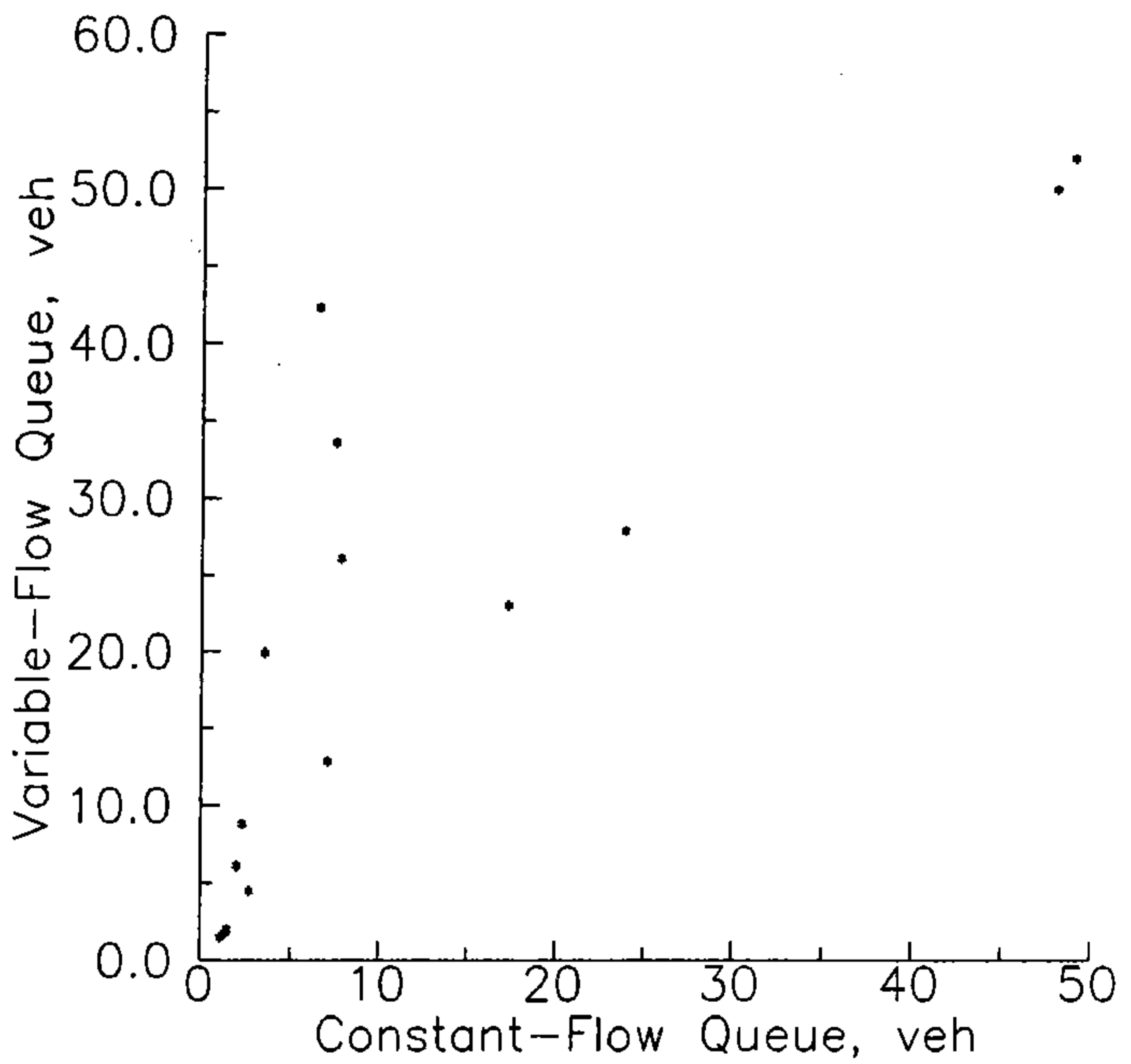


Fig. 20 Effects of Temporal Variation in Flow Rate on Average Queue Length

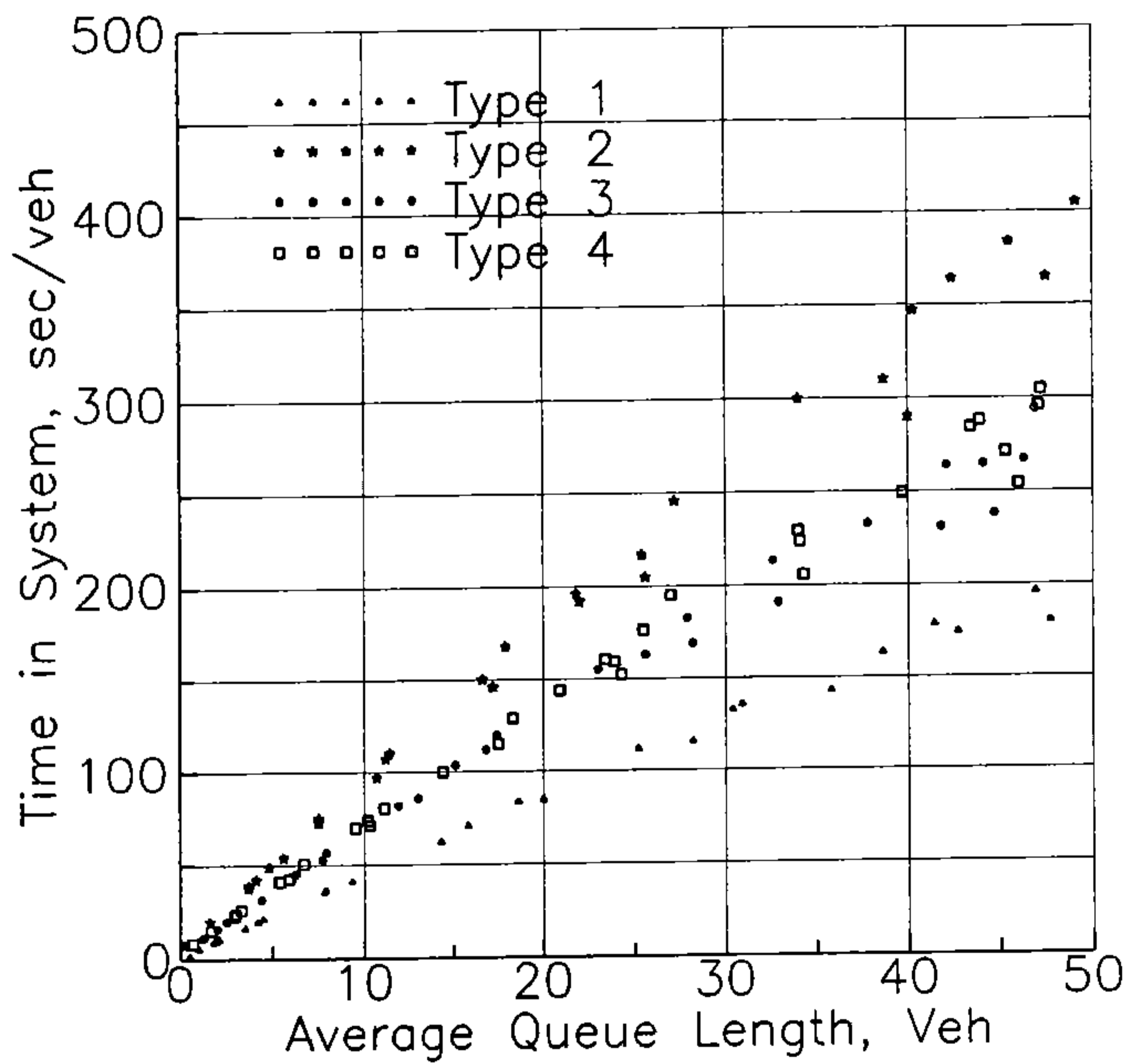


Fig. 21 Relationship between Average Queue Length and Average Time in System

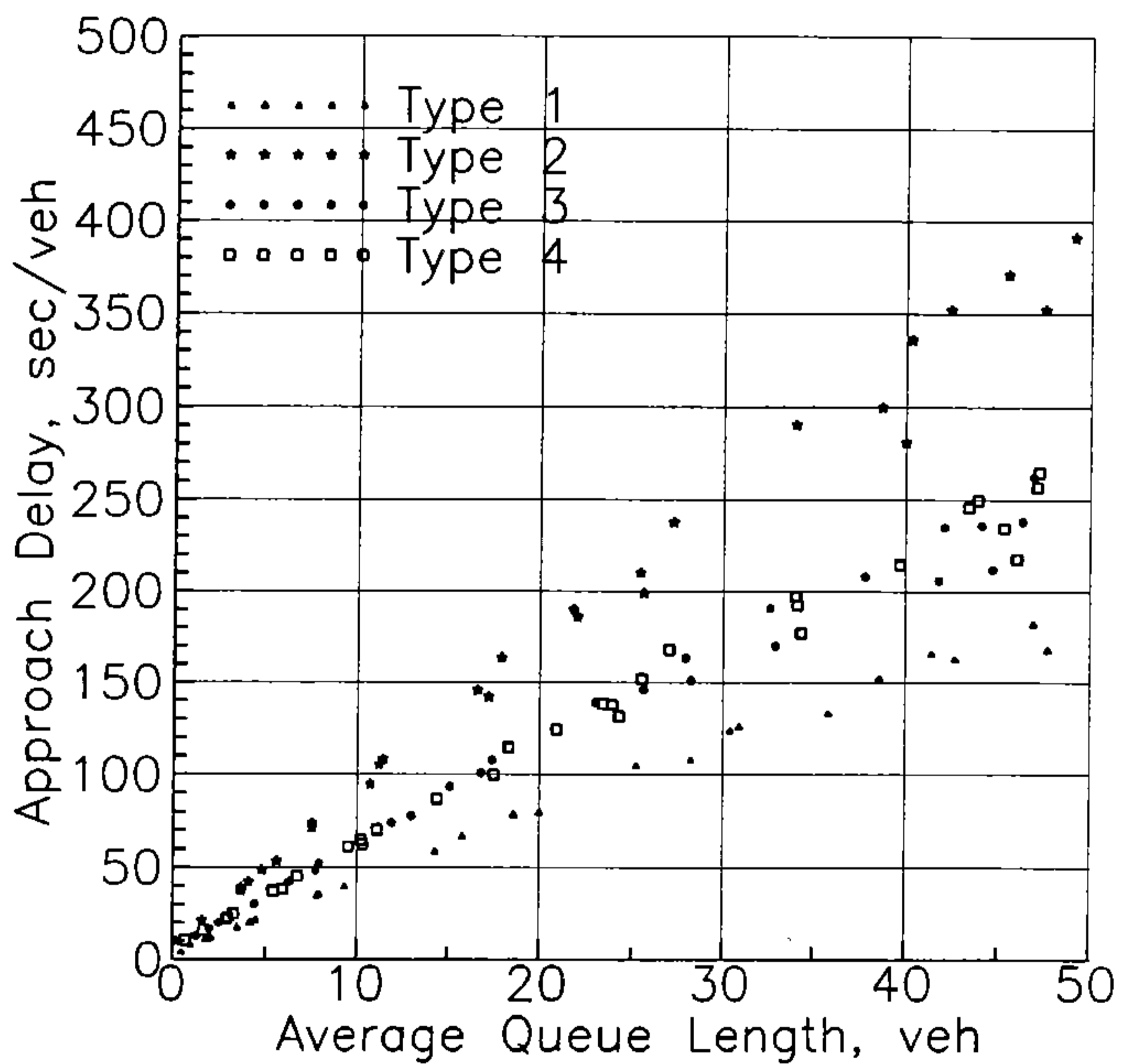


Fig. 22 Relationship between Average Queue Length and Average Approach Delay

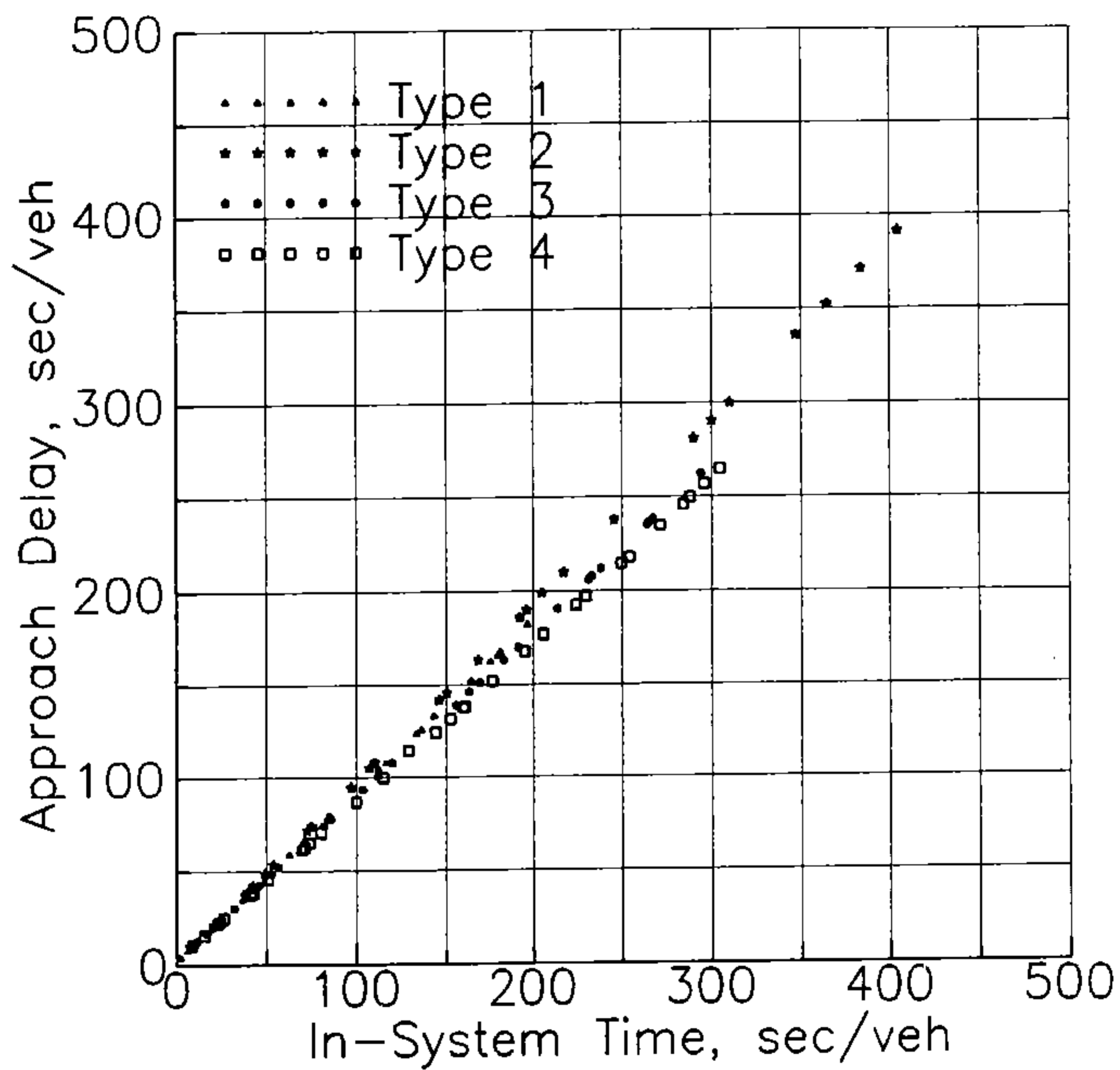


Fig. 23 Relationship between Average Time in System and Average Approach Delay

and

$$T = \frac{8748 + 2776 L}{C} \quad \text{for } L > 15 \quad (14)$$

where C = gate capacity, in vph.

Similarly, average approach delay can be estimated from average queue length as

$$d = \frac{2060 + 2980 L}{C} \quad \text{for } L \leq 15 \quad (15)$$

and

$$d = \frac{8244 + 2570 L}{C} \quad \text{for } L > 15 \quad (16)$$

2.4 Methodology of Level-of-Service Analysis

The quality of the traffic operations at toll gates can be classified into several levels of service to serve as a basis for the planning, design, and operation of toll plazas. Furthermore, since toll plazas are a component of a freeway, it is also desirable to be able to evaluate the level of service of a toll plaza relative to those of other freeway components. Therefore, the level-of-service analysis of toll plazas envisioned in this project encompasses two tasks. One task involves a microscopic analysis of the operations of toll gates. This type of analysis provides useful information to assist in the planning and operation of toll plazas. The other task concerns the aggregated performance of an entire plaza in relation to the traffic operations on freeway mainline. Its purpose is to facilitate a system-wide evaluation of a freeway on the basis of uniform criteria. The aggregated performance of a toll plaza is to be measured in terms of the average vehicle speed through the plaza.

The criteria for system-wide evaluation of freeways are still being investigated. Therefore, only microscopic level-of-service analysis is discussed herein.

2.4.1 Measures of Effectiveness (MOE)

Freeways are expected to provide high-speed, uninterrupted travel. From this perspective, such MOE's as average approach delay, average time in system, and average queue length are obviously important concerns because they affect driver behavior and emotions. Approach delay and time in system are similar in nature and are strongly correlated. For this reason, it is not necessary to use both for classifying levels of service. Since time in system can be more easily measured in the field than approach delay, the former is a logical choice for level-of-service classification.

Queue length is perhaps a quality of traffic operation that drivers can most easily related to under interrupted flow conditions, particularly when congestion is serious [6]. It is also a measure that is relatively easy to quantify in the field. Currently, the Chung-San Freeway Bureau uses queue length as the basis to adjust the gate operations. In addition, the geometric design of a toll plaza can affect and be affected by queue length. Therefore, it is logical to use queue length to classify levels of service. Queue length, however, cannot adequately reflect time in system that is also important to motorists. This limitation is manifested in Fig. 21 in that a given queue length may be associated with a wide range of time in system because of the variation in gate capacity. Therefore, average time in system should also be considered for level-of-service classification.

Traffic density has been suggested for use in gauging the level of service of toll plaza [7, 8]. In such an application, density is defined as the number of vehicles per lane per mile within the boundary of a plaza. One weakness of using density as a level-of-service indicator is that density is an aggregated measure and, as such, cannot reveal the gate-to-gate variation in traffic operation. As a result, it is not useful for a planning or operational analysis that has to estimate the performance of each type of gates. For example, in a planning analysis one may want to know how many gates are needed for each gate type and how long the approach lanes should be. Traffic density cannot provide meaningful information to deal with these planning problems.

Volume to capacity ratio (V/C), aggregated for all the toll gates at a toll plaza and derived from traffic density, is another MOE that has been suggested [7,8]. The use of V/C ratio has appealing features. V/C is easier to measure in the field than most other MOE's. Furthermore, in a planning process the estimation of design volume and vehicle mix is a routine task. Therefore, V/C data are readily available and convenient to use. But there are weaknesses in using V/C ratio. First, volume is a point measure, i.e., it is defined with respect to a reference line. If the movement of vehicles across the reference line is interfered with by downstream congestion, the resulting V/C ratio will be small while severe congestion is present. In other words, V/C ratio based on measured

data may not allow one to identify the actual flow conditions. This is not a problem for planning applications because the V/C ratios in such applications are based on projected traffic demand. For operational analysis, the measurement of arrival volume over a time period must be conducted at a point far away from the downstream queues so that downstream congestion can be appropriately highlighted with a V/C ratio greater than 1.0. Another weakness of using V/C ratio is that the desirability of a given V/C is in fact reflected in queue length and time in system, but the related queue length and time in system can vary substantially not only with the duration of the arrival flow but also with the gate type (see Figs. 12 through 15). As a result, the use of V/C ratio for level-of-service classification will either lead to oversimplification of reality or result in a cumbersome classification scheme.

