

1. Report No. FHWA/TX-85/44+304-2F		2. Government Accession No.		3. Recipient's Catalog No.	
4. Title and Subtitle CHARACTERIZING THE QUALITY OF TRAFFIC SERVICE IN URBAN STREET NETWORKS				5. Report Date November 1984	
				6. Performing Organization Code	
7. Author(s) Robert Herman and Siamak Ardekani				8. Performing Organization Report No. Research Report 304-2F	
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin Austin, Texas 78712-1075				10. Work Unit No.	
				11. Contract or Grant No. Research Study 3-8-80-304	
12. Sponsoring Agency Name and Address Texas State Department of Highways and Public Transportation; Transportation Planning Division P. O. Box 5051 Austin, Texas 78763				13. Type of Report and Period Covered Final	
				14. Sponsoring Agency Code	
15. Supplementary Notes Study conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration. Research Study Title: "Quality of Traffic Service"					
16. Abstract The characterization of the quality of traffic service in urban street networks has been made according to the Two-Fluid Model of Town Traffic. A comparison of traffic related characteristics in various cities around the world using the Two-Fluid methodology has given insight into the physical network features which most strongly affect the quality of traffic service and the model parameters. Through ground experiments and simultaneous aerial observations, it has been shown that the model assumptions are reasonable. The Two-Fluid model has also been used in before/after studies in Dallas, Lubbock and San Antonio, where changes in traffic control strategies and mix of vehicles had taken place. The sensitivity of the model parameters to the vehicle type used in the data collection has also been investigated. Finally, time-lapse aerial photographs of traffic in Austin and Dallas have been reduced and analyzed to establish relations among network-wide averages of fraction of vehicles stopped, speed, concentration and flow, hence improving the Two-Fluid methodology by allowing the comparison of the quality of traffic service in various networks to be made under similar vehicular concentrations.					
17. Key Words Two-Fluid Model, ergodicity, urban traffic network, running time per unit distance, vehicular concentration, vehicular flow, chase car method			18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.		
19. Security Classif. (of this report) Unclassified		20. Security Classif. (of this page) Unclassified		21. No. of Pages 202	22. Price

CHARACTERIZING THE QUALITY OF TRAFFIC SERVICE
IN URBAN STREET NETWORKS

by

Robert Herman
Siamak Ardekani

Research Report Number 304-2F

Quality of Traffic Service
Research Project 3-8-80-304

conducted for

Texas
State Department of Highways and Public Transportation

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

by the

Center for Transportation Research
Bureau of Engineering Research
The University of Texas at Austin

November 1984

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PREFACE

The research leading to this report was supported in part by the Federal Highway Administration of the U.S. Department of Transportation through the Highway Planning and Research funds administered by the Texas State Department of Highways and Public Transportation. The aerial photography carried out by the Texas State Department of Highways and Public Transportation has also been a vital part of this research.

The authors are grateful to Professor Hani Mahmassani for fruitful discussions. The authors also expressed their gratitude to Professor Herbert H. Woodson for providing a grant from the Center for Energy Studies, The University of Texas at Austin, through which the principal test vehicle was obtained.

Thanks are also due many volunteer students at The University of Texas at Austin who participated in the collection of a major portion of the data used in the study. Additional thanks are extended to Mark Pierce and Lindsey C. Morrisett for their efforts in many respects, particularly for their diligent reduction of aerial photographic data.

The authors are also grateful to James C. Williams, Eric Nudell and Steven D. Williams for traffic observations in Albuquerque and to Bahman Izadmehr for assistance in the data collection in Lubbock.

Finally, the efforts of Candace A. Gloyd in typing and redaction of this report are greatly appreciated.

LIST OF REPORTS

1. Report No. 304-1, "Quality of Traffic Service," by Siamak Ardekani and Robert Herman, presents a discussion of the theoretical Two-Fluid model, the physical interpretation of the model and its parameters, the calibration of the model for the cities of Austin and Dallas, and the results of experiments to verify the model assumptions.
2. Report No. 304-1(S), "Quality of Traffic Service," by Siamak Ardekani and Robert Herman, summarizes report No. 304-1 emphasizing the practical significance of the Two-Fluid approach.

ABSTRACT

The characterization of the quality of traffic service in urban street networks has been made according to the Two-Fluid Model of Town Traffic. A comparison of traffic related characteristics in various cities around the world using the Two-Fluid methodology has given insight into the physical network features which most strongly affect the quality of traffic service and the model parameters. Through ground experiments and simultaneous aerial observations, it has been shown that the model assumptions are reasonable. The Two-Fluid model has also been used in before/after studies in Dallas, Lubbock and San Antonio, where changes in traffic control strategies and mix of vehicles had taken place. The sensitivity of the model parameters to the vehicle type used in the data collection has also been investigated. Finally, time-lapse aerial photographs of traffic in Austin and Dallas have been reduced and analyzed to establish relations among network-wide averages of fraction of vehicles stopped, speed, concentration and flow, hence improving the Two-Fluid methodology by allowing the comparison of the quality of traffic service in various networks to be made under similar vehicular concentrations.

KEY WORDS: Two-Fluid Model, Ergodicity, Urban Traffic Network, Collective Effects, Quality of Traffic Service, Level of Service, Fraction of Time Stopped, Fraction of Vehicles Stopped, Stop Time per Unit Distance, Trip Time per Unit Distance, Running Time per Unit Distance, Average Running Speed, Vehicular Concentration, Vehicular Flow, Aerial Photography, Chase Car Method

SUMMARY

The Two-Fluid Model of Town Traffic has been shown to be a feasible means of characterizing the quality of traffic service in urban street networks. The assumptions of the model have been verified through the analysis of extensive data obtained from experiments and observations conducted on the ground and from the air. The Two-Fluid model has been calibrated for various urban networks with diverse features in cities in Texas and around the world. Furthermore, the model has been used in before/after studies in various cities. Calibration of the model for various networks and the before/after studies have helped in identifying those physical network features that appear to most strongly affect the quality of traffic service and thus the Two-Fluid model parameters. In addition, data from buses and passenger cars obtained on the same routes have been compared in order to study the effect on the model parameters for a given network of the particular vehicle types used in the data collection.

Time-lapse aerial photography over Austin and Dallas has been employed to determine the averages of concentration, speed and fraction of vehicles stopped and to examine the relations among such network-wide averages including the flow which was measured on the ground simultaneously. The results of this phase of the study have indicated that the average flow in a street network can indeed be expressed as the product of the space mean speed and concentration. Moreover, relations between the fraction of vehicles stopped and concentration as well as between speed and concentration have resulted in the need to define two additional Two-Fluid parameters. Consequently, the Two-Fluid model may be used to predict, for a given change

in vehicular concentration in a street network, the resulting changes in the averages of speed, running speed, fraction of vehicles stopped, flow, etc. This is particularly useful as a performance model in urban planning where for a given demand it is desirable to predict the resulting traffic conditions.

Such simple macroscopic descriptions of the character and quality of the traffic service in urban networks provide the practitioner with a feasible methodology for addressing not only specific engineering questions such as the impact of changes in a network on the quality of service but also broader issues related to urban planning, transportation economics, as well as environmental and energy questions.

IMPLEMENTATION STATEMENT

The Two-Fluid Model of Town Traffic has been shown to be a feasible means of characterizing the quality of traffic service in urban non-freeway street networks. The model has been shown to be reliable and easy to establish. The Two-Fluid model can be used in comparing the quality of traffic service in various urban networks as well as in the same network before and after major changes in control strategy, travel patterns, vehicle mix, network topology, etc. The comparison is made through four Two-Fluid model parameters,

- T_m = an estimate of the average minimum trip time per unit distance
- n = parameter relating running speed to fraction of vehicles running
- $f_{s,min}$ = an estimate of the average minimum fraction of vehicles stopped to concentration
- π = parameter relating fraction of vehicles stopped to concentration

It is desirable to have a network with low values of the parameters T_m , n , and $f_{s,min}$ and a high value of the parameter π .

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CHAPTER 1. INTRODUCTION

TRAFFIC SCIENCE - ITS DEVELOPMENT AND DIRECTIONS

Traffic problems are by no means a modern phenomenon. Transportation of goods and products to city dwellers is as old a practice as city dwelling itself. In ancient Rome, for example, according to the *Corpus Inscriptionum Latinarum* there existed traffic ordinances restricting the use of the city street network to authorized wagons only. Permission to enter and use the city network between the time of sunrise and the tenth hour of the day was granted only to wagon drivers with narrowly defined legitimate business in the area.

While mankind has struggled with traffic problems for a long time, it was only half a century ago that a fundamental scientific approach to these problems began to be taken. Since then significant advances have been made in our understanding of various aspects of the young science of vehicular traffic. It is remarkable that vehicular traffic processes which concern the interaction of human beings through a machine and a control system can be described by a rational approach [Refs 1 and 2]. Such an approach, however, involves the application of knowledge in many diverse disciplines such as mathematics, theoretical and experimental physics, engineering, psychology, and medicine but to mention a few.

During the past thirty years, a wide variety of questions have been examined in this relatively new science, with special attention given to the details of the specified traffic processes. The subjects investigated have ranged from the basic traffic flow relations and studies of capacity and level of service along highways, arterials and intersections, to

car-following theory in single lane flow and a kinetic theory of multilane traffic, to shock waves and the propagation of perturbations in traffic streams, to network traffic studies including fuel consumption studies in urban networks, and to studies in the economics of transportation and its environmental impacts.

One of the main objectives of this body of scientific work has been to understand traffic in urban networks. It is difficult to contemplate how this can be achieved on a purely microscopic basis since as a minimum it would involve knowledge concerning the detailed movement of the traffic on all the links and through all the intersections of the network under consideration. It is then to be hoped that the examination of relations among the averages of pertinent variables in such a highly complex system might lead to some simple functional dependencies as a consequence of collective effects.

Problems of this type, i.e. problems complex in the interrelational sense can be termed symplectic problems [Ref 3]. The word symplectic derives from the Greek word symplektikon meaning intertwined or complex precisely in the interrelational sense. Many modern problems are, indeed, of this symplectic type. One of the main reasons for mentioning this class of problems is that a purely theoretical approach to the solution of a highly complex problem is not at all hopeful. The science of traffic is basically an empirical science that can go forward effectively when there is a union of theory with observation and experiment [Ref 3].

PROBLEM DEFINITION

City street networks are subsets of a larger network of roads designed and maintained to be responsive to the vital need in modern society for the mobility of people and goods. In traffic engineering practice there is the

continuing problem of determining the quality of traffic service offered to the users of an urban street network and along with it to evaluate relative traffic quality of a traffic system as a whole.

Such problems have been addressed to some extent in limited situations at intersections or along arterials, and on occasion in limited city networks where area-wide traffic control studies, involving substantial resources and manpower, have been undertaken. While it is possible to determine the effectiveness of various approaches to traffic control on a rather limited basis, such as at an intersection or along an arterial, there has not been comparable success for area traffic control, where the interactions among intersections play such a vital role. In general it seems that before-and-after studies of the effectiveness of a given control scheme in a sizeable traffic network have not led to definitive answers to the question, "How effective has the new control strategy been?"

The question therefore remains, "How can the traffic engineer determine the quality of traffic in a sizeable city network or portion thereof, and what tools exist to determine the effect of changes in traffic control strategy as well as street and highway modifications on the effectiveness of the traffic system and on the quality of the traffic itself?"

More specifically a practical methodology for assessing the overall quality of traffic service provided by existing street networks in urban areas is needed. Assessment should be made on an area-wide basis and should complement the traditional techniques that are used to evaluate the level of service on street links and at intersections. Feasible ways of obtaining the data required for applying the methodology in evaluating changes in traffic system performance with time must be developed and validated.

PREVIOUS WORK

Early attempts to assess the quality of traffic service have been generally limited to intersections and single arterials and roads. As early as 1950 the Highway Capacity Manual [Ref 4] had put forth guidelines to determine the performance quality of single arterials and roads through the concepts of capacity and level of service. Six levels of service (A through F) were defined in terms of the speed-flow relation. The Manual has also defined a performance quality measure at intersections, i.e. the maximum number of cars which can be processed per hour of green at a given intersection. Contributions of experimental and observational work by Greenshield, et al on traffic performance at intersections [Ref 5] were used in the development of intersection capacity concepts in the Highway Capacity Manual. Experiments and observations were also conducted in England by Wardrop and his co-workers on the capacity and traffic performance quality along highways and arterials as well as at intersections. These early works included the development of relations among flow, speed and street widths in connection with roadway capacity [Refs 6 and 7] as well as relations between delay at intersections and flow through intersections [Refs 7 and 8].

Among attempts to characterize the quality of traffic service in urban street networks one of the most common methods employed has been to dispatch experimental vehicles radially away (or toward) the central part of a network with each vehicle recording its travel time and distance at specified times or locations. As a result contour travel time and speed maps of the area are established. Such multi-dimensional traffic maps of a city traffic network would then provide knowledge of the distribution of travel times and mean speeds over the network (see for example Ref 9). Such studies, however,

require substantial amounts of manpower and with limited resources are difficult to perform on a regular basis.

Moreover, while a pertinent variable, the average speed cannot by itself be an accurate measure of the quality of traffic service. For example, in two networks with the same average speed during a given time period, the average number of stops per unit distance and/or the average running speed may be different, thus signifying different qualities of traffic service.

In the early 1970's, a growing number of theoretical and experimental efforts were directed toward describing and characterizing the quality of traffic service in urban street networks. Following the speed contour map concept, Hutchinson [Ref 10] suggested that the mean speed increases approximately as the cubic root of distance from the city center. Vaughan, et al [Ref 11] described the traffic intensity as a function of distance from the city center, where traffic intensity is the total distance traveled by all vehicles on major roads per unit area of road per unit time at that distance. In addition Smeed [Ref 12] and Blumenfield [Ref 13], among others, were concerned with the effects of road network design features on variables such as mean speed, concentration, and flow which was defined as the product of mean speed and concentration over the network under consideration.

Another means of evaluating the quality of traffic service has been suggested by Zahavi [Ref 14] through his so-called α -parameter concept. He suggested that the length-weighted mean flow, q , is inversely proportional to the space-mean speed, v , i.e. $q = \alpha/v$. In this relation Zahavi suggested that α is a parameter describing the quality of traffic service for a specified section or a complete road network. He then proceeded to construct maps of equi-value α -lines for cities of London, England and Meridian, Mississippi to show the spatial distribution of the traffic performance in

these two cities [Ref 15]. Examining Zahavi's α -concept, Buckley and Wardrop [Ref 16] have suggested that the traditional speed-flow relations might be at least as helpful and meaningful as the α -concept. They also showed that Zahavi's α is strongly related to the value of the space-mean speed. The α -concept has also been examined through the use of aerial photographic data in the work reported here.

Much of the work in the arena of network-wide characterization of the traffic quality of service over the past ten years has been focused on attempts to examine general traffic characteristics in various cities with the goal of relating the character and quality of traffic to pertinent traffic variables, such as average speed, stop time per unit distance, standard deviation of acceleration, etc., as well as to the associated fuel consumption and exhaust emissions [Refs 17 and 18]. An analysis by Chang and Herman [Ref 17] of chase car data generated by Johnson et al [Ref 19] in nine metropolitan areas in the United States showed that variables such as stop time per unit distance, levels of acceleration or braking, as well as the magnitude of the speed variations, are all highly dependent on the average speed, i.e., the reciprocal of the average trip time per unit distance. The effects of traffic concentration and roadway types on the speed distribution function and the acceleration distribution function have also been examined.

As is now well known it was found that as a result of collective effects the average speed is a traffic variable that can be used rather well to characterize urban traffic in many respects. For example, in terms of trip time, stop time, acceleration noise, etc., the higher the average speed in an urban network the better the quality of traffic service in general. It is interesting to note that there are rather clear trends among various of the traffic variables in many cities around the world. This has been an

encouraging sign to the traffic scientists who have continued to pursue this and other aspects in order to reach for some global representation of traffic in large city networks [Refs 20 and 21]. The main thrust of much of the research reported in this paper derives from the Two-Fluid model, proposed by Herman and Prigogine in 1979 [Ref 21].

Another excellent example of collective effects arose in studies of the interrelation of fuel consumption and urban traffic [Refs 22, 23 and 24]. Early studies in the Detroit metropolitan area showed that the fuel consumed per unit distance, ϕ , was, in first approximation, linearly related to the trip time per unit distance, T . This result was predicated on the test vehicle sampling the entire network, for example, by means of chase car techniques. It is noteworthy that many of the other variables correlated with fuel consumption are also strongly correlated with the average speed, e.g. the number of stops per unit distance. Evans, Herman and Lam [Ref 22] examined the effect on fuel consumption of a large number of variables and found the above-mentioned simple relation by means of a multivariate analysis. They pointed out [Refs 22 and 23] that it is possible to interpret such a linear relation between ϕ and T in terms of a model of the engine-vehicle system developed by Amann et al [Ref 25]. This simple result was first indicated by Pelensky et al [Ref 26] as well as by Everall [Ref 27] and Roth [Ref 28] on the basis of their observations.

It is important to emphasize, for such a complex system with perhaps two dozen or more variables operating, that there are likely to be collective effects which reduce the complicated non-linear interrelations to relatively simple functional dependences. Furthermore, traffic observations of trip time per unit distance and stop time per unit distance for vehicles driven in urban non-freeway networks have indicated a consistent simple relation

between these two variables in a number of cities around the world. The Two-Fluid model of town traffic was developed by Herman and Prigogine [Ref 21] in an attempt to establish a theoretical basis for this particular observational relation.

OBJECTIVES OF THE PRESENT STUDY

As a continuing effort to characterize the quality of traffic service in non-freeway urban street networks, the reported research work has addressed the following main objectives:

- (1) To experimentally establish the reasonableness and validity of the underlying assumptions of the Two-Fluid model which is a macroscopic model formulated to characterize the quality of traffic service in urban non-freeway networks.
- (2) To study the feasibility of the use of the Two-Fluid model to characterize the quality of traffic service in urban non-freeway street networks.
- (3) To investigate the effects of vehicle types used in data collection on the parameters of the Two-Fluid model.
- (4) To extend the Two-Fluid model theory to allow the comparison of the quality of traffic service in various networks under similar vehicle concentrations.
- (5) To examine relations that may exist among network-wide averages of pertinent traffic variables such as speed, concentration and flow.

STUDY APPROACH

The work reported here is largely an experimental and observational attempt to explore and develop ways of characterizing the quality of traffic service in urban street networks. The theoretical basis for the work has been the Two-Fluid model of Town Traffic [Ref 21]. Chapter 2 describes the Two-Fluid model theory and in addition includes a discussion of postulated relations among network-wide averages of speed, concentration and fraction of vehicles stopped.

The reasonableness of the underlying assumptions of the Two-Fluid model is discussed in Chapter 3. The Chapter presents a theoretical discussion of the Two-Fluid model assumptions along with the results of experiments and observations carried out to examine the validity of these assumptions. The ergodic character of traffic is further explored by comparing the time averages of variables obtained with a single test vehicle with averages taken over all the test vehicles involved in the experiments conducted.

A considerable amount of data have been collected in various cities to evaluate the parameters of the Two-Fluid model for different cities. The results are documented in Chapter 4. Comparison of these cities and their Two-Fluid model characterization has led to a preliminary list of network features, discussed in this chapter, that may affect the model parameters and the quality of traffic service in a network. Observations have also been made to investigate how the results depend on the particular type of test vehicle used in making the observations. Chapter 4 also includes the results of before/after studies conducted in cities which had undergone modifications in traffic control schemes or other aspects of the urban traffic system. The results reported in Chapter 4 have also been published in two separate papers [Refs 29 and 30]. The results of a closely related study addressing the energy aspects of the quality of traffic service have been presented in a paper by Herman, et al [Ref 31].

Time-lapse aerial photographs have been taken over the CBD networks of Austin and Dallas to study relations that may exist among traffic flow variables pertinent to the quality of traffic service. Chapter 5 discusses the techniques used in reducing and analyzing the aerial photographs including a discussion of sources of measurement errors.

Chapter 6 contains the aerial photographic results and their comparison with the results obtained from the ground observations made simultaneously with the aerial photographs. Aerial photographic data have been used in Chapter 6 to establish relations among network-wide averages of vehicular concentration, flow, speed and fraction of vehicles stopped. The Two-Fluid model has been improved through addition of these relations, allowing the comparison of the quality of traffic service in Austin and Dallas under the similar vehicular concentrations.

Chapter 7 summarizes the results of the study along with a discussion of the limitations of these results. Recommendations for possible implementation and further research and extensions are also presented in Chapter 7.

CHAPTER 2. THE TWO-FLUID THEORY

This chapter presents a discussion and expansion of a theoretical model known as the Two-Fluid Model of Town Traffic which is formulated to characterize the quality of traffic service in non-highway urban networks. The significance of the model parameters with regard to the quality of traffic service is also discussed and the quality of traffic service of various hypothetical networks are compared using the Two-Fluid methodology.

The traffic in an urban street network can be considered to consist of two "fluids": one fluid composed of the moving vehicles and the other of vehicles that are stopped. The stopped "fluid" includes vehicles stopped as a result of congestion, traffic signals and other control devices, as well as obstructions caused by construction, accidents, etc.; but it does not include parked vehicles since they are not a component of the traffic, but rather form a part of the geometric configuration of the streets. Herman and Prigogine [Ref 21] first proposed the concept of a Two-Fluid model as an extension of their kinetic theory of multilane highway traffic [Ref 31].

In the kinetic theory of multilane traffic [Ref 31] the average speed, v , is expressed in terms of the average vehicular concentration, k , as $v = [kt_R(1-P)^{-1}]$ where P is the probability of passing and t_R is the relaxation time to traffic perturbations, i.e. the time required for the effect of traffic perturbations to dissipate. Such a relation is strongly mechanism-dependent since both P and t_R depend on k as well as on quantities such as lane configuration, driver behavior, etc. The relaxation time, t_R , also depends on the average fraction of vehicles stopped, f_S , which itself is a complicated function of the concentration.

In the kinetic theory, at sufficiently high vehicular concentrations the speed distribution of the vehicles includes a delta function at $v=0$ representing the motionless fluid or the fraction of the vehicles that are stopped, f_s . This is analogous to the Bose-Einstein condensation, which at sufficiently low temperatures leads to the splitting of the distribution function into two parts, one representing the molecules in the ground state and the other, the molecules in the excited states [Ref 2]. The thermal energy of the excited molecules is proportional to a power of the fraction of the excited molecules present. Similarly, the kinetic theory of multilane highway traffic suggests that, at sufficiently high vehicular concentrations at which $f_s \rightarrow 0$, the average speed of vehicles, V , is a function of the fraction of vehicles in motion $f_r = 1 - f_s$, namely $V = U_0(1 - f_s) = U_0 f_r$, where U_0 is a characteristic quantity depending on the "desired speed" distribution function.

In the Two-Fluid model the ideas in the kinetic theory of traffic are followed by assuming that the average speed of the moving cars, v_r , depends on the fraction of the cars that are moving, f_r , in the following form:

$$v_r = v f_r^{-1} = v_m f_r^n = v_m (1 - f_s)^n \quad n > 0 \quad (2.1)$$

where v_m is the average maximum running speed in the network system, v is the space-mean speed of the traffic and n is a parameter whose significance will be discussed later. The boundary conditions are satisfied since for $f_s = 0$ and one, the running speed is v_m and 0, respectively. Note that

$$f_r + f_s = 1 \quad , \quad (2.2)$$

$$v_m = 1/T_m \quad , \quad (2.3)$$

$$v_r = 1/T_r \quad , \quad (2.4)$$

and $v = 1/T \quad , \quad (2.5)$

where f_s and f_r are the fractions of the vehicles stopped and moving, respectively, T_r is a parameter representing the average minimum trip time per unit distance, T_s is the average running time per unit distance and T_r is the trip time per unit distance. If, in addition, the stop time per unit distance is denoted by T_s it follows that

$$T = T_s + T_r \quad . \quad (2.6)$$

In the model it is also assumed that over a sufficiently long period of time the fraction of the time stopped for the i th vehicle circulating in a network, $(T_s/T)_i$, is equal to the time-mean fraction of the homogeneous population of cars stopped, $\langle f_s \rangle_p$, over the same time period, namely,

$$\langle f_s \rangle_p = (T_s/T)_i \quad . \quad (2.7)$$

For convenience the symbol $\langle x \rangle$ is used interchangeably with the notation \bar{x} to represent the average value of x .

The above relation (Eq 2.7) is known as the ergodic assumption. An ergodic traffic system is one in which the time-mean performance of any single vehicle over a sufficiently long period of time would be identical to the mean performance of all the vehicles in the ensemble over the same period. A more detailed discussion of the ergodicity property and ergodic systems is given in the section on ergodic experiments in Chapter 3.

The relation in Eq (2.7) can be theoretically proven for steady state traffic conditions. To do this, we will first prove that even under non-steady state conditions the mean fraction of time stopped for all the vehicles in a network is equal to the mean fraction of all the vehicles stopped in that system.

Assume a traffic network with a population of N vehicles. Over an observation period τ , the probability of the i th vehicle of this population being stopped, P_i , can be expressed as the fraction of time stopped for this vehicle, namely,

$$P_i = (T_s/T)_i \quad . \quad (2.8)$$

Therefore, we can write for the entire population of vehicles that

$$\sum P_i = \sum (T_s/T)_i \quad , \quad i = 1, 2, \dots, N \quad . \quad (2.9)$$

Dividing both sides of Eq (2.9) by the total number of vehicles we obtain

$$\frac{1}{N} \sum P_i = \frac{1}{N} \sum (T_s/T)_i \quad , \quad i = 1, 2, \dots, N \quad . \quad (2.10)$$

The left-hand side of Eq (2.10) is the mean probability of finding any vehicle of the population stopped, or in other words it is the mean fraction of vehicles stopped. On the other hand, the right-hand side of Eq (2.10) is the mean fraction of time stopped for the population of the vehicles. Therefore, over the observation period τ we can write that

$$\langle f_s \rangle_p = \langle T_s/T \rangle_p \quad (2.11)$$

under any conditions.

In steady state conditions and over a sufficiently long period of time, each of the N vehicles will have fully sampled the network area. Therefore,

the fraction of time stopped for any vehicle of the population, $(T_s/T)_i$, will be equal to that of any other vehicle and thus identical to the mean fraction of time stopped, namely,

$$(T_s/T)_i = \langle T_s/T \rangle_p, \quad i = 1, 2, \dots, N \quad (2.12)$$

Combining Eqs (2.11) and (2.12) we will have for the steady state case that,

$$\langle f_s \rangle_p = \langle T_s/T \rangle_p = (T_s/T)_i \quad (2.13)$$

Equation (2.13) contains the second assumption of the Two-Fluid model as expressed in Eq (2.7).

The ergodic experiments, described in Chapter 3, were designed to examine the extent to which the fraction of the time stopped for a single test vehicle in a real traffic network satisfactorily approaches the mean value of this fraction taken over N_t test vehicles for reasonable times, i.e., times short enough so that traffic conditions are not changing too much but long enough so that a test vehicle can sample all the streets in the area with frequencies corresponding to the traffic utilization of those streets and to sample all the different driver behaviors. On the other hand, if the time period τ during which the traffic conditions are reasonably stable is not long enough to allow a test vehicle to properly sample the test area, then daily repetitive observations made by a single test vehicle during the period τ should on the average correspond to the averages obtained over the entire population on a typical day during this particular time period.

Adopting Eqs (2.1) and (2.7), we are now able to derive the theoretical relation between the trip time and the stop time per unit distance predicted by the Two-Fluid model. Let T be the trip time per unit distance, T_r be the running time per unit distance, T_s be the stop time per unit distance, and T_m

be an estimate of the average minimum trip time per unit distance or the reciprocal of the average maximum speed, V_m , in the system. Thus, Eq (2.1) can be rewritten as

$$T_r = T_m f_r^{-n} \quad (2.14)$$

Substituting for f_r in Eq (2.14) in terms of f_s yields

$$T_r = T_m (1 - f_s)^{-n} \quad (2.15)$$

Combining Eqs (2.7) and (2.15) we have

$$T_r = T_m [1 - (T_s/T)]^{-n} \quad (2.16)$$

Rearranging Eq (2.16) gives

$$T_r = T_m T^n (T - T_s)^{-n} \quad (2.17)$$

and knowing that

$$T_r = T - T_s \quad (2.18)$$

Eq (2.17) can be rewritten as

$$T_r^{n+1} = T_m T^n \quad (2.19)$$

or

$$T_r = T_m^{\frac{1}{n+1}} T^{\frac{n}{n+1}} \quad (2.20)$$

Combining Eqs (2.17) and (2.20), we have

$$T - T_s = T_m^{\frac{1}{n+1}} T^{\frac{n}{n+1}} \quad (2.21)$$

or finally the Two-Fluid model formulation

$$T_s = T - \frac{1}{T_m^{n+1}} \frac{n}{T^{n+1}} . \quad (2.22)$$

It follows from Eq (2.19) that

$$\log T_r = \frac{1}{n+1} \log T_m + \frac{n}{n+1} \log T , \quad (2.23)$$

or $\log T_r = A + B \log T , \quad (2.24)$

with $n = B/(1 - B) , \quad (2.25)$

and $\log T_m = A/(1 - B) . \quad (2.26)$

The parameters n and T_m associated with a traffic network can be obtained from Eqs (2.25) and (2.26) by collecting trip time versus stop time data for a test vehicle circulating in that traffic network and performing a linear regression between $\log T_r$ and $\log T$ to determine A and B .