Based on the previous discussions, it is recommended that average queue length and average time in system be used for level-of-service classification.

2.4.2 Level-of-Service Criteria

Level of service is subjective in nature and, thus, the selection of criteria will inevitably have to rely heavily on human perceptions. Nevertheless, current practices of toll plaza management and the performance characteristics of toll gate operations can serve as a useful guide to prevent the selection of criteria from becoming a nebulous exercise.

There are no existing criteria based on time in system for the analysis of toll plaza operations. On the other hand, stopped delay, which is similar to time in system for a toll plaza operation, has been commonly used in defining the levels of service associated with interrupted flow conditions. For example, both Taiwan's HCM and the U. S. HCM have used stopped delay to classify the levels of service at signalized intersection. In terms of average stopped delays, Taiwan's criteria are as follows: up to 15 sec/veh for Level of Service A; between 15 and 30 sec/veh for Level B; between 30 and 45 sec/veh for Level C; between 45 and 60 for Level D; between 60 and 80 for Level E; and above 80 sec/veh for Level F. It is possible that motorists on freeways are less tolerant to delays than those at signalized intersections. But there is a lack of understanding of the extent of such a difference. Therefore, it is suggested that, as an interim measure, average time in system be divided into the same ranges for classifying the levels of service at a toll plaza.

Regarding queue length, Figs. 12 through 15 and Figs. 18 and 19 reveal that the traffic operation at a toll plaza is generally stable until average queue length reaches about 3 vehicles. Stable operations are normally assigned a level of service A, B, or C for the operations of various highway facilities. Therefore, it is

suggested that the following average queue lengths be used to define these three levels of service: up to 1 vehicle for Level A; greater than 1 vehicle but no more than 2 vehicles for Level B; and greater than 2 vehicles but no more than 3 vehicles for Level C.

An average queue length of one vehicle implies that at any given point in time, there is an average of one vehicle at the gate. With such an average queue length, an arriving vehicle would suffer only slight delay. The V/C ratio under such a condition is less than 0.75. For multiple-gate operations, the corresponding average time in system is less than 7 sec/veh for Type 1 gates and less than 14 sec/veh for Type 2 gates. With an average queue length of 2 vehicles, the average time in system is raised to about 11 sec/veh for Type 1 gates and 23 sec/veh for Type 2 gates. These operating conditions are associated with a V/C ratio of about 0.85. With an average queue length of 3 vehicles, the V/C ratio is about 0.87 to 0.93, and the average time in system is about 15 sec/veh for Type 1 gates and 32 sec/veh for Type 2 gates.

Once the average queue length exceeds three vehicles, the traffic operation at a toll gate can become either metastable or unstable. The average queue length associated with a metastable state is generally between 3 and 6 vehicles, although long queues may occasionally exist. This range of average queue length may be assigned Level D. Within this range of average queue length, the average times in system for Type 1 gates and Type 2 gates can exceed 30 sec/veh and 60 sec/veh, respectively. The corresponding V/C ratio varies with gate type, number of gates, and flow duration; it can be greater than 0.95.

At the present time the Chung-San Freeway Bureau would consider opening another gate for a certain class of vehicles if the queue lengths at available gates are about ten vehicles and there is a possibility that the queue lengths may grow. This implies that a persistent queue of about ten vehicles or more is very undesirable. It is interesting to note that an average queue length of about 10 vehicles is associated with an unstable state that has a V/C ratio close to or exceed 1.0 (see Figs. 12 through 15 and Figs. 18 and 19). The corresponding average time in system can be more than 45 sec/veh for Type 1 gates and 100 sec/veh for Type 2 gates. Therefore, it appears logical to assign a level of service of F to any gate operation that has an average queue length of over ten vehicles, regardless of the corresponding average time in system. This also means that an average queue length greater than 6 vehicles but no more than 10 vehicles can reasonably be assigned a LOS of E.

The level-of-service criteria suggested on the basis of the discussions given above are summarized in Table 2.

Table 2 Level-of-Service Criteria

LOS	Average Queue Length L vehicles	Average Time in System T sec/veh
A	≤ 1	≤ 15
B	$1 < L \leq 2$	$15 < T \leq 30$
C	$2 < L \leq 3$	$30 < T \leq 45$
D	$3 < L \leq 6$	$45 < T \leq 60$
E	$6 < L \leq 10$	$60 < T \leq 80$
F	>10	> 80

2.4.3 Planning Analysis

In planning for a toll plaza, it is necessary to know the number of gates needed to accommodate a projected service flow and the required length of the toll plaza. The number of gates that should be provided depends on such factors as types of gates to be provided, gate capacities, expected service flow for each type of gates, etc. The length of a toll plaza should be based on the expected queue length and the distance needed to channel approaching vehicles safely to and from gates. For the section of the plaza upstream the gates, this means that a full-width approach lane of sufficient length should be given to each gate to store queuing vehicles and that a taper is needed to allow approaching vehicles to maneuver to target gates.

The planning analysis as shown in Fig.24 is focused on the determination of the number of gates and the lengths of full-width approach lanes upstream of the gates. This procedure requires the use of approximate analytical models to obtain a preliminary design and the use of the TPS simulation model to provide a more detailed and reliable evaluation of the design for possible modifications. In a planning analysis, it is logical to use a design flow pattern that has a constant flow rate as the basis for determining geometric design and gate requirements. Given that the design flow rate is constant, the planning analysis can make use of the various equations described in Section 2.3.

The components and the application of the planning analysis are described below with the help of an example.

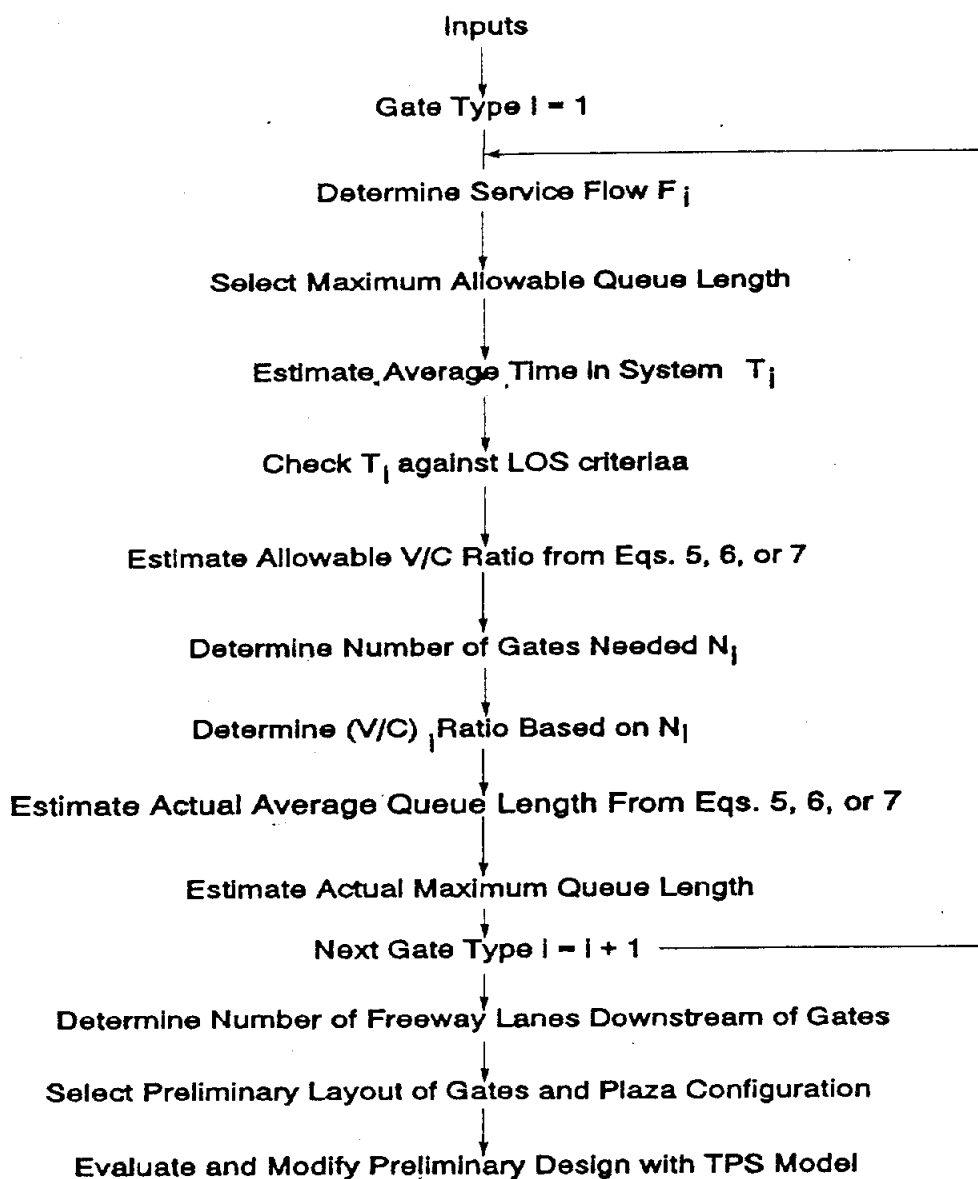


Fig. 24 Procedure of Planning Analysis

Inputs

Types of gates i : small vehicles with exact changes ($i = a$);
small vehicles without exact changes ($i = b$); all other vehicles ($i = c$).

Gate capacity C_i : 800 vph for Type a gates; 300 vph for Type b gates; and 500 vph for Type c gates. Service time distribution are similar to those shown in Figs. 2 and 3.

Service flow on freeway Q : 3,000 vph, total for two lanes.

Duration of service flow t : 2 hours (assume 4 hours if unknown).

Vehicle mix P_i : 72 % Type a; 8 % Type b; and 20 % Type c.

Average vehicle length at Type i gates w_i : 4.5 m for Type a;
4.5 m for Type b; and

Fig.24

15 m for Type c.

Desired level of service : LOS = D

Analysis Procedure

Given the inputs, the planning analysis is to follow the procedure outlined below.

1. For each gate type i , perform the following tasks:

(1) Determine the design service flow F_i for the gate type being analyzed. The service flow for Type i gates is the total service flow Q times the proportion of vehicles P_i that are expected to use Type i gates. Results: $F_a = 2,160$ vph; $F_b = 240$ vph; and $F_c = 600$ vph.

(2) Select a maximum allowable queue length L_i for design. For a LOS = D, the allowable average queue length is greater than 3 vehicles but no more than 6 vehicles. For the example, choose a maximum of 6 vehicles for all gate types, i.e., $L_a = L_b = L_c = 6$ vehicles.

(3) Use either Eq. 13 or Eq. 14 to estimate average time in system based on the design average queue length L_i . Results: $T_a = 26.4$ sec/veh; $T_b = 70.4$ sec/veh; and $T_c = 42.2$ sec/veh.

(4) Check the estimated average times in system for conformance with LOS D criteria. For the example problem, $T_b = 70.4$ sec/veh will result in a LOS of E. Therefore, the design

average queue length for Type b gates should be reduced and a new estimate made for the corresponding average time in system. If L_b is reduced to 5 vehicles, the average time in system becomes 59.5 sec/veh, which is in the range of LOS D. Therefore, change the design average queue length of Type b gates to $L_b = 5$ vehicles.

- (5) Assuming multiple gates are needed, use Eqs. 5, 6, or 7 to estimate the allowable $(V/C)_i$ ratio for the combination of gate capacity C_i , design average vehicle length L_i , and flow duration t . Eqs. 5 or 6 should be used if the design average vehicle length does not exceed 3 vehicles. Otherwise, use Eq. 7. Results: 0.966 for Type a; 0.982 for Type b; and 0.974 for Type c.
- (6) Determine the number of gates needed, N_i , according to the following equation:

$$N_i = \frac{F_i}{C_i \left(\frac{V}{C}\right)_i} \quad (17)$$

Results: $N_a = 2.8$ or 3 gates; $N_b = 0.8$ or 1 gate; and $N_c = 1.2$ or 2 gates. The total number of required gates is 6.

- (7) If the number of gates determined previously for a given gate type is one, reduce the allowable V/C ratio by 0.03 and use the resulting V/C ratio in Eq. 17 to obtain a revised estimate. For the example problem, the adjusted allowable V/C ratio is 0.952 for Type b gates. This leads to the same required number of gates. Therefore, The number of Type b gate is one.
- (8) Determine actual V/C ratio based on the number of available gates. Results: $(V/C)_a = 2,160/(3 \times 800) = 0.90$; $(V/C)_b = 240/300 = 0.80$; and $(V/C)_c = 600/(2 \times 500) = 0.60$.
- (9) Use the estimated V/C ratio in Eqs. 5, 6, or 7 to estimate the actual average queue length. In using these equations, increase the V/C ratio of single-gate operations by 0.03. Results: $L_a = 7 \times 0.90 - 3.5 = 2.8$ vehicles; $L_b = 2.3$ vehicles; and $L_c = 0.7$ vehicles.
- (10) Use the estimated actual average queue length in either Eq. 10 or Eq. 11 to determine maximum queue length $(L_{\max})_i$. Results: $(L_{\max})_a = 7 + 1.7 \times 2.8 = 11.8$ vehicles; $(L_{\max})_b = 10.9$ vehicles; and $(L_{\max})_c = 8.2$ vehicles.
- (11) Determine the minimum required length W_i of full-width

approach lanes based on maximum queue length (L_{\max}), average vehicle length, and a gap of 2 m between vehicles. $W_a = (4.5 + 2) \times 11.8 = 77$ m; $W_b = 71$ m; and $W_c = 139$ m.

2. Determine the number of freeway lanes, N_f , needed at the downstream junction between freeway and toll plaza. If current practice is followed, N_f can be determined as the total number of toll gates divided by 2.5. A more logical method is to determine the lane requirement based on the maximum flow that may be released from all the gates and the capacity of the plaza-freeway junctions downstream of the gates. For the example problem, the maximum flow released by 6 gates is $800 \times 3 + 300 \times 1 + 500 \times 2 = 3,700$ vph. If the downstream junction has a capacity of 2,000 vph per lane, then the minimum required number of lanes is $3,700/2,000 \approx 2$. To provide a better level of service, additional lanes may be needed. For example, if the freeway lanes at the downstream junctions are to be operated at a V/C ratio of below 0.8, then 3 lanes would have to be provided. At the present time, the capacities of the plaza-freeway junctions downstream of toll gates are unknown.
3. Select a preliminary layout of gates and plaza configuration based on the previous analysis. A preliminary design for the example problem is shown in Fig. 25.
4. Use TPS simulation model to evaluate the preliminary design. Gate-specific outputs for the example problem are given in Table 3. In this case, the flow at Gate 4 is much smaller than those at Gate 5 and 6 because drivers tend to avoid lane changes.

Table 3 Simulated Gate Performance for Example Problem

Gate	Gate Type	Flow, vph	Average Queue, veh	Average Time in System, sec/veh	Maximum Queue, veh (m)	Level of Service
1	c	276	0.5	6.3	7 (119)	A
2	c	322	0.6	7.1	7 (119)	A
3	b	240	1.4	20.8	10 (65)	B
4	a	562	0.6	4.0	9 (59)	A
5	a	794	2.5	11.4	13 (85)	C
6	a	802	3.7	16.4	13 (85)	C

Remark: 1. No queue formation downstream of gates.
2. Average speed through plaza is 41 kph.

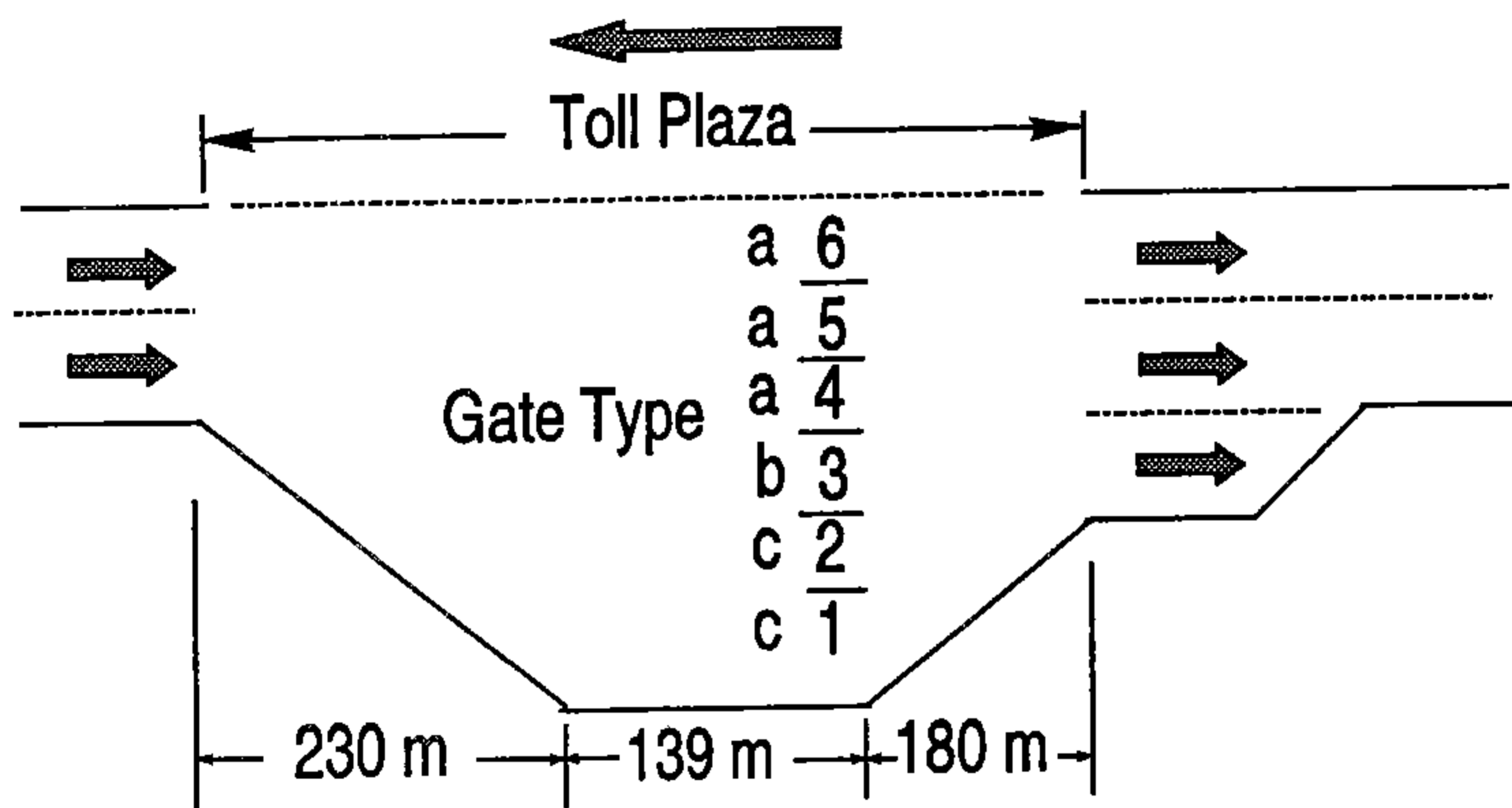


Fig. 25 Layout of Gates and Plaza for Example Problem

2.4.4 Operational Analysis

Operational analysis of toll plazas may involve the evaluation of either existing operations or certain modifications of existing operations. In such an undertaking the vehicle arrival pattern, the layout of the toll plaza, and the gate facilities are given. The purpose of the analysis is to determine the level of service provided.

Since the gate performance is highly influenced by the temporal variation in flow pattern, the equations described in Section 2.3 may result in large estimation errors for operational analysis unless the vehicle arrival rate is more or less uniform over the analysis period. Therefore, it is suggested that operational analysis be based either on field studies or computer simulation. If these options are not feasible due to resources constraints, then a crude analysis can be performed according to the procedures described below.

1. Choose a flow rate and flow duration for analysis. This analysis flow may be determined in three steps. The first step is to divide the analysis period into small time intervals, each of which has a more or less uniform arrival rate. The second step is to determine the average flow rate for the entire analysis period. And, the last step is to treat the analysis flow as the average of the flows rates in various time intervals that are above the overall average rate. The analysis flow determined in this manner is between the overall average rate and the maximum rate of any time interval. Based on the analysis flow and the vehicle mix, determine the service flow for each gate type.
2. Use Eqs. 5, 6, or 7 to estimate the average queue length for each gate type. In using these equations, increase the V/C ratio by 0.03 if a given gate type has only one gate available.
3. Use the estimated average queue length in Eq. 13 or Eq. 14 to estimate average time in system.
4. Use the estimated average queue length in Eq. 10 or Eq. 11 to estimate maximum queue length for each gate type.
5. Use the estimated maximum queue length and average vehicle length to estimate the required length of full-width approach lane for each gate type. Check to see if the required length exceeds the available length.
6. Determine the level of service based on the criteria given in Table 2. If the required storage length is longer than the available length. The actual LOS may be lower than that determined from Table 2.

3.0 FREEWAY MAINLINE SECTIONS

The freeways in Taiwan have 3.75-m wide lanes and 3-m wide outside shoulders and 1-m wide clearance between their median barriers and the edges of traffic lanes. About 85% of the existing mileage has four lanes and 8-lane sections account for about 7% of the mileage. Out of the total mileage, 73.5% has grades under 1% and 0.3% has grades between 5% and 6%. The speed limit is 90 kph near major cities and 100 kph elsewhere.

Freeway mainline sections are to be classified into the following categories for analysis: basic sections; ramp sections; weaving sections; and tunnels. There are no rigid boundaries among these sections because how the operation of one section may be affected by those of others depends also on traffic volume. Past research on freeway flows has formed a basis for level-of-service analysis of freeways. Controversies, however, still linger regarding what representative relationships among traffic parameters and geometric design features should be used for the level-of-service analysis. For example, questions have been raised concerning the capacities and the flow-speed relationships described in the U.S. HCM. Several researchers [9, 10, 11, 12] have provided valuable insights into many of the issues involved.

The controversies surrounding the selection of representative relationships for the level-of-service analysis of freeways may have been caused in part by the possibility that flow characteristics are site specific. The lack of uniformity in data collection and analysis is another contributing factor. The advancement in vehicle design over the years may have also added uncertainty to our understanding of the characteristics of freeway flows. To provide a good foundation for developing a methodology for analyzing mainline sections, the Planning Division of IOT has launched a major data collection effort. Data are being collected at several sites with different geometric design features, speed limits, and distances from ramp junctions. All the traffic data are recorded with video cameras that allow time be encoded on tapes at a resolution of 0.01 sec. Due to the difficulties in using cameras to record the details of traffic movement in all the traffic lanes in a given direction, only the shoulder lane and the inside lane adjacent to it are the subjects of data collection. The video data are reduced to reveal the time series of arrival time, departure time, vehicle speed, occupancy of 1.83-m or 2-m long detection area, and vehicle length. Each time series covers a period between one to two hours. The data reported herein are based on 1.83-m detection areas. Due to equipment limitations in data reduction, the arrival times and departure times with respect to a series of reference lines can only be estimated to within 0.015 sec of the true values, and the estimated vehicle lengths have errors of up to 0.5 m.

So far, the data collected at two sites have yielded information important to the development of a framework for level-of-service analysis. One site is at a location 500 m upstream of an off-ramp at Sanchung; the other is at a location 2 km upstream of an off-ramp at Sichu. Both sites are on level sections and have a speed limit of 90 kph. The flow characteristics at these sites and the methodological framework being considered are discussed below.

3.1 Passenger Car Equivalent

Vehicles are usually converted into equivalent passenger car units (PCU) to facilitate capacity and level-of-service analysis. In estimating PCU, constant passenger car equivalents are often assigned to various types of vehicles for a given terrain condition. This approach is convenient but may mask the real relationships among such freeway traffic parameters as flow rate, speed, occupancy, and density. As an alternative, PCE is considered to be a function of vehicle length, vehicle speed, and grade.

Consider a platoon of vehicles moving across a reference line on a level section that has ideal lane width and lateral clearance to roadside obstacles. The speeds of various types of vehicles in such a platoon would be about the same. Let v be the expected average speed of that platoon of vehicles on the ideal level section and $H_p(v)$ be the corresponding average headway of passenger cars, measured from rear end to rear end. At a certain point downstream, Type i vehicles in that platoon cross a reference line at an expected average speed v_i and an average headway $H_i(v_i)$. The average headway represents the average time needed to process a given type of vehicles through a specified location on a freeway. It also represents the approximate length of a lane a vehicle will occupy at a speed v_i . Therefore, the PCE of Type i vehicles at any location can be defined as

$$PCE = \frac{H_i(v_i)}{H_p(v)} \quad (18)$$

PCE as defined in the above equation is based only on the headways of vehicles in platoons. This avoids the inclusion of long headways that exist because of the lack of vehicle supply. A platoon refers to a stream of vehicles of which the vehicles in front affect the movement of the vehicles following behind. A 5-sec headway has been used [2] as a threshold value to differentiate the vehicles in platoons from those that are considered to be outside the influence of the vehicles ahead. This threshold headway, however, is not applicable to low-speed or stop-and-go conditions. The time series data collected in this study show that, when the speed of a vehicle

over a detection area of about 2-m long drops below 20 kph, the headway of the vehicle following behind may sometimes exceed 10 sec although the two vehicles are clearly in a densely packed platoon. At the other extreme when vehicle speeds are very high, vehicle interactions may still exist at a headway of slightly longer than 5 sec. Based on these considerations, a vehicle on a level section is considered to be in a platoon if the elapsed time between the departure of the rear end of the vehicle in front from a reference line and the arrival of the front end of the subject vehicle does not exceed the threshold value specified below:

$$T = 5 + 0.5 (20 - v_{j-1}) \quad \text{if } v_{j-1} < 20 \text{ kph} \quad (19a)$$

$$T = \text{Minimum of } 5 + 0.1(v_{j-1} - 80) \text{ and}$$

$$5 + 0.1(v_j - 80) \quad \text{if } v_{j-1} \text{ and } v_j > 80 \quad (19b)$$

and

$$T = 5 \quad \text{otherwise} \quad (19c)$$

where T = threshold elapsed time (rear end to front end), sec; and v_{j-1} = speed of the vehicle in front, kph; and v_j = speed of the subject vehicle.

In order to identify the nature of the average headways of vehicles in platoons, the headways in platoons are cross-classified with vehicle length and speed. For this purpose, vehicle speed is divided into 5-kph intervals and vehicle length is grouped into the following categories: ≤ 6.5 m, $6.5 \sim 11.5$ m; $11.5 \sim 16.5$ m; and > 16.5 m. Vehicles that are not longer than 6.5 m are considered to be passenger cars; their PCE is 1.0. The resulting headways, based on the data collected at Sanchung, are shown as a function of speed and vehicle length in Figs. 26 and 27. A comparison of the average headways of passenger cars identified respectively from the Sanchung data and Sichu data is given in Fig. 28. For passenger cars, which have an average length of about 4.5 m, the average headways for the various 5-kph speed intervals are derived from sample sizes ranging from 31 vehicle to 1,653 vehicles. The sample sizes for longer vehicles are much smaller. For vehicles longer than 16.5 m, for example, the sample size for each speed interval is between 11 and 221 vehicles. For speeds between 20 kph and 100 kph, the standard errors of the estimated average headways are about 0.035 sec for passenger cars, and they reach as high as 0.43

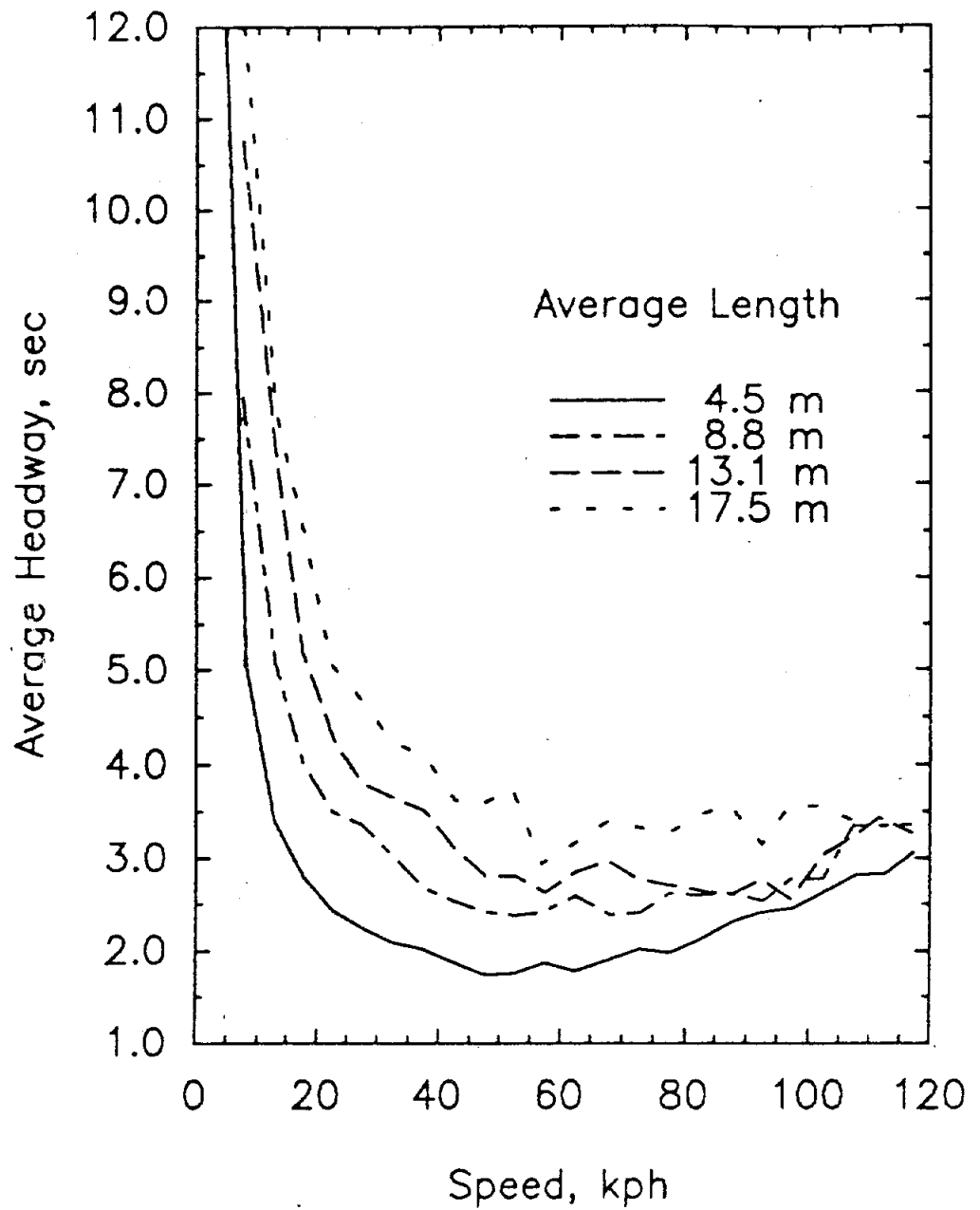


Fig. 26 Average Headway as a Function of Vehicle Speed and Vehicle Length in Sanchung Shoulder Lane

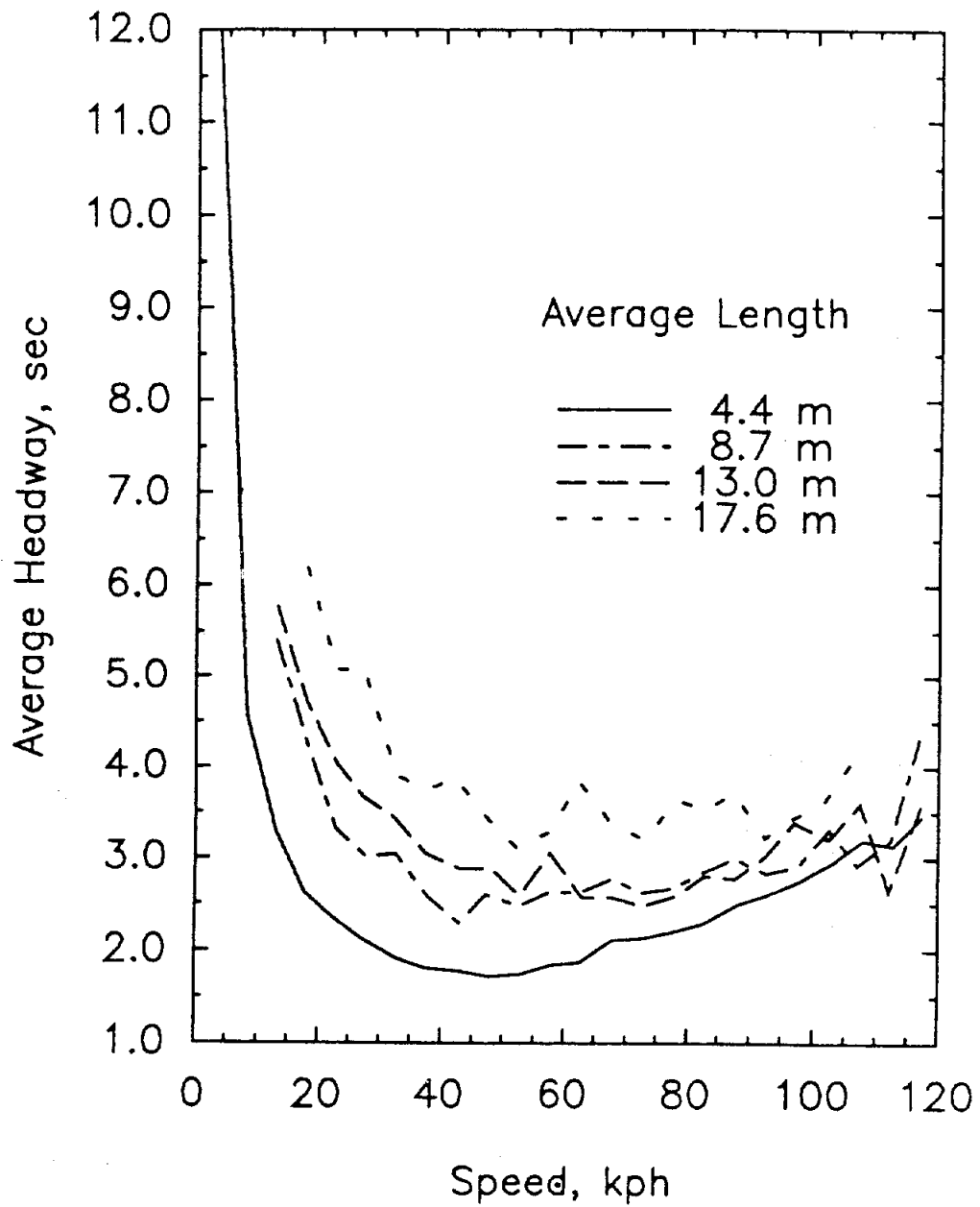


Fig. 27 Average Headway as a Function of Vehicle Speed and Vehicle Length in Sanchung Inside Lane

sec at speeds between 50 kph and 55 kph for vehicles longer than 16.5 m. This implies that the estimates for passenger cars are much more reliable than those for longer vehicles.

On level sections, the differences between the speeds of light vehicles and those of heavy ones in platoons are small. Therefore, the PCE can be estimated as the ratio of the average headway of Type i vehicles to that of passenger cars at the same speed. The PCE values determined from the field data for level sections are shown in Fig. 29. Based on this figure, the PCE of a vehicle on a level section can be estimated from the following equation:

$$PCE = 0.6 + 0.12 L - (0.12 L - 0.39) \frac{V}{120} \quad (20)$$

The determination of the PCE of the vehicles at a specified location on a grade requires the following information:

1. The relationship between the average speed of a given type of vehicle at a specified location and the initial speed before grade.
2. The headway and speed of individual vehicles measured at the specified location.

A vehicle on a grade may slow down and, thus, prompt the drivers following behind to change lanes. Furthermore, the speeds of light vehicles and those of heavy vehicles may differ considerably on grades. Therefore, the headway of a vehicle on a grade may be significantly affected by the type of vehicles immediately ahead. For this reason, the headway of a vehicle at a specified location on a grade should be classified in terms of the type of vehicle immediately ahead. Another complication in analyzing headways on grades is the need to determine whether or not a long headway belongs to a platoon. Ideally, all the headways that reflect the combined effects of grade and vehicle type should be included in the analysis. This may be difficult to achieve based on what one can observe in the field, because a platoon before a grade may disperse after it enters the grade. The data collected at Sanchung on level sections indicate that at an average speed of 90 kph the longest 10% to 20% of the headways do not belong to a platoon. At an average speed of 50 kph, this percentage drops to about 2%. Based on this understanding, a portion of the observed headways on a grade may be excluded from analysis in accordance with the expected speed before the grade. For example, if the expected speed before a grade is about 90 kph, the longest 15% of the observed headways may be excluded from analysis.