It is to be emphasized that in the Two-Fluid theory the variables are always meant to be time averages taken over the entire system. Observationally, we have determined the values of these parameters for various cities using T, T_r data corresponding to trips one to two miles long. The above observations are not averages over the entire system. Therefore, the estimates of A and B through regressing $\log T_r$ against $\log T$ as obtained observationally do not necessarily correspond to the estimates of A and B obtained by the regression of $\log T_r$ against $\log T$ as specified by the theory for system-wide averages of T_r and T . However, the magnitude of the error introduced is not expected to be significant since the non-linearity of the

Two-Fluid relation (Eq 2.22), as will be shown later, is slight so that the model trend can be approximated by a linear regression of T against T_s .

To further assess the magnitude of the errors involved, 354 two-mile trips in the Austin CBD1 have been aggregated into 59 twelve-mile trips. The parameters A and B in Eq (2.24) were then estimated using both sets of data, resulting in $(A_1, B_1) = (0.0923, 0.620)$ for the unaggregated two-mile data and $(A_1, B_1) = (0.0837, 0.624)$ for the aggregated twelve-mile data. Two-sample t -tests were performed to test the hypotheses that $H_0: A_1 = A_2$ versus $H_1: A_1 \neq A_2$ and $H_0: B_1 = B_2$ versus $H_1: B_1 \neq B_2$. The t -statistics computed for the tests of intercepts and slopes were $t_A = 0.628$ and $t_B = -0.175$, respectively, each with 409 (352 + 57) degrees of freedom. The above statistics indicate that the difference between A_1 and A_2 is significant at 47 percent confidence level, and the corresponding level of confidence for the difference between B_1 and B_2 is only 14 percent. These results show that there is virtually no change in estimates of A and B when T, T_r data are averages over two or twelve-mile trips. Similar results were obtained when calibrating the model for one-kilometer versus two-kilometer trips in Mexico City or one-mile versus two-mile trips in San Antonio. These results begin to indicate that the magnitude of the errors in using T, T_r averages of microtrips one to two miles long instead of system-wide averages of T and T_r are not significant enough to affect the estimates of the model parameters.

The Two-Fluid model represented by Eq (2.22) yields a curvilinear relation between T and T_s as shown in Fig 2.1 for T_m values of 1.5 and 3.0 minutes per mile and for n values of one, two, and three. All of the data we have examined from cities around the world appear to fit the model which predicts a slightly convex upward curve for T versus T_s . However, the

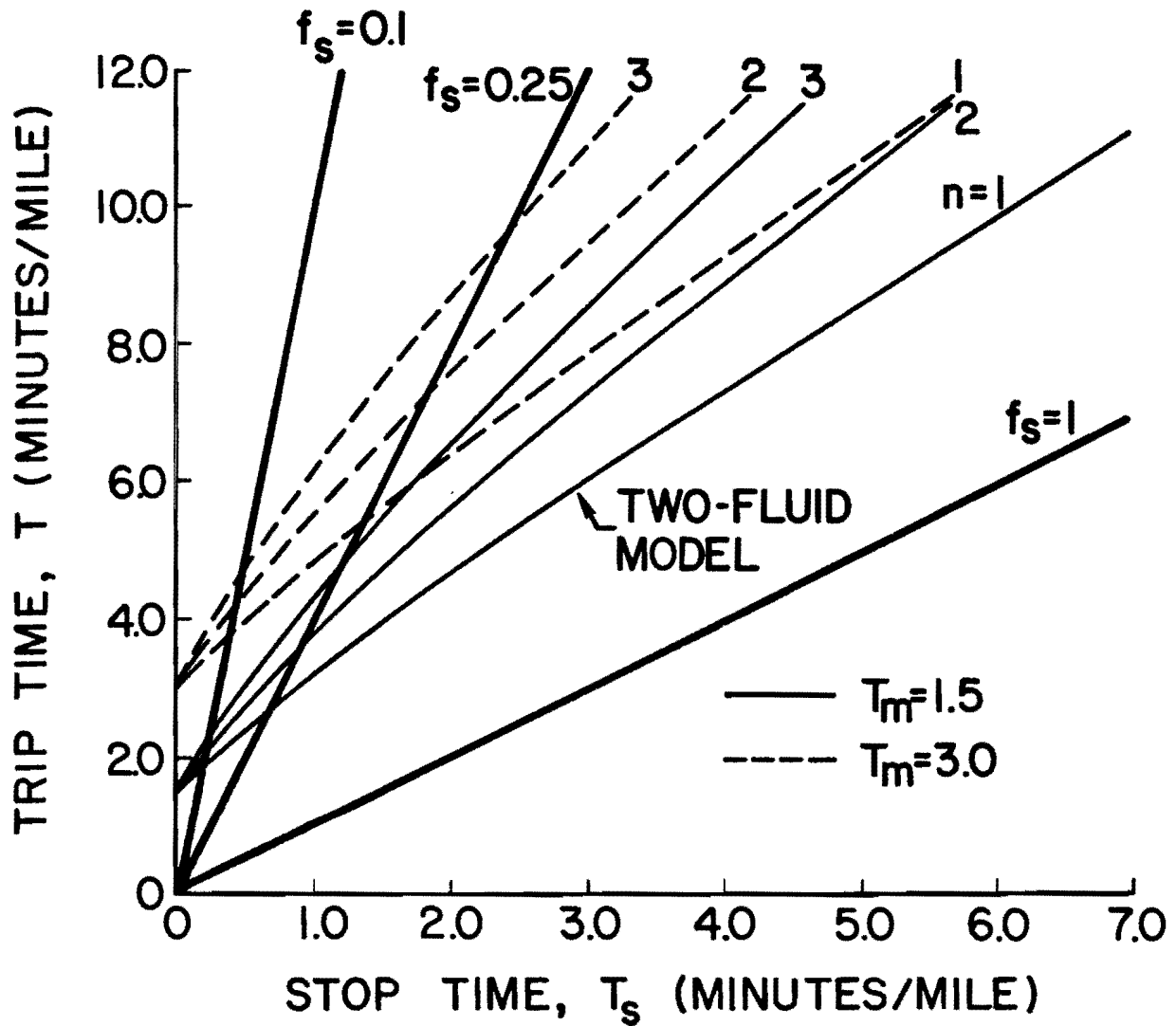


Figure 2.1. The trip time versus stop time relation of the Two-Fluid model for $n = 1, 2, 3$ and $T_m = 1.5$ and 3 min/mile. The lines radiating from the origin correspond to lines of constant fraction of vehicles stopped, $f_s = 0.1, 0.25, 1.0$. The area below the line $f_s = 1$ (all vehicles stopped) is physically inadmissible.

curvature is small in all cases so that for some purposes we may approximate the model curve with a linear regression between trip time and stop time.

The slope dT/dT_s and the intercept T_m of the curve represented by Eq (2.22) have been used as indicators of the relative quality of traffic service in an urban street network. As will be seen later, the quality of traffic service refers to the ability of the system to carry traffic with a better system in general having smaller T , T_s and T_r for the same demand. The slope dT/dT_s is itself a function of T_m and n as can be seen from the following relation which can be derived from Eq (2.22) following Chang and Herman [Ref 20]:

$$\frac{dT}{dT_s} = \left\{ 1 - \left[\frac{n}{n+1} \right] \left(\frac{T_m}{T} \right)^{1/(n+1)} \right\}^{-1} . \quad (2.27)$$

Therefore, the parameters T_m and n are useful in representing the quality of traffic service in a network. For the sake of completeness we list below the various derivatives [Ref 20] which show the manner in which T , T_r , and T_s vary with f_s . In other words this will allow us to study the effect of increasing or decreasing the fraction of vehicles stopped on trip time and stop time. From Eq (2.22) and the relation

$$f_s = 1 - \left(\frac{T_m}{T} \right)^{1/(n+1)} \quad (2.28)$$

the following derivatives can be obtained:

$$\frac{dT}{df_s} = (n+1) T_m^{-1/(n+1)} T^{(n+2)/(n+1)} , \quad (2.29)$$

$$\frac{dT_r}{df_s} = nT , \quad (2.30)$$

$$\text{and} \quad \frac{dT_s}{df_s} = (n+1) T_m^{-1/(n+1)} T^{(n+2)/(n+1)} - nT . \quad (2.31)$$

Since, as mentioned above, a given traffic network can be represented, approximately by a linear fit between T and T_s , namely, $T = a + bT_s$, the entire situation can be viewed in a somewhat simpler manner. In this linear representation we have $T_s = (T-a)/b$, $T_r = T(1-1/b) + a/b - a/(bT)$ so that the various derivatives can be written as follows: $dT/df_s = (b/a)T^2$, $dT_r/df_s = [(b-1/a)T^2]$, and $dT_s/df_s = T^2/a$.)

It can be seen analytically from Eq (2.27) or graphically from Fig 2.1 that for a given value of T_s or T , the curve with the larger value of n and/or T_m has a steeper slope. At a fixed value of T the effect of n on the slope is considerably larger than that of T_m . A change in the stop time per unit distance, T_s , produces a greater change in the trip time per unit distance, T , for the curve having a steeper slope which then implies a poorer quality of traffic service for the larger n . Furthermore, the parameter T_m is by itself an estimate of the average minimum trip time per unit distance or the reciprocal of the average speed that can be achieved in a network under the lightest traffic conditions. Therefore, the larger T_m the less efficient will be the control system of the network. In short, a desirable traffic system is one with a smaller value of T_m , a smaller parameter n , and thus a smaller slope dT/dT_s .

The straight lines radiating from the origin in Fig 2.1 correspond to lines of constant fraction of vehicles stopped, f_s , since $f_s = T_s/T$. The region below the line $f_s = 1$ (all vehicles stopped) is physically inadmissible. Note that as the T versus T_s curves are traced to higher values the slopes of the curves will eventually approach unity and thus will become parallel to the line $f_s = 1$. It is interesting to point out that during the rush-hours when f_s can be as high as 40 percent or more [Ref 29], the stop time and trip time corresponding to this value will be higher for

larger values of n . In this connection it must be emphasized that in this representation the data sampled for a system under a variety of traffic conditions has a greater range in T and T_S along the Two-Fluid model trends with larger values of n . This effect has already been observed for example, in a comparison of the Milwaukee and London data [Ref 20].

To further clarify the physical interpretation of the Two-Fluid model parameters it is especially instructive to introduce a variable ΔT_R , the deviation of the running time from T_m , the average minimum trip time (or running time, since $T_S = 0$ at $T = T_m$) per unit distance, i.e. $\Delta T_R = T_R - T_m$. Then, the trip time per unit distance, T can be said to be composed of three parts: T_m , T_S , and ΔT_R , namely,

$$T = T_m + T_S + \Delta T_R \quad . \quad (2.32)$$

The quantity ΔT_R can be thought of as representing the effect of vehicular interactions on the running time per unit distance. As such it is expected that ΔT_R , increases with an increase in the vehicular concentration, since a greater concentration implies a greater level of interaction among vehicles.

It can be shown, for example, that at a given T_S larger values of ΔT_R are produced in networks with greater T_m and n . Consider two trends each with a $T_m = 2.0$ min/mile but different n values of 1.5 and 2.5. As shown in Fig 2.2, for a T_S of 3.0 min/mile, for example, the Two-Fluid curves with n values of 1.5 and 2.5 yield incremental running times, ΔT_R , of 2.4 and 3.8 min/mile, respectively. The corresponding ΔT_R values at a $T_S = 6.0$ min/mile are 3.9 and 6.2 min/mile, respectively. It must first be noted that as the T_S value increases from zero to 3.0 min/mile, the trip time is not only increased by an amount $T_S = 3.0$ min/mile but also by an additional amount ΔT_R . Furthermore, the additional ΔT_R increase in the trip time is larger

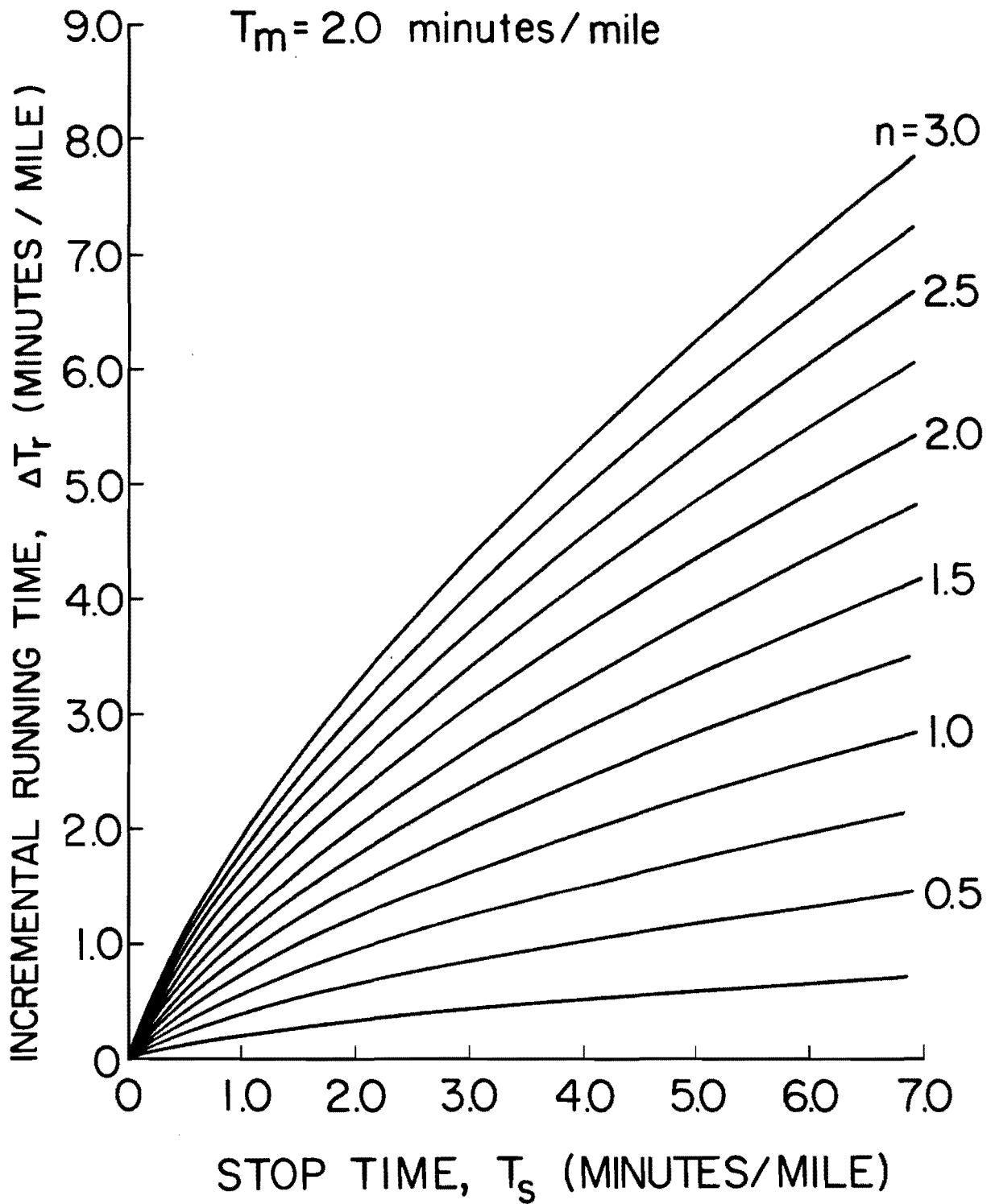


Figure 2.2. Incremental running time, $\Delta T_r (= T_r - T_m)$, versus stop time for $n = 0.25$ to 3.00 in 0.25 increments and $T_m = 2$ minutes per mile.

for greater values of n . For example, for $T_s = 3.0$ min/mile and $T_m = 2.0$ min/mile, the value of ΔT_r is 58 percent larger for $n = 2.5$ compared to $n = 1.5$. The corresponding percent increase in ΔT_r for $T_s = 6.0$ min/mile is 59 percent. The above numerical examples further indicate the importance of having a street network with a smaller value of the parameter n .

Another aspect of the quality of traffic service is how the fraction of vehicles stopped is related to the quality of the road facilities and traffic control system. The concentrations of the vehicles stopped, k_s , and moving, k_r , are related through $k_s + k_r = k$, and by definition $f_s = k_s/k$. It has been suggested that the single independent variable to which traffic variables such as T , T_s , k , v and the flow, q , might be related is the fraction of vehicles stopped and it has been suggested that

$$f_s = \left(\frac{k}{k_m} \right)^p \quad (2.33)$$

where k_m is the average maximum concentration at which the traffic jams in the system [Ref 21]. The parameter p then, in some sense, will measure the quality of the traffic network system as it determines the fraction of vehicles stopped for a given partial concentration. Since k/k_m is less than one, the higher p the smaller f_s will be. Thus, in addition to the parameters T_m and n there is another parameter that might be useful in describing the character of a traffic network system.

Noting from Eqs (2.3), (2.5) and (2.15) that

$$f_s = 1 - (v/v_m)^{\frac{1}{n+1}} \quad (2.34)$$

and combining this with the postulated relation for f_s in terms of k in Eq (2.33), the concentration has the following form [Ref 21]:

$$k = k_m \left[1 - (v/v_m)^{\frac{1}{n+1}} \right]^{\frac{1}{p}} \quad (2.35)$$

The average flow in the network would follow if it were legitimate to assume that the average flow were equal to the product of the averages of the concentration and the speed in the network. It is not clear that the flow would be given by the usual relation since it would depend on how averages are taken. This matter is investigated in Chapter 6 through the analysis of data from simultaneous ground and aerial traffic studies in the Austin CBD network.

The postulated relation in Eq (2.33) implies that as $k \rightarrow 0$, $f_s \rightarrow 0$. However, observations in the Austin CBD under very light traffic concentrations have resulted in non-zero values of

$$(T_s/T)_{k \rightarrow 0} \approx f_s = f_{s,\min} \neq 0 .$$

A non-zero average minimum fraction of vehicles stopped, $f_{s,\min}$, is expected since even at very light concentrations ($k \rightarrow 0$) the control devices continue to function. Thus, if a test vehicle were one of a small number of vehicles in a network system it could still measure $f_{s,\min} = (T_s/T)_{\min} \neq 0$. The above suggests that the relation in Eq (2.33) can be written more properly as,

$$f_s = f_{s,\min} + (1 - f_{s,\min}) (k/k_m)^\pi , \quad (2.36)$$

in which the parameter π replaces the parameter p as a measure of the quality of traffic service; and $f_{s,\min}$, the average minimum fraction of vehicles stopped, is a new parameter also related to the quality of traffic service. Once again, since k/k_m is less than one, the higher π the smaller f_s and the better the quality of traffic service. Therefore, in describing the character of an urban traffic network we now have four parameters T_m , n ,

$f_{s,\min}$, and π ; with a desirable network being one for which T_m , n , and $f_{s,\min}$ are relatively smaller and π is relatively larger.

The new postulated relation (Eq 2.36) yields the following relation between k and v , namely,

$$k = k_m \left[1 - \frac{(v/v_m)^{\frac{1}{n+1}}}{(1 - f_{s,\min})} \right]^{\frac{1}{\pi}} \quad (2.37)$$

A more elaborate discussion of the relations in Eqs (2.36) and (2.37) is presented in Chapter 6.

CHAPTER 3. VERIFICATION OF THE TWO-FLUID MODEL ASSUMPTIONS

The derivation of the Two-Fluid model is based on two assumptions. One of the assumptions referred to as the ergodic assumption states that the fraction of time a single vehicle circulating in a network is stopped, over a sufficiently long period of time is identical to the mean fraction of vehicles stopped (Eq 2.7). The second assumption of the model postulates that the space mean running speed is equal to an average maximum speed in the network times the fraction of vehicles running to a power n (Eq 2.1).

In this chapter the results of four traffic experiments referred to as ergodic experiments are presented. The experiments were performed in the Austin CBD in order to verify the assumptions of the Two-Fluid model. The description of the experimental details and its results are preceded by a discussion of the word "ergodic" and its use in the context of network vehicular traffic.

ERGODICITY AND ERGODIC SYSTEMS

The word "ergode" was first introduced by Boltzmann in Physics; although his use of the word was somewhat different from the context in which it is presently used in Statistical Mechanics. Ergode derives from the Greek words for energy and path. According to Webster the word "ergodic" is an adjective relating to a process in which every sequence or sizable sample is equally representative of the whole. In physical systems it is often not possible to compute time averages because the equations of motion are generally not solvable. Quasi-ergodic hypotheses are then invoked to circumvent these obstacles by attempting to prove that time averages may be replaced by ensemble averages.

In the context of a traffic network, it can be said that the system is ergodic if the performance of a single vehicle, as measured through its fraction of time stopped $(T_s/T)_i$, over a sufficiently long period of time would be identical to the ensemble mean performance of all the vehicles in the system as measured through the time average fraction of time stopped over the same population of vehicles, which is identical to the ensemble mean fraction of the population of vehicles stopped.

It must be emphasized that in the ergodic assumption (Eq 2.7), $(T_s/T)_i$ refers to the time average of the fraction of time stopped associated with any vehicle i of the population. However, in the course of experiments conducted to examine the reasonableness of the ergodic assumption, we have measured the time averages of the fractions of time stopped for the test vehicles circulating in the network and not those of the individual vehicles in the population. In what follows we attempt to establish the reasonableness of the ergodic assumption by showing that $(T_s/T)_i$ for the i th test vehicle, and not for the i th vehicle of the population as hypothesized by the ergodic assumption, has a small variance across the test vehicles. Furthermore, we show that this variance approaches zero for long observation times so that $(T_s/T)_i$ for any test vehicle i approaches its mean, $\langle T_s/T \rangle_t$, taken over all the test vehicles which is then identical to the average fraction of test vehicles stopped, $\langle f_s \rangle_t$. In Chapter 6 we then set out to show that $\langle f_s \rangle_t$ taken over as few as ten test vehicles is a close estimate of the average fraction of population of vehicle as determined from aerial photographs taken simultaneously with the ground experiments.

It is important to note that unlike a system of physical particles such as identical atoms or molecules, the particles in traffic, i.e. the driver-car units are similar but certainly not identical. Thus, if the

traffic system were to be sampled by examining the time averages of various traffic variables as determined from different test cars, different averages would be expected since, for example, the average value of the fractional stop time would depend on driver behavior, the characteristics and performance of the vehicle, etc. both across the test cars and the chased vehicles. There would, indeed, be a distribution of the individual averages of T_s/T around the mean of the individual means with a dispersion depending on how dissimilar the drivers and vehicles might be.

In the case of a test vehicle i sampling the network by passively following 'random' vehicles j a number of observations (or realizations) of the random variable $(T_s/T)_{ij}$, the fraction of time a test vehicle i has been stopped while following a vehicle j , will be obtained. The mean fraction of time stopped for the i th test vehicle, $(T_s/T)_i$, is then the mean of the sample obtained by that vehicle and is, as such, an estimator for the mean $\langle T_s/T \rangle_t$ over all the test vehicles. Providing that a sufficient number of test vehicles N_t have been used, then $\langle T_s/T \rangle_t$ is itself a reasonable estimator of the mean T_s/T over the entire population of vehicles using the system, $\langle T_s/T \rangle_p$.

Implicitly assumed in the above interpretation is that the probability distribution of the random variable $(T_s/T)_i$ remains invariant over the time period during which the sample is collected. It will be shown below that this would be the case over a long period of time providing that each test vehicle i continues to follow random vehicles while circulating in the network. If the test vehicle i followed each vehicle j for an equal time duration, then at the end of the observation period each test vehicle would report a fraction of time stopped, $(T_s/T)_i$, which is an average over the N_i random vehicles it has followed, namely,

$$(T_s/T)_i = \frac{1}{N_i} \sum_{j=1}^{N_i} (T_s/T)_{ij} \quad (3.1)$$

where $(T_s/T)_{ij}$ is the fraction of time stopped reported by the i th test vehicle while following the j th random vehicle. Denoting the variance across $(T_s/T)_{ij}$ for the i th vehicle by σ_i^2 , then according to the Central Limit Theorem the variance of $(T_s/T)_i$ across the vehicles followed is of the form

$$\sigma_{(T_s/T)_i}^2 = \frac{\sigma_i^2}{N_i} \quad (3.2)$$

where N_i as before is the number of random vehicles which were followed by the i th test vehicle. Note that since the same population of vehicles is being sampled by each test vehicle, it is expected that σ_i^2 be constant across all the test vehicles, i.e. $\sigma_i^2 = \sigma^2$, $i=1, 2, \dots, N_t$. The average fraction of time stopped taken over all the test vehicles, $\langle T_s/T \rangle_t$, can be written as

$$\langle T_s/T \rangle_t = \frac{1}{N_t} \sum_{i=1}^{N_t} (T_s/T)_i \quad (3.3)$$

with its variance across test vehicles,

$$\sigma_{\langle T_s/T \rangle_t}^2 = \frac{\sum_{i=1}^{N_t} (\sigma^2/N_i)}{N_t^2} \quad (3.4)$$

where N_t is the number of test vehicles involved in the experiment and providing that $(T_s/T)_{ij}$ $i=1, 2, \dots, N_t$ are uncorrelated random variables.

Over a long period of time Eqs (3.2) and (3.4) imply that the variances $\sigma_{(T_s/T)_i}^2$ and $\sigma_{\langle T_s/T \rangle_t}^2$ tend to zero since the test vehicle i will have

sampled many vehicles, i.e. $N_i \rightarrow \infty$. The above implications are consistent with the Two-Fluid model ergodic assumption that over a sufficiently long period of time, the fraction of time a single vehicle circulating in the system is stopped is equal to the mean fraction of vehicles stopped in that network over the same time period. Note that the above conclusion is also subject to the limitation that the quantity $(T_s/T)_i$ as determined by the test vehicle i is a reasonable estimate of the average fraction of time stopped associated with any vehicle of the population. However, this does not appear to be a restrictive assumption since the chase-car technique has been used to randomize the routes taken by each test driver during the course of the experiment rather than to reproduce the trip time history of the "chased" vehicle. Moreover, the drivers participating in the experiments were untrained drivers who were not able to accurately reproduce trip time history even if instructed to do so. Therefore, each test driver in the course of the experiment maintains his or her "identity" so that the variance in $(T_s/T)_i$ across ten test drivers is not expected to be significantly different from the variance across ten drivers in the population. It is also to be noted that each test driver in an experiment using the chase-car technique spends a significant fraction of the duration of the experiment in transition from one chased vehicle to another. During such transitions the individual character of each test driver definitely prevails. It must be noted that we speak of ergodic traffic systems in the context of non-freeway street networks since freeway traffic is of an entirely different character with generally few stoppages and would require a different treatment [Ref 32].

EXPERIMENTAL DETAILS

In order to investigate the Two-Fluid assumptions, a series of controlled experiments were conducted in a delineated area of downtown Austin, Texas. The first of these experiments was carried out during the evening rush-hour (5:00-6:00 p.m.) of January 15, 1981, in an area of downtown Austin approximately 1.5 miles wide and three miles long. The experiment involved eight automobiles, each carrying a driver and an observer with a digital watch. The odometer of each test vehicle was calibrated, and all the watches were synchronized prior to the experiment using radio time signals.

The observations for each vehicle were made continuously over a period of approximately one hour in the test area. The one-hour trip for each test vehicle was divided into two-mile segments to obtain average values of trip time, stop time, and running time, all per unit distance. To randomize the routes taken by each driver, the well-known chase-car technique was employed. The test vehicles, which were originally outside the test area, entered the area and circulated so that they were essentially at random places at the start of the observations. Then, each test vehicle followed some car in the test area until it either left the area or parked, in which case the next nearest convenient vehicle was followed. It is expected that each test vehicle, in this manner, will sample the test area according to how it is being used by the customers who enter the network in terms of their travel time history and the routes they take. This will be the case if the period of the ergodic experiment is sufficiently long and if so, one must then consider whether the traffic conditions are remaining essentially constant.

The task of the observers included recording the odometer readings and the absolute times corresponding to the start and the end of each two-mile

trip as well as noting the absolute times associated with every stop and the subsequent resumption of motion of the test vehicle. For the purposes of these experiments a stop is considered to be the absolute cessation of motion of the test vehicle. Naturally the specific pattern of stops would, among other things, depend on the behavior of the various drivers who were instructed only to follow vehicles in a safe manner.

In addition, three other ergodic experiments (the second, third, and fourth) were also performed in downtown Austin. The second experiment was carried out during the noon rush-hour ($\sim 12:00$ - $\sim 1:00$ p.m.), February 24, 1981, in a sub-area of experiment one, $\sim 0.7 \times \sim 1.5$ miles. The third experiment took place during the afternoon peak hour ($\sim 4:30$ - $\sim 5:30$ p.m.), September 11, 1981 in the same area as for the first ergodic experiment. Finally the fourth experiment was carried out during two periods, one from $\sim 3:45$ - $\sim 4:30$ p.m., and the other from $\sim 4:45$ - $\sim 5:45$ p.m., November 12, 1982 in another sub-area of the first experiment, $\sim 1.25 \times \sim 1.5$ miles. Simultaneous time-lapse aerial photographs of the network traffic were also taken during the course of the fourth experiment. The four experiments involved eight, five, ten, and ten test vehicles, respectively.

ANALYSIS AND RESULTS

The analysis [Ref 33 and 34] proceeded by first entering into the computer all of the absolute start and stop times recorded in the various ergodic experiments. A computer program was written to synchronize, normalize and access this data at various specific times in order to determine, for each time entry, which vehicles were moving and which were stopped. In addition, the program generated fractional stop times for each of the test vehicles in the four experiments.