Given the data collected before a grade and at a specified location on the grade. The PCE of a given type of vehicles at the specified

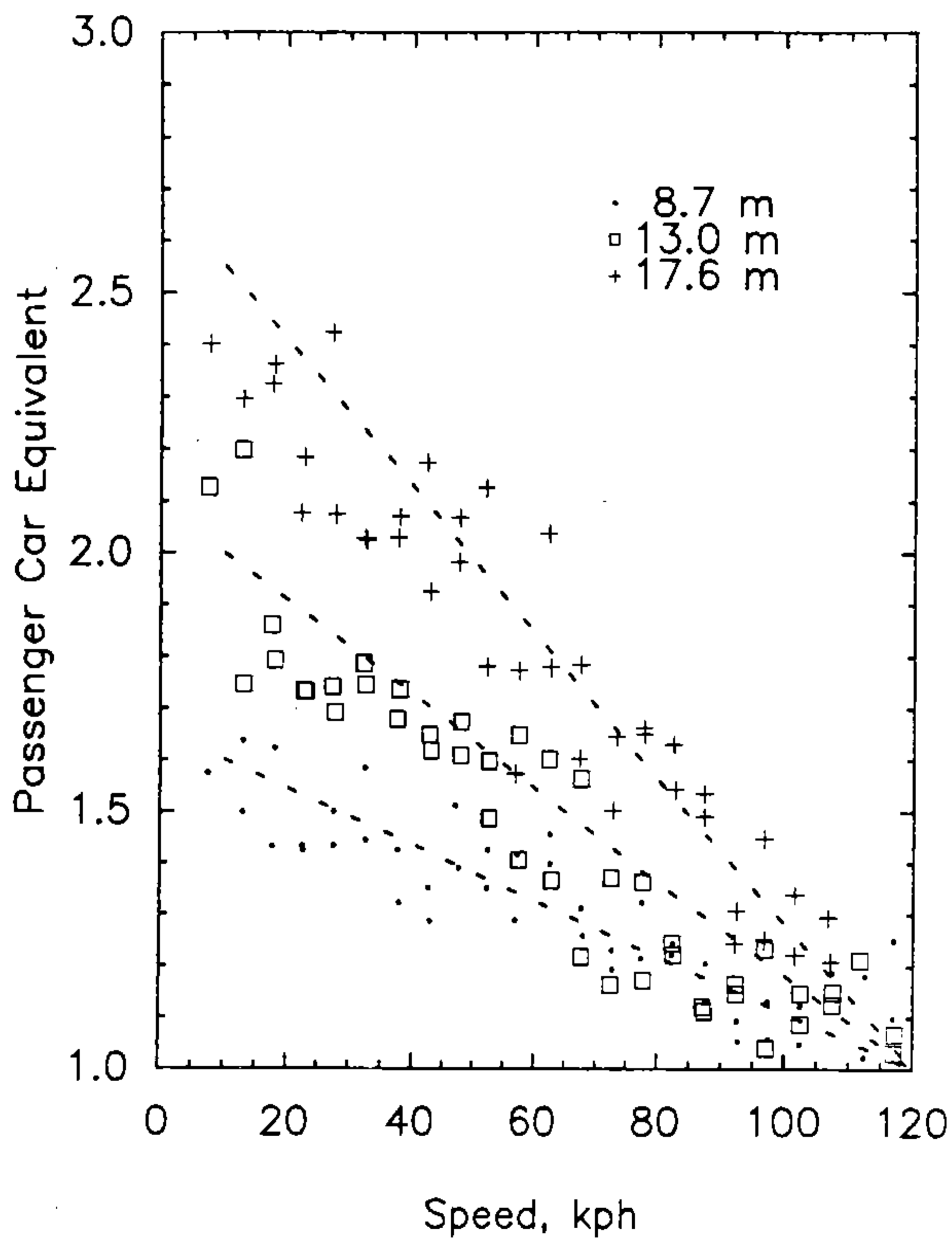


Fig. 29 Passenger Car Equivalent as a Function of Vehicle Speed and Length

location can be determined as a function of the proportion of that type of vehicle in a traffic stream. The procedure for deriving the PCE can be illustrated with an example. Let us assume that the trucks at the end of a 2-km grade of 5% have an average speed of 40 kph, an average headway of 6 sec if the vehicles ahead is a passenger car, and an average headway of 4 sec if the vehicle ahead is also a truck. Let us also assume that the 40-kph speed on the grade corresponds to an initial speed of 80 kph before the grade. The initial speed is associated with an average headway of about 2.3 sec for passenger cars (see Fig. 28). If a platoon of vehicles consists of 20% trucks and 80% passenger cars, the chance that a truck will be following another truck is 20% and the chance that a truck will be following a car is 80%. Therefore, the average headway of the trucks on the grade is $6 \times 0.8 + 4 \times 0.2 = 5.6$ sec. The PCE for the trucks can then be determined as $5.6/2.3 = 2.4$. The PCE values obtained in this fashion can be used to relate PCE to vehicle type, vehicle mix, grade, length of grade, and the flow conditions before grade. It should be noted that it is also necessary to determine the PCE of passenger cars on grades, unless it is known that none of the grades on the existing freeways affects the speeds of passenger cars.

3.2 Speed-Flow Relationships and Service Flow Rates

Mean speed can be classified into time-mean speed and space-mean speed [13]. Space-mean speed is the harmonic mean of the spot speeds of individual vehicles; it is usually used to establish the speed-flow relationships of freeway flows. As shown in Figs. 30 and 31, time-mean speed is always greater than space-mean speed. When the speeds of the individual vehicles in a sample differ only slightly from each other, the difference between time-mean speed and space-mean speed is usually less than 3 kph. When very low speeds exist among relatively high speeds in a sample, large discrepancies between space-mean speed and time-mean speed emerge. The presence of both low speeds and high speeds in a sample implies an unstable speed profile over the sampling period. Therefore, a time-mean speed much larger than the corresponding space-mean speed indicates certain discontinuities in the traffic movement. At the Sanchung site, serious discontinuities in traffic movement appear to occur when the space-mean speed drops below 70 kph.

The relationships between space-mean speed and flow rates identified for the shoulder lane and the inside lane at Sanchung are shown respectively in Fig. 32 and Fig. 33 based on data aggregated at 1-min intervals. It is clear from these figures that considerable variations in flow rate exist at a given level of speed. Therefore, an issue can be raised as to how such relationships as shown in the figures can be used to guide the planning and design of freeways. To address this issue, it should be noted that the lack of vehicle supply accounts partially for the variations in flow rates at a given level of speed. This portion

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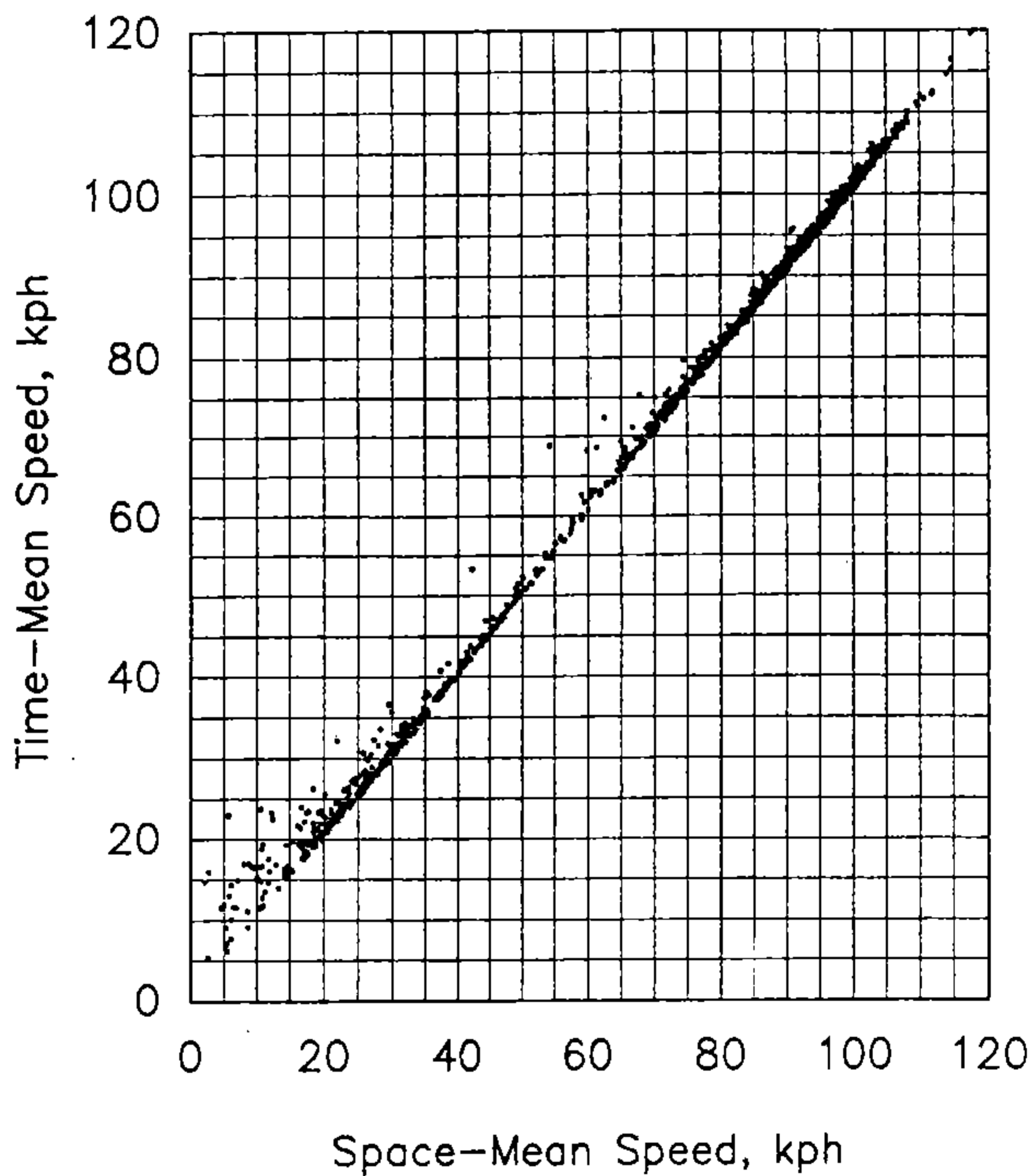


Fig. 31 Time-Mean Speed vs. Space-Mean Speed, Sanchung Inside Lane

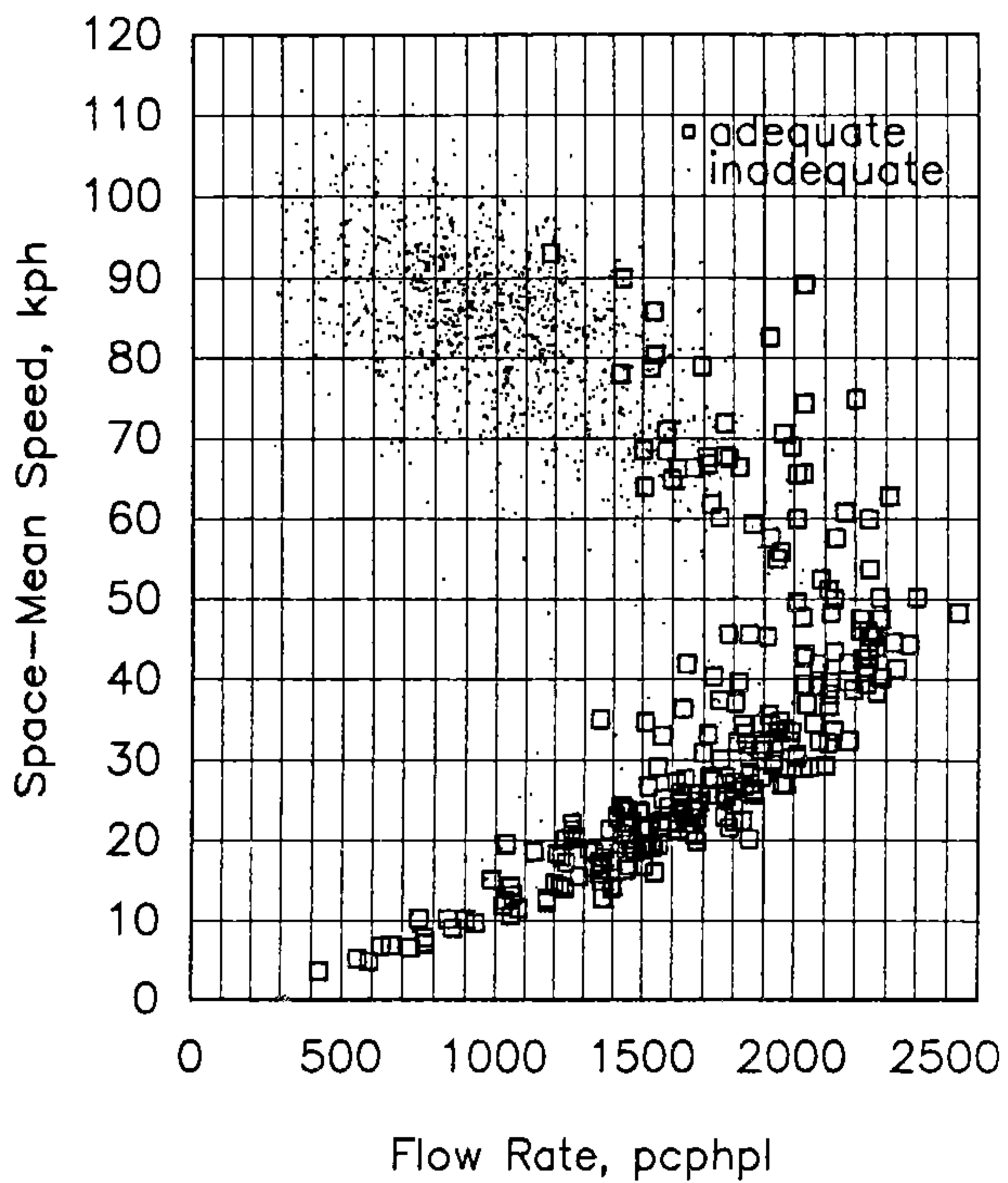


Fig. 32 Space-Mean Speed and Flow Relationship, Sanchung Shoulder Lane

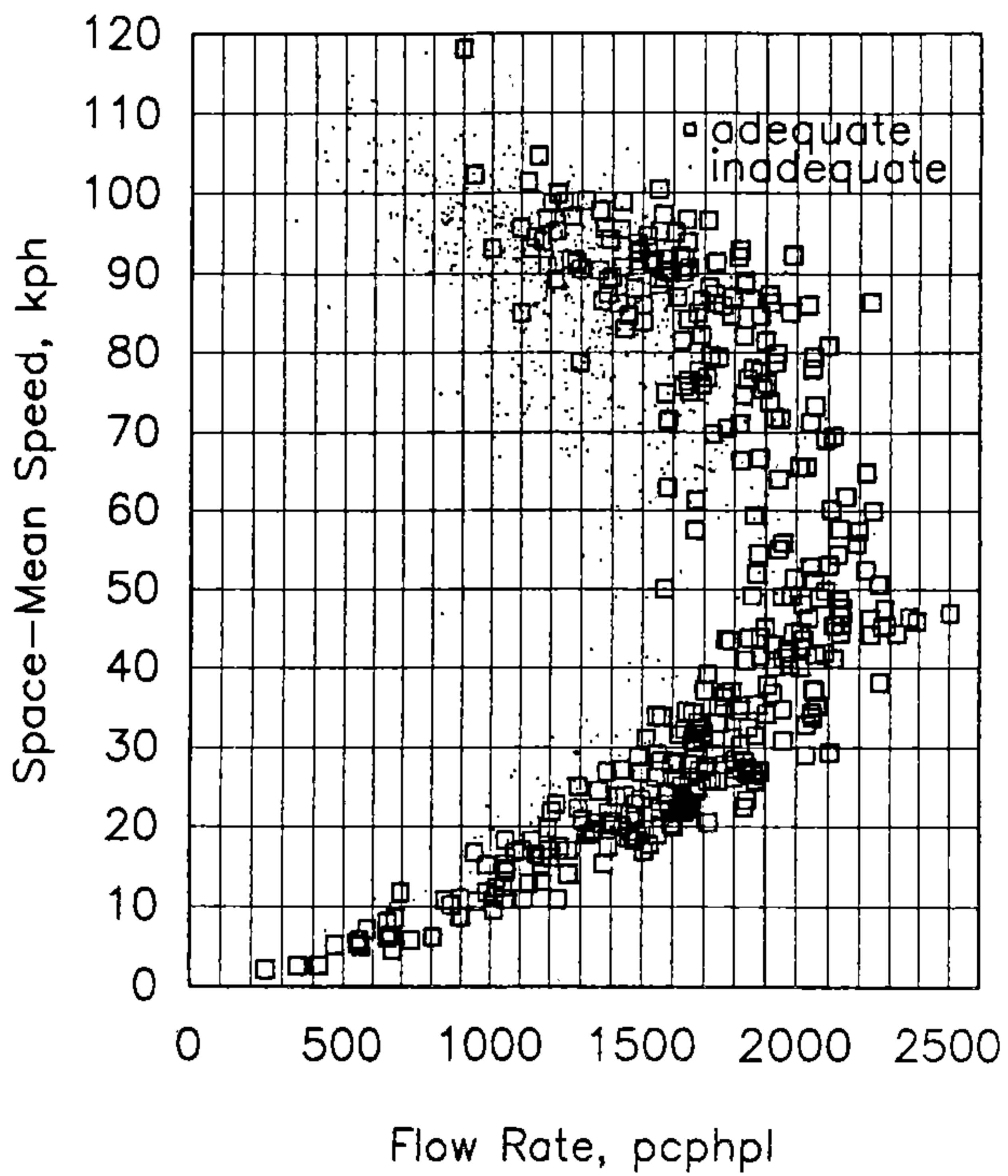


Fig. 33 Space-Mean Speed and Flow Relationship, Sanchung Inside Lane

of the variations can be removed by discarding samples that contain vehicles not belonging to a platoon. In Figs. 32 and 33, samples that contain only vehicles in a platoon are marked as having an "adequate" vehicle supply, while those containing vehicles not in a platoon are marked as having an "inadequate" vehicle supply. To determine the service flow rate a freeway section can be expected to deliver at a specified speed, only those samples containing adequate vehicle supplies should be considered. Based on such samples, the expected service flow rates at various levels of speed are determined for both Sanchung and Sichu sites. The results are shown in Fig. 34. The standard error of the expected service flow rate at a given speed ranges from about 50 pcphpl (passenger cars per hour per lane) to 150 pcphpl. It should also be noted that the service flow rates shown in Fig. 34 approximate the flow rates that can be determined from the average headways shown in Fig. 28. At the Sanchung site, the maximum expected service flow rate (i.e., capacity) reaches 2,210 pcphpl for the shoulder lane and 2,140 pcphpl for the inside lane. These rates appear to occur at a space-mean speed between 45 kph and 50 kph. In theory, the maximum service flow rate can be maintained indefinitely as long as the average speed remains at the same level. In reality, the disturbances downstream may cause a drop in speed and, thus, reduce the flow rate. The data collected at Sanchung show that a service flow rate between 2,100 pcphpl and 2,200 pcphpl can last for at least 18 minutes.

Whether a lane is a shoulder lane or an inside lane seems to have some impact on the speed-flow relationship. At the Sanchung site, the service flow rates at speeds between 60 and 90 kph can be as much as 150 pcphpl higher for the inside lane than those for the shoulder lane. At speeds between 25 kph and 50 kph, the shoulder lane has higher service flow rates. These characteristics can also be revealed by comparing the respective average headways shown in Fig. 28. The distance from a ramp appears to have quite a substantial impact on the speed-flow relationship for inside lanes. The data collection at Sichu has not been completed. Nevertheless, the speed-flow relationship identified from 7 hours of data for the inside lane at that site, at speeds between 70 kph and 90 kph, deviates dramatically from those identified for the Sanchung site. On the other hand, the speed-flow relationship for the shoulder lane at Sichu resembles that of the shoulder lane at Sanchung. The vehicle mix in the inside lanes at the study sites cannot account for such deviations. The proportion of passenger cars in the inside lane is 79% at Sichu and 76% at Sanchung.

3.3 Sampling Period

The choice of sampling period for aggregating data will affect the scattering of data as shown in Figs. 32 and 33. In theory, as long as the vehicles in a sample are all in a platoon, the time interval used for data aggregation should not affect the values of expected

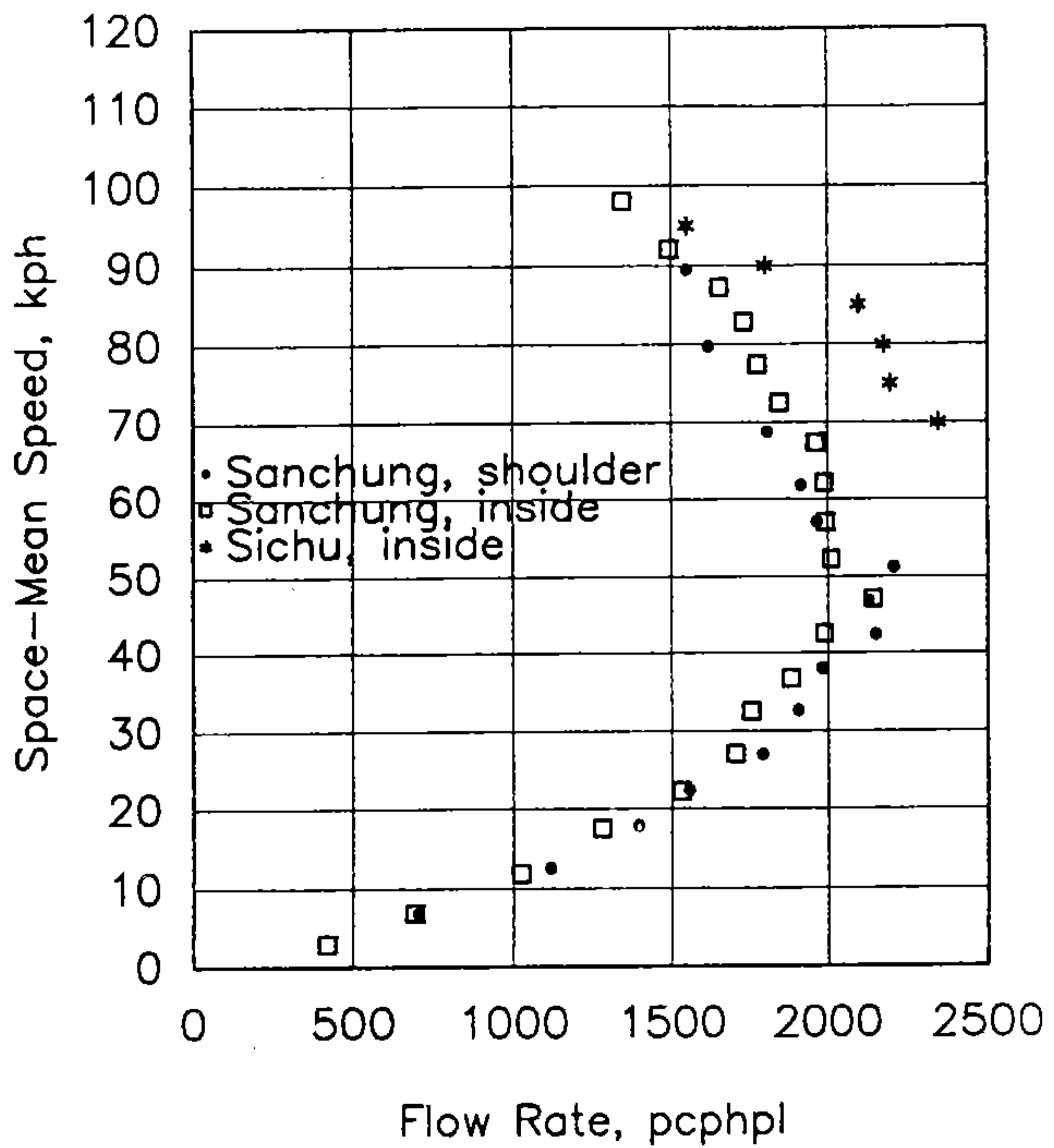


Fig. 34 Expected Service Flow Rates at Different Levels of Speed

service flow rates. In reality, a longer sampling period tends to contain more vehicles with dissimilar speeds. Therefore, the use of longer time periods in an analysis may produce lower expected service flow rates. To examine how sampling period can affect the speed variation in a sample, the standard deviations of the individual vehicle speeds observed at Sanchung are determined for sampling periods ranging from 0.5 min to 5 min. The average standard deviation for each 5-kph interval of space-mean speed is then determined and shown in Fig. 35. This figure reveals several interesting flow characteristics at the Sanchung site. First, longer sampling periods (e.g., 2.5 min or more) tend to contain larger within-sample speed variations. Therefore, data aggregated over longer sampling period will be less capable of revealing the true nature of traffic flow. Second, short sampling periods of 1 min or less reveal that the speed variation changes only slightly for space-mean speeds either above 70 kph or below 50 kph. In contrast, the speed variation nearly doubles when the speed is increased from 50 kph to 70 kph. This phenomenon suggests that, for the Sanchung site, the speed range between 50 kph and 70 kph is a transitory state between stable traffic movement and congested conditions. And, finally, for speeds below 70 kph, the speed variations based on a 5-min sampling period are much larger than those based on a 0.5-min sampling period. This implies that the flow conditions within this speed range may change substantially from one minute to another.

Overall, the aggregation of data over longer sampling periods will result in a greater loss of useful information. Too short a sampling period may not contain enough vehicles for a meaningful analysis. It appears that a sampling period of 1 minute is a reasonable choice.

3.4 Occupancy, Density, And Flow Rate

For the inside lane and the shoulder lane at Sanchung, the relationship between flow rate and occupancy is relatively well defined up to an occupancy of just under 25%. This is illustrated in Figs. 36, 37, 38, and 39. At an occupancy exceeding 25%, flow rate can vary over a wide range. Therefore, the flow conditions at this site can be considered to be approaching saturation at an occupancy of 25%. The maximum 1-min flow rate tends to exist at an occupancy of about 35%. Such a flow rate may be associated with queue discharge.

Density cannot be easily determined in the field; it is often derived from flow rate and space-mean speed according to the following equation:

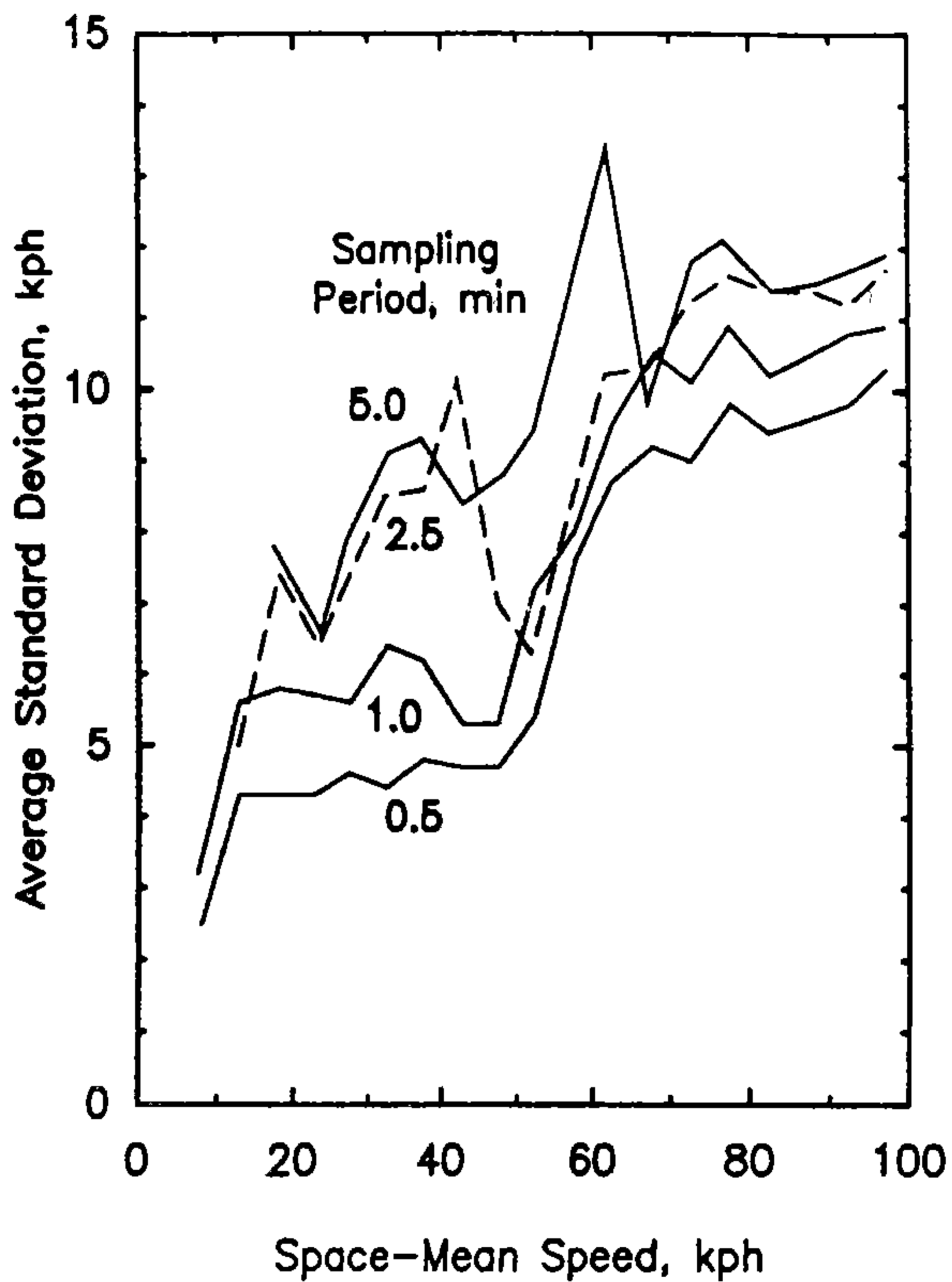


Fig. 35 Average Standard Deviation of Flow Rate as a Function of Speed and Sampling Period

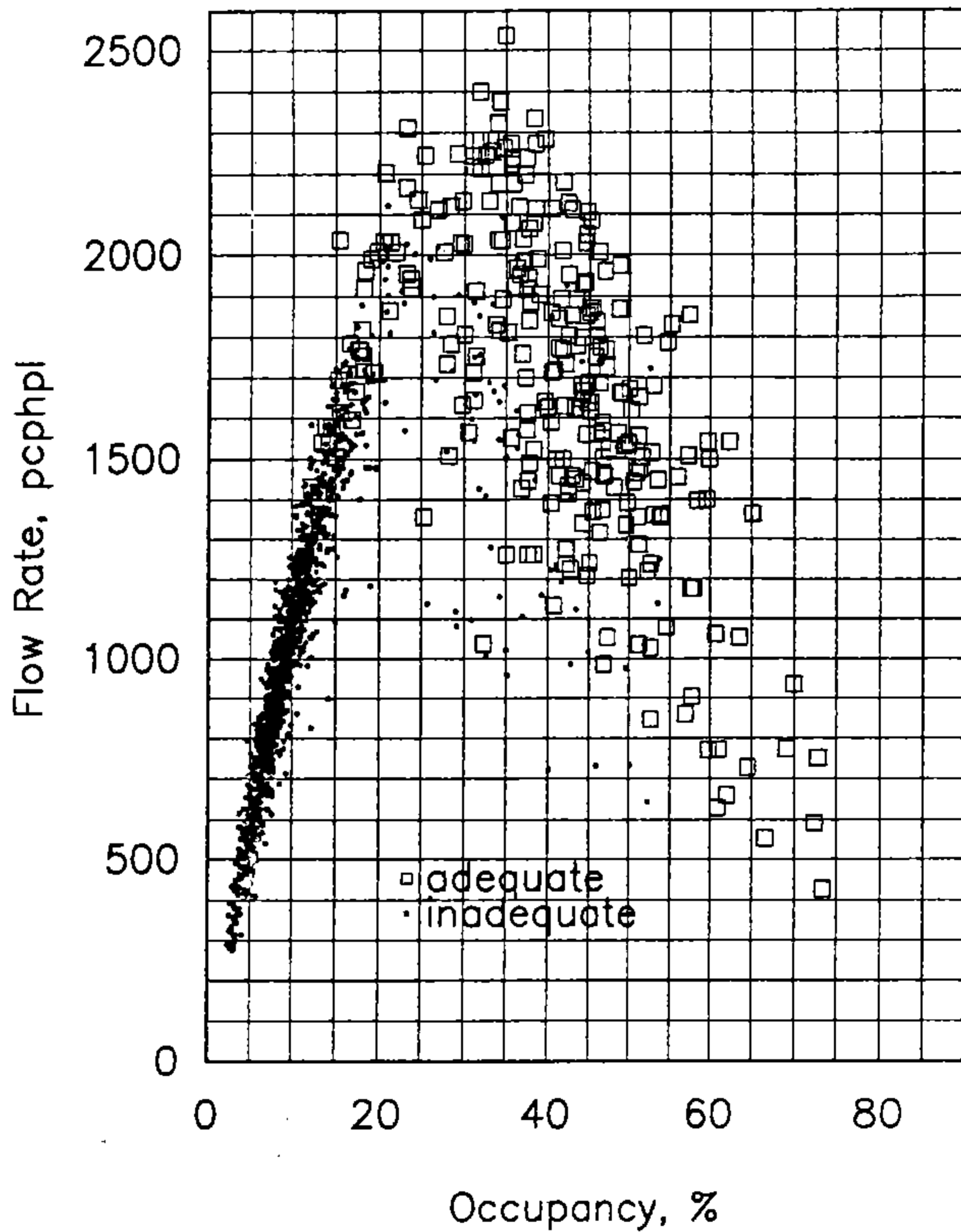


Fig. 36 Flow Rate and Occupancy Relationship, Sanchung Shoulder Lane

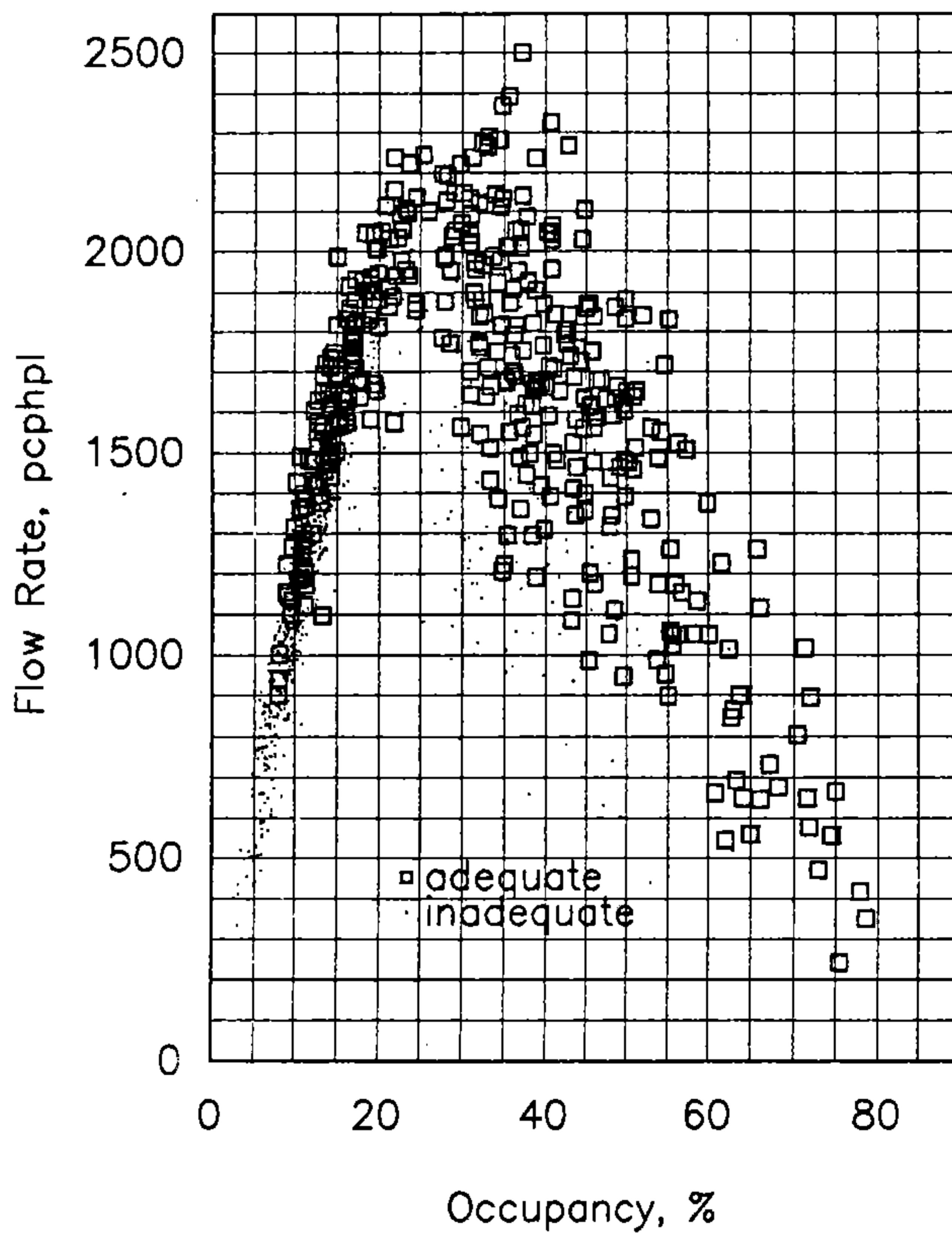


Fig. 37 Flow Rate and Occupancy Relationship, Sanchung Inside Lane

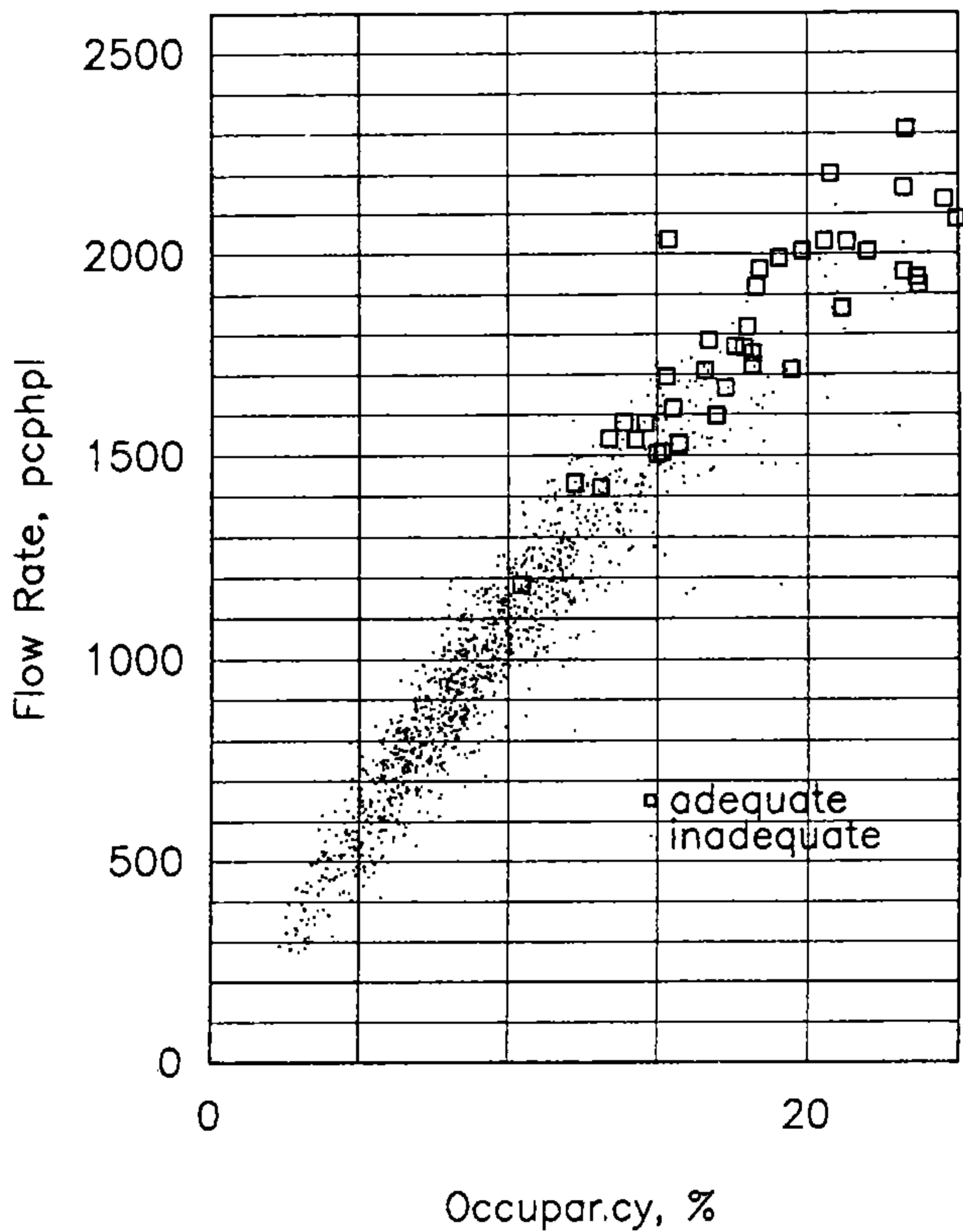


Fig. 38 Flow Rate and Occupancy Relationship under Uncongested Conditions, Sanchung Shoulder Lane