The fraction of vehicles stopped at a given instant of time (the j th time entry), $f_{s,j}$, was determined at three-second intervals starting from the beginning of each experiment. The mean fraction of vehicles stopped for the first 15-minute period of each experiment, for example, could then be obtained by averaging the first 300 values of $f_{s,j}$.

The uniform three-second sampling interval was used to ensure that almost all of the stopping events were counted at least once. Sampling intervals of one and two seconds have also been explored, and it is found that using intervals shorter than three seconds does not significantly change the average value of the fraction of vehicles stopped, $\langle f_s \rangle$. An examination of this question for four five-minute periods in the 1st ergodic experiment shows that f_s changes by 0.5 - 1.0 percent in going from a one-second to a three-second uniform sampling interval. Moreover, for the second ergodic experiment the influence of sampling size up to ten-second intervals has been examined for the entire data set. It is found that in going from a three-second interval to an eight-second interval f_s is reduced by about one to two percent. For a ten-second interval f_s is reduced by about four percent.

The advantage of using uniform sampling intervals is that it provides an average value for the fraction of vehicles stopped in any sub-period of the test data. The disadvantage is that the average value for f_s over the entire test period can be obtained only after all the data is accessed. This is in contradistinction to random sampling in which case rather rapid convergence to the final average occurs after only 300 to 400 entries. This method, however, does not provide any information other than the overall average.

VERIFICATION OF THE ERGODIC ASSUMPTION

In Table 3.1 the fractions of time stopped for each test vehicle for the indicated periods of the four ergodic experiments have been recorded. In addition, the average value of T_s/T taken over all test vehicles in a given experiment as well as the observed fraction of the vehicles stopped for the same experiment and time period are also shown.

It is first noted that $\langle T_s/T \rangle_t$, the mean T_s/T taken over all test vehicles is very close to $\langle f_s \rangle_t$, the mean value of f_s taken over all test vehicles. Thus,

$$\langle f_s \rangle_t = \langle T_s/T \rangle_t . \quad (3.5)$$

A further analysis of the data shows that the result stated in Eq. (3.5) is independent of the duration of the observational period and the number of test vehicles in the experiment. We have shown in Chapter 2 that the above result (Eq. 3.5) can be theoretically proven for any time period τ as long as the observations are made over the same population of vehicles and during the same period. Indeed, a comparison of the mean fractional stop time with the mean fraction of vehicles stopped for the five observational time periods in the four ergodic experiments shows that they agree to 0.2 percent. This discrepancy may result from round-off errors.

It must be emphasized that Eq. 3.5 is not the Two-Fluid model ergodic assumption, rather the ergodic assumption states that,

$$\langle f_s \rangle_p = (T_s/T)_i \quad (3.6)$$

where $\langle f_s \rangle_p$ is the ensemble population average fraction of vehicles stopped during an observation period, and $(T_s/T)_i$ is the fraction of time each vehicle i of the population of vehicles circulating in the network during the

TABLE 3.1. COMPARISONS OF THE INDIVIDUAL FRACTIONS OF TIME STOPPED WITH THE MEAN VALUE FOR THE INDICATED PERIODS OF THE FOUR ERGODIC EXPERIMENTS IN AUSTIN, TEXAS

Vehicle	Mean Fraction of Time Stopped in the Ergodic Experiments				
	First Jan. 15, 1981 17:00:21-17:58:12	Second Feb. 24, 1981 12:10:09-12:54:16	Third Sept. 11, 1981 16:30:39-17:40:58	Fourth Nov. 12, 1982 15:49:12-16:28:28 16:48:22-17:40:55	
1	0.2008	0.3072	0.2868	0.2793	0.3061
2	0.2578	0.2452	0.2468	0.3001	0.2656
3	0.2060	0.2830	0.3152	0.1948	0.2797
4	0.2598	0.2602	0.2330	0.2177	0.2839
5	0.2227	0.2311	0.3378	0.2589	0.2753
6	0.2838		0.3046	0.2466	0.2908
7	0.2377		0.2993	0.2950	0.2743
8	0.2674		0.3145	0.2156	0.2518
9			0.2721	0.2755	0.2496
10			0.2624	0.3179	0.3076
<T _g /T>	0.2420	0.2653	0.2872	0.2601	0.2785
Std. Dev.	0.0301	0.0303	0.0332	0.0408	0.0198
<f _g >	0.2430	0.2656	0.2873	0.2604	0.2779
Std. Dev.	0.1609	0.2040	0.1527	0.1434	0.1422

same time period is stopped. Using the information in Table 3.1 we can investigate whether the mean fraction of vehicles stopped taken over the test vehicles, $\langle f_s \rangle_t$, is equal to $(T_s/T)_i$ as determined by the test vehicle i , namely,

$$\langle f_s \rangle_t = (T_s/T)_i \quad . \quad (3.7)$$

As shown in Table 3.1 the fractions of time stopped for the individual test vehicles, $(T_s/T)_i$, are rather narrowly distributed around the mean value taken over all of the test vehicles in a particular experiment. The standard deviations for the five independent periods range from 0.02 to 0.04. The variations around the mean decrease considerably as the duration of the analysis period increases, as long as the traffic conditions during the entire experiment are not changing too radically. In the first ergodic experiment, for example, the average standard deviation using five-minute analysis intervals is 0.128 compared to averages of 0.070, 0.048, and 0.030, for 15, 30, and 58-minute periods, respectively. If the average fraction of the time stopped is normalized to unity then all of the data given in Table 3.1 can be combined. A histogram of the normalized fractions of the time stopped for all the test vehicles in the ergodic experiments is given in Fig 3.1. As shown, most of the observations for the individual test vehicles lie within 11 percent of the normalized mean, the region lying between the heavy $\pm 1 \sigma$ pip marks. It is essential to recognize that the dispersion in the histogram comes not only from variations in sampling of the system by the test cars over the finite test time period but also from differences in the behavior of the test vehicle drivers in how they may carry out the chase-car technique.

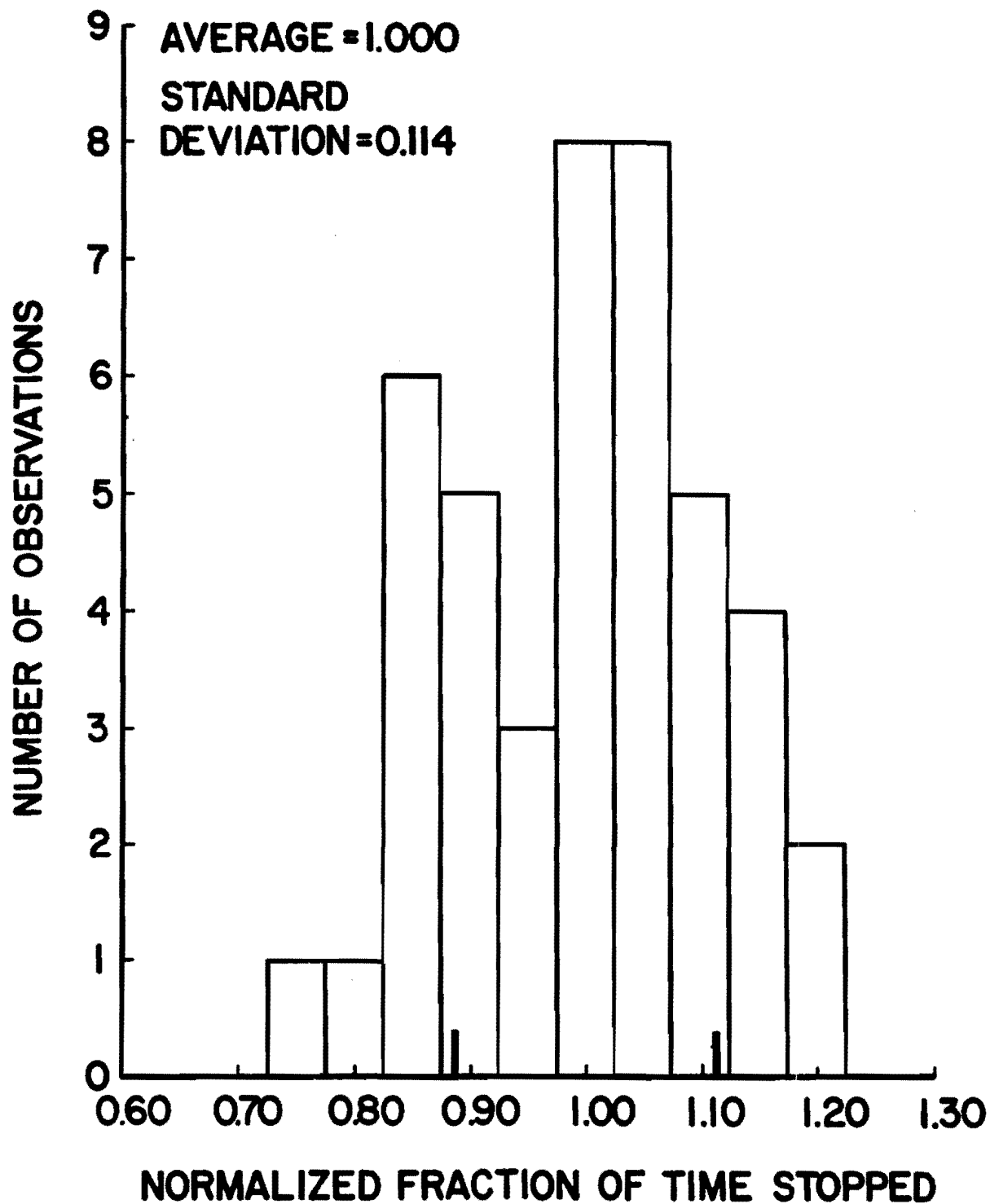


Figure 3.1. Histogram of the normalized fraction of time stopped for the data obtained with 33 test-cars in four ergodic experiments conducted in Austin, Texas, 1981-1982. The pip marks indicate the $\pm 1\sigma$ limits. Note that there are 43 observations because ten vehicles in one ergodic experiment were employed in two phases.

The narrow distribution of $(T_s/T)_i$ around its mean (Fig 3.1) is an indication of the reasonableness of Eq (3.7). What still remains to be answered is whether the mean fraction of test vehicles stopped, $\langle f_s \rangle_t$, is a reasonable estimator of the mean fraction of population of vehicles stopped, $\langle f_s \rangle_p$. Time-lapse aerial photographs taken over the Austin CBD during the fourth ergodic experiment have been used to estimate $\langle f \rangle_{s p}$ which can then be compared to $\langle f_s \rangle_t$. The methodology used in measuring the fraction of vehicles stopped from aerial photographs is outlined in Chapter 5. A thorough discussion of the results are presented in Chapter 6 in the section entitled Aerial Versus Ground Observations (see for example Table 6.1).

VERIFICATION OF THE SECOND TWO-FLUID MODEL ASSUMPTION

The results of the four experiments described earlier may also be used in verifying the second assumption of the Two-Fluid model. This assumption states that $V_r = V_m f_r^n$, remembering that $V = V_r f_r = 1/T$ and $V_m = 1/T_m$. In Table 3.2 all the information relevant to this question has been organized from the results of various ergodic experiments. The parameters T_m and n have been separately determined from the data obtained in each experiment.* The fraction of vehicles stopped and moving as well as the average speed and average running speed when observed are indicated by $f_s(\text{obs})$, $v(\text{obs})$, etc.** The objective is to compare the observed average running speed and the observed average speed with values calculated from the assumption $v_r = v_m f_r^n$ employing the Two-Fluid model parameters determined from the data.

* It is to be noted that for the first and third independent ergodic experiments, which were carried out in the same area of Austin, the parameters T_m and n are very nearly the same.

** In Table 3.2 the ratio $\langle v(\text{obs}) \rangle / \langle v_r(\text{obs}) \rangle = \langle T_r(\text{obs}) \rangle / \langle T(\text{obs}) \rangle$ is not precisely equal to $\langle f_r(\text{obs}) \rangle \equiv \langle T_r / T \rangle_{\text{obs}}$. It would only be the case if the trip time and running time per unit distance were identical for all the test vehicles.

TABLE 3.2. COMPARISON OF OBSERVED AND PREDICTED AVERAGE SPEEDS AND RUNNING SPEEDS FOR FOUR ERGODIC EXPERIMENTS PERFORMED IN AUSTIN

Experiment (No. of Vehicles)	Observation Period	f_r (obs)	Two-Fluid Parameters		Theory $v_r = v_m f_r^n$		v_r (obs) (mph)	Theory $v = v_m f_r^{n+1}$		v (obs) (mph)
			T_m , min/ mile (v_m , mph)	n	$v_m=30$ (mph) Speed Limit	$v_m=60/T_m$ (mph)		$v_m=30$ (mph) Speed Limit	$v_m=60/T_m$ (mph)	
First (8)	17:00:21 to 17:58:12	0.758	1.88 (31.9)	1.50	19.8	21.1	20.4	15.0	16.0	15.2
Second (5)	12:10:09 to 12:54:16	0.735	2.31 (26.0)	1.02	21.9	19.0	19.4	16.1	14.0	13.1
Third (10)	16:30:39 to 17:40:58	0.713	1.82 (33.0)	1.34	19.1	21.0	21.7	13.6	15.0	15.6
Fourth (11)	15:49:12 to 16:28:28	0.740	1.67 (35.9)	2.15	15.7	18.8	18.9	11.6	13.9	13.9
Fourth (12)	16:48:22 to 17:40:55	0.722	1.67 (35.9)	2.15	14.9	17.8	18.5	10.8	12.8	13.4

The average speed limit posted in the Austin areas used in the ergodic experiments is 30 mph. In Table 3.2 we have used this value as well as the value $1/T_m$ obtained from the parameter determination for v_m . We have also listed values of v_r and v calculated from the first assumption in the Two-Fluid model. A comparison of these values with those observed in each of the experiments is made. The average percent deviation between the observed and calculated values of both v_r and v are ~ 13 percent, when v_m is taken as 30 mph, and ~ 3 percent, when v_m is determined from the model parameter T_m .

The improvement in using T_m which gives values of the mean free speed higher than the speed limit, may stem from the fact that drivers in this system may well not observe the 30-mph speed limit if it is possible to go faster. If we accept this interpretation then the second assumption is rather well validated.

SUMMARY AND DISCUSSION

Four experiments have been conducted in the Austin CBD to investigate the reasonableness of the Two-Fluid model assumptions. Following a discussion of the background in Physics relating to ergodic theories, the ergodic character of network traffic has been examined. It has been mathematically shown that for a set of test vehicles over a very long observation period it is possible to replace the ensemble averages by the time averages, namely that $(T_s/T)_i$ for any test vehicle i can be said to equal $\langle f_s \rangle_t$. Such a result, while not a verification of the ergodic assumption, testifies to its reasonableness. This is particularly the case if $(T_s/T)_i$ for any test vehicle i were a reasonable estimate of the fraction of time stopped associated with any vehicle of the population circulating in the area during the same time period. Also to be shown is whether or not

$\langle f \rangle_{s t}$ is a good estimate of $\langle f \rangle_{s p}$, an issue to be addressed in Chapter 6 on the aerial photographic results

The results of the ergodic experiments further show that even for short observation periods of no more than one hour, the above mathematical results hold observationally as well, namely that $(T/T)_i$ for test vehicles are narrowly distributed about their mean $\langle T/T \rangle_s = \langle f \rangle_{s t}$. This, in turn, is an indication that the ergodic assumption is a reasonable one subject to the limitations discussed above.

The experimental results have also been used to verify the model assumption that $v_r = v_m f_r^n$ or $v = v_m f_r^{n+1}$ (Table 3.2). In this endeavor the parameters V_m and n have been obtained through the calibration of the Two-Fluid model for the trip time - stop time data resulted from each experiment. A second value of v_m corresponding to the average speed limit in the network has also been used.

In the course of these verifications both experimental results and the theory suggest that regardless of the length of the observation period, $\langle f \rangle_{s t}$ and $\langle T/T \rangle_t$ are essentially identical variables by means of which the quality of traffic service in a network of streets can be expressed. Furthermore, the results indicate that a single vehicle circulating in a network during an observation period long enough to allow the vehicle to properly sample the network can by itself provide a good estimate of $\langle f \rangle_{s t}$ through its fraction of time stopped (Fig 3.1). As an example, in Table 3.3 the mean fraction of the time stopped in Austin is compared with those in a number of other cities in the United States. All the values noted in the table were obtained during peak traffic conditions in the central business districts of their respective cities. The fraction of the time stopped during the peak period is relatively high in Dallas, Houston and New York/Newark, reaching over 40

TABLE 3.3. AVERAGE FRACTION OF THE TIME STOPPED DURING PEAK TRAFFIC IN THE BUSINESS DISTRICTS OF VARIOUS CITIES

City CBD	Fraction of Time Stopped
Austin ^a	0.242
Dallas	0.378
Houston	0.413
Los Angeles ^b	0.217
New York/Newark ^b	0.368
San Antonio	0.300

^a From the first ergodic experiment during the late afternoon peak period

^b Reference 17 in which the fraction of the time stopped for Los Angeles is given incorrectly

percent in Houston compared to 20-30 percent in Los Angeles, Austin and San Antonio. The fraction of the time stopped is an important traffic variable through which various other variables may be determined and it also undoubtedly has a marked impact on the detailed character of the traffic as well as the behavior of the drivers. It would be of great interest to obtain the data required for the expansion of Table 3.3 and include other major metropolitan areas around the world.

CHAPTER 4. RESULTS OF TWO-FLUID MODEL GROUND STUDIES

During the course of our studies, through observations of trip time and stop time per unit distance, the Two-Fluid model has been calibrated for twelve street networks in eight cities around the world. Trip time-stop time observations have been made in Austin, Dallas, Houston, San Antonio and Lubbock, Texas as well as in Albuquerque, Matamoros and Mexico City. Data have also been available in the literature for five other city networks in Milwaukee, Brussels, London, Sydney, and Melbourne.

The quality of traffic service in these city networks have been compared. A comparison of the results for cities with highly varying characteristics has provided insight into understanding which network features may affect the parameters of the Two-Fluid model. A further examination of the sensitivity of the model parameters to changes in a street network has been made through conducting 'before/after' studies in Dallas, Lubbock and San Antonio. Finally, in order to investigate the effect of vehicular mode on the value of the parameters T_m and n in a particular network, trip time and stop time observations were made during 1979 and 1980 aboard both the student shuttle buses of the University of Texas at Austin and passenger cars driven on the same fixed shuttle bus routes.

OBSERVATIONAL DETAILS

Traffic observations, consisting of recording the trip time and stop time of automobiles traveling known distances, were made in the Central Business Districts (CBD) of five major cities in Texas, namely, Austin, Dallas, Houston, Lubbock and San Antonio, as well as in the CBD networks of Matamoros, Mexico City, Albuquerque, Tehran, and London. The trip time and

stop time data were generally taken for micro-trips two miles long. However, the micro trips were one mile long whenever fuel consumption data was also taken. The data in Mexico City and Tehran were for two-km and one-km trips, respectively.

The data collection procedure was similar to the one employed in the ergodic experiments. As before, a test vehicle carried a driver and an observer. The driver was responsible for the following: informing the observer of the odometer readings at the beginning and end of each two-mile trip, using the chase car technique, obeying all traffic regulations, and remaining within the boundaries of the designated street network as much as possible. The observer, on the other hand, was responsible for recording the odometer readings and the absolute times corresponding to the beginning and end of each two-mile trip and for noting the absolute times associated with each stop made by the test vehicle and the subsequent resumption of motion. In these cases, time was measured with Casio Digital watches, Module No. 79. Table 4.1 shows a typical data sheet.

In Austin and Dallas two sub-areas of the CBD (referred to as CBD1 and CBD2) were also examined. The purpose of these observations was to establish the Two-Fluid curvilinear relation between trip time per unit distance and stop time per unit distance, Eq (2.22), by determining the model parameters T_m and n from the data through Eqs (2.25) and (2.26). Trip time - stop time data were also collected in three non-CBD networks: two in San Antonio and one in Houston. The boundaries of each study area are given in Appendix A.

The Austin data were collected during the years 1980-1983, the Dallas and Matamoros data during 1981-1983, the Lubbock and Mexico City data during 1983-1984, the San Antonio data during 1981-1984, the Houston data in 1982, and the Albuquerque data in 1983. The data in all cases were obtained during

TABLE 4.1. A TYPICAL DATA SHEET FOR A ONE-MILE TRIP TIME - STOP TIME
OBSERVATION IN AUSTIN

Date: February 28, 1983
 Driver: Lin Morrisett
 Observer: Mark Pierce
 Starting Mileage: 2069.0
 Starting Time: 22:57:00

Time of Stop	Time of Start	Comments
22:57:27	57:28	yield sign
58:21	59:13	left-turn signal
23:00:04	00:08	stop sign
00:27	00:31	stop sign
00:48	01:09	signal
01:28	01:42	signal

End of Trip: Time 23:01:58
 Mileage Reading 2070.0

the morning, noon and evening rush-hours as well as the morning, afternoon, and nighttime off-peak hours.

COMPARISON OF THE QUALITY OF TRAFFIC SERVICE IN VARIOUS NETWORKS

In earlier studies [Refs 17 and 20] attempts were made to examine general traffic characteristics in various cities with particular interest in relating the character and quality of traffic to pertinent traffic variables, such as average speed, stop time per unit distance, standard deviation of acceleration, etc., as well as fuel consumption and exhaust emissions [Ref 18]. When the consequences of the Two-Fluid approach were first examined, essentially linear trends for trip time versus stop time were found for various cities around the world including nine metropolitan areas in the United States, Detroit, London, and Melbourne [Ref 32]. The results for the nine metropolitan areas came from an analysis [Refs 17 and 20] of chase-car data collected by Johnson et al [Ref 19]. Although there is a considerable amount of scatter in the data and the trend lines vary in slope and intercept, they do fall in a fairly consistent overall trend. This encouraged looking further to determine if there were consistency among data taken at various times and under various traffic conditions for a given city when examined in terms of T versus T_s . This certainly appeared to be the case in the present work which led to a further examination of these questions in terms of the quality of traffic service in a number of cities around the world.

The Two-Fluid model predicts a linear relation between the logarithm of the running time per unit distance and the logarithm of the trip time per unit distance, Eq (2.24); thus, $\log T_r$ was linearly regressed against $\log T$, and the values of the parameters T_m and n were determined for each case. In Table 4.2 are listed the Two-Fluid model parameters T_m and n together with

TABLE 4.2. TWO-FLUID MODEL PARAMETERS FOR VARIOUS CITIES AROUND THE WORLD

City Network	No. of Points (2-mile/1-mile)	Linear Representation			Two-Fluid Model Parameters		
		Intercept (min/mile)	Slope	R ²	T _m (min/mile)	n	R ²
Austin CBD1	354/ -	2.32	1.54	0.87	1.75	1.63	0.82
Austin CBD2	122/90	2.68	1.43	0.80	1.95	1.58	0.72
Austin CBD1+2	476/90	2.43	1.50	0.84	1.78	1.65	0.78
Dallas CBD1	124/ -	2.53	1.49	0.88	1.79	1.62	0.82
Dallas CBD2	- /136	2.62	1.53	0.89	2.12	1.36	0.79
Dallas CBD1+2	124/136	2.59	1.50	0.88	1.97	1.48	0.80
Houston CBD	- /159	3.07	1.30	0.90	2.70	0.80	0.63
Houston Non-CBD	- /51	2.18	1.35	0.93	1.92	0.98	0.75
Lubbock CBD	- /190	2.29	1.39	0.88	2.03	0.97	0.74
San Antonio CBD	47/55	2.54	1.48	0.92	1.96	1.38	0.85
San Antonio Non-CBD1	14/42	2.25	1.16	0.81	1.94	0.72	0.45
San Antonio Non-CBD2	- /39	2.24	1.31	0.83	1.90	1.12	0.66
Milwaukee [Ref 20]	24*	1.81	1.61	0.84	1.58	1.41	0.81
Albuquerque	- /68	2.61	1.43	0.76	1.93	1.62	0.70
Average Over Nine Major U.S. Cities [Ref 17]	32*	2.17	2.16	0.95	1.74	2.83	0.97
Matamoros, Mexico	- /88	4.23	1.62	0.82	2.98	2.10	0.88
Mexico City [30]	101**	2.87	1.34	0.95	1.72	1.63	0.84
Brussels [Ref 37]	90*	2.13	1.80	0.87	1.26	2.76	0.92
London [Ref 28]	39*	2.74	2.00	0.90	1.93	3.02	0.97
Melbourne [Ref 26, 35]	34*	2.00	1.62	0.98	1.74	1.41	0.95
Sydney [Ref 26,36]	95*	2.06	1.81	0.92	1.85	1.68	0.88

* These points refer to various trip lengths

** Two-kilometer trips

the correlation coefficient, the intercept and the slope for the linear regression of T against T_s . In addition, the numbers of two-mile and one-mile trips used in the calculations are given. For purposes of comparison the parameters for other cities around the world are listed in addition to those for the Texas cities. The data from Albuquerque were collected by volunteers interested in the study. The old data from London, Mexico City, Milwaukee, Brussels, Sydney and Melbourne were obtained from the literature and the respective sources are given in Table 4.2.

In examining the parameters in Table 4.2 note that the intercept in the linear regression corresponds to the intercept T_m in the Two-Fluid model, and the slope in the former case corresponds to the average slope over the range of interest in the latter case. The slope in the Two-Fluid case depends on both T_m and n , but mainly on n . Thus, the curves for fixed n but various values of T_m are essentially parallel. The linear regression for the average over nine U.S. cities is given by $T = 2.17 + 2.16 T_s$. For the Two-Fluid case, $T_m = 2$ min/mile and $n = 2$, Eq (2.27) yields in the range from $T = 2$ to 8 min/mile an average slope of 2.07, in relatively good agreement with the slope of the linear regression, 2.16.

In order to discuss a comparison of cities in Table 4.2 we have chosen Austin and Houston. The largest amount of data has been obtained for the Austin CBD1+2 for which $T_m = 1.78$ min/mile and $n = 1.65$, with $R^2 = 0.78$; for Houston these values are 2.70, 0.80 and 0.63, respectively. The Two-Fluid model equations for Austin and Houston corresponding to the above-stated values of T_m and n are as follows:

$$T_s = T - 1.24 T^{0.623}, \quad \text{Austin CBD1 + 2} \quad (4.1)$$

and
$$T_s = T - 1.74 T^{0.444}, \quad \text{Houston CBD} \quad (4.2)$$

Figures 4.1 and 4.2 are plots of T versus T_s data obtained in the Austin and Houston CBD's, respectively, in which the solid curves are given by the equations directly above. Not shown are the best-fit linear regressions which closely approximate the data and the slightly convex upward Two-Fluid curves. The relatively high correlation coefficients for both the Two-Fluid and linear representations indicate that the trends in the data collected in Austin and Houston can indeed be modelled by the Two-Fluid theory or its linear approximation. Moreover, Figs 4.1 and 4.2 show that the trends do not depend on the level of traffic congestion in the networks; rather, as the traffic density fluctuates with time of the day, etc., the data points move along the trends.

An examination of the data acquired so far indicates that while there is considerable scatter, there generally appears to be a tendency for the data to curve slightly as the theory suggests or to have a linear trend in the logarithmic representation as shown in Fig 4.3. If the curves for Austin and Houston were overlaid one would find that the Houston curve lies above that for Austin in the light traffic region indicating that the traffic in Austin moves at significantly higher average speed in the off-peak hours. On the other hand, the curves are essentially overlapping in the congested traffic region. While this implies that the quality of traffic service would be the same during congestion, it must be noted that considerably more traffic is processed in Houston during the congested periods. The data in the Dallas CBD also indicates better quality of traffic service there during the light-traffic nighttime period than in the CBD of Houston. These findings have been corroborated by various traffic engineers. This situation in Houston during uncongested periods may be improved by modifications in the signal timing patterns set to serve the peak-period traffic.