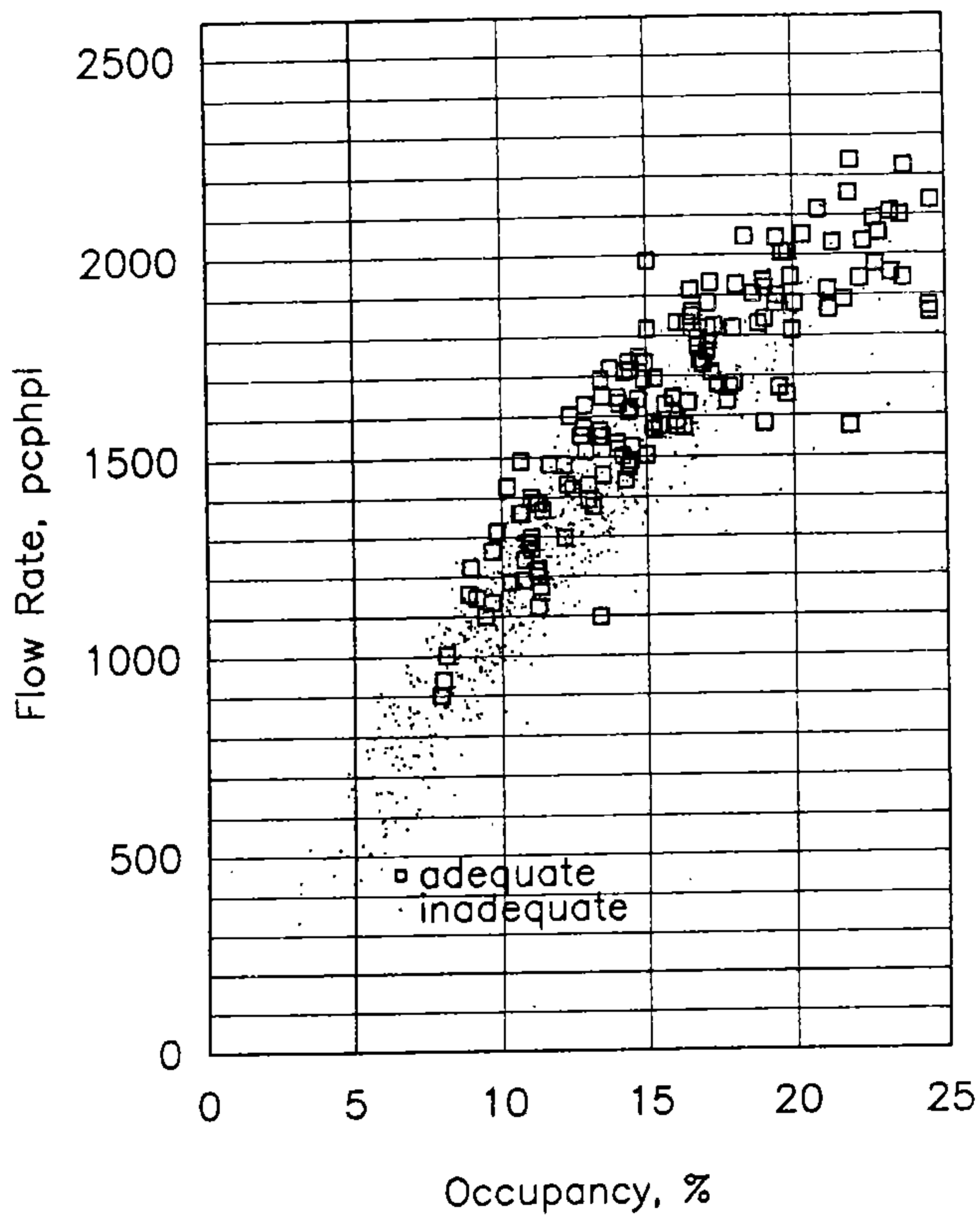


Fig. 39 Flow Rate and Occupancy Relationship under Uncongested Conditions, Sanchung Inside Lane

$$Q = K S \quad (21)$$

where Q = flow rate, vph; K = density, vehicles per lane per kilometer; and S = space-mean speed, kph.

As shown in Fig. 40, the relationship between flow rate and density resembles that between occupancy and flow rate. This is because occupancy and density have similar qualities. Given vehicles are traveling at a uniform speed, density can be determined from occupancy according to the following equation:

$$K = \frac{10 P}{L_e} \quad (22)$$

where P = occupancy, in %; and L_e = effective vehicle length, in m, equal to the sum of detector length and average vehicle length. The occupancy-density relationships derived respectively from the above two equations are shown in Figs. 41 and 42. These figures indicate that Eqs. 21 and 22 are consistent up to an occupancy of about 30%.

3.5 Flow Rate, Occupancy, and Space-Mean Speed

Under congested conditions, flow rate correlates poorly with such parameters as speed, density, and occupancy. This is a reason why little is known about the nature of the flow rates of congested freeways. The lack of strong correlations between flow rate and individual traffic parameters, however, does not imply the absence of strong relationships between flow rate and some combinations of traffic parameters.

The Sanchung data, for example, reveal the existence of a rather well defined relationship between 1-min flow rate and the following speed-occupancy function:

$$\phi = P (S - \xi) \quad (23a)$$

where P = occupancy, %; S = space-mean speed, kph; and ξ is a speed reduction factor defined as follows:

$$\xi = \begin{cases} 1.33 [S - (100 - 1.25 P)] & \geq 0 \quad \text{if } P \leq 25\% \\ 0 & \text{if } P > 25\% \end{cases} \quad (23b)$$

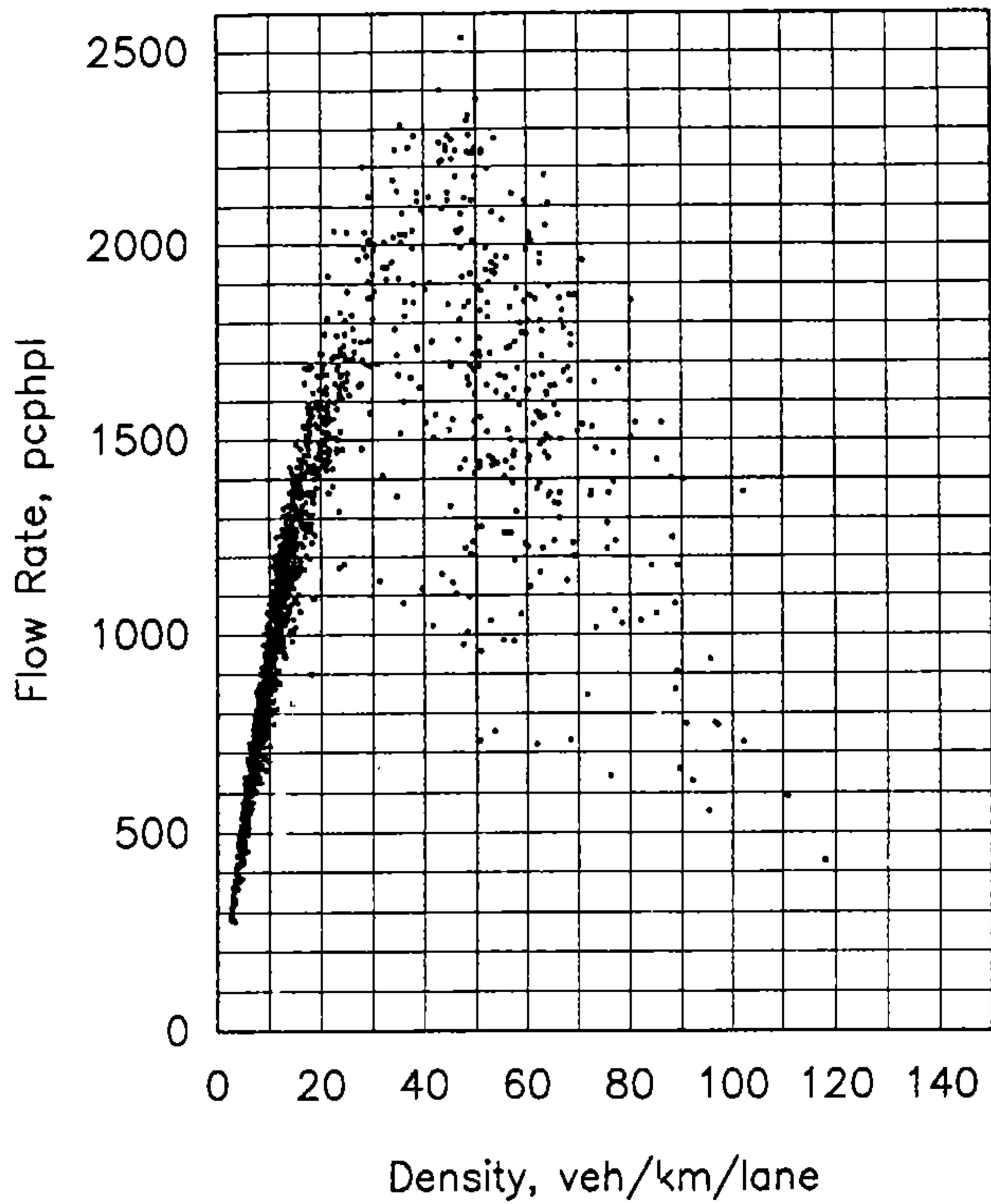


Fig. 40 Flow Rate and Density Relationship, Sanchung Shoulder Lane

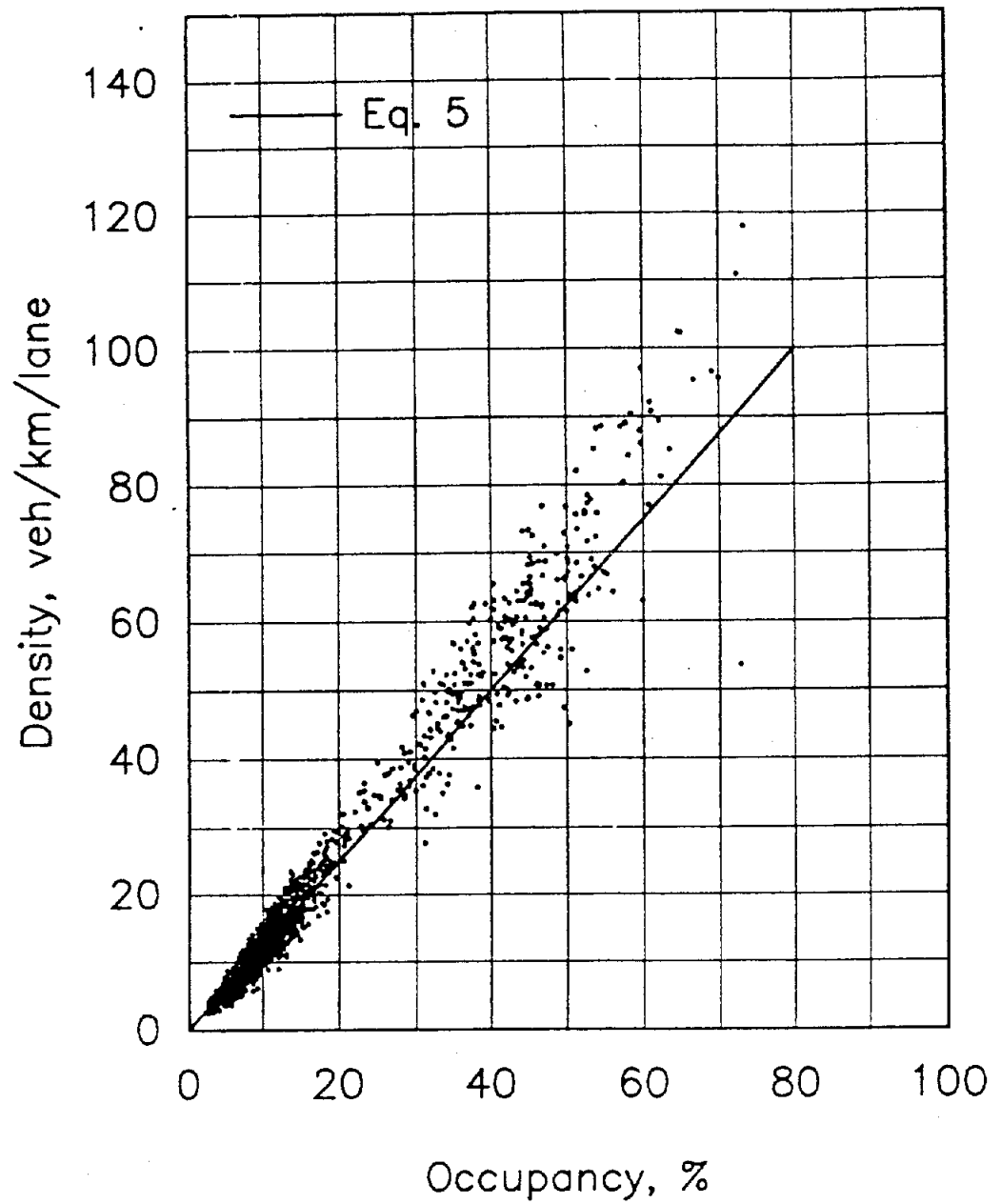


Fig. 41 Density and Occupancy Relationship, Sanchung Shoulder Lane

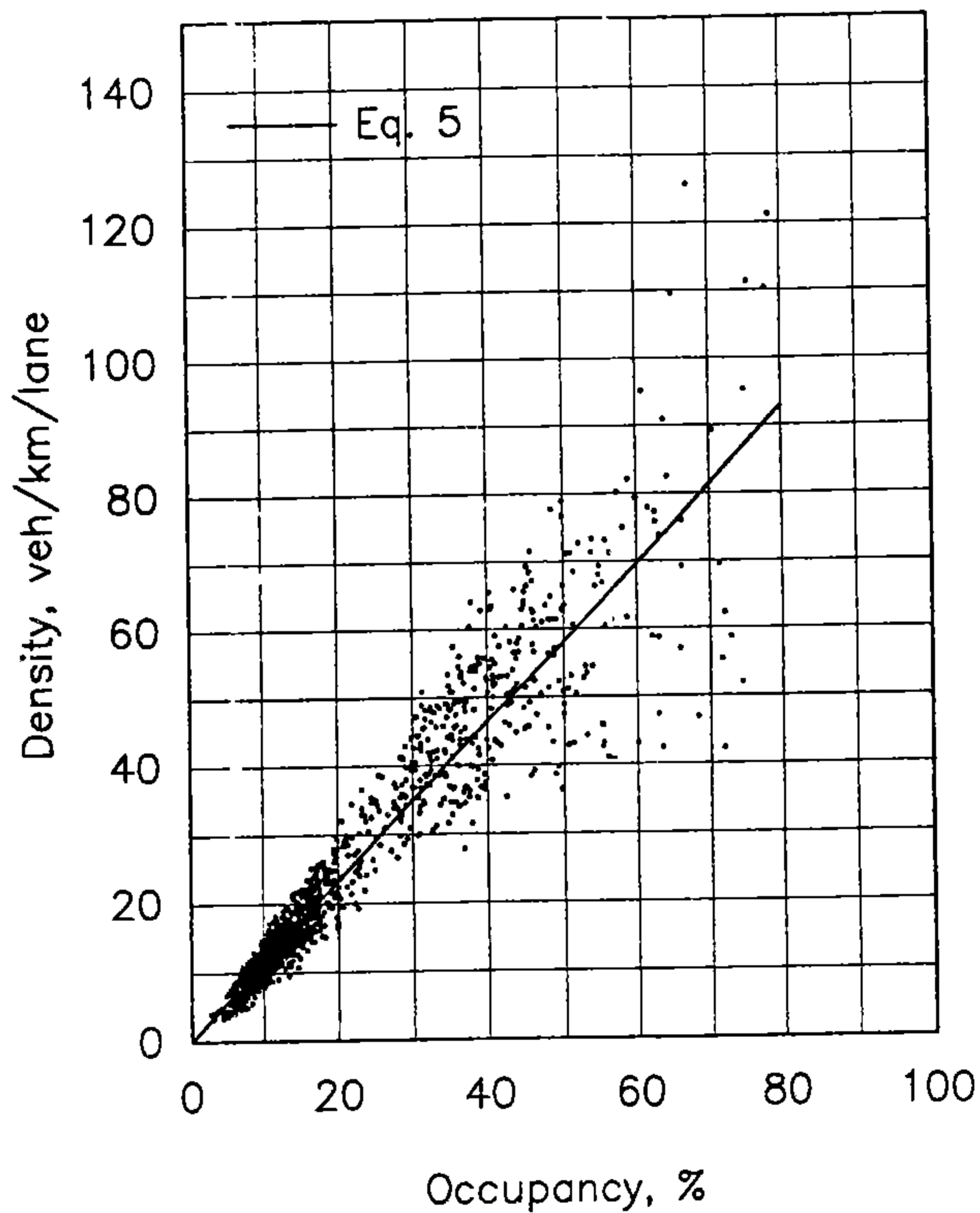


Fig. 42 Density and Occupancy Relationship, Sanchung Inside Lane

The relationships between flow rate and this speed-occupancy function are shown in Fig. 43, and 20 based on the Sanchung data. As shown in Table 4, the standard deviations of flow rate when the speed-occupancy function F is used as a predictor are much smaller than those when either occupancy or space-mean speed is used, particularly under congested conditions ($P > 25\%$).