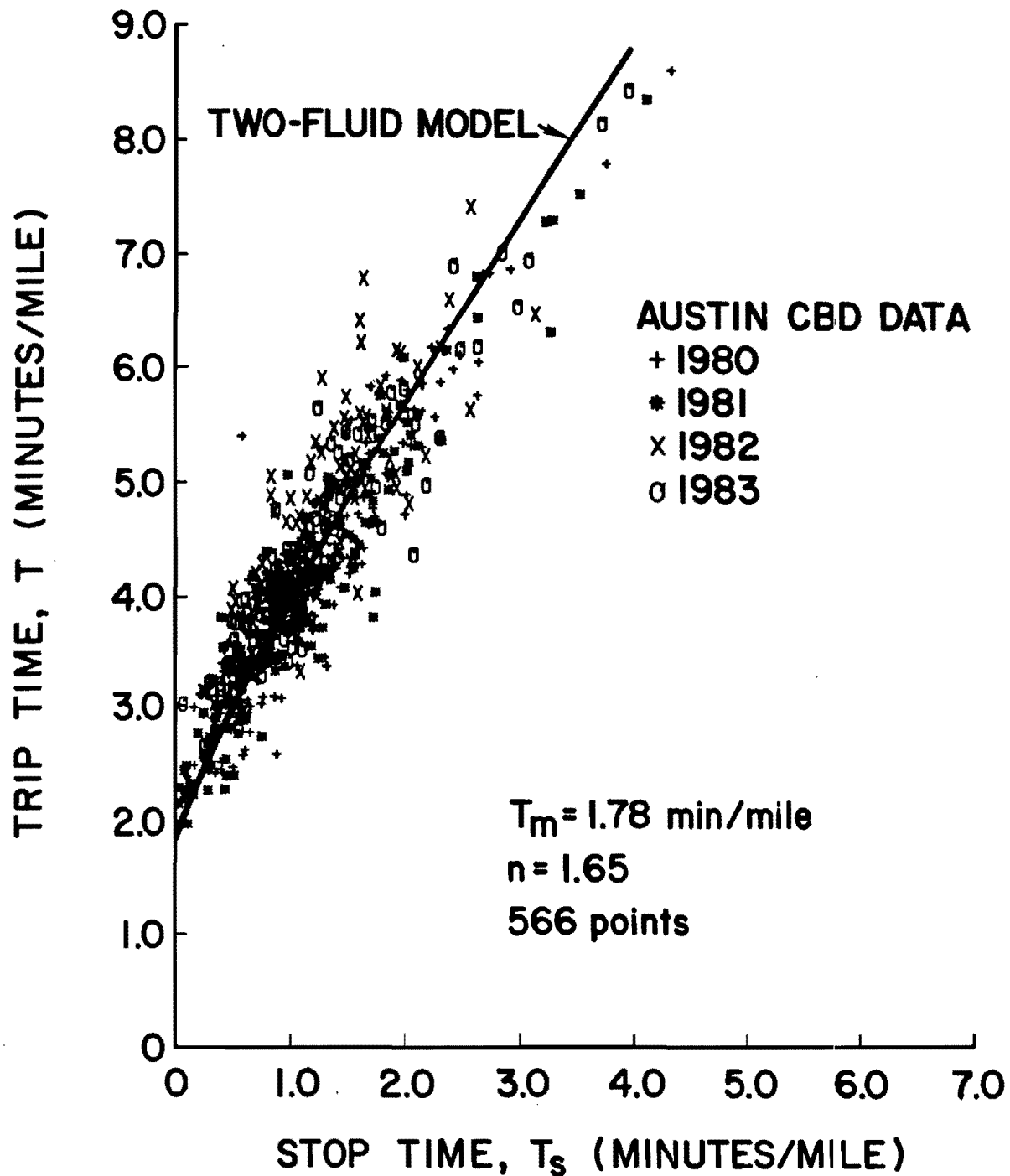


Figure 4.1. Trip time versus stop time for the non-freeway street network of the Austin CBD. Each point represents one test run approximately one or two miles long. The Two-Fluid model curve is the regression fit of the data to Eq (2.22). The parameters n and T_m are obtained using Eqs (2.25) and (2.26).

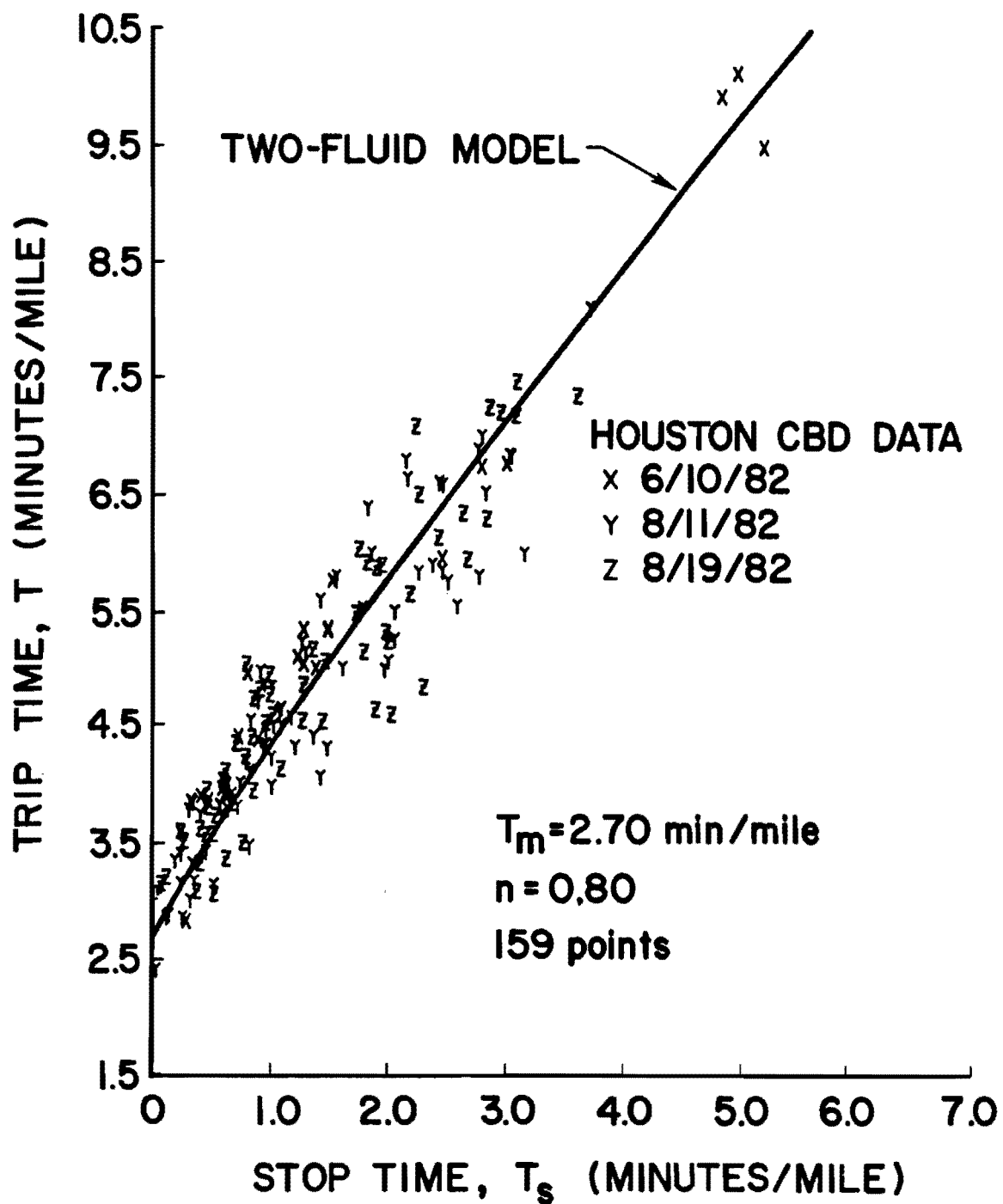


Figure 4.2. Trip time versus stop time for data collected in the CBD of Houston, Texas. The Two-Fluid model curve is the regression fit of the data to Eq (2.22).

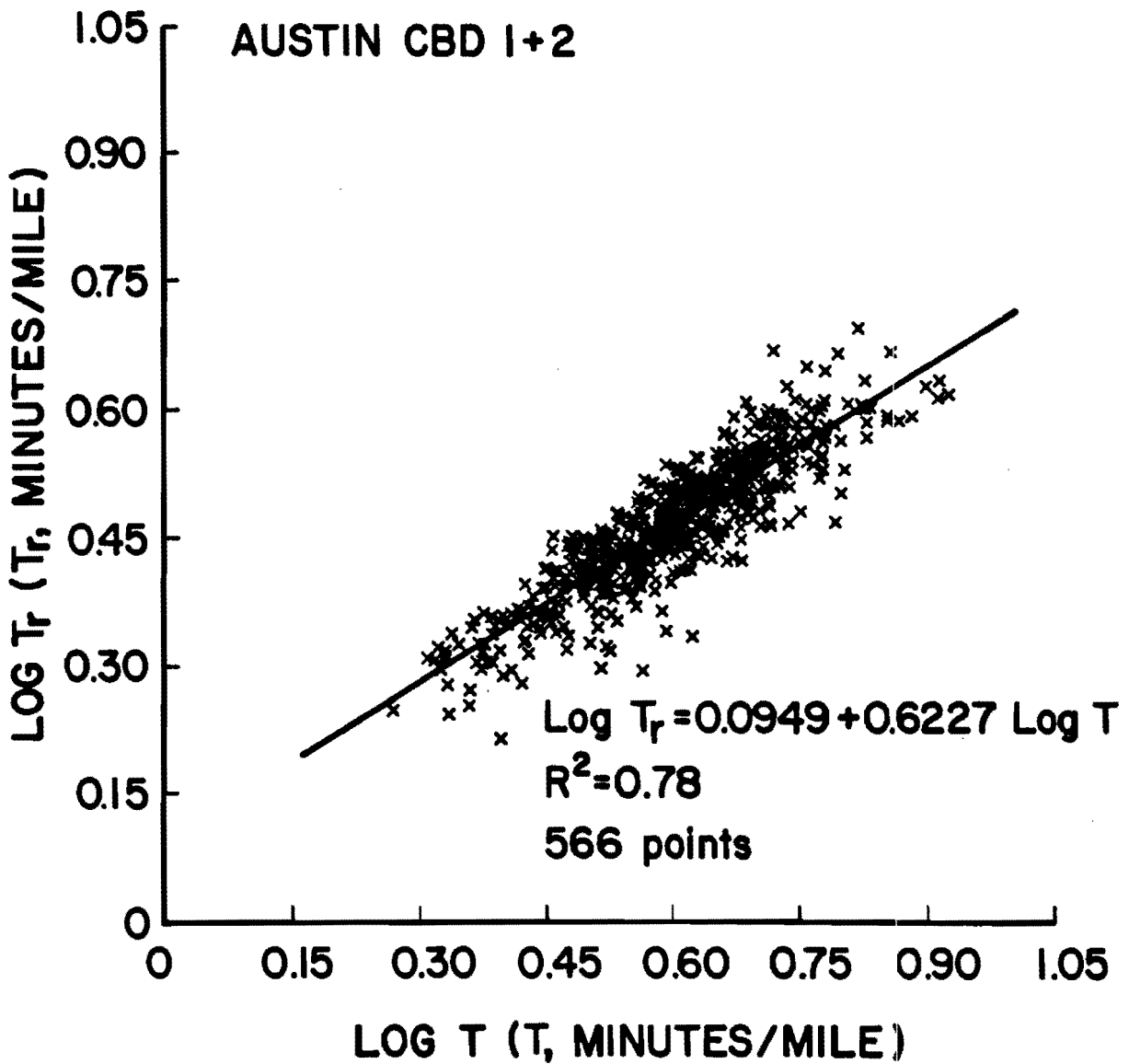


Figure 4.3. The logarithm of running time, $\log T_r$, versus the logarithm of trip time, $\log T$ for the CBD data in Austin, Texas. The line is the regression fit of the data to Eq (2.23) for the Two-Fluid model.

The parameters for various other cities in Texas and around the world are also listed in Table 4.2. In previous publications the data for the nine metropolitan areas in the United States [Ref 17] and also detailed data for Milwaukee [Ref 28], London [Ref 28], Melbourne [Ref 26 and 35], Sydney [Ref 26 and 36] and Brussels [Ref 37] have been presented. The slope dT/dT_s of the linear regression approximation represents the increase in the trip time per unit distance for a unit increase in the stop time per unit distance. Thus, from Table 4.2 it would appear that the street systems of Austin, Dallas, Houston and Milwaukee in a general sense possibly have better traffic control, geometric and other features than those of, say, Sydney, Brussels and London. For example, the traffic network of old central London with narrow one- and two-way streets, long-cycled traffic signals, heavily used pedestrian crossings with pedestrian-actuated controls, etc. has a slope in the linear representation of 2.00 while Dallas with many wide one-way streets, short-cycled traffic signals, moderate pedestrian crossings, etc. has a slope of 1.50.

The slope of the Two-Fluid model curve is given by Eq (2.28); thus, for a value of $T = 3.0$ minutes per mile or a mean speed of 20 miles per hour, old London has a slope of 3.060 and Matamoros a slope of 3.137, while the corresponding values for Austin, Melbourne and Milwaukee are 2.046, 1.875 and 1.813, respectively. At lower speeds, say, $T = 4.0$ or 15 miles per hour, where the slopes are lower, the slope for old London is 2.679 and 2.672 for Matamoros while for Houston and Milwaukee they are 1.556 and 1.719, respectively. In other words, when T is 3 minutes per mile dT/dT_s is 1.5 times greater for London and Matamoros compared to Austin, Melbourne and Milwaukee. Likewise at $T = 4$ minutes per mile, there are significant differences between Matamoros and London compared to Houston and Milwaukee.

The above examples show that for a given change in T_s there is a smaller change in T for networks with smaller n values. Examining the relative slopes dT/dT_s appears to be a way to compare the quality of traffic service in various cities. The significantly larger values of the parameter n for London and Matamoros compared to the other cities in the above examples is in agreement with the above results.

In order to further illustrate the role of the parameters T_m and n as characterizers of the quality of traffic service, the Two-Fluid model assumption of Eq (2.1) can be rewritten in the form

$$v = v_m (1 - f_s)^{(n+1)} \quad , \quad (4.3)$$

or
$$T = T_m (1 - f_s)^{-(n+1)} \quad . \quad (4.4)$$

From Eq (4.4) it can be seen that for a given fraction of vehicles stopped, the network with the larger value of T_m or n is the one with the lower average speed. Since in the urban regime the mean speed is directly related to the average delay and fuel utilization, a lower average speed indicates a poorer quality of traffic service. Therefore, the higher values of n for London, Brussels and Matamoros represent less desirable traffic qualities than those, for example, in Austin, Dallas, Melbourne, etc. given that the same f_s in different networks corresponds to the same level of concentration. The relation between f_s and the concentration is discussed in Chapter 6 on aerial photographic results.

Figures 4.4 and 4.5 are graphical representations of the Two-Fluid trends for some of the city networks in Table 4.2. In Fig 4.4, the Two-Fluid model trends for the Texas cities of Austin, Houston, Lubbock, San Antonio as well as Matamoros, Mexico are presented. The curve for Dallas lies very close to that of Austin and has been omitted for the sake of clarity. In Fig

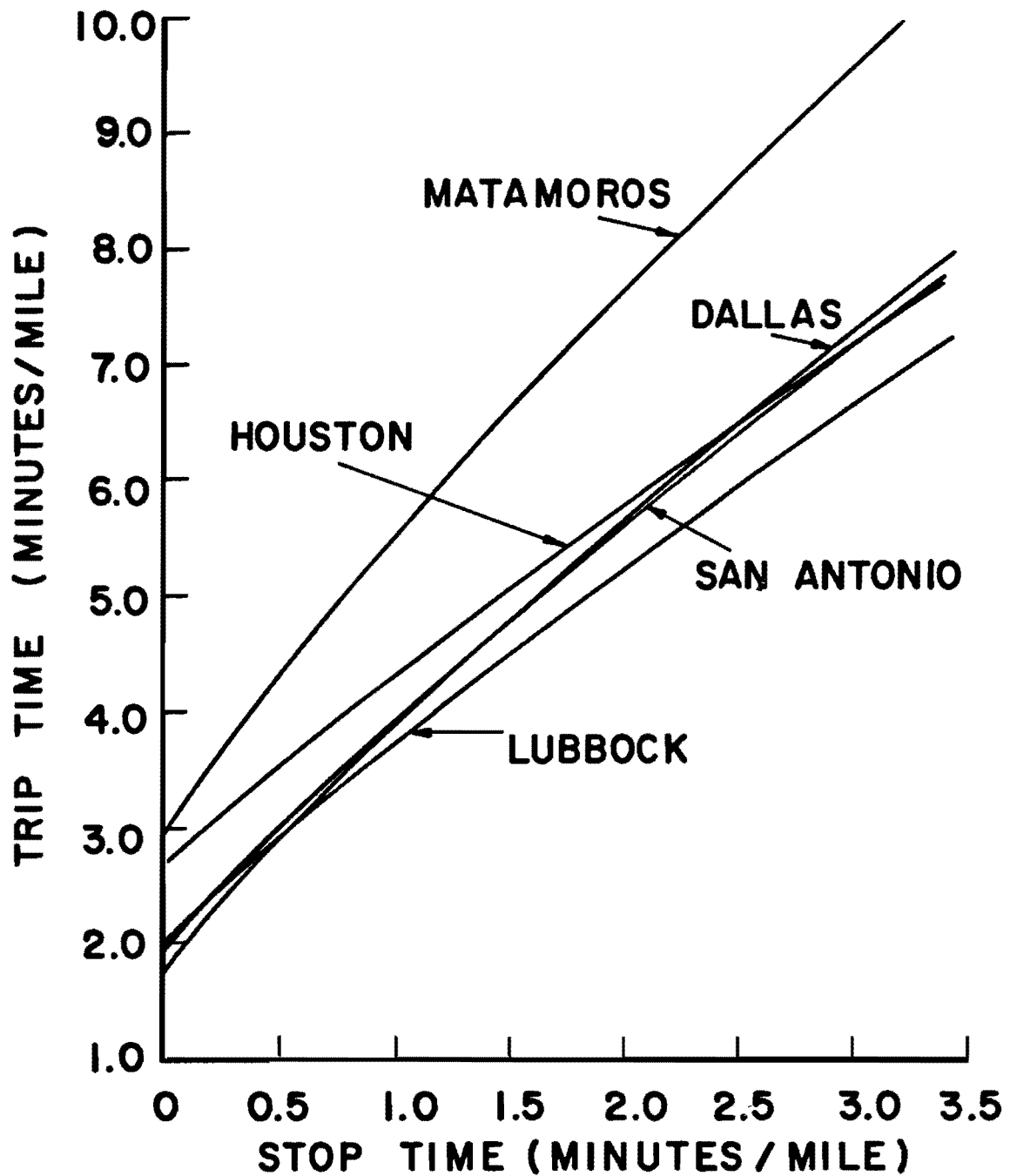


Figure 4.4. Trip time versus stop time Two-Fluid model trends for CBD data from the cities of Dallas, Houston, Lubbock, San Antonio, Texas, as well as for Matamoros, Mexico.

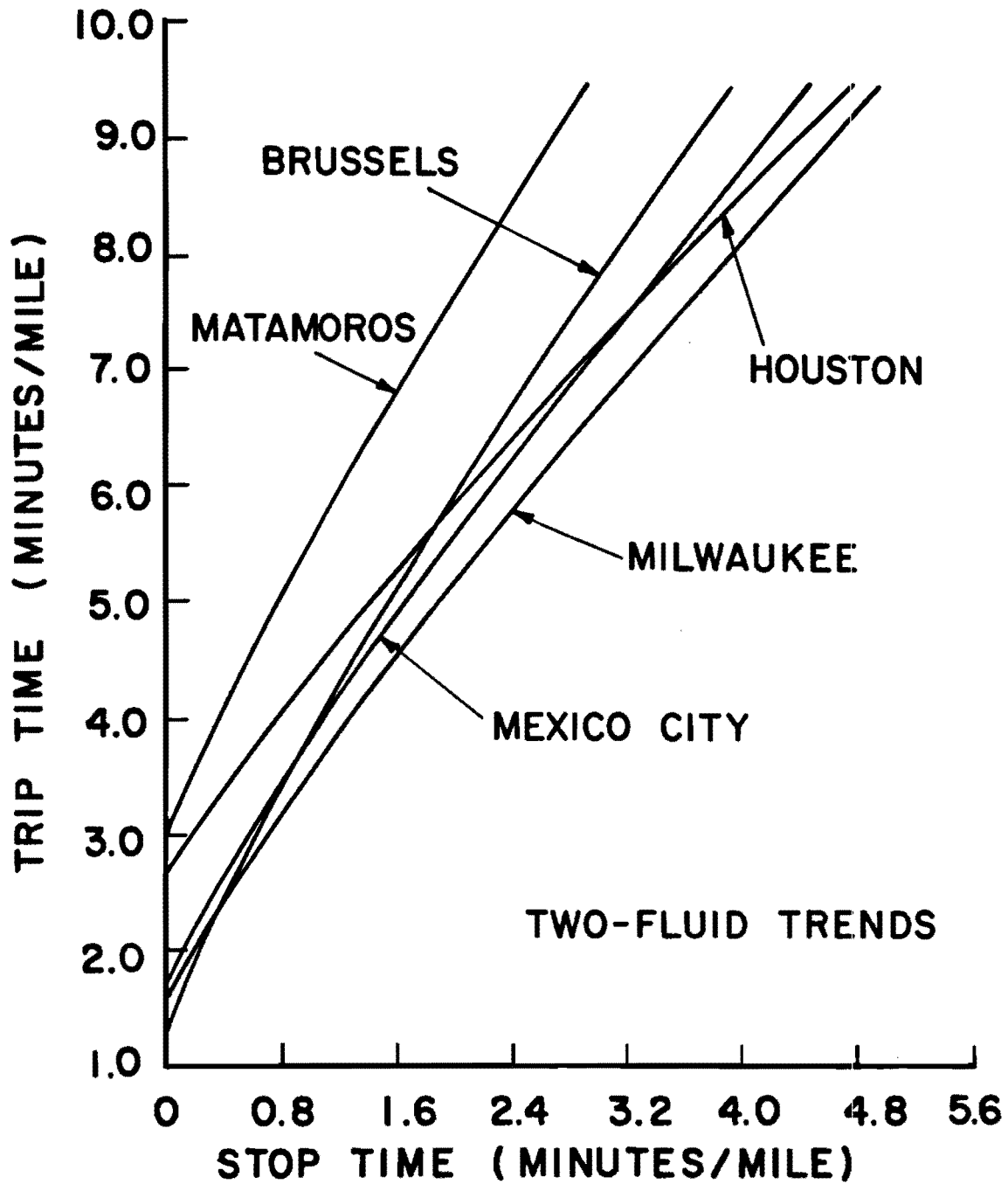


Figure 4.5. Trip time versus stop time Two-Fluid model trends for Houston, Milwaukee, Matamoros, Mexico City, and Brussels.

4.5 the trends for Mexico City, Matamoros, Brussels, Houston and Milwaukee are plotted. Many of the points discussed earlier are exemplified by these trends. A question that arises at this juncture is whether there are significant differences among the various trends for different cities.

In order to address this question the confidence limits for the linear regression to the Austin data are shown in Fig 4.6. These are ± 1 and ± 2 standard deviation limits. The observed data points plotted are those for the city of Houston. Note that most of the Houston points lie outside the $\pm 1 \sigma$ bound especially in the region of small T , T_s . In Fig 4.7 we have shown the confidence limits for the Dallas CBD Two-Fluid trend along with the observed data for Matamoros, Mexico. In this case almost all the data fall outside $\pm 1 \sigma$ limits and most of the data even lie outside the $\pm 2 \sigma$ limits.

Figure 4.8 shows the $\pm 1 \sigma$ and $\pm 2 \sigma$ confidence intervals of the Two-Fluid trend for Matamoros along with the data from Mexico City. It can be seen from Fig 4.8 that not only does nearly all the Mexico City data fall outside the -1σ limit of the Matamoros trend but also a great number of the points fall well outside the -2σ limit as well. It can then be concluded that the Two-Fluid trends for these two cities are statistically different.

Furthermore, as shown in Table 4.2 the Two-Fluid model parameters associated with the Mexico City study network ($T_m = 1.72$ min/mile, $n = 1.63$) are comparable to those of some of the better traffic networks listed in the table. This is not surprising since the study network in Mexico City with its mostly wide one-way streets and well-designed traffic control system appear to efficiently serve the types of vehicular concentrations which are considered peak traffic in most of the other cities studied. The traffic problems in the Mexico City central network appear to result from the great

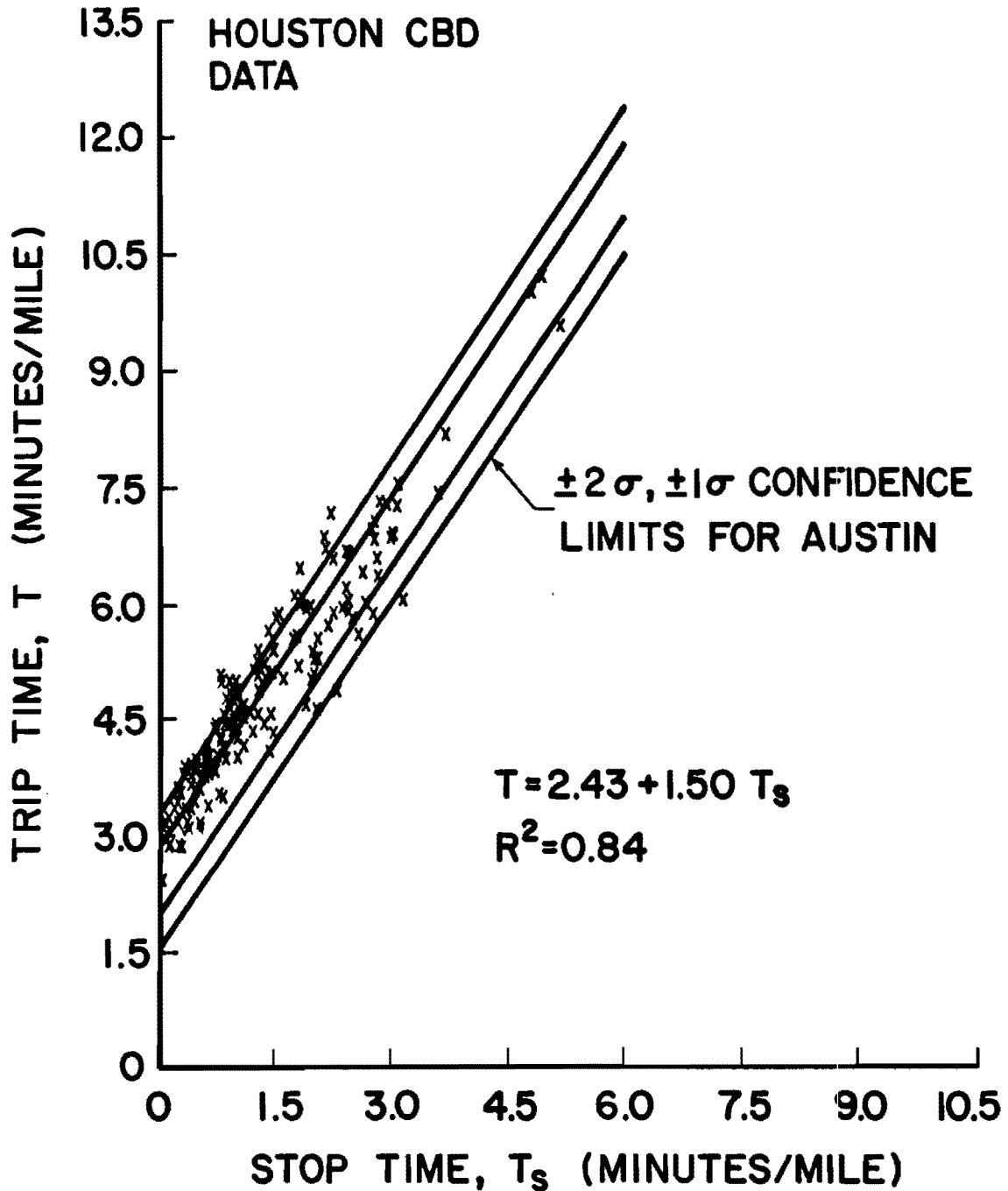


Figure 4.6. Trip time versus stop time data for the Houston CBD. The lines represent the $\pm 1\sigma$ and $\pm 2\sigma$ confidence bands associated with the linear regression to the CBD data of Austin which are not shown. Note that most of the Houston data lie outside the $+1\sigma$ limit in the region of the low trip time values.

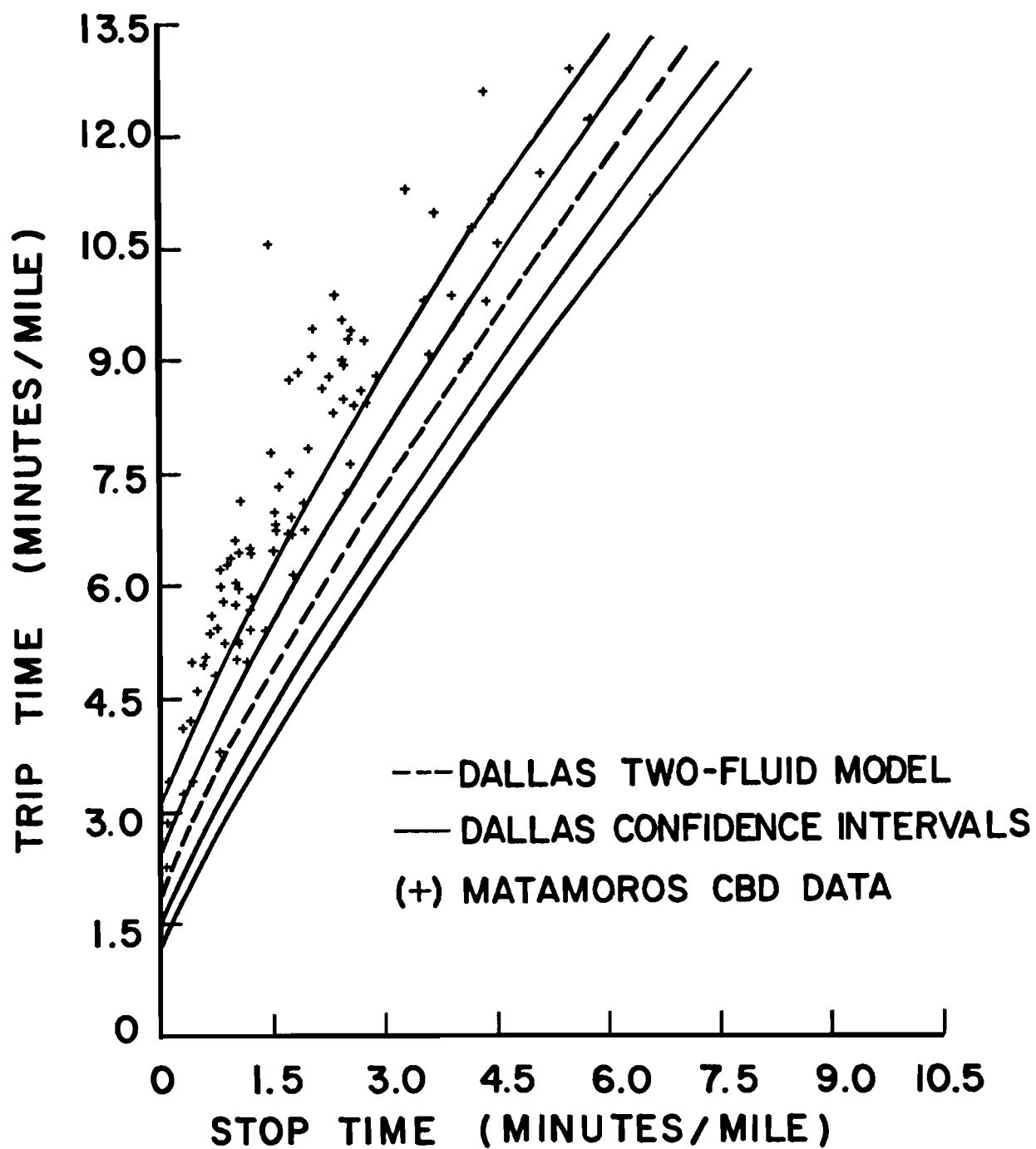


Figure 4.7. Trip time versus stop time data for the Matamoros CBD data. The solid curves represent the $\pm 1\sigma$ and $\pm 2\sigma$ confidence bands associated with the Two-Fluid curvilinear regression to the CBD data of Dallas which are not shown. Note that most of the Matamoros data lie outside $+2\sigma$ limit of Dallas.