Table 4 Standard Deviations of Flow Rate (pcphpl) in Relation to Predictors based on Sanchung Data Aggregated at 1-min Intervals

Predictor	Occupancy < 25%		Occupancy $\geq 25\%$	
	Shoulder Lane	Center Lane	Shoulder Lane	Center Lane
Occupancy P	136.4	143.2	308.3	292.3
Speed S	232.3	210.7	271.2	247.1
Function Φ	124.2	110.1	93.0	109.4

It is also found that the 1-min service flow rates observed at Sanchung under varying conditions can be estimated from a simple model containing space-mean speed and occupancy as the governing variables. Let Q be the service flow rate, then the model can be represented by

$$Q = 900 (1 - e^Y) + \delta P \leq 122 P \quad (24a)$$

where

$$Y = -0.06 (S - 30) \leq 0 \quad (24b)$$

and

$$\delta = \begin{cases} 1.5 S & \text{if } S \leq 30 \\ 38 + 0.24 S & \text{if } S > 30 \end{cases} \quad (24c)$$

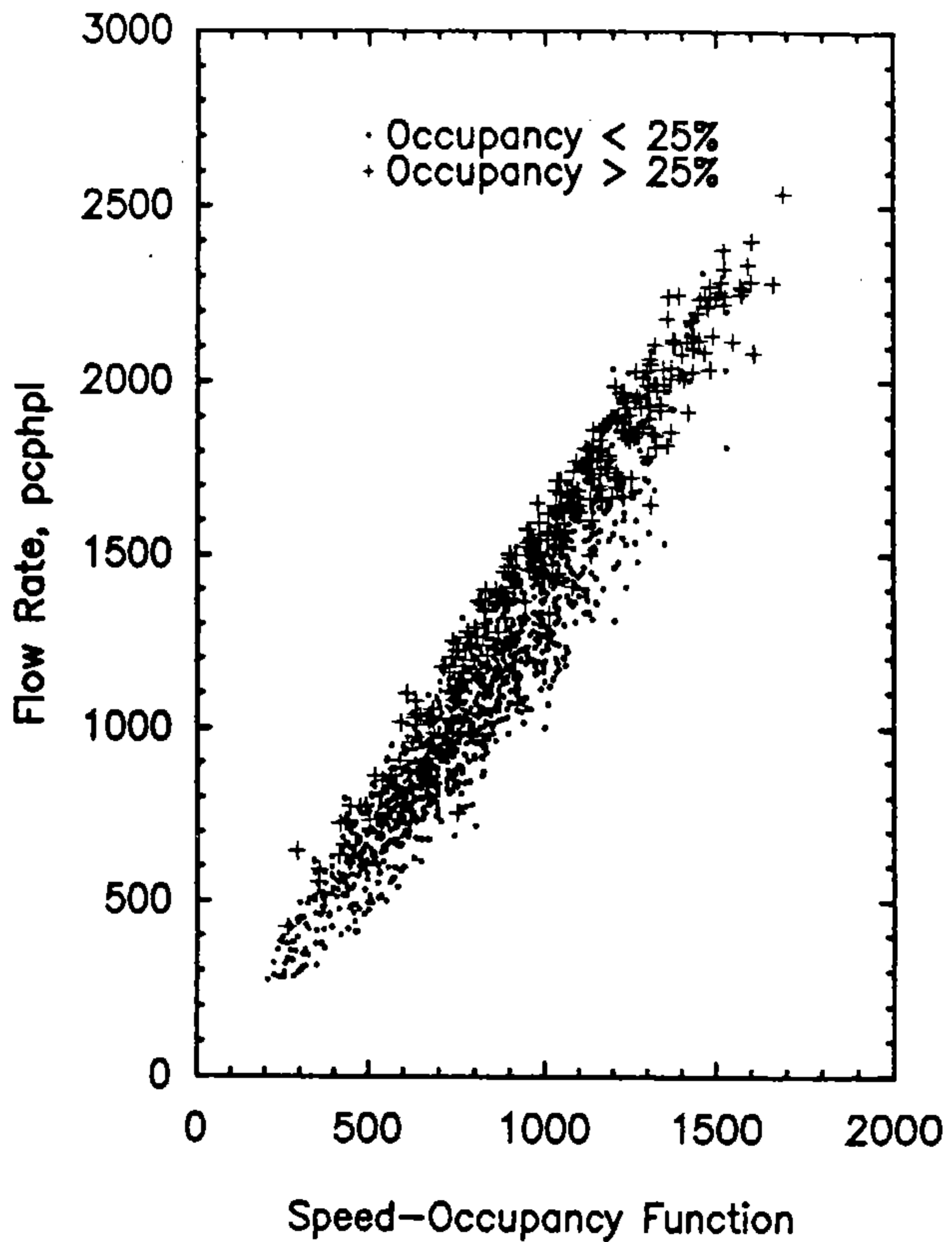


Fig. 43 Flow Rate as a Function of Occupancy and Speed, Sanchung Shoulder Lane

A comparison of the observed and the estimated 1-min service flow rates is shown in Fig. 44.

It is not known whether congested flows at different locations have similar characteristics. If they do, then it will be possible to classify congested operations into several meaningful levels of service by using a combination of occupancy and speed.

Space-mean speed is a function of occupancy. Their relationships at the Sanchung site are shown in Figs. 45 and 46. The expected occupancy for a given space-mean speed for vehicles in platoons are shown in Fig. 47.

3.6 Development of Analysis Methodology

The methodology to be developed for the analysis of mainline sections will consider the following elements: traffic demand, geometric design features, passenger car equivalent, measures of effectiveness, level-of-service criteria, and procedures for planning analysis and operational analysis. The details of the methodology cannot be finalized until the representative characteristics of the various sections are identified. In principle, the development of the methodology will be guided by the following concerns:

1. The same level-of-service criteria should be used to evaluate all mainline sections in order to promote design consistency.
2. The relationships among traffic parameters can be expected to vary with the location of a lane. The methodology should reflect such a relationship.
3. Congested conditions can be associated with a wide range of speed. Assigning a single level of service (i.e., LOS F) to congested conditions makes it difficult to assign intelligently priorities for traffic improvement. Therefore, there is a need to classify congested conditions into several levels of service.

Space-mean speed and occupancy are two major candidate measures of effectiveness being evaluated. It is premature to establish level-of-service criteria because much of the operating characteristics of the various freeway components are still unknown. Additional data are being collected to address this problem.

For sections adjacent to ramps, the data collection effort is directed toward identifying the relationship between freeway flow, ramp flow, and the critical operating characteristics adjacent to the freeway-ramp junctions. The critical operating characteristics are those associated with the worst quality of service. They will also be examined in terms of the interactions among speed, flow, and occupancy.

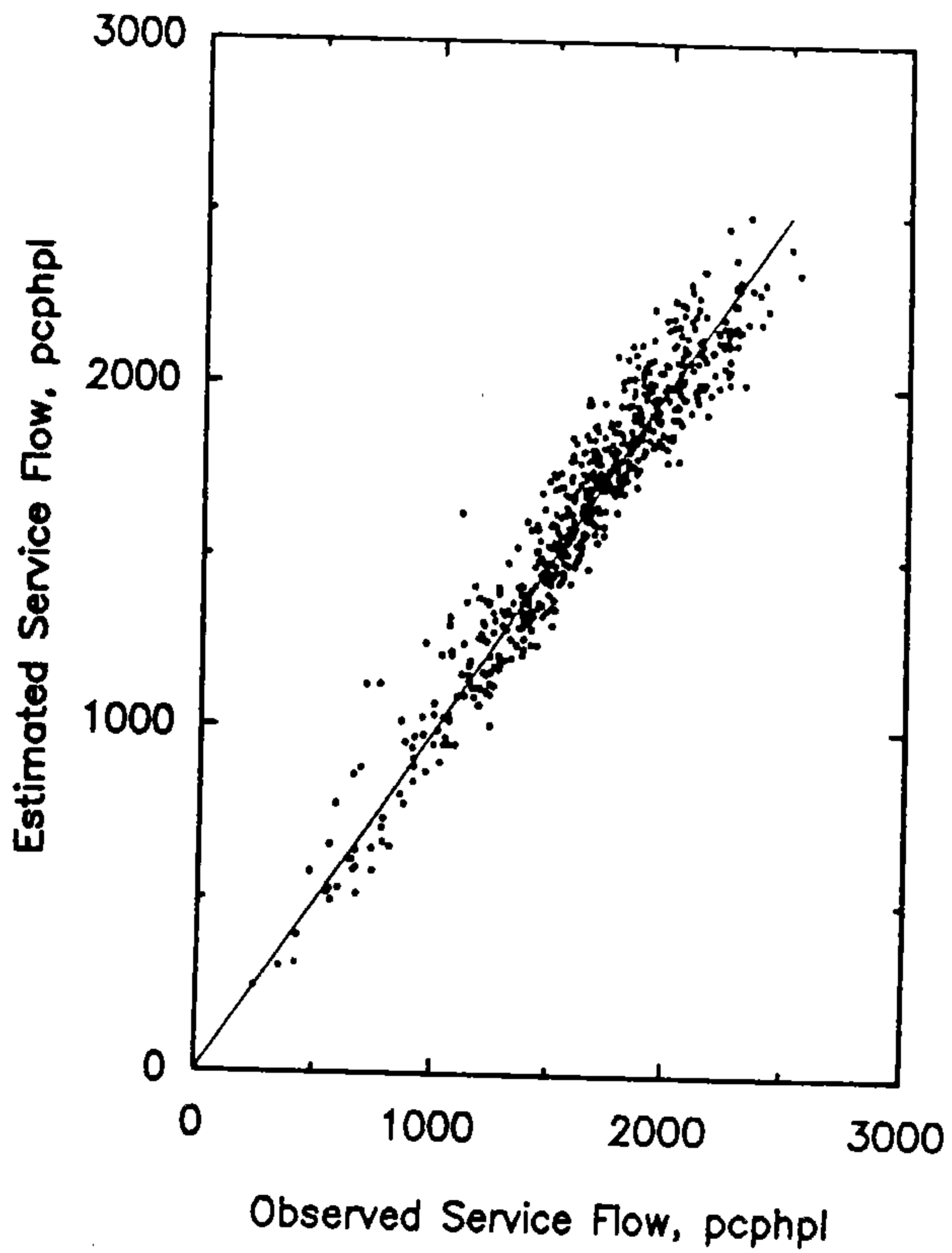


Fig. 44 Observed Service Flow Rates and Estimates

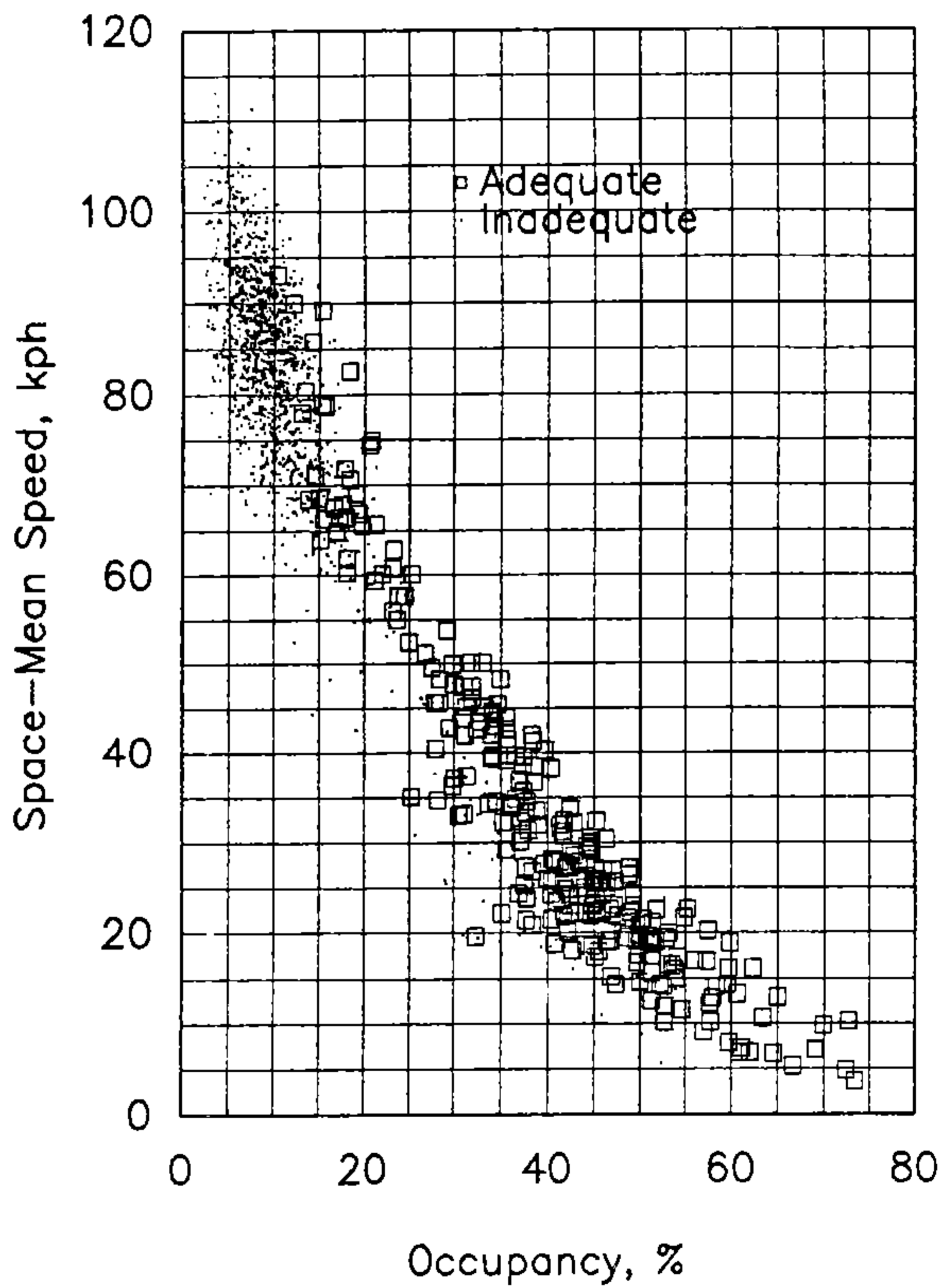


Fig. 45 Variation of Speed with Occupancy, Sanchung Shoulder Lane

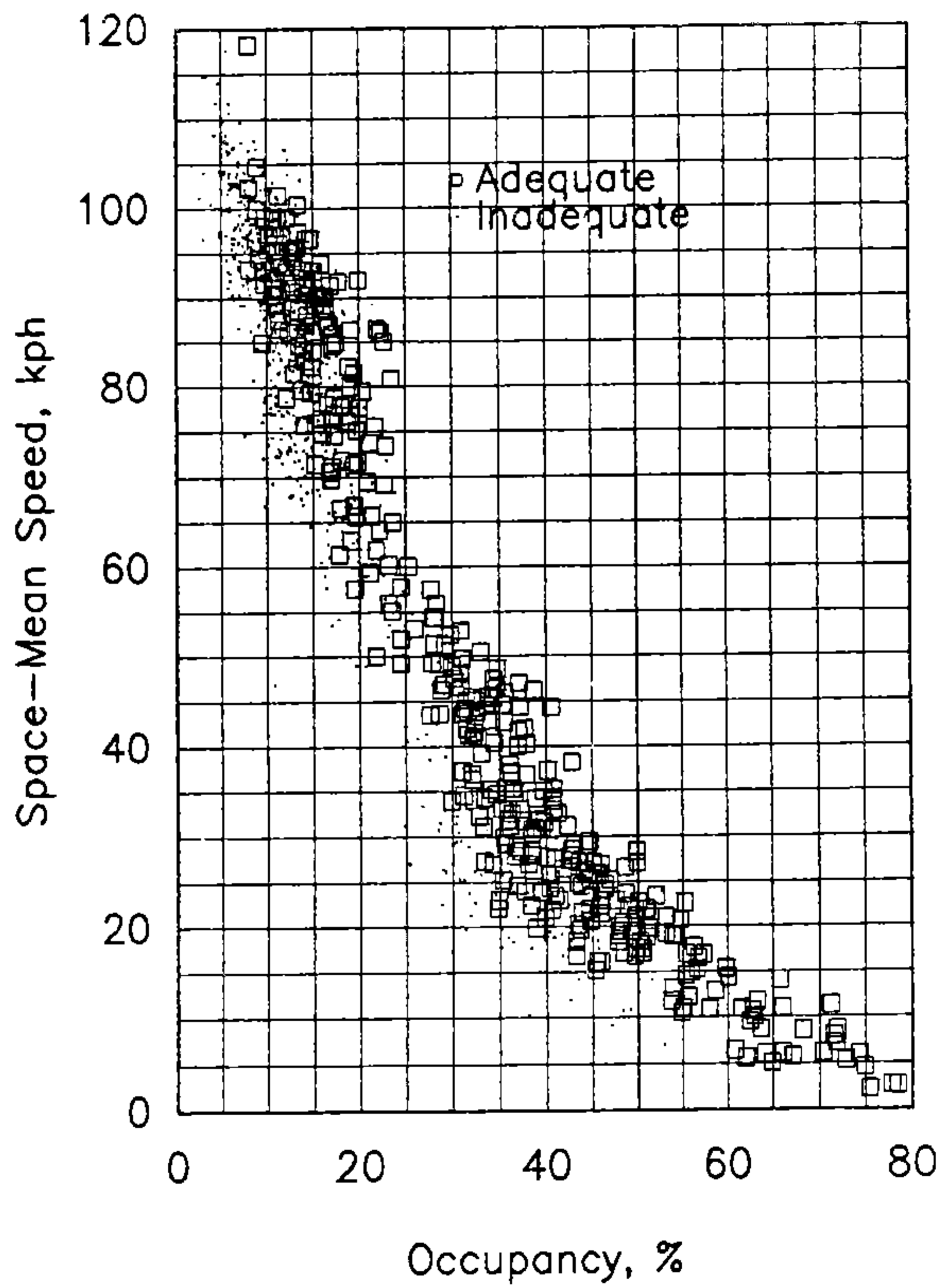


Fig. 46 Variation of Speed with Occupancy, Sanchung Inside Lane

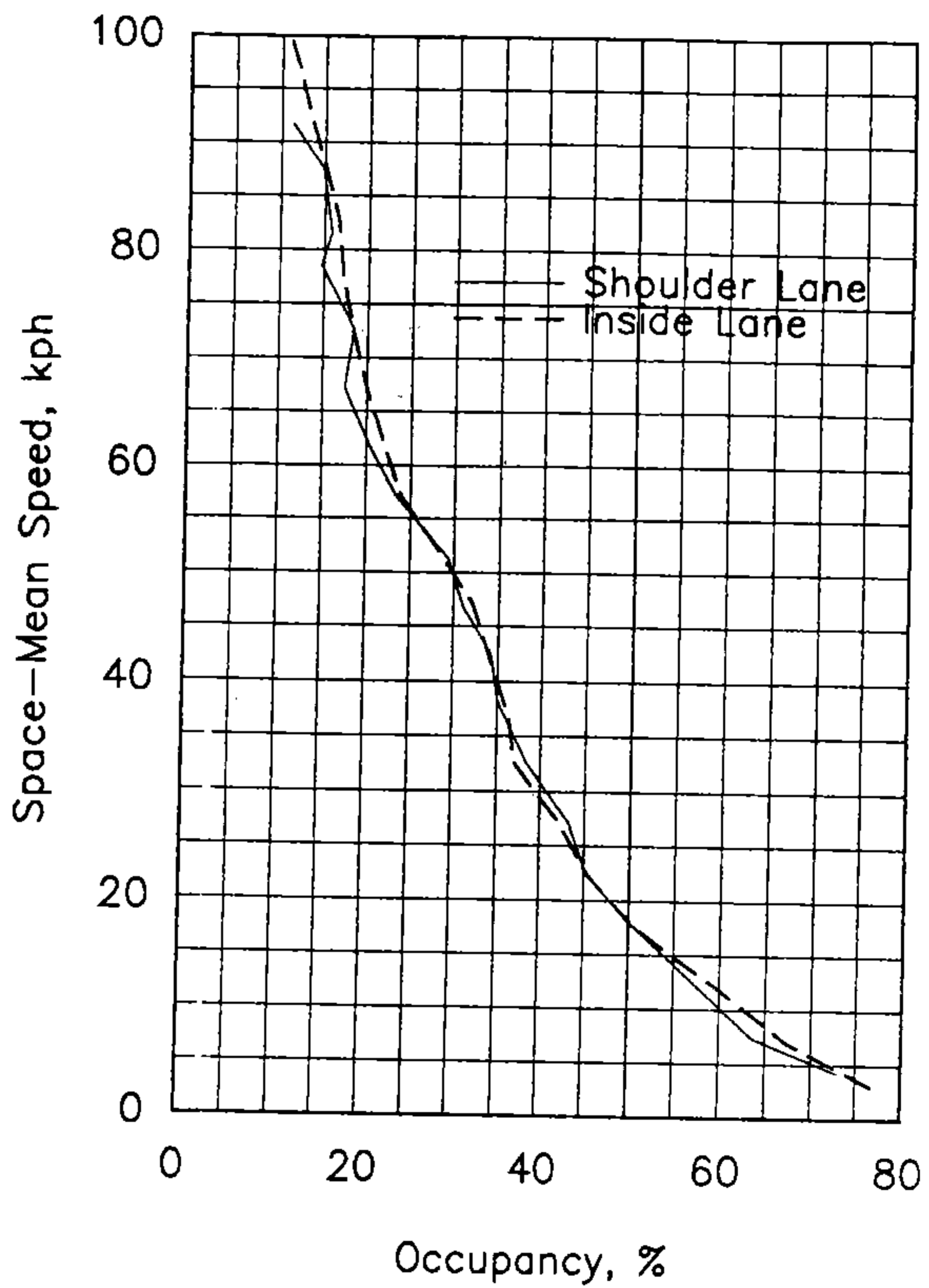


Fig. 47 Expected Relationship between Speed and Occupancy for Vehicles in Platoons