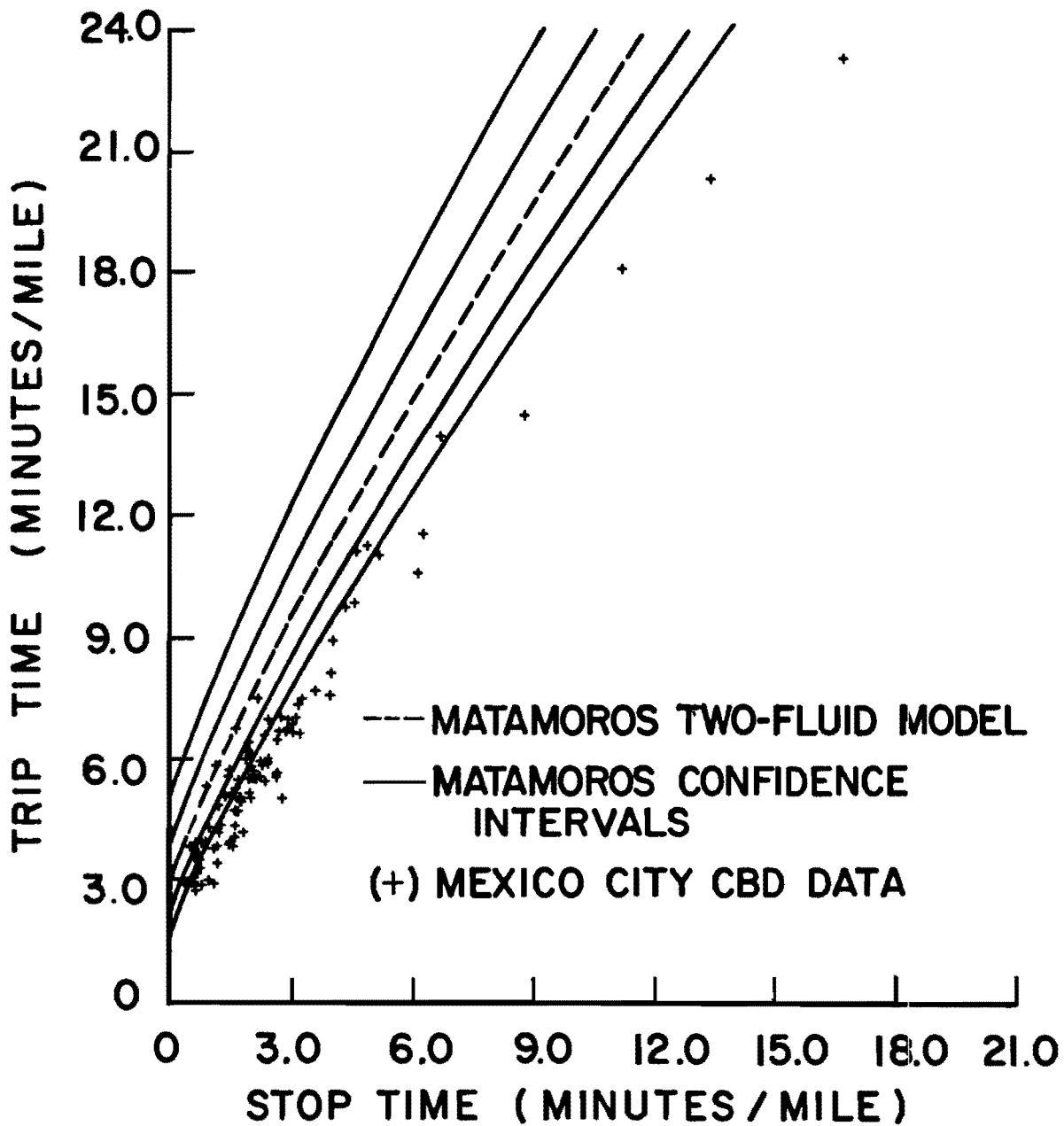


Figure 4.8. Trip time versus stop time data for the Mexico City CBD. The solid curves represent the $\pm 1\sigma$ and $\pm 2\sigma$ confidence bands associated with the Two-Fluid curvilinear regression to the CBD data of Matamoros which are not shown. Note that most of the Mexico City data lie outside the -2σ limit of Matamoros.

traffic demand imposed on the network and not a result of a poor quality of traffic service.

In understanding the effects of various physical features of a network on the parameters T_m and n , it would be instructive to compare networks with significantly different T_m and n values. Among the ten cities studied Matamoros ($T_m = 2.98$ min/mile, $n = 2.10$) and Lubbock ($T_m = 2.03$ min/mile, $n = 0.97$) ranked among the worst and best, respectively, in terms of the quality of traffic service. A comparison of physical features of these two networks may then be useful in identifying these features which influence the values of T_m and n the most.

The Matamoros CBD is a grid network of approximately 1.2 by 1.2 miles, where the majority of the streets (~75 percent) are one-way streets about 30 feet in width with parallel parking allowed on both sides. This leaves an effective 14-foot wide through lane. The street pavements in this area are marred with potholes, frequent patches, and severe rutting and warpings. At intersection areas usually the crowns of the major streets remain unaltered, thus resulting in the slow down of the crossing traffic stream.

In contrast to Matamoros, the Lubbock CBD is a grid network of approximately one by two miles, where the majority of the streets (~80 percent) are two-way streets. A considerable fraction of the streets are very wide, having five to seven through lanes. Unlike Matamoros, pavement surfaces in Lubbock are generally surfaced with brick and offer a high riding quality. However, as is the case in Matamoros, the crowns of the major streets remain unaltered at most of the intersection areas.

In terms of control devices, in Matamoros about half of the intersections in the CBD area are signalized and the remainder have two-way or four-way stop signs. Traffic signals have fixed timing patterns and the

same timing scheme remains in effect throughout the day. At intersections with stop-sign control, it is not clear whether the control is a two-way or a four-way stop. Stop signs are also posted at the signalized intersections. This is partially because there are frequent power failures. These auxiliary stop signs also serve as a warning to the drivers facing a green display to beware of the widely-practiced running of the red. Right-turn and 'straight-through' movements on red are common occurrences even though they are both prohibited in Mexico.

In the Lubbock CBD, containing some 476 intersections (not counting intersections with small alleys) more than half are controlled by stop sign with the majority of them being two-way stop signs. Among signalized intersections about half have timing patterns with more than two phases. The cycle lengths are generally long and mostly exceed 90 seconds in duration. Cycle lengths as long as 150 seconds with red phases lasting more than one minute have also been encountered. Long cycles and red phases are in operation partly to allow pedestrians to cross streets which are at times as wide as 100 feet. While the very wide streets may minimize the vehicular interaction at relatively high concentrations, thus contributing to a small value of n , the consequent long cycle lengths and red phases may result in a high T_m value.

Among the twenty-one networks listed in Table 4.2, the T_m value of 2.98 minutes per mile for Matamoros is by far the highest. This implies high values of running time per mile even under very light traffic conditions. A contributing factor to a high minimum running time is that even at very light traffic concentrations in Matamoros, a driver must approach a green signal indication with extreme caution. This is partly due to the lack of adequate sight distance for cross streets caused by the proximity of buildings to

intersection corners. Inadequate sight distance at intersections combined with the common practice of vehicles running the red as well as rough pavement surfaces in the intersection areas are major factors in reduction of the speed and flow of vehicles through intersections in Matamoros. Such reductions in speed would force further increases in values of T_m and n .

Another important characteristic of any network is the ease with which it handles a given level of traffic. In this regard, the geometry of streets, the mix of vehicles using the network, and the socio-economic characteristics of the locale and the character of its population are key factors. The narrow streets of Matamoros are utilized by highly diverse modes of transportation, each noticeably present. These include transit buses, mini-buses, taxis, passenger cars, delivery and service trucks, hand-pulled and animal-pulled carts, motorcycles, bicycles, etc. In the interaction of these modes, aggressivity of the driver and size of the vehicle play a major role in the determination of the right-of-way. Crowded buses, emitting black exhaust fumes into the atmosphere, indulge in evasive maneuvers and nudge their way through the intersections while drivers of smaller vehicles can only honk in frustration. To add to the problems, pedestrians also encroach on the path of the vehicular traffic when the narrow sidewalks become over-saturated. Overfilled sidewalks or not, J-walking is a frequently occurring "random" event. Such conditions force intimidated vehicle operators to drive cautiously, stop frequently, and maintain a low running speed even though they may have the opportunity to drive faster.

In the Lubbock CBD, on the contrary, wide intersections with adequate sight distance allow vehicles facing green to clear the intersections without reductions in their approach speeds. However, at intersections where the

pavement crown of the major street is unaltered through the intersection area a hump is generated which often acts as a 'speed bump' for drivers familiar with the area. The interaction of vehicles is damped not only as a result of wide streets which leave ample room for maneuvering but also due to a mix of vehicles consisting mostly of passenger cars and pick ups with occasional delivery vans, transit buses, motorcycles and bicycles. In addition to the above, pedestrian crossing usually occurs at intersections except near the Texas Tech campus area. All these conditions appear to contribute to relatively lower T_m and n values.

The average stop time per mile in Matamoros is one of the highest among the cities studied. The stop delay is high even when there is little or no traffic in the network provided that one obeys the traffic ordinances. A major factor is the high density of traffic signals in this area coupled with the unusually long cycle lengths of about 90-150 seconds. Although a major portion of this area is geometrically ideal for a network-wide signal coordination plan (a grid network of one-way streets), any signal coordination plan is unlikely to succeed mainly because there are a great number of interactions among the vehicles along the links, and these interactions build up rapidly with a slight rise in the vehicular concentration. For example, the frequent stopping of buses, mini-buses, and service trucks to load and unload often results in the blockage of the only through lane available, thus rendering any signal progression plan useless.

Another cause of long stop times in Matamoros might be traced to the transit agents, i.e. traffic officers. During peak traffic conditions, they often override the signal timings. They then assign priority to the approach whose queue is the longest until the queue is emptied or until the outbound approaches to the vehicles in that queue are saturated. The agents overlook

the adverse effects of this prioritization rule, for example, on the neighboring intersections. As a result, they often aggravate the traffic conditions as a whole. The only contributions they might possibly make are to prevent the spill back of outbound approaches into their local intersection area and to quickly clear the intersections in the event of minor accidents.

The high level of vehicular interaction and long stop times, caused by the above-mentioned phenomena in Matamoros, result in a Two-Fluid model trend with high values of the parameters T_m and especially n . A higher value of n means a steeper T, T_s trend. As the stop time per unit distance is increased by an amount ΔT_s , the vehicles are also penalized through an increase in their running time per unit distance, ΔT_r . For the same ΔT_s , the magnitude of ΔT_r is greater in the networks whose Two-Fluid trends have greater n values. For example, a T_s increase from zero to 2.0 minutes/mile would mean a ΔT_r of +1.2 minutes/mile in the Lubbock CBD ($n = 0.97$) but a ΔT_r of +2.7 minutes/mile in Matamoros ($n = 2.10$).

While the T_m values for Matamoros and Lubbock are 2.98 and 2.03 minutes/mile, respectively, the value of n for the latter network is more than two times smaller. In light of the above, a comparison of the physical features of the Matamoros and Lubbock CBD's can be most instructive in identifying those network characteristics that affect the Two-Fluid model parameters. In the past, attempts have been made to explore differences and similarities between metropolitan areas. For example, Golob et al [Ref 40] have used statistical methods such as stepwise discriminant analysis on 15 variables selected out of 53 variables initially considered in order to investigate similarities and differences between metropolitan areas with respect to their arterial transportation needs and requirements. For our

purposes we have proposed the following preliminary list of network components pertinent to the quality of traffic service:

1. Fraction of one-way streets
2. Density of traffic signals, stop signs, etc.
3. Average number of lanes per street
4. Average cycle length
5. Fraction of protected left-turn signals
6. Average block length
7. Percent buses
8. Percent trucks
9. Percent motorcycles/bicycles
10. Pedestrian density
11. Fraction of streets with parallel parking allowed
12. Grades
13. Degree of success in signal progression
14. Degree of success in demand responsive signal timing
15. Level of driver aggressivity
16. Level of driver obedience to traffic ordinances
17. Diversity of vehicular modes

The above 17 features can then be rated as being high (H), medium (M), or low (L) for each study network. As an alternative to the above qualitative rating scheme, it is possible to quantify most of these features. For example, the density of traffic signals may be expressed as the number of signals per mile of roadway and the fraction of protected left-turn signals may be expressed as the ratio of number of signalized intersections with protected left-turn phase (excluding the intersections of two one-way

streets) to the total number of signalized intersections. While determining some of the above network features such as the average block length or the fraction of one-way streets is not very involved, considerable effort is required in quantifying a majority of the features listed above. It is however possible to develop efficient sampling and measurement techniques for these purposes.

Network comparisons such as the above comparison between Lubbock and Matamoros begin to indicate that it is possible to capture differences and similarities among the various cities through the Two-Fluid representation. Furthermore, a systematic comparison of features and characteristics of various networks, as suggested above, would enable traffic engineers to identify the network elements that affect the quality of traffic service the most. In this connection it should be noted that data taken in a given city area day after day fall well on the trend line that is established for the area unless the study area undergoes major changes in its geometric or control features. The following section presents the results of before/after studies performed in the cities of Dallas, Lubbock, and San Antonio to examine the effect of changes in network timing patterns and geometric features on the parameters of the Two-Fluid model and the distribution of the T, T_s data along their respective trends.

'BEFORE' AND 'AFTER' STUDIES

A question of special interest in any characterization of the quality of traffic service in a network is whether the approach is sufficiently sensitive to detect effects arising from changes in the traffic system, such as modifications in the signalization, alterations in one-way street patterns, etc. As an initial attempt to answer this question, before/after studies have been performed in Dallas, Lubbock and San Antonio.

In the Dallas CBD prior to early October 1982, there were three timing plans in operation for the morning peak (6:15-9:00 a.m.), afternoon peak (3:30-6:15 p.m.), and off-peak hours and weekends. During late October 1982 through late January 1983, the timing plan was changed so that there were different timing schemes for the following periods: morning peak (6:15-9:00 a.m.), mid-day and weekends (9:00-3:30 p.m.), afternoon peak (3:30-6:15 p.m.), evening (6:15-8:00 p.m.) and deep night (8:00 p.m.-6:15 a.m.).

The above timing plan changes in the CBD of Dallas provided us with an opportunity to see whether the Two-Fluid approach would be useful in detecting changes in the quality of traffic service associated with these changes. The T , T_s data were taken, in the manner already described, in the before study on September 23, 1982, and on January 6 and 7, 1983 for the after study in the CBD area where the timing changes had been made. In Table 4.3 the pertinent information is recorded, namely, \bar{T} , the average trip time per mile for the number of one-mile trips indicated in parenthesis; \bar{T}_s , the average stop time per mile; and the various standard deviations for the five time periods during which the before/after data were taken. The timing scheme for the later period, 9:34-11:12 p.m., was the same for the before/after studies since those changes were made in late January.

The results for the morning period, 7:50-8:26 a.m., indicate that both \bar{T} and \bar{T}_s are larger after than before. This result corresponds to the qualitative assessments of the Dallas traffic engineers. We understand that the timing scheme for this period will be changed again to attempt to improve the traffic service. In the case of the next three time periods of the observations, namely 10:01-10:45 a.m., 12:02-12:50 p.m., and 4:53-5:50 p.m., both \bar{T} and \bar{T}_s are reduced; this result was also in agreement with the conclusions of the Dallas traffic engineers. In the last period, 9:34-11:12

TABLE 4.3. COMPARISON OF THE TRIP TIME AND STOP TIME PER UNIT DISTANCE IN THE DALLAS CBD NETWORK BEFORE AND AFTER MAJOR MODIFICATIONS IN THE TRAFFIC SIGNAL TIMING PATTERNS

Period	Statistics	Dallas 'Before' Study September 23, 1982		Dallas 'After' Study January 6, 1983	
		\bar{T} , min/mile (No. of Points)	\bar{T}_S , min/mile (No. of Points)	\bar{T} , min/mile (No. of Points)	\bar{T}_S , min/mile (No. of Points)
7:50- 8:26	Mean	4.72	1.72	5.94	2.00
	Std. Dev.	1.01 (7)	0.73 (7)	1.34 (6)	1.00 (6)
10:01-10:45	Mean	5.38	1.70	4.45	1.22
	Std. Dev.	1.29 (8)	0.81 (8)	1.17 (16)	0.76 (16)
12:02-12:50	Mean	6.08	2.07	4.67	1.39
	Std. Dev.	2.08 (8)	1.37 (8)	1.47 (9)	0.99 (9)
16:53-17:50	Mean	6.26	2.62	5.58	1.94
	Std. Dev.	1.45 (9)	1.10 (9)	2.00 (10)	1.07 (10)
21:34-23:12*	Mean	3.58	0.68	3.49	0.62
	Std. Dev.	0.84 (13)	0.44 (13)	0.62 (12)	0.47 (12)

* The signal timing scheme for this period was the same for both the before and after studies.

NOTE: The study is bounded by Thornton Freeway, Houston Street, Woodall Rodgers, and Hall Street.

p.m., for which there were no timing changes, the results are essentially the same before and after.

A one-sided two-sample t-test was carried out for the before/after study with the following results. The differences in the average trip times per unit distance, in the Dallas before/after study, for the first through the fifth periods were significant at the following levels: 0.96, 0.95, 0.94, 0.78, 0.61, respectively. Therefore, at the 90 percent level the worsening of the average trip time per unit distance, \bar{T} , in the first time period is found to be significant. The improvement in \bar{T} for the second and third time periods is also significant at the 90 percent level while for the fourth period there is no significant difference at this level. Finally, no significant difference is found for the fifth time period at the 90 percent level. The statistical test applied assumes that the standard deviations for the various quantities are the same before and after. Since this assumption is not necessarily valid we also performed a variation, namely, the Behrens-Fisher approximate t-test, in which the standard deviations are assumed to be different. This test generated significant levels of 0.95, 0.95, 0.93, 0.79 and 0.62 for the first through the fifth study periods, respectively. These results were very close to those of the previous test and did not change any of the results mentioned above.

We are inclined to place more reliance on the fact that the results appear to correspond well with other observations regarding perceived improvement or worsening of the traffic. The before and after data along with the Two-Fluid trend for the combined before/after data are shown in Fig 4.9. The parameters for this set of observations correspond to Dallas CBD2 for which $T_m = 2.12$ min/mile and $n = 1.36$. These parameters have also been obtained by dividing the data into before and after cases resulting in

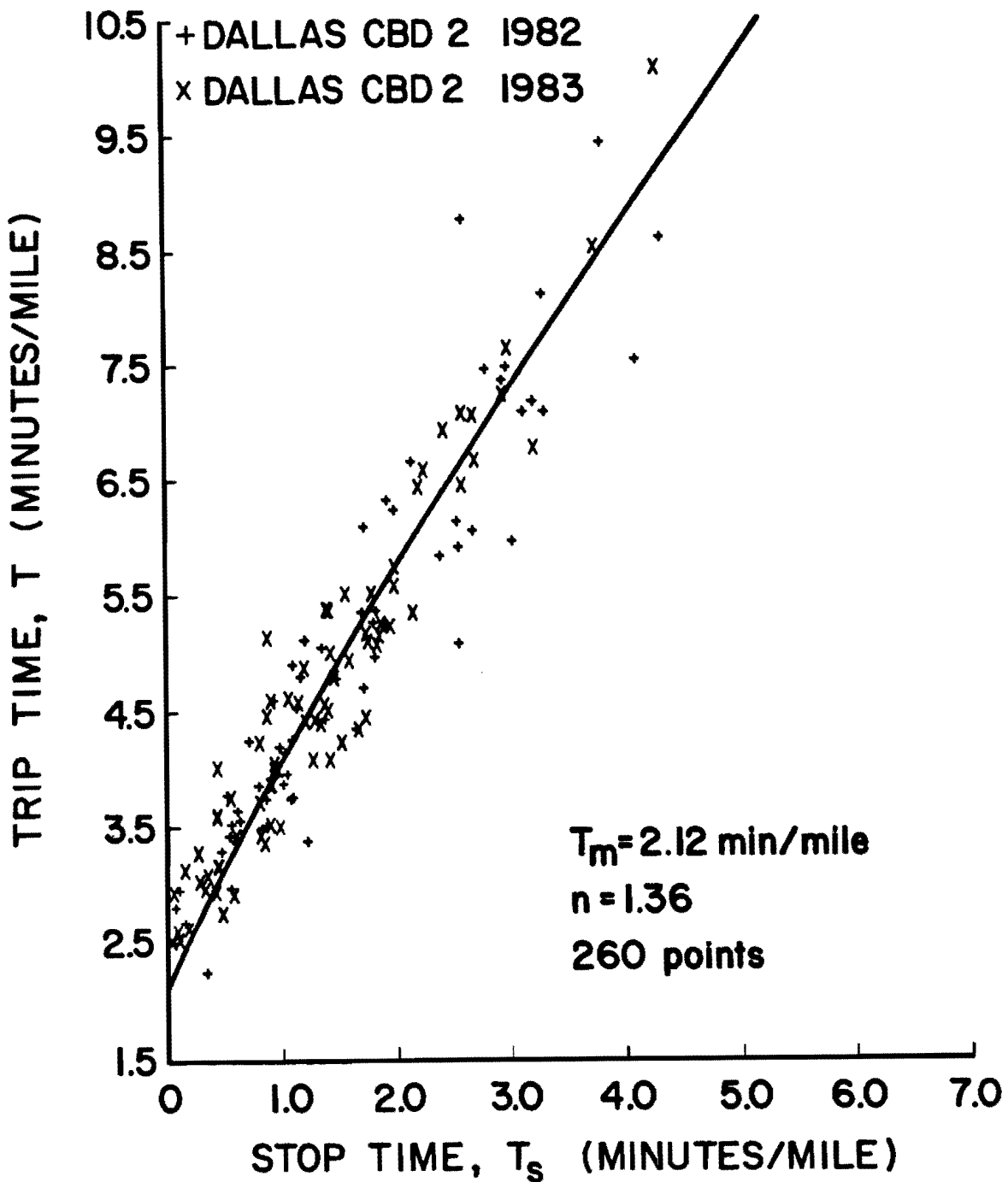


Figure 4.9. Trip time versus stop time for CBD data in Dallas before and after changes in signal timing plans. The curve is for the Two-Fluid model, $T_m = 2.12$ min/miles and $n = 1.36$. Note that the points associated with the after study, X, appear generally to have shifted with respect to the before study points, +, toward lower trip time values.

$(T_m, n) = (2.03, 1.43)$ and $(T_m, n) = (2.17, 1.34)$ for the before and after cases, respectively. These values are quite close to the average and to each other. This can also be shown through performing t-tests on the intercept A and slope B of the regression of $\log T_r$ against $\log T$, from which the value of the parameters T_m and n are calculated. In comparing the intercept of the before case to that of the after case in Dallas a value of -0.745 was computed for the t-statistic, indicating a two-sided confidence level of about 54 percent; i.e. the difference in A for the before and the after cases was not found to be significant. The corresponding values of t and the confidence level for the slope B were 0.420 and about 33 percent, respectively, again indicating no significant difference in B for the two cases. This implies that the Two-Fluid model representation for the Dallas CBD2 area is much the same before and after the timing changes. Undoubtedly the values of T_m and n depend on many other factors.

Although the changes in signal timing did not modify the model parameters, an improvement is apparent in terms of the distribution of the T , T_s data along the Two-Fluid curve. It appears from Fig 4.9 that the data for the after study lie further down the curve closer to the intercept T_m than the before data. This is in agreement with the conclusion of the t-tests discussed above that there is an improvement in the quality of the traffic service.

A study similar to the Dallas before/after study was performed in the Lubbock CBD area in which major network-wide changes in signal timing patterns were scheduled to take place. In September 1983 when the before study was performed there were two timing patterns, a peak and an off-peak plan. The off-peak plan was in operation during all periods except from 7:45 a.m. to 8:30 a.m. and from 5:00 p.m. to 6:45 p.m. when the peak timing plans

would be invoked providing that the traffic volumes were heavy enough. On the other hand, in late June 1984 when the after study took place five new timing patterns, three peak and two off peak plans, were implemented. The three peak plans were morning peak (7:00-8:30 a.m.), noon peak (11:00 a.m.-1:30 p.m.), and afternoon peak (4:30-6:15 p.m.). An off-peak plan was in operation from 8:30-11:00 a.m., 1:30-4:30 p.m. and 6:15-7:30 p.m.. After 7:30 p.m. all signals except those on Broadway Avenue were put on the flashing mode with drivers on major approaches facing the flashing amber.

The analysis of the before and after data resulted in $(T_m, n) = (1.93, 1.13)$ and $(T_m, n) = (2.14, 0.82)$, respectively, where T_m is in minutes per mile. The R^2 values were 0.78 and 0.70 for the before and the after cases, respectively. As shown by the above results, the Two-Fluid trend for the after case in Lubbock had an 11 percent increase in T_m and a 27 percent decrease in n . The significance of these differences can be statistically established through performing t-tests to compare the intercepts and slopes of the regression of $\log T_r$ against $\log T$ for the before and the after cases in Lubbock. A t-value of -2.903 corresponding to a level of confidence of 99.6 percent was obtained in comparing the intercept A of the before case to the after case, indicating that the difference in A for the two cases is highly significant. The corresponding t and confidence level values for the slope B were +2.682 and 99.3 percent, respectively. Therefore, the difference in B for the two cases was also found to be highly significant. A significant increase in A and reduction in B in the after case imply a worsening of the service during off-peak periods and an improvement of service during peak traffic periods. Figure 4.10 further illustrates this point by showing the one and two standard deviation confidence bands of the Two-Fluid trend for the before case along with the after study T, T_s data. As

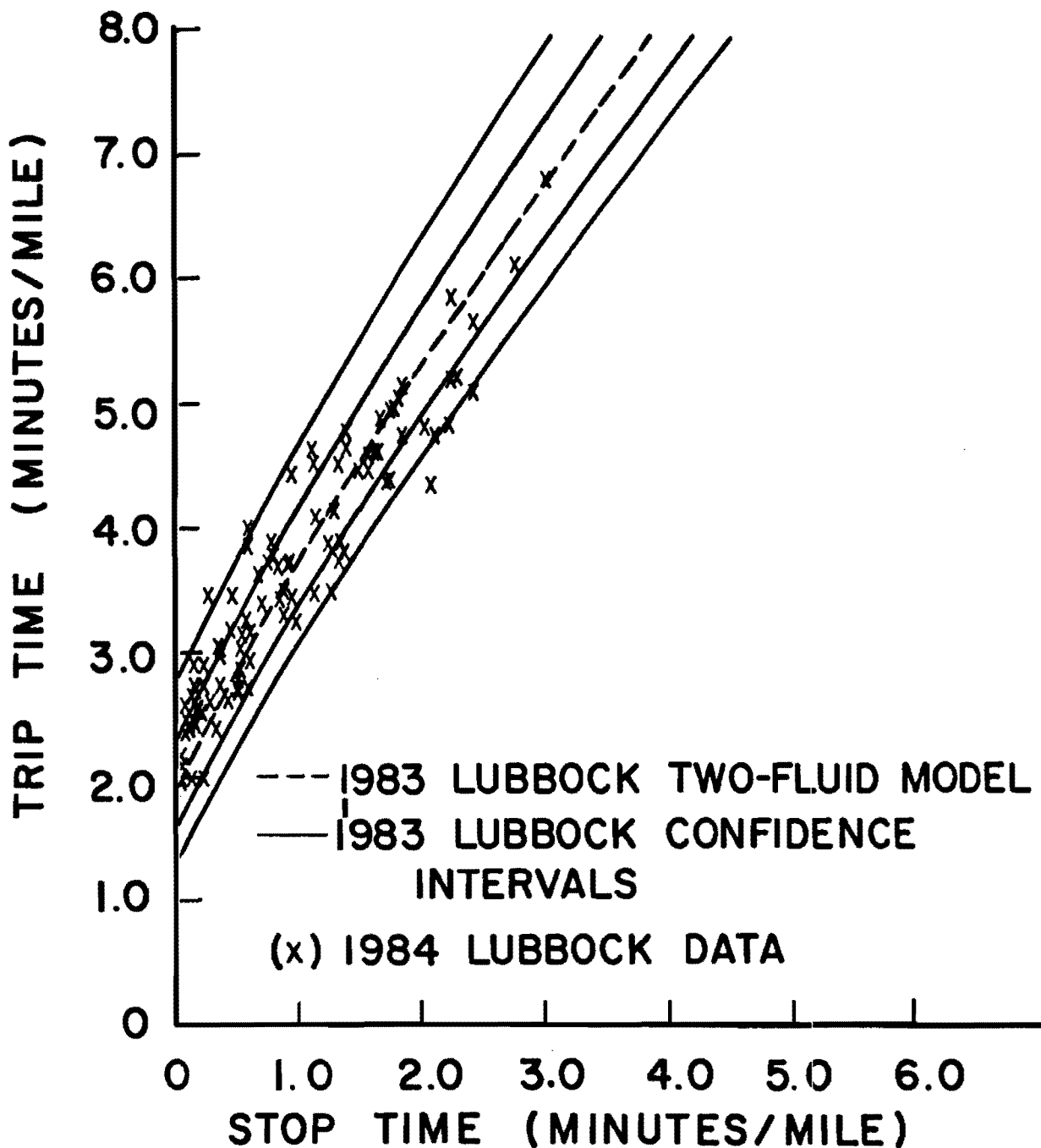


Figure 4.10. Trip time versus stop time for the Lubbock CBD after study along with the $\pm 1\sigma$ and $\pm 2\sigma$ confidence bands associated with the Two-Fluid curvilinear regression to the Lubbock CBD before study. The before data is not shown.

can be seen, in the low T, T_s regime ~ 20 percent of the after data falls outside $+1 \sigma$ limit of the before trend while in the high T, T_s regimes ~ 10 percent of the data fall outside -1σ limit out of which ~ 4 percent fall even outside -2σ limit of the before trend. Such differences are not readily apparent when the before and after data are combined, as shown in Fig 4.11 which also includes the Two-Fluid trends for both before and after case separately. However, a closer examination of the data points reveals that at regions of relatively high T, T_s , the after data points by and large lie below the before data points indicating a slight improvement for the after case. It must be noted that even improvements as moderate as these could be significant if they occur in the high trip time - stop time regime, as is the case here. This is so because high T, T_s regimes generally imply heavy vehicular concentrations. An elaborate discussion of the latter point is given in Chapter 6 on the aerial photographic results.