4.0 RAMPS

Ramps are designed to bridge traffic movement between freeways and surface streets. Therefore, their levels of service should be judged in terms of the efficiency at which a given traffic demand can be processed through them. The major concerns about the traffic operations at ramps include capacity, queue length, and delay. The capacity of an on-ramp is a function of ramp geometric design and freeway traffic conditions. At a metered on-ramp, the capacity also depends on the metering rate. The capacity, in turn, affects queue length and delay. The capacity of an off-ramp is often dictated by the capacity of the junction between the ramp and the surface streets. As an initial effort, only on-ramps that are not metered will be considered. Off-ramps will be analyzed in the future.

4.1 Estimation of Ramp Capacity

To identify the capacities of an on-ramp for a given flow condition on the freeway, it is necessary to have a continuous supply of ramp vehicles while the flow condition on the freeways remain unchanged for an extended period of time. Such a situation may exist only when both freeway and ramp are congested. Therefore, it is extremely difficult to rely on field data alone to quantify the capacities of on-ramps over a wide range of freeway conditions. Because of this difficulty, little is known about the real nature of the capacities of on-ramps.

Geometric design features have a major influence on the capacities of on-ramps. Such features include lane and shoulder widths, curvature, angle of convergence with freeway, and type and length of acceleration lane. When there are no conflicting vehicles approaching the ramp junction from the freeway, these geometric design features will determine the speed-flow relationship on an on-ramp. There are no data to show how the speed and flow rate on an on-ramp are related. Nevertheless, the speed-headway relationship as shown in Fig. 28 may offer a clue. This figure shows that, for passenger cars on freeways moving in platoons at speeds between 30 kph and 60 kph, their average headways vary from about 1.73 sec to about 2.00 sec. It is likely that the average headways of platoon vehicles on an on-ramp for the same speed range would be longer because of more restricted ramp design and the need for drivers to check the potential conflicts with freeway vehicles. The presence of two or more lanes can further increase the headways. How much longer such headways are can be determined from field data. For the time being, let us assume that, if the capacity of a ramp is governed only by the ramp geometric design, the average headway on an on-ramp for speeds ranging from 30 kph to 60 kph is 2.5 sec. This would imply a capacity of 1,440 pcphpl. A more reliable estimate can be made based on the headways of platoon

vehicles on ramps.

When vehicles are present on the shoulder lane of a freeway, the capacity of an on-ramp will also be affected by the way the drivers on the ramp accept or reject the gaps or lags in the freeway traffic. An example of the gap acceptance behavior of the drivers on Chung-San Freeway is shown in Fig. 48 . This figure reveals the probability that a gap equal to or shorter than a specified length will be accepted and the probability that a gap longer than a specified length will be rejected. Theoretically, the probability that a given gap will either be accepted or rejected is 1.0. The field data deviate from this theoretical expectation because of the bias in sampling accepted gaps and rejected gaps. Therefore, the observed probabilities are adjusted and the resultant probabilities are also shown in the same figure. Both the observed and the adjusted probabilities have a median accepted gap of about 3 sec. This value is for an average freeway speed of 78 kph and a flow rate of 1,521 vph; the average speed of the ramp vehicles is unknown. The median accepted gap is sometimes referred to as critical gap. Critical gap has been known to vary with the angle of convergence, the type and length of acceleration lane, and the number of vehicles accepting a gap. Existing data [5] show that in the cases of multiple entries each additional merging vehicle consumes about 2.4 sec of an available gap. The portion of a gap consumed by each additional vehicle can be expected to increase if the entry speeds of ramp vehicles decrease. Since the average portion of an available gap consumed by an additional merging vehicle should not be longer than the average headway between ramp vehicles, let us use a value of 2.5 sec for ramp vehicle with entry speeds of 60 kph and 3.0 sec for ramp vehicles with 30-kph entry speeds.

Given that the critical gap is α , we may assume that an available gap shorter than α will be rejected while a gap longer than α will be used by at least one ramp vehicle. If the portion of a gap consumed by each additional merging vehicle is β , then each gap longer than α can be accepted by J ramp vehicles, where J is the integer value of $1.0 + (\text{gap size} - \alpha) / \beta$. If we further assume that the minimum headway of freeway vehicles is τ , the flow rate on freeway shoulder lane is Q , and the freeway vehicles arrive at the ramp junctions at random, then the ramp capacity can be estimated as

$$(Q_{\max})_r = Q \frac{e^{-\frac{\alpha - \tau}{H - \tau}}}{1 - e^{-\frac{\beta}{H - \tau}}} \quad (9)$$

where $(Q_{\max})_r$ = ramp capacity and H = average headway in freeway shoulder lane, i.e., $1/Q$.

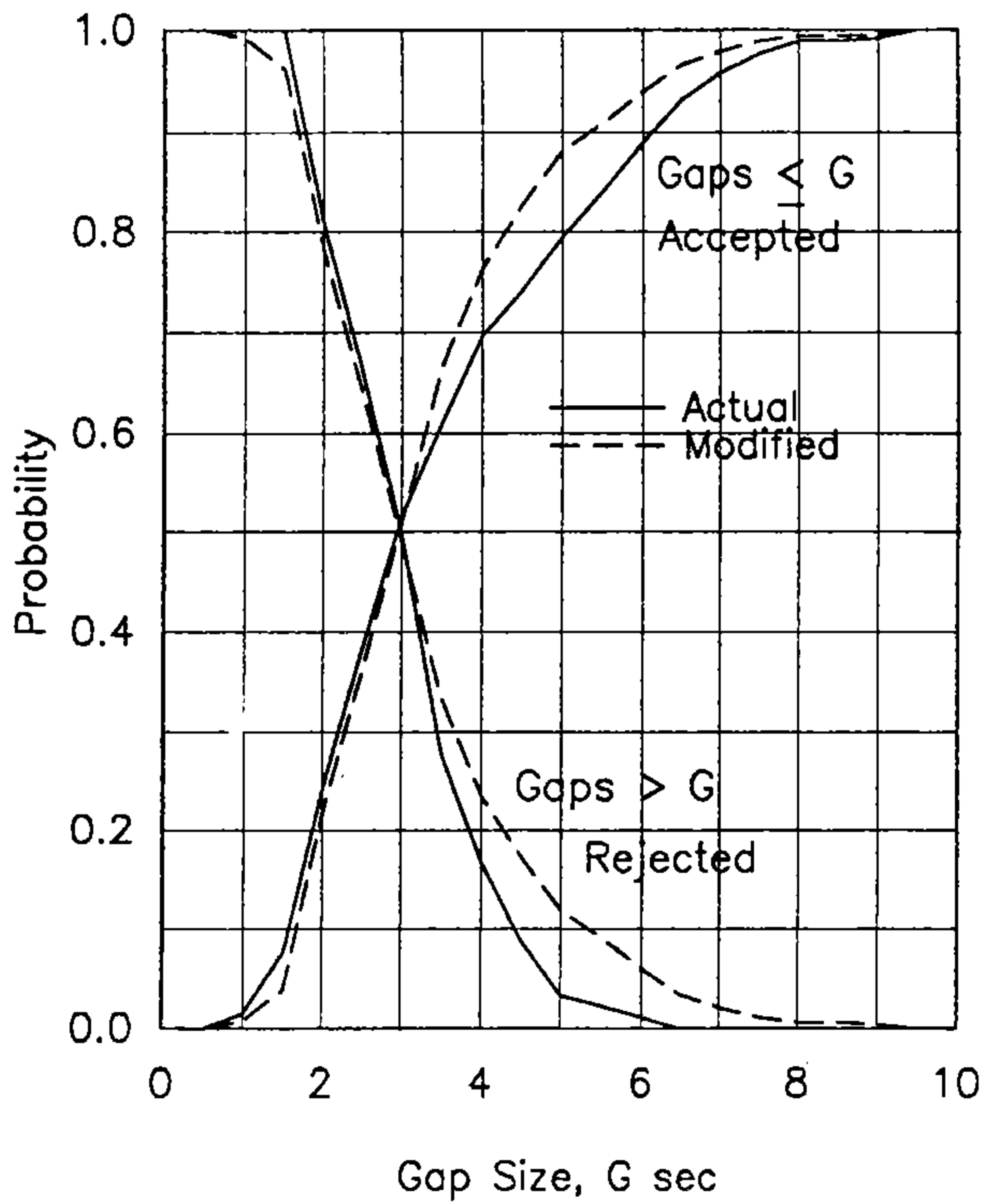


Fig. 48 Gap Acceptance Behavior of Ramp Drivers on Chung-San Freeway

The ramp capacities estimated from this equation for entry speeds of 30 kph and 60 kph are shown in Fig. 49. The minimum headway on the freeway used in this analysis is assumed to be 0.5 sec. It should be noted that, when the freeway flow is heavily congested (space-mean speed drops below 30 kph), gap acceptance behavior may no longer apply for the estimation of ramp capacity. Under such a condition, field observation is the best way of determining the rate at which ramp drivers can force their way onto the freeway. More field data concerning the characteristics of critical gap, speed-headway relationship, and the gaps required for multiple entries are also needed to modify the capacity estimates shown in Fig. 49.

4.2 Modeling Average Queue Length and Delay

For level-of-service analysis, the average queue length and delay of ramp vehicles need to be modeled. The factors that may be considered in modeling these measures of effectiveness include but are not limited to ramp geometric design (e.g., average curvature, lane and shoulder width, angle of convergence, and type and length of acceleration lane), vehicle speed and flow rate in freeway shoulder lane, the rate at which vehicles enter a ramp, vehicle mix, and ramp capacity.

Field data and simulation data will be used to develop the needed model.

4.3 Development of Analysis Methodology

The analysis methodology will use the geometric design of ramp and the traffic conditions at the ramp junction as the input. Based on the input data, capacity and the related average queue length and delay can be estimated. By examining the characteristics of queue length and delay, level-of-service criteria will be established. For planning applications, high degree of accuracy may not be necessary in estimating the measures of performance. Therefore, separate procedures may be developed for planning applications and for operational analysis.

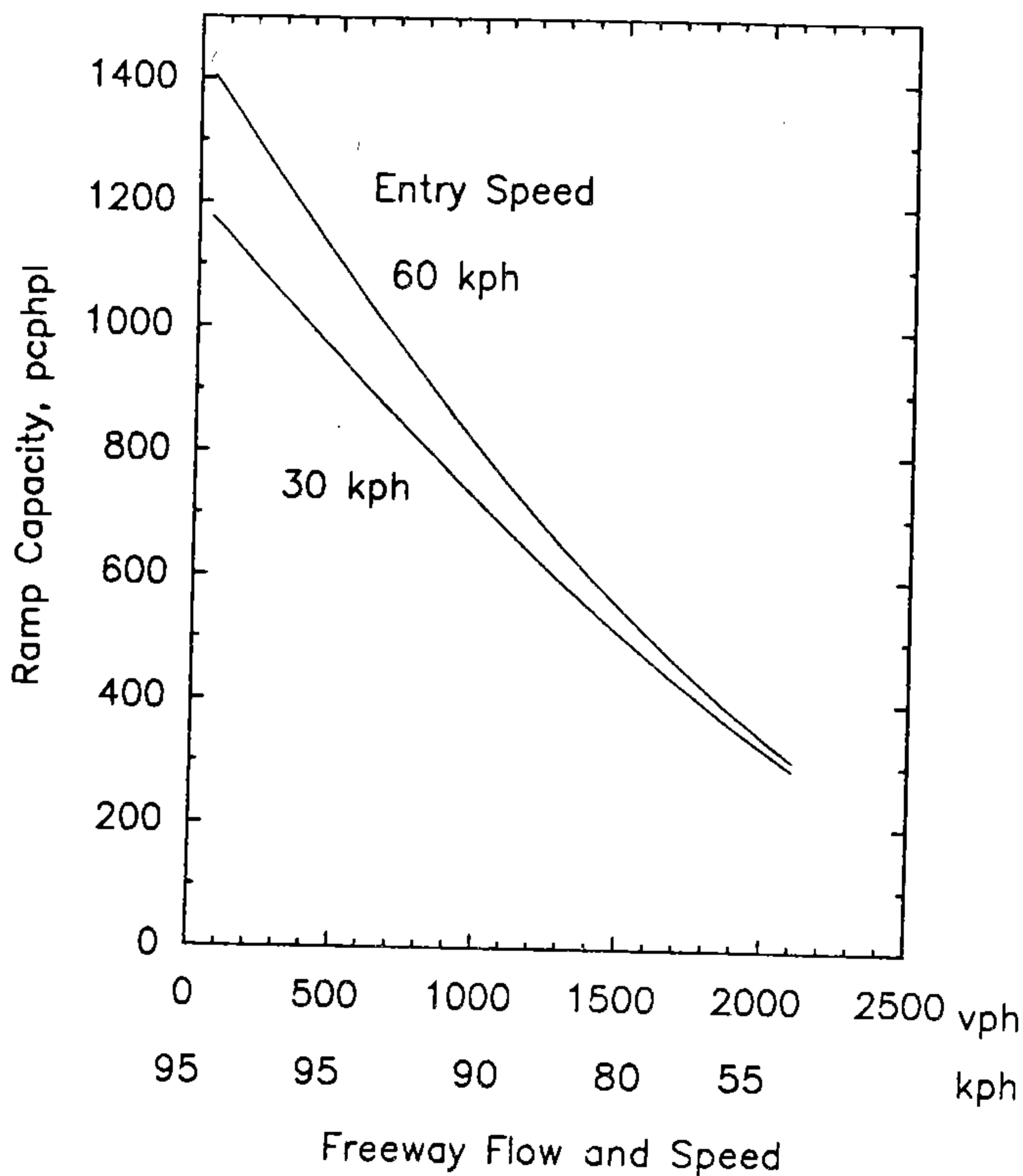


Fig. 49 Estimated Capacities of On-Ramps

5.0 CONCLUSIONS

For level-of-service analysis, freeways are divided into mainline sections, toll plazas, on-ramps, and off-ramps. The mainline sections include weaving sections, ramp sections, tunnels, and basic sections not affected by ramps and weaving sections. Weaving section, tunnels, and off-ramps are beyond the scope of this research effort.

The methodologies being developed will emphasize the need to promote design consistency. They will also recognize the unique features of each freeway component. Therefore, a set of uniform criteria will be established for evaluating mainline sections, and additional criteria may also be used to guide the planning, design, and operation of other freeway components.

The traffic operations at toll plazas can be affected by the vehicle arrival pattern, driver behavior, number and capacities of toll gates, and the geometric designs of plazas. Because of the large number of influencing factors involved, analytical models have limited applications in the analysis of toll plaza operations. Therefore, Toll Plaza Simulation (TPS) model is developed to assist in the planning, design, and operation of toll plazas. TPS model, written in FORTRAN 77, is intended for personal computer applications. The Institute of Transportation, Ministry of Transportation and Communications is in the process of preparing a user manual for public distribution.

TPS model is used in this project to analyze the characteristics of traffic operations at toll gates. The simulation results reveal that when the volume to capacity ratio for a toll gate is less than about 0.93, the gate performance as measured in terms of queue length has a good chance of being stable. Under such a state, the average queue length at any moment in time is generally under 3 vehicles, but the maximum queue length may exceed 10 vehicles. With a volume to capacity ratio of greater than 0.93, the gate performance is likely to become metastable or unstable. Under an unstable state, queue length may grow over time if the arriving flow rate remains the same.

Based on existing practices in toll plaza management and the understanding of the characteristics of traffic operations at toll plazas, a level-of-service analysis methodology is recommended for planning and operational applications. This methodology uses average queue length and average time in system as the measures of effectiveness for classifying level of service. Level of service is classified into six levels in accordance with the criteria given in Table 2. Procedures for planning and operational analyses are described in this report. For planning analysis, it is suggested that analytical models be used to develop a preliminary design concerning the required lengths of full-width approach lanes and

the number of gates needed for each gate type. TPS Model is to be used to evaluate the preliminary design for possible modifications. For operational analysis, analytical models have uncertain reliability and, therefore, it is suggested that the level-of-service analysis be based either on field studies or TPS model. If these options are not practical due to resources constraints, then a shortcut procedure described in this report can be employed to estimate level of service.

It is expected that the TPS model will be enhanced in the future. One area of enhancement lies in the treatment of automatic toll collection and toll-free operations. Due to the lack of behavioral data, the current model provides only crude simulation of such operations. Another area of enhancement concerns the expansion of the model to address fuel consumption, air pollutant emissions, and total vehicle operating costs for benefit and cost analysis.

The development of the methodologies for analysis of freeway mainline sections and ramps is still on-going. Space-mean speed and occupancy are two major measures of effectiveness being considered for establishing the mainline level-of-service criteria. These two parameters, when used simultaneously, are found to be able to predict rather reliably the service flow rate of a mainline section. Queue length and delay are important concerns of ramp operations; they will be modeled on the basis of field data and computer simulation data. The characteristics of queue length and delay will be analyzed to establish level-of-service criteria for analysis of ramps.

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