In the San Antonio CBD, data were collected in November 1981 and later in April 1984. Over the three-year period no major changes were made in geometry or control of the network. However, in 1984 there were 194 more transit buses (total of 471 buses) in the vehicle mix [Ref 38]. In addition, for tourism purposes a number of horse-drawn coaches and 20 rubber tired street cars were also in operation in the CBD area in 1984. In April 1984, there also was a noticeable reduction in the level of construction activities as compared to November 1981.

The Two-Fluid model fit to the old and the new sets of data from San Antonio resulted in $(T_m, n) = (1.89, 1.53)$ and $(T_m, n) = (1.99, 1.33)$, respectively, where T_m is in minutes per mile. The R^2 values were 0.84 and 0.81 for the 47 old and 55 new data points, respectively. Figure 4.12 is a plot of the old and new data and their respective Two-Fluid trends in San

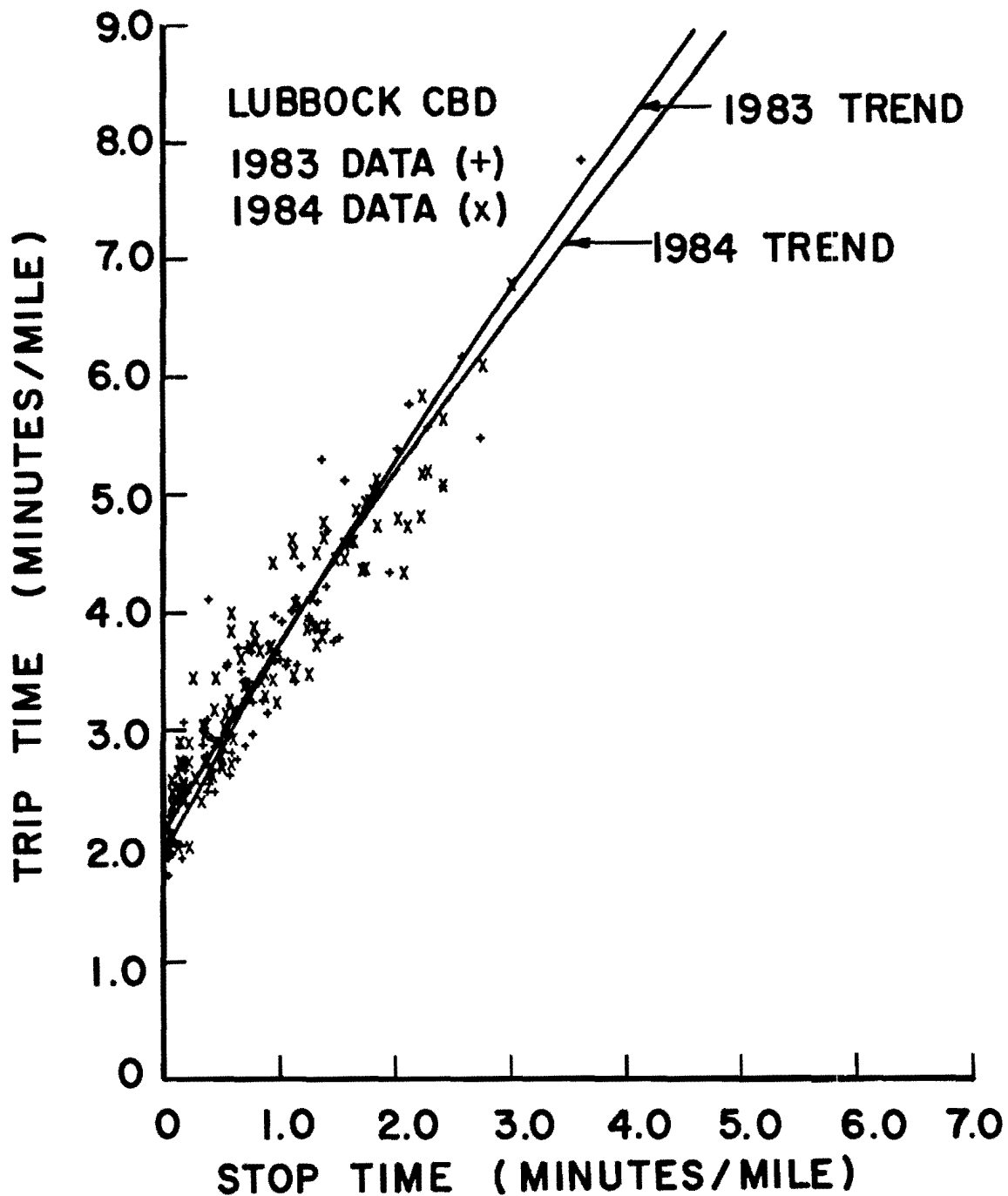


Figure 4.11. Trip time versus stop time for CBD data in Lubbock before and after changes in signal timing plans. Also shown are the Two-Fluid trends associated with each of the two data sets.

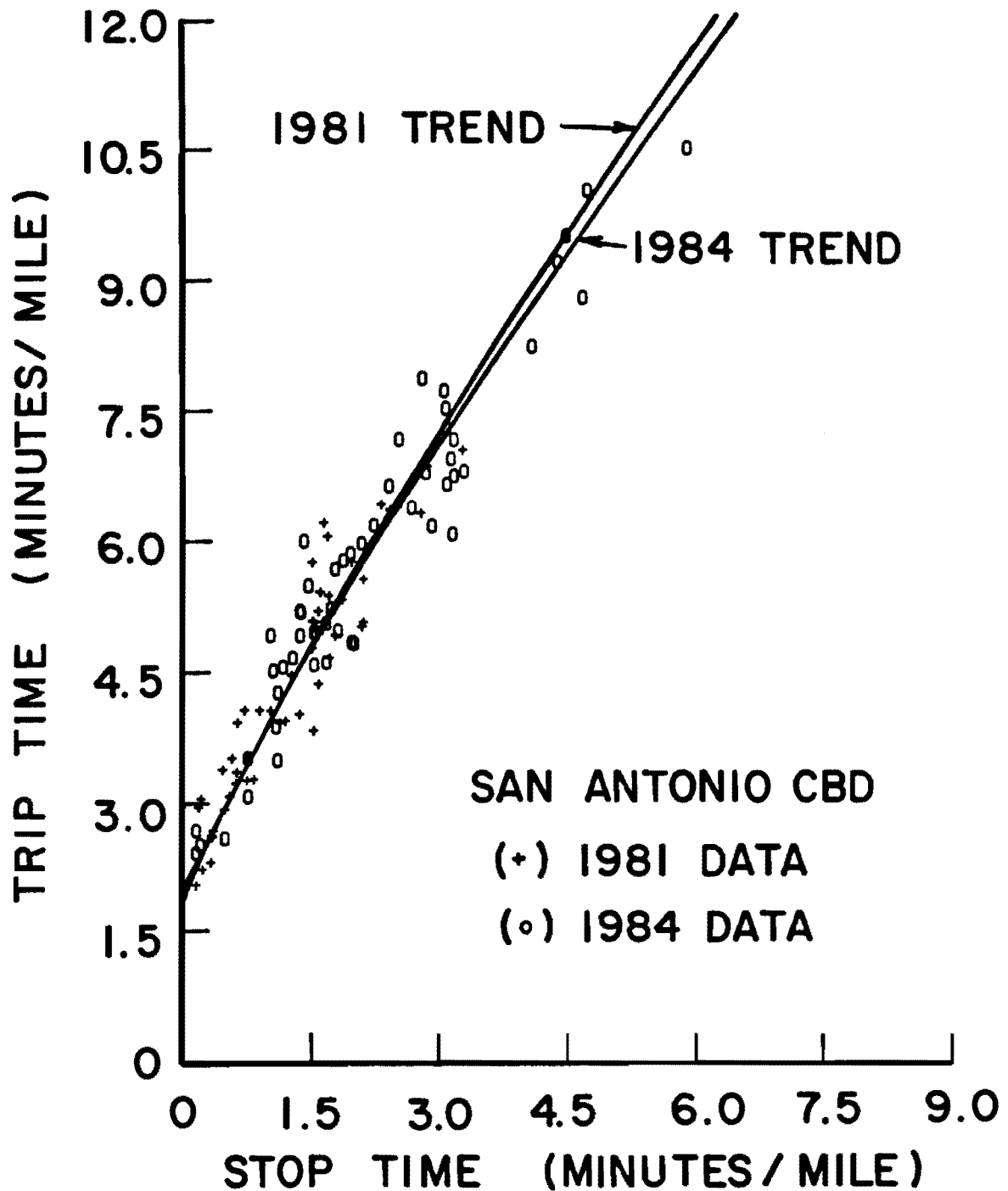


Figure 4.12. Trip time versus stop time for 1981 and 1984 data in San Antonio CBD. Also shown are the Two-Fluid trends associated with each of the two data sets.

Antonio. As can be seen from Fig 4.12, the new data is virtually along the old trend and the differences between the two trends are not significant. This is verified by a two-sided t-test showing confidence levels of about 52 percent and 63 percent for differences in intercept and A and slope B (Eq 2.24), respectively, indicating that these differences are not significant. The major difference between the two data sets, obtained over comparable time periods during the day, is that the ranges in T and T_s are, respectively, ~ 50 percent and ~ 84 percent greater for the new data set relative to the old one. The highest (T_s , T) observations in minutes per mile were (3.2, 7.0) and (5.9, 10.5) for the old and the new data, respectively. It is, however, important to note that these substantial increases in T_s and T for the new observations have been, as predicted, along the old Two-Fluid trend. These increases may be attributed, among other factors, to the growth of traffic in terms of vehicular concentration as well as to changes in the vehicular mix resulting in a greater number of vehicles with relatively slower performance characteristics. This is not to suggest that introducing transit buses in the traffic stream will be detrimental to the quality of traffic service. It would not be so if the buses were well utilized, in which case significantly more passengers are processed through the traffic network per unit time despite higher trip times and stop times. A separate study has been conducted in Austin to investigate the effect of vehicle performance characteristics on the Two-Fluid model parameters. The results are discussed later in this chapter.

The before/after studies in Dallas, Lubbock, and San Antonio have shown that the trip time - stop time and the Two-Fluid model representations can be used as means of assessing the impact of major changes in urban traffic networks. These studies have also provided information regarding some of the

vehicular, geometric and control features in a street network which could affect the Two-Fluid model parameters. Such before/after studies may, for example, be conducted in a network such as Vasastaden street network in Stockholm, Sweden where minor streets have been blocked off to the traffic to provide a smoother flow in the major streets [Ref 38].

COMPARISON OF BUS AND CAR-BASED TWO-FLUID MODELS

To examine the dependency of the Two-Fluid model parameters on the vehicular type used in collection of trip time - stop time data, an experiment involving passenger cars and buses was carried out. Trip time - stop time data were collected aboard buses and passenger cars on same bus routes. The Two-Fluid model was then calibrated for the routes studied once using the car data and a second time using the bus data. The results are reported in this section.

The University of Texas shuttle buses operate on fixed routes and schedules. Trip time and stop time data were collected on three of the routes which run through non-freeway street networks including low speed-limit campus streets as well as residential and other non-arterial streets. The round-trip lengths of the routes were 2.64, 4.66 and 2.77 miles. The shuttle buses were International Harvester chassis and motor with a Wayne coach; they were 33.5 feet long with a 44-seat capacity.

The bus data were collected by an observer equipped with three stop watches. The total trip time, traffic stop time and the unloading/loading time were accumulated and recorded for each round trip. It was found that for the particular bus routes under study the chances were small of an overlap occurring between the traffic stop time and the loading/unloading time since all bus stops were located at mid-blocks and not at intersections. Therefore, the overlap time was neglected in the analysis. Trip time and

stop time data were also collected on the same three fixed bus routes in the usual fashion using a passenger car whose driver was instructed to drive with the traffic.

The bus and passenger car data were obtained during 1979 and 1980. An analysis of the data is shown in Table 4.4 where the parameters of the linear representation and the Two-Fluid model are displayed. The linear regression of T against T_s for the bus data has a relatively low R^2 of 0.60 because the routes consist of different types of streets. One route consists of mainly residential streets, the second is on arterials and the third mostly on campus streets with a high level of pedestrian interaction, and as a consequence there appear to be some systematic differences in the data for the three routes. In the Two-Fluid model the value of n is considerably greater for the buses and T_m is smaller possibly as a result of the steep slope.

Figure 4.13 shows the $\pm 1 \sigma$ and $\pm 2 \sigma$ confidence bands and the Two-Fluid trend for the car data. Also shown in Fig 4.13 are the bus data and their corresponding Two-Fluid trend. Note that most of the bus data lie outside $\pm 2 \sigma$ limit of the car trend indicating statistically significant differences between the trends of the two data sets.

The significant differences between the Two-Fluid trends of the bus and car data may be addressed in the following fashion. Buses are vehicles with low performance, i.e. low acceleration and deceleration characteristics and also low maneuverability. Although the loading/unloading time is subtracted from the trip time of the shuttle bus, the effect of these loading/unloading stops on the remaining trip time (the automobile equivalent trip time) is not completely removed. This is so because every stop made for the purpose of passenger loading and unloading introduces an additional deceleration-

TABLE 4.4. RESULTS FOR BUSES AND AUTOMOBILES ON THE SHUTTLE BUS ROUTES OF THE UNIVERSITY OF TEXAS AT AUSTIN

Vehicular Mode	No. of Points	Linear Representation			Two-Fluid Parameters		
		Intercept (min/mile)	Slope	R^2	T_m (min/mile)	n	R^2
Buses	179	3.21	1.81	0.60	1.47	4.88	0.87
Cars	96	2.77	1.66	0.75	2.20	1.92	0.76

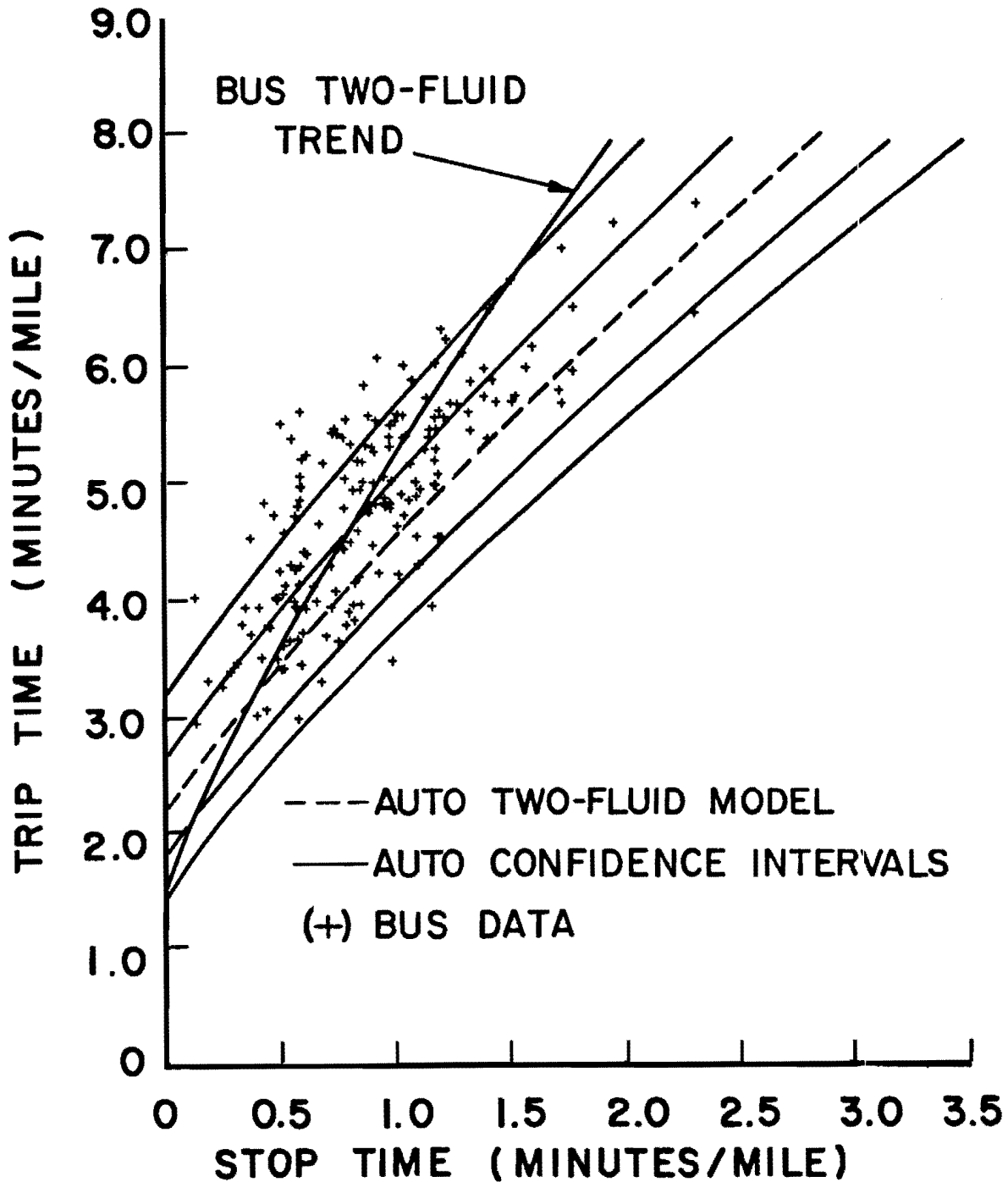


Figure 4.13. Trip time versus stop time University of Texas shuttle bus data along with the $\pm 1\sigma$ and $\pm 2\sigma$ confidence bands associated with the Two-Fluid curvilinear regression to the passenger automobile data taken on the bus routes studied. Also shown is the Two-Fluid trend for the bus data. The automobile data is not shown.

acceleration cycle to the trip history of the bus. Thus, the automobile equivalent trip time per unit distance for a bus is considerably longer than that for an automobile driven on the same route and subject to identical traffic conditions. In other words the running time per unit distance for the bus is greater; however, the traffic stop time per unit distance for the bus and automobile will be essentially equivalent. Consequently, the trip time - stop time linear representation and the Two-Fluid model fit for a bus will be steeper than that for an automobile driven on the same route. The value $n = 4.88$ for the bus compared to $n = 1.92$ for the car further illustrates the above point.

In conclusion, this portion of the study, i.e. the comparison of a bus with a passenger car on the same routes, shows that the parameters obtained for a given network differ in value depending on the vehicular mode used in the collection of the data from which they are determined. It is also important to take into account the manner in which the data is taken. The data from the bus corresponds to the special properties of that vehicle as a member of the population, whereas the data from the test car is taken from a vehicle driven in a manner to represent an average member of most of the population.

DISCUSSION

In Chapter 2 on the Two-Fluid theory a term $\Delta T_r = T_r - T_m$, the incremental running time per unit distance was introduced. It was argued that as the stop time per unit distance increases from zero to T_s , the trip time per unit distance departs from its minimum value T_m by an amount $T_s + \Delta T_r$, i.e. $T = T_m + T_s + \Delta T_r$, where the magnitude of T_m and ΔT_r varies from network to network and as such is a measure of the quality of traffic

service in the street system. The Two-Fluid model simply predicts, for a given network, what ΔT_R is for a given value of T_S .

The Two-Fluid model is a logarithmic linear relation between the trip time $T = T_S + T_m + \Delta T_R$ and the running time $T_R = T_m + \Delta T_R$. The strength of the correlation between $\log T_R$ and $\log T$, therefore, depends on how well T_S and ΔT_R are correlated. In other words, for a network with high correlation between T_S and ΔT_R , $\log T_R$ and $\log T$ are also highly correlated. On the other hand, if in a network there is no correlation between ΔT_R and T_S , i.e. if ΔT_R remains constant with changes in T_S , then $\log T_R$ will be constant and totally uncorrelated with $\log T$. Consequently, the slope B of Eq (2.25) will be zero resulting in a value of zero for n . In such a network, where $n = 0$, ΔT_R and therefore the running speed of vehicles are both independent of T_S . But T_S is a function of vehicular concentration and increases for an increase in concentration due to longer queue lengths at intersections, etc. Then it can be said that in a network for which $n = 0$, the running speed of vehicles is constant and not a function of the level of concentration.

The above arguments begin to indicate that the correlation between $\log T_R$ and $\log T$ of the Two-Fluid model is stronger, the greater the value of n . Table 4.2 verifies this conclusion since for the twenty one networks listed in this table it can be seen that the R^2 to the Two-Fluid model fit is in general higher for the networks with higher values of n .

A greater interaction among vehicles is more likely in networks with features that restrict the maneuverability and relaxation opportunities of individual vehicles; features such as narrow streets, parking on both sides, short block lengths, a mix of vehicles with highly varying size and characteristics, frequent occurrence of turning movements, etc. On the other hand, calibration of the Two-Fluid model for the networks listed in Table 4.2

has shown, in general, that higher values of T_m and especially n are obtained in those networks which possess some of the physical features described above. Therefore, a higher value of n implies a greater level of interaction among vehicles; and since a desirable network is one in which the interaction among vehicles is minimal, it is then desirable to have a network with relatively small values of T_m and especially n .

The trip time - stop time data used in determining the Two-Fluid parameters of cities of Austin, Dallas, Houston, Lubbock, San Antonio, Matamoros, Mexico City, and Albuquerque also included information on the number of stops per unit distance, N_s , for each micro-trip. An examination of the data revealed that T , T_s and T_r all increased with increasing N_s and that the general rate of increase as determined through weighted linear regressions of T , T_s and T_r against N_s varied significantly for cities with different T_m and n values. The linear regressions were weighted according to the value of N_s since the variance in T , T_s and T_r appeared to be greater for greater N_s values as shown, for example, in Figure 4.14 for the Houston CBD.

The results of these weighted regressions are summarized in Table 4.5. Also given in this table are τ_s , the average stop duration in minutes per stop and the parameter T_m and n for each of the eight cities. The regression of T_s against N_s was forced through the origin to satisfy the boundary condition that when there are no stops ($N_s = 0$) the stop time per unit distance is zero ($T_s = 0$).

Let the resulting three weighted linear regressions be in the forms:

$$T = a + b N_s \quad (4.5a)$$

$$T_r = \alpha + \beta N_s \quad (4.5b)$$

and $T_s = \gamma N_s$, (4.5c)

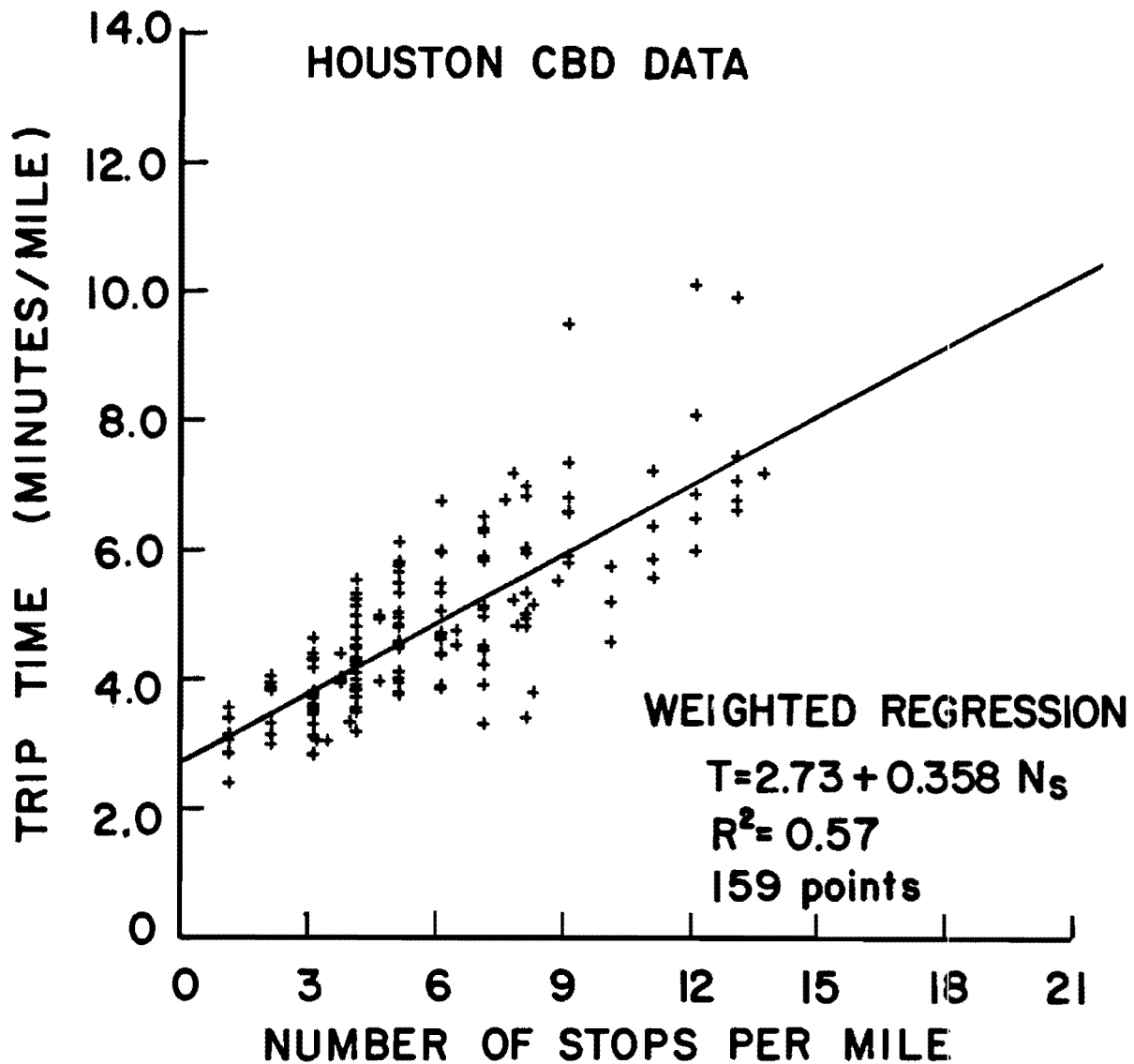


Figure 4.14. Trip time per mile versus number of stops per mile in the Houston CBD. The line is the weighted regression fit to the data. Note that for a given value of trip time, T , there is a considerable spread in the number of stops, N_s . Each data point has been weighted according to its value of N_s .

TABLE 4.5. WEIGHTED LINEAR REGRESSION PARAMETERS FOR REGRESSIONS OF TRIP TIME, RUNNING TIME AND STOP TIME PER MILE AGAINST THE NUMBER OF STOPS PER MILE FOR EIGHT CITIES AROUND THE WORLD. ALSO GIVEN ARE THE AVERAGE DURATION OF STOP, τ_s , AND VALUES OF PARAMETERS T_m AND n FOR EACH CITY.

City CRD	No. of Points	$T = a + bN_s$			$T_r = \alpha + \beta N_s$			$T_s = \gamma N_s$		τ_s (min/ stop)	T_m (min/ mile)	n
		a (min/ mile)	b (min/ stop)	R^2	α (min/ mile)	β (min/ stop)	R^2	γ (min/ stop)	R^2			
Austin	545	2.32	0.426	0.76	2.48	0.137	0.56	0.262	0.87	0.275	1.78	1.65
Dallas	259	2.41	0.447	0.79	2.32	0.188	0.65	0.272	0.90	0.271	1.97	1.48
Houston	159	2.73	0.358	0.57	2.61	0.147	0.66	0.229	0.79	0.228	2.70	0.80
Lubbock	190	2.49	0.279	0.52	2.15	0.133	0.69	0.207	0.72	0.238	2.03	0.97
San Antonio	102	2.26	0.542	0.73	2.32	0.208	0.65	0.325	0.88	0.326	1.96	1.38
Mata- moros	79	2.96	0.431	0.67	3.77	0.167	0.39	0.194	0.88	0.171	3.98	2.10
Mexico City	101	2.04	0.765	0.94	2.86	0.187	0.78	0.486	0.92	0.460	1.72	1.63
Albu- querque	68	2.53	0.336	0.73	2.31	0.162	0.55	0.213	0.87	0.218	1.93	0.62

where the parameters a and α are in minutes per mile and b , β and γ are in minutes per stop. The values of these parameter for each of the nine cities are given in Table 4.5. Note in Table 4.5 that the parameters obtained through regressions to the data for each city appear to be related to one another as follows,

$$a \approx \alpha \quad (4.6a)$$

and $b \approx \beta + \gamma$. (4.6b)

with a deviating from α by an average of 11.6 percent with a standard deviation of 9.5 percent and b deviating from $\beta + \gamma$ by an average of 9.1 percent with an associated standard deviation of 6.8 percent. Equations (4.6a) and (4.6b) need to be met approximately since $T \equiv T_s + T_r$. Furthermore, the parameter γ is an estimate of the average duration of a stop and therefore is approximately equal to τ_s .

In connection with the determination of the quality of traffic service, it must be noted that the parameters β and γ signify the average penalties in minutes per mile which are imposed on the running time and stop time for each additional stop. Likewise, the parameter $b \approx \beta + \gamma$ is the penalty to the trip time in minutes per mile for each additional stop made. On the other hand, $a \approx \alpha$ is an estimate of the minimum trip time per unit distance since $T = a$ when $N_s = 0$.

Also note that the parameters a and b in Table 4.5 vary widely in value for different cities with a ranging from 2.04 to 3.20 minutes/mile and b ranging from 0.279 to 0.765 minutes/stop. As may be expected, the value of the parameter a in Table 4.5 is greater for cities with greater T_m values and likewise the parameter b is greater for cities with greater values of n . Therefore, the number of stops per unit distance appears also to be an

important macroscopic variable in the determination of the quality of traffic service in urban street networks.

CHAPTER 5. AERIAL PHOTOGRAPHIC OBSERVATIONS

The reduction and analysis of aerial photography, although cumbersome and time consuming, is a means of recording a very large amount of traffic data in a short period of time. Time-lapse aerial photography has been used over the years in various traffic studies by many investigators.

In freeway studies aerial photography has been employed, for example, to determine the effect of bottlenecks on freeway traffic [Refs 41, 42 and 43], to make estimates of travel time delay and accident experience due to freeway congestion [Ref 44], to study merging freeway operations [Ref 45] and freeway interchange operations [Ref 46], to determine the traffic flow characteristics of a facility [Refs 47, 48, 49 and 50], and to study headway and speed distributions and their correlation in freeway traffic [Ref 51].

In the arena of non-freeway traffic, aerial photographs have been used, for example, to measure the vehicular concentration in a network of streets [Ref 52], to conduct origin-destination surveys [Ref 53], to perform parking studies [Ref 54], and to measure the effectiveness of traffic control systems in a network [Ref 55].

In the present work time-lapse aerial photographs have been analyzed to investigate the accuracy of simultaneous ground observations in estimating network-wide variables related to the quality of traffic service. These variables include the average fraction of vehicles stopped, the space mean speed, and the mean vehicular concentration. The work reported here does not intend to promote aerial photographic traffic surveys as a routine means of collecting traffic data or determining the quality of traffic service; rather the objective is to develop simple means of estimating values of pertinent

network-wide traffic variables on the ground as an alternative to aerial photographic surveys.

SPECIFICATIONS

The aerial photographs were taken by the Texas State Department of Highways and Public Transportation over the Austin CBD on November 12, 1982. The area of study was bounded by Martin Luther King Boulevard on the north, first Street on the south, Red River Street on the east and Lamar Boulevard on the west.

The photography was done in four periods: 8:00 a.m., 12:00 noon, 3:30 p.m., and 5:00 p.m. A Cessna 206 turbo-engine aircraft, a 153.28 mm RC10 Wild Lens cone camera, and a 9" by 9" diapositive color film were used. The aircraft was flying at 120 mph at an altitude of 3000 feet above street level. The photographs were taken on the average 2.66 seconds apart (with elapsed time standard deviation of 0.40 seconds). The above conditions resulted in 90 percent longitudinal overlap on successive frames. To cover the study area, the photographs were taken along two parallel strips with 5 percent lateral overlap.

REDUCTION

The first step in the reduction of the aerial photographs was to assign a scale to each frame. It must be noted that aerial photographs are perspective projections so that a location with higher elevation is closer to the camera lens and thus its image has a greater scale [Ref 56]. However, for relatively flat topography one may assume a constant average scale for each photographic frame. As will be discussed in the section on sources of errors, this is not an unreasonable assumption for the study area in Austin.

The scale determination for each frame was made through measuring photo distances between fixed monuments on the ground (control points) as well as the ground distances between those points. The control points were selected after the photographs were taken. The ground measurements were made using a Keuffel & Esser Electronic Distance Measurement apparatus. On the average for each frame four pairs of control points were used for scale determination resulting in four independent scale measurements, the average of which was used as the scale for that frame. For the 71 frames reduced, the average scale ranged from 1" = 471.2' to 1" = 537.3' reflecting the various sources of errors discussed later in this chapter.

A Keuffel and Esser Mono-Digital comparator (Fig 5.1) was used to determine the Cartesian coordinates of the location of each vehicle. The position of each vehicle was represented by the x,y coordinates of its front left corner which were recorded on a computer card. The Cartesian coordinate system for each frame was arbitrarily defined by setting the coordinates of the lower left corner of the photograph at $x = 10^5$, $y = 10^5$ microns and selecting an ordinate approximately parallel to the left edge of the photograph.

In addition to the coordinates of the position of each vehicle on the photo, each computer card also contained the frame serial number and a three-digit vehicle identification number. The three digits in the identification code represented, from left to right, the size, color, and direction of travel, respectively. Travel directions were coded from one to eight, with one indicating a northerly travel direction, two representing a north-easterly direction, three indicating an easterly travel direction etc. The codes are shown in Table 5.1, according to which a subcompact silver car

Mono Digital Comparator

Resolution of one micrometer.

Repeatability of \pm one micrometer.

Fully air bearing supported and guided X-Y stages for rapid and effortless positioning of stages.

8 operator-selectable reticles with variable illumination control.

Projected reticle system mounted independent of viewing system for increased accuracy and stability.

Binocular, stereo microscope viewing system with 8.8X to 52.5X zoom enlargement range.

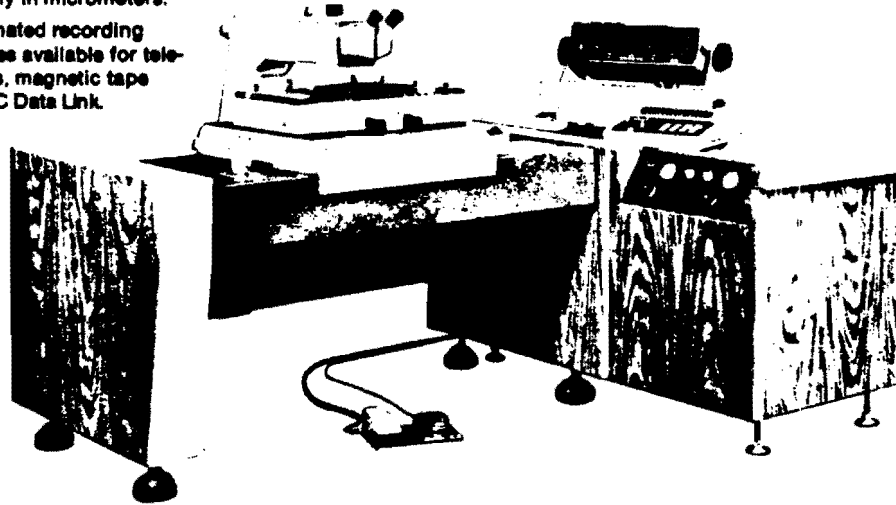
Accepts film or glass plates up to 9" X 18" (229mm X 458mm).

MetriGraphic Terminal with 2 axes and 6-digit display per axis for semi-and automatic output recording of points and associated identifiers.

Display reads directly in micrometers.

Programmable, formatted recording output with interfaces available for teletype, punched cards, magnetic tape cassette or RS-232-C Data Link.

The Mono Digital Comparator is primarily a first order photogrammetric measuring instrument for analytical aerotriangulation, which may be effectively used to back-up first order plotters, freeing them for large scale mapping assignments. However, it is also ideally suited for diverse applications in the fields of astronomy, nuclear physics, medical analysis, microcircuit design. In fact, the Comparator is an effective and economical choice whenever precision measurement of X-Y coordinates, with high productivity, repeatability and computer compatible micrometer recording of point image data is required.



A Subsidiary of Keuffel & Esser Company

Figure 5.1. Keuffel and Esser Mon-Digital comparator used in reduction of aerial photographs to determine the cartesian coordinates of the location of each vehicle.

TABLE 5.1. INTERPRETATION OF THE 3-DIGIT CODE NUMBER USED IN IDENTIFYING EACH VEHICLE ON AN AERIAL PHOTOGRAPH

Left Digit		Center Digit		Right Digit	
Code	Size	Code	Color*	Code	Travel Direction
1	Pick ups, Jeeps, Station Wagons	1	Red**	1	Northerly
2	Motorcycles and Small Cars (e.g. Honda, Chevette, Toyota)	2	Yellow	2	Northeasterly
3	Medium Cars (e.g. Mustang, Camaro, Fairmont)	3	Blue	3	Easterly
4	Buses	4	Green	4	Southeasterly
5	Large Cars (e.g. Cadillac, Continental)	5	Orange**	5	Southerly
6	Vans	6	Black	6	Southwesterly
7	Delivery Vans	7	White**	7	Westerly
8	Single-frame Trucks	8	Silver or Grey**	8	Northwesterly
9	Trailer Trucks	9	Brown		

* Color of the hood except for vehicles with very small hood area, in which case the color of the roof is considered

** Colors used in vehicle sampling

NOTE: The three digits represent, from left to right, the size, color, and direction of travel, respectively.

travelling eastward, for example, is coded as 283. This three-digit number was then followed by the frame serial number on which the vehicle appeared.

Using the above coding convention, a Fortran program was written to trace a vehicle from one frame to the next (see Appendix B). For a pair of time-lapse photographs consisting of a master frame (the first photograph taken) and a conjugate frame, the program searches the conjugate frame for a match to each vehicle appearing on the master frame. The search on the conjugate frame is confined to a circular area centered at the position of the vehicle on the master frame whose location on the conjugate frame is determined. The radius of this search area is taken arbitrarily to be the photo distance in microns travelled by the vehicle at 60 mph, a speed rarely exceeded by the downtown traffic.

Once the search is narrowed to all the vehicles on the conjugate frame that fall within this limited circular area, the program then selects the match vehicle with an identical code as the key vehicle. If more than one potential match vehicle exists, all the alternatives and their coordinates are printed. If no such matches exist, the codes and x,y positions of all the vehicles contained in the circular area of search are printed.

When a direct match is not possible, either because there are more than one possible matches or because there are no direct matches, the data analyst would then try to make the match manually. First, all the vehicles in the list for which the shifts in x and y from the master to the conjugate frame are not consistent with the coded travel direction are eliminated. In doing so, a turning movement is considered as a possibility. Then the size and color codes of the vehicles remaining as potential matches are examined for operator inconsistencies in coding. For example, due to changes in orientations of shadows of buildings and other objects as a result of the

change in position of the aircraft a white vehicle on the master frame could be coded as silver or gray on the conjugate frame and vice versa; or a medium size car near the center of a frame due to relief displacement effects discussed later in this chapter could be coded as a large car once it has moved farther away from the center and vice versa.

A manual matching process such as the above could result in more than one possible match for a key vehicle. In such cases, for vehicles coded as travelling east or west the match vehicle with minimum change in its y coordinate is chosen. Likewise, for vehicles travelling northward or southward the match vehicle with minimum change in its x coordinate is chosen. For vehicles travelling in any of the other four directions no match is made.

SAMPLING BY COLORS

In the reduction of the first 25 percent of the frames all the vehicles were considered and their locations were determined. In the reduction of the remaining 75 percent of the frames a sample of only white, silver, gray, red and orange vehicles were considered, since vehicles with these five colors were more easily recognizable and together constituted 45 percent of the population of vehicles. As shown in Table 5.2 the average speed determined using such a sample deviated from the population average speed by ~ 0.1 percent to ~ 6.6 percent. In general it appeared that such a sampling strategy did not result in either a significant deviation of the sample mean from that of the population or a bias in these deviations.

TABLE 5.2. A COMPARISON OF THE POPULATION AVERAGE SPEEDS OF VEHICLES WITH THE CORRESPONDING SAMPLE AVERAGE SPEEDS USING ONLY RED, ORANGE, WHITE, SILVER AND GREY VEHICLES

Photo Pair	Population		Sample		% Difference in Means
	Average (mph)	No. of Vehicles	Average (mph)	No. of Vehicles	
7282 - 83	12.04	293	12.68	132	- 5.3%
7283 - 84	14.23	284	13.92	136	+ 2.2%
7288 - 89	14.59	243	14.00	108	+ 4.0%
7289 - 90	15.31	332	16.28	166	- 6.3%
7323 - 24	15.83	210	15.09	94	+ 4.7%
7324 - 25	16.30	283	15.23	125	+ 6.6%
6583 - 84	13.01	527	12.98	296	+ 0.2%
6584 - 85	12.24	578	12.25	336	- 0.1%
6654 - 55	13.22	263	12.93	170	+ 2.2%
6655 - 56	14.40	224	14.72	135	- 2.2%

SPEED MEASUREMENTS

To determine vehicle speed, the photo displacement of each vehicle, $\Delta\ell$, was measured as $\Delta\ell = \left[(x_1 - x_2)^2 + (y_1 - y_2)^2 \right]^{1/2}$ where (x_1, y_1) and (x_2, y_2) are the coordinates of the vehicle on the master and the conjugate frames, respectively. The above computation assumes that vehicles travel on a straight line during the elapsed time between two consecutive exposures.

Given the short elapsed time, Δt , between successive frames, the above assumption does not introduce a significant error in the measurements. For example, for a Δt of ~ 2.5 seconds and a scale of $1" = 500'$, a vehicle moving at 30 mph would travel only ~ 110 feet corresponding to a displacement of ~ 5588 microns on the photographs. Let us now assume that the above vehicle has been actually travelling at 30 mph along the zigzag path ABCDE, rather than the straight path ACE, as shown in Figure 5.2 which schematically depicts a hypothetical case of an extreme lane-changing maneuver. Then the actual distance travelled in 2.5 seconds is $AB + BC + CD + DE = 110'$ while the photographic estimate of the travelled distance is $ACE = 108.72'$, a discrepancy of only 1.3 percent.

Once a $\Delta\ell$ is determined in microns using the above procedure, it is converted to the distance travelled on the ground, ΔL , through the relation $\Delta L = \Delta\ell \times (\text{scale in feet per microns})$. However, in measuring $\Delta\ell$, it is, of course, necessary that the coordinates of a vehicle on the master and the conjugate frames be measured with reference to a common coordinate system. This was achieved through the transferring of the Cartesian coordinate system of the conjugate frame to that of the master frame, using the fixed control points that appeared on both frames. The transfer of coordinates allowed for not only a shift in x and y but also a rotation of the conjugate coordinate system relative to the master coordinate system, as shown schematically in

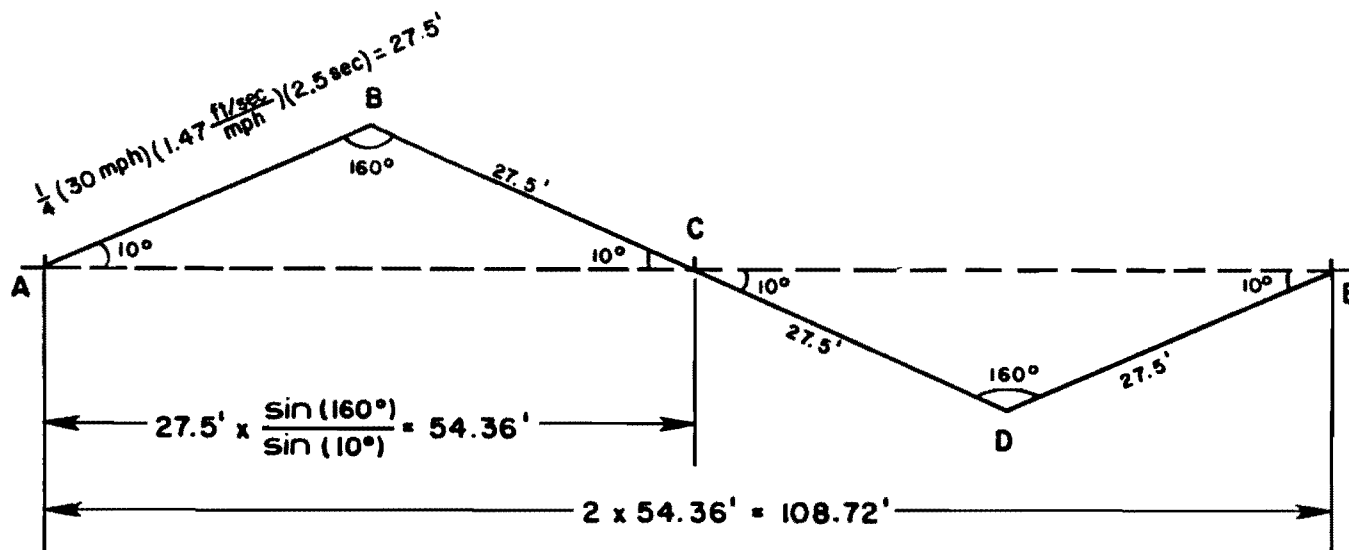


Figure 5.2. The schematic path of a vehicle conducting an extreme lane-changing maneuver during the 2.5 second elapse time between two successive photographs. The hypothetical vehicle has travelled the zigzag path ABCDE (a distance of 110 feet at running speed of 30 mph) while the photo reduction procedure has measured the length ACE = 108.72 feet.

Figure 5.3.

Once the three transformation parameters x_0 , y_0 and θ were determined the vehicle coordinates on the conjugate frame, (x', y') , were expressed relative to the master frame coordinate system, namely,

$$x = x' \cos \theta - y' \sin \theta + x_0 \quad (5.1a)$$

$$y = x' \sin \theta + y' \cos \theta + y_0 \quad (5.1b)$$

SOURCES OF ERRORS

The vehicle speeds obtained in the manner described in the previous section are subject to errors from a number of different sources [Ref 57]. The more significant of these sources are the non-level topography of the test area, parallax, relief displacement, tip and tilt of the airplane, and the operator.

The non-flat topography of the area is a major source of error. Unlike a map which is an orthographic projection and has a uniform scale, an aerial photograph is a perspective view and its scale varies from point to point due to variation in terrain elevation in addition to parallax, etc. For example, the areas on the photograph with higher ground elevations are closer to the lens of the camera and thus have a larger scale, since $\text{scale} = (\text{focal length of the camera}) / (\text{aircraft altitude} - \text{ground elevation})$. As a result of variation in terrain elevation, using a constant average scale for an aerial photograph is bound to produce errors so that a vehicle travelling on top of a hill at speed v appears to have moved a longer distance during the time t than a vehicle travelling at the same speed v at the bottom of that hill, i.e. $v(\text{bottom}) < v < v(\text{top})$ (Fig 5.4).

Parallax is another systematic source of error in photographic reduction. Parallax is defined as the apparent displacement of the position

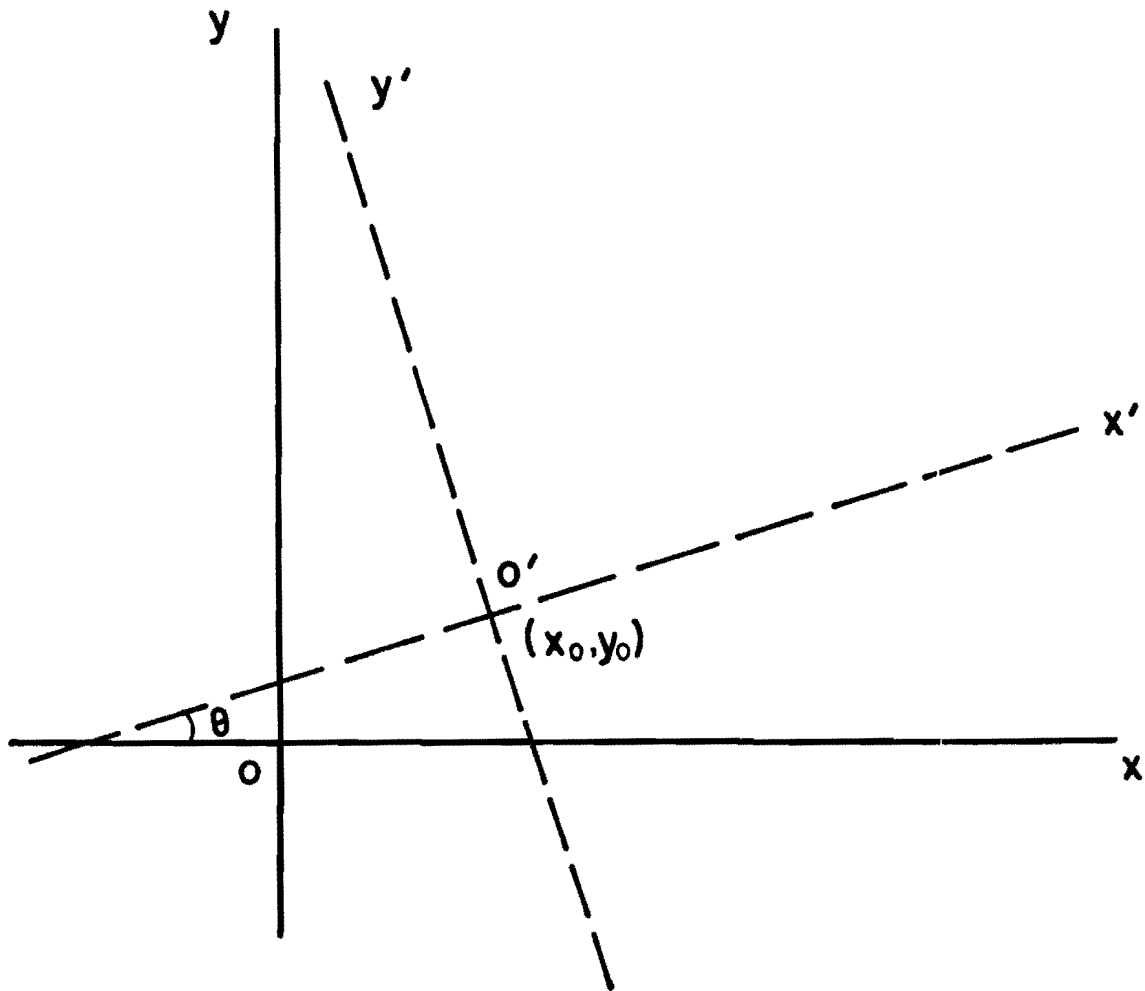


Figure 5.3. Schematic diagram showing the relative positions of cartesian coordinate systems of a pair of successive aerial photographs. In transferring the coordinates of one frame to the other a shift in origin of (x_0, y_0) as well as a rotation θ of the conjugate coordinate system relative to the master coordinate system was assumed.

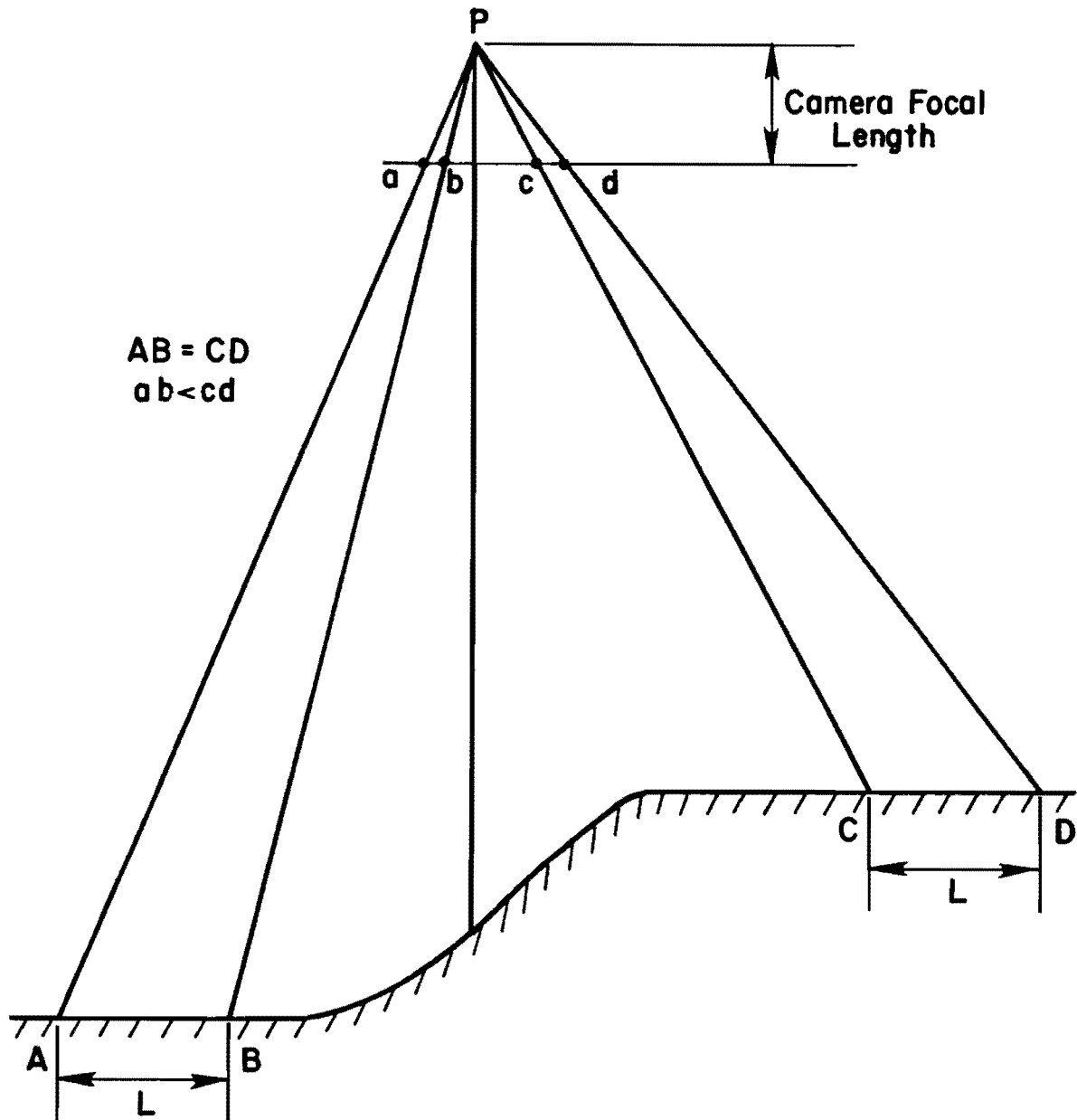


Figure 5.4. Schematic diagram showing the effect of variations in terrain elevation on the scale of an aerial photograph. Note that while ground distances AB and CD are equal, due to variations in terrain elevation their images on the photographic plate abcd are not of equal length, hence $(ab/AB) \neq (cd/CD)$. By the same token, a vehicle travelling the length $AB = L$ during Δt would appear to have moved a shorter distance on a pair of time-lapse photographs than a vehicle travelling the same length $CD = L$.

of an object with respect to a frame of reference due to a shift in the point of observation [Ref 56]. Using the aerial photographic plane as a reference frame, parallax exists for all images appearing on successive photographs and is larger the greater the elevation of the point. This apparent movement between successive exposures takes place parallel to the direction of flight.

The parallax phenomenon affects the transformation of coordinate systems since the fixed control points used in the transformations are assumed to have the same elevation. Having this in mind, the control points used in these transformations were chosen to not vary substantially in elevation and at the same time be widely scattered in the network area so that at least three control points appear on each photograph. A total of 28 fixed control points were used for transformation purposes. The relief displacement, while not as major an error source as the parallax, does generate problems such as the masking due to highrise buildings. The relief displacement is defined as the shift in position of an image caused by the relief or the height of the object [Ref 56]. In vertical photographs, i.e. those taken when the focal plane of the camera is parallel to the ground, the relief displacement occurs along radial lines through the point in the photograph located directly below the camera lens (the principal point).

The relief displacement is greater, the farther the object is from the principal point and the greater the height of the object. Consequently, the determination of the positions of vehicles which have greater heights or are further away from the principal point is subject to greater magnitude of error. However, since the vehicle heights are negligibly small compared to the flight altitude, the errors due to relief displacement are not considerable. Nonetheless, the relief displacement due to tall buildings often obscures parts of streets, a problem often referred to as masking.

An assumption in the reduction of the aerial photographs is that they are vertical photographs. Such is not the case if the aircraft is tipped or tilted with respect to the plane of its altitude at the time of exposure. Since tips and tilts are usually no greater than three degrees, the errors due to tip and tilt can be considered negligible [Ref 58].

The non-systematic operator errors are present in all steps of the photo reduction process but the two most error-prone steps have been the proper placement of the comparator reticle on the desired points and the reading of the clock image on each frame. To measure the degree of consistency of an operator to place the reticle of the mono-digital comparator on the same point, five repetitive measurements were made on a set of twenty vehicles. The five measurements on a given vehicle were not made successively. These results showed mean standard deviations of 13.5 microns for measurements in x and 11.6 microns in y in determining the position of a given vehicle. Assuming a scale of 1" = 500' and a $\Delta t = 2.5$ seconds, these deviations correspond to 0.07 and 0.06 mph respectively. The greatest error ranges encountered in measurements made on the same vehicle were 85 and 56 microns in the x and y directions, respectively. Using the same scale and Δt as above, these maximum errors correspond to speed values of 0.46 mph and 0.30 mph respectively.

The determination of the elapsed time between two consecutive photographs, Δt , was made by reading the image of a clock on each frame. The close divisions were to the nearest second whereas the clocks were to be read to the nearest tenth of a second. Three independent readings were made for each frame. The lowest and highest readings never differed by more than 0.2 seconds while often all three readings were the same. However an error

of 0.1 seconds in Δt would result in a 3 to 5 percent error in the value of mean speed for the corresponding pair of frames.

To correct for each of the above-stated systematic errors individually requires the lay-out of many control points prior to photography as well as a tremendous increase in the level of effort required for reduction of the photographs. Moreover, for the purposes of these studies, where one often deals exclusively with averages, such tedious efforts to secure high levels of accuracy are not warranted. The question that arises is whether or not the directions and magnitudes of errors from these sources are random enough to yield meaningful averages.

To investigate the above question a study of the apparent 'speed' of parked cars was undertaken. On three pairs of frames the apparent 'speed' and the apparent 'travel' direction (drift angle) of parked cars were determined. Figures 5.5 to 5.7 show the vector fields of velocities for the three pairs of frames studied. In these diagrams the lengths of the velocity vectors are exaggerated with respect to the scales of the abscissa and the ordinate for visual clarity. Each figure also includes histograms of the magnitude and the direction of the velocity vectors corresponding to the respective pair of frames. The velocity directions (drift angles) are measured counter-clockwise from the abscissa of the Cartesian coordinate system for each master frame. The angles range from 0-360 degrees.

An examination of these vector fields (Figs 5.5 to 5.7) shows that for each pair of frames there are local patterns in the magnitude and the direction of the velocity vectors. Because of these local patterns it is necessary for the observed parked cars to be uniformly scattered in the frames studied if one is to examine whether or not the drift angles for each pair of frames are uniformly distributed. Otherwise, as is the case here,

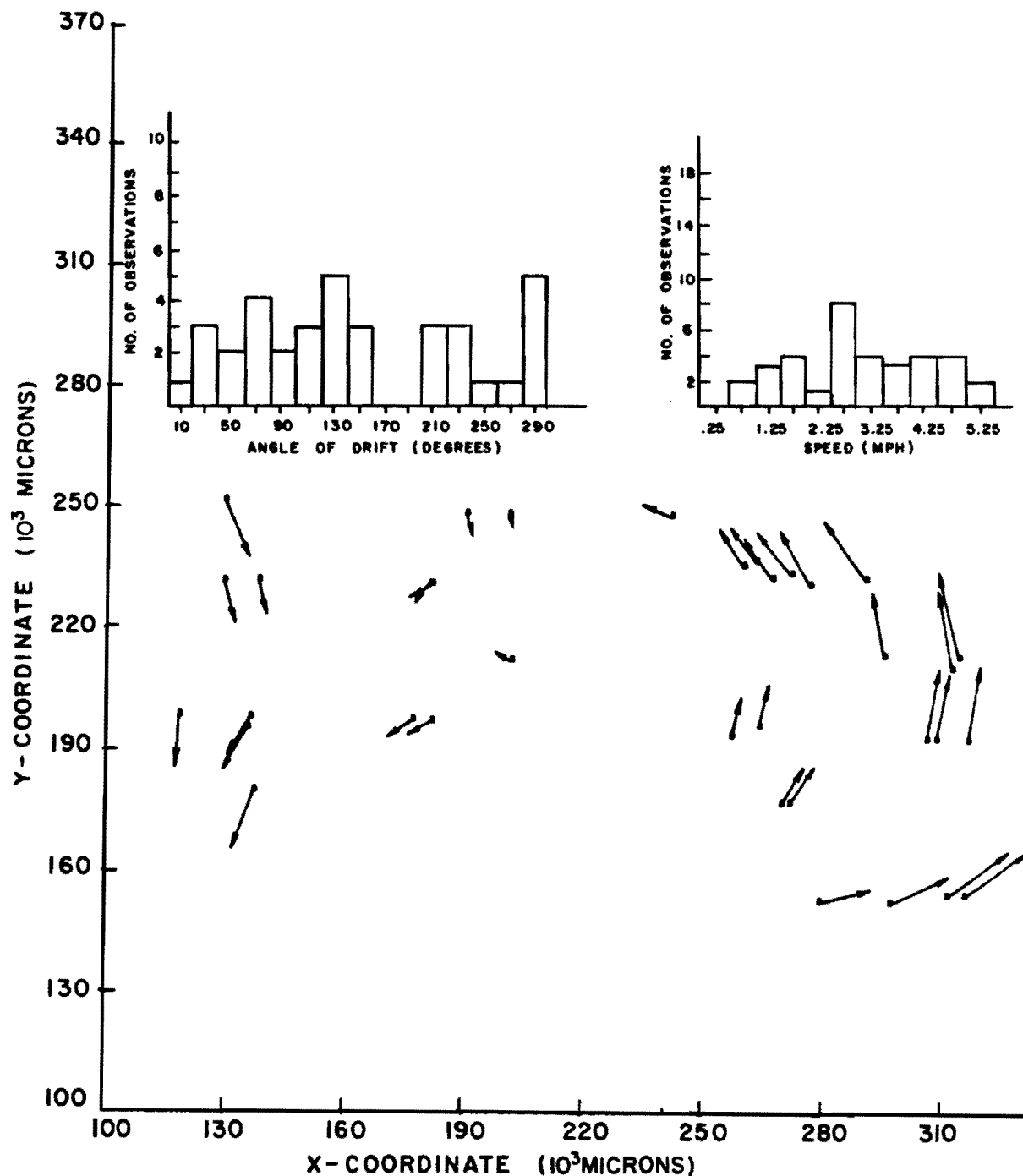


Figure 5.5. Vector field of apparent velocities for parked cars from the pair of frames 6587-6588 in the Austin CBD. The lengths of the velocity vectors are 25 times exaggerated with respect to the scales of the abscissa and the ordinate for visual clarity. Also shown are the histograms of the magnitude (apparent speed) and the direction (angle of drift of the parked cars) of the velocity vectors. The angles are measured counter-clockwise from the abscissa of the cartesian coordinate system of the master frame (frame 6587).

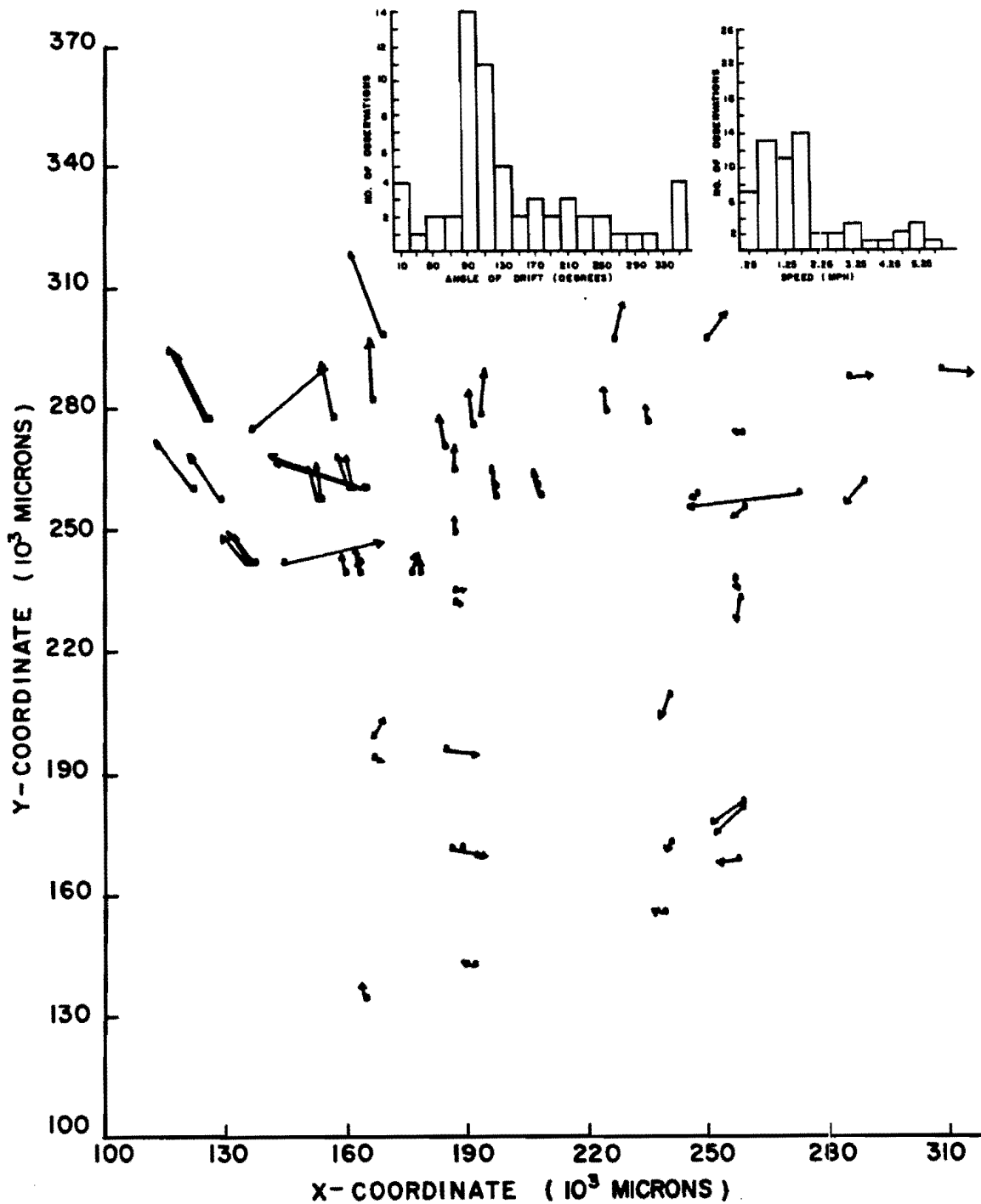


Figure 5.6. Vector field of apparent velocities for parked cars from the pair of frames 7246-7247 in the Austin CBD. The lengths of the velocity vectors are 25 times exaggerated with respect to the scales of the abscissa and the ordinate for visual clarity. Also shown are the histograms of the magnitude (apparent speed) and the direction (angle of drift of the parked cars) of the velocity vectors. The angles are measured counter-clockwise from the abscissa of the cartesian coordinate system of the master frame (frame 7246).

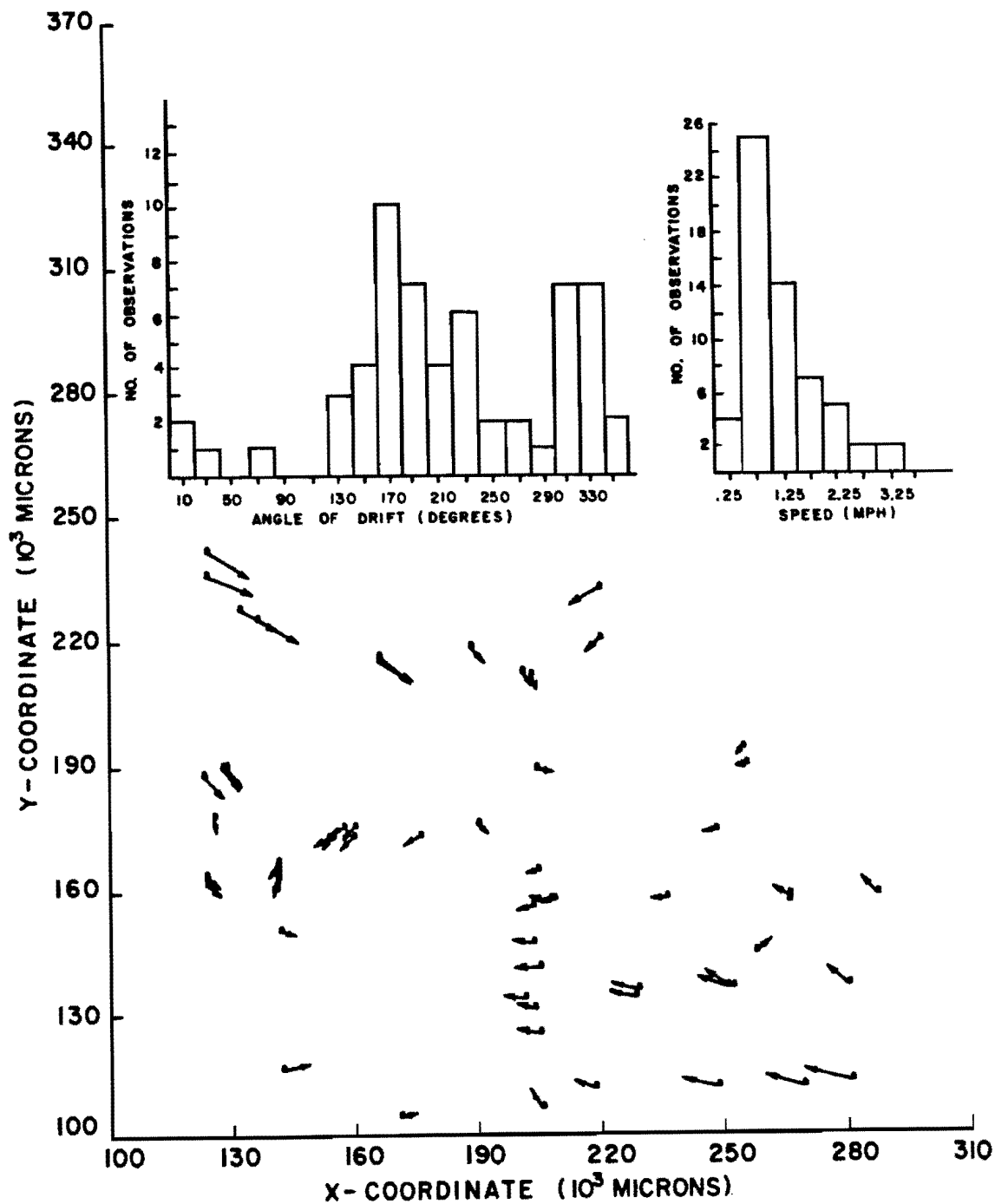


Figure 5.7. Vector field of apparent velocities for parked cars from the pair of frames 6660-6661 in the Austin CRD. The lengths of the velocity vectors are 25 times exaggerated with respect to the scales of the abscissa and the ordinate for visual clarity. Also shown are the histograms of the magnitude (apparent speed) and the direction (angle of drift of the parked cars) of the velocity vectors. The angles are measured counter-clockwise from the abscissa of the cartesian coordinate system of the master frame (frame 6660).

the peaks in the histograms of the drift angles will be associated with the locations with greater concentrations of parked cars.

To reduce such local effects, the three vector field diagrams (Figs 5.5 to 5.7) have been combined into one (Fig 5.8) resulting in a more uniform spatial distribution of parked cars. Also shown in Fig 5.8 are the speed and drift angle histograms corresponding to the velocity vectors. As can be seen from the histogram of the drift angles in Fig 5.8, the angles are rather uniformly distributed. Such global randomness of the drift angles suggests that in measuring the velocities of the moving vehicles, v_r , the errors in magnitudes of these velocities are essentially random and the resulting estimate of the average running speed, \bar{v}_r , can be assumed unbiased. On the contrary, the measured mean overall speed, $\bar{v}(\text{estimate})$, is likely to be greater than its actual value \bar{v} since the stopped vehicles contribute to this average by an amount \bar{v}_s , the average apparent speed of the stopped vehicles, i.e.,

$$\bar{v}(\text{estimate}) = \frac{N_r \bar{v}_r + N_s \bar{v}_s}{N_r + N_s} \quad (5.2)$$

where N_r and N_s are the number of vehicles running and stopped, respectively.

On the other hand, the unbiased estimate of the mean overall speed, \bar{v} , is

$$\bar{v} = \frac{N_r \bar{v}_r}{N_r + N_s} \quad (5.3)$$

Therefore, to obtain an accurate estimate of the mean overall speed it is necessary to develop a criterion for defining the stopped vehicles and set their apparent speeds to zero.

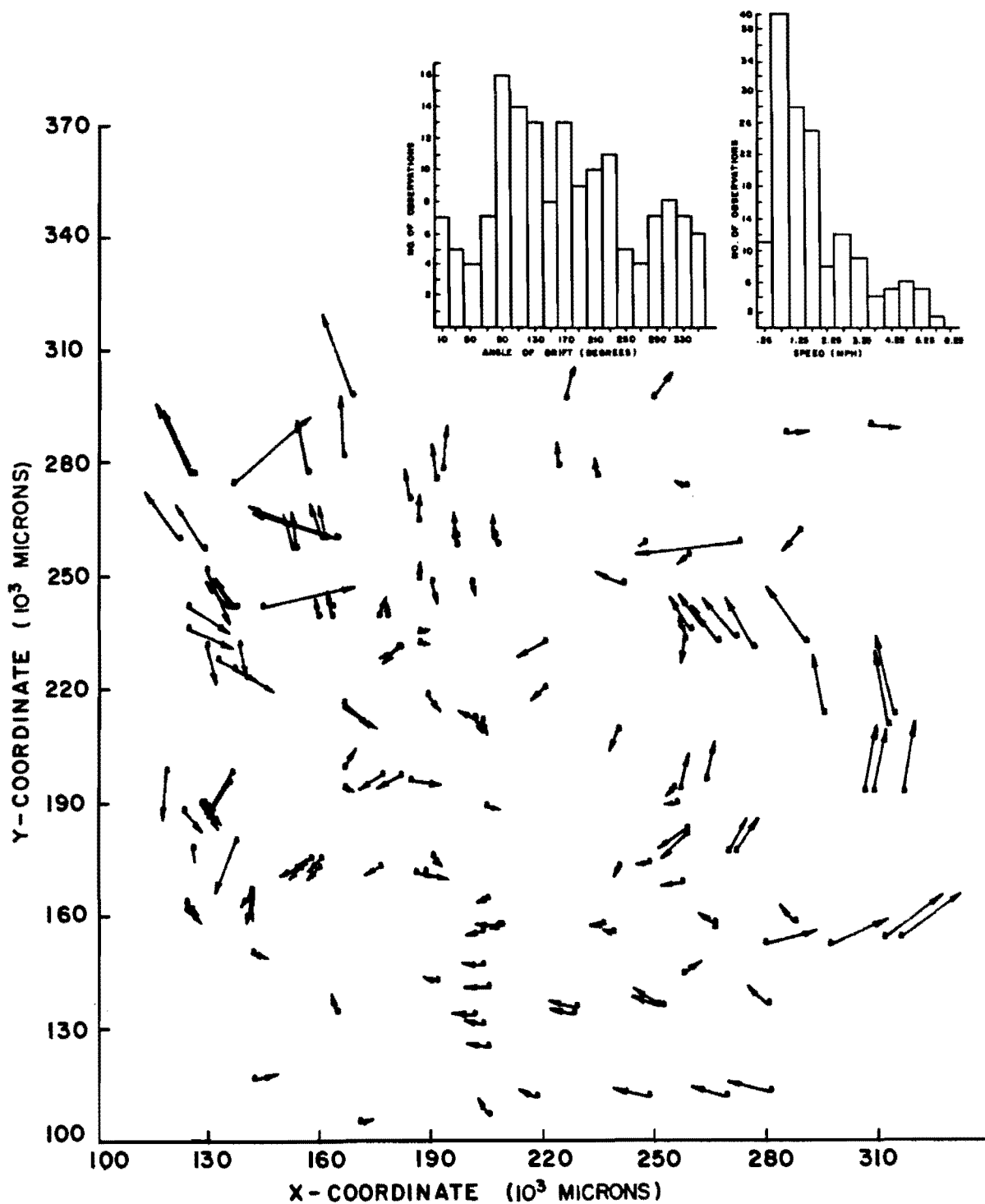


Figure 5.8. Combination of the three vector field diagrams of Figs 5.5, 5.6 and 5.7. Also shown are the combined histograms of speeds and drift angles. Note that the drift angles are rather uniformly distributed.

DEFINING A STOPPED VEHICLE

Stopped vehicles were defined as all vehicles travelling below a cutoff measured speed, C . To establish a reasonable cutoff speed, the speed of all the vehicles in the overlapped areas on four selected pairs of frames were determined. In addition, all the vehicles situationally perceived to be stopped were coded as stopped. Consequently, for each pair of frames a perceived fraction of vehicles stopped, a distribution of apparent speed of stopped vehicles, $f(v_s)$, and a distribution of speeds of vehicles perceived to be in motion, $f(v_r)$, were obtained.

In defining the cutoff speed it was assumed that the perceived fraction of vehicles stopped was the "true" fraction of vehicles stopped. The reasonableness of this assumption depends on the accuracy of the operators' perception of whether a vehicle on the photographic frame being reduced is stopped or in motion.

To examine the relative accuracy of the operators' perception of the status of a vehicle (stopped, running, or parked), a study was conducted involving three operators. In this study the three operators recorded their opinion of the status of 200 randomly selected vehicles. Each vehicle was to be classified as running, stopped in traffic or parked. The results, shown in Table 5.3, indicate perceived fractions of vehicles stopped of 0.358, 0.367 and 0.361 for operators A, B and C, respectively; i.e. a maximum deviation of only 2.5 percent in the perceived f_s . Note that a given vehicle could be classified in $3^3 = 27$ different combinations by the three operators. Operator A can classify a vehicle in three ways and this is the case for operators B and C, as well. For example, a vehicle can be called running by A, stopped by B and running by C or parked by A, parked by B and stopped by C, etc. As shown in Table 5.4, only 14 of the 27 possible

TABLE 5.3. RESULTS OF A STUDY INVOLVING THREE OPERATORS TO DETERMINE OPERATOR'S ACCURACY IN JUDGING WHETHER A VEHICLE ON A PHOTOGRAPH WAS RUNNING, STOPPED IN TRAFFIC OR PARKED

Operator	Number of Vehicles			Fraction of Vehicles Stopped
	Running	Stopped	Parked	
A	115	64	21	0.358
B	114	66	20	0.367
C	115	65	20	0.361

NOTE: The three operators examined the status of 200 vehicles on three aerial photographs.

TABLE 5.4. VARIOUS OUTCOMES OF A STUDY IN WHICH THREE OPERATORS CLASSIFIED THE STATUS OF 200 VEHICLES ON AERIAL PHOTOGRAPHS AS RUNNING (R), STOPPED IN TRAFFIC (S), OR PARKED (P)

Operator A	Operator B	Operator C	Number of Occurrences
R	R	R	109
S	S	S	56
P	P	P	15
R	R	S	1
R	S	S	3
R	S	P	1
R	P	P	1
S	R	R	2
S	R	S	2
S	S	R	3
S	P	S	1
P	S	P	3
P	P	R	1
P	P	S	2
R	R	P	-
R	S	R	-
R	P	R	-
R	P	S	-
S	R	P	-
S	S	P	-
S	P	P	-
S	P	R	-
P	S	R	-
P	S	S	-
P	R	S	-
P	R	P	-
P	R	R	-

combinations actually occurred in the course of the study. There were 180 unanimous classifications by the three operators. A stopped car was perceived to be in motion three times by operators A and C and twice by B. On the other hand, a running car was called stopped only twice by operator A and once by C. A car was considered stopped or running when two of the three operators classified it as stopped or running.

To summarize the results of this study, the fractions of vehicles stopped as perceived by the three operators were very close to each other with a maximum deviation of 2.5 percent. Moreover, in 180 out of 200 cases (90 percent of the time) there was unanimous agreement regarding the status of vehicles. In the remaining 20 cases of disagreement the judgement of operators A and B was a majority judgement 13 times while operator C was in majority 12 times. In general, it appeared that the fraction of vehicles stopped as perceived by any of the three operators could be used to represent the 'true' fraction of vehicles stopped in the following analysis which led to a definition of the cutoff speed.

A cutoff speed, C , was to be chosen so that for each pair of frames the value of the fraction of vehicles with speeds less than C was a close estimate of its corresponding value of the perceived f_s for that pair. Furthermore, the sum of apparent speeds of stopped vehicles exceeding C was not to be greatly different from the sum of running speeds less than C , i.e.

$$\Sigma (v_s > C) \approx \Sigma (v_r < C) \quad (5.4)$$

The latter condition (Eq 5.4) was required to ensure that the calculated mean overall speeds based on setting all the speeds less than C to zero, $\bar{v}(C)$, was not a biased estimator of the actual \bar{v} , namely,

$$\bar{v}(C) = \frac{N_r(C)\bar{v}_r(C)}{N} \approx \frac{N_r\bar{v}_r}{N} \quad (5.5)$$

where $N_r(C)$ and $\bar{v}_r(C)$ are the number and mean speed of vehicles with speeds greater than C , respectively. N is the total number of vehicles.

For the four pairs of frames studied eight cutoff speeds of 1.75 mph to 3.50 mph at intervals of 0.25 mph were examined. The results, as tabulated in Table 5.5, show that for the four pairs of frames at the cutoff speed of 2.5 mph the deviations of $f_s(C)$ from the perceived f_s are 1.6 percent, zero percent, 1.3 percent, and 9.7 percent. The above deviations are individually small and their sum is also minimum compared to those at other cutoff speeds. Furthermore, $\Sigma(v_s > 2.5)$ is not highly different from $\Sigma(v_r \leq 2.5)$ and the mean overall speed, $\bar{v}(C)$, obtained by setting all speeds less than C to zero does not vary more than 3.1 percent for the wide range of cutoff speeds examined.

On the basis of the above results, a cutoff speed of 2.5 mph has been selected and for each pair of frames analyzed all the speeds less than 2.5 mph were set to zero. Estimates of f_s , \bar{v} , and \bar{v}_r were made in the following manner:

$$f_s = \frac{N(v \leq 2.5)}{N} \quad (5.6)$$

$$\bar{v} = \frac{\Sigma(v > 2.5)}{N} \quad (5.7)$$

$$\bar{v}_r = \frac{\Sigma(v > 2.5)}{N(v > 2.5)} \quad (5.8)$$

where $N(v > 2.5)$ and $N(v \leq 2.5)$ are the numbers of vehicles with speeds greater than and less than or equal to 2.5 mph, respectively and N is as before the total number of vehicles.

TABLE 5.5. AN EXAMINATION OF THE FRACTION OF VEHICLES STOPPED AND THE MEAN OVERALL SPEED FOR FOUR PAIRS OF FRAMES AT EIGHT DIFFERENT CUTOFF SPEEDS

Frames 6654 - 55						Frames 6655 - 56				
Cutoff Speed (mph)	Perceived	$f_s(C)$	$\sum(v_s > C)$ (mph)	$\sum(v_r \leq C)$ (mph)	$\bar{V}(C)$ (mph)	Perceived	$f_s(C)$	$\sum(v_s > C)$ (mph)	$\sum(v_r \leq C)$ (mph)	$\bar{V}(C)$ (mph)
	f_s					f_s				
3.50	(66/263) = 0.251	0.278	25.7	22.7	13.16	(53/224) = 0.237	0.317	20.7	49.1	14.17
3.25		0.274	25.7	19.4	13.17		0.312	20.7	45.8	14.19
3.00		0.274	25.7	19.4	13.17		0.304	23.9	42.7	14.22
2.75		0.262	28.6	13.6	13.20		0.286	23.9	31.3	14.27
2.50		0.255	34.0	13.6	13.22		0.237	44.6	23.4	14.40
2.25		0.247	36.3	11.2	13.24		0.183	70.9	21.2	14.52
2.00		0.232	40.6	6.8	13.28		0.152	79.6	14.7	14.59
1.75		0.202	53.7	4.8	13.33		0.138	85.2	14.7	14.62
Frames 6656 - 57						Frames 6657 - 58				
Cutoff Speed (mph)	Perceived	$f_s(C)$	$\sum(v_s > C)$ (mph)	$\sum(v_r \leq C)$ (mph)	$\bar{V}(C)$ (mph)	Perceived	$f_s(C)$	$\sum(v_s > C)$ (mph)	$\sum(v_r \leq C)$ (mph)	$\bar{V}(C)$ (mph)
	f_s					f_s				
3.50	(123/527) = 0.233	0.247	42.6	33.8	12.96	(155/578) = 0.268	0.313	4.3	68.0	12.04
3.25		0.247	42.6	33.8	12.96		0.298	4.3	37.8	12.09
3.00		0.237	51.9	27.5	12.99		0.280	32.4	34.7	12.14
2.75		0.235	54.8	27.5	12.99		0.265	46.8	23.1	12.19
2.50		0.230	60.0	24.9	13.01		0.242	75.4	17.9	12.24
2.25		0.194	96.0	17.7	13.10		0.222	101.3	15.6	12.29
2.00		0.159	130.1	11.2	13.17		0.196	128.7	11.3	12.35
1.75		0.138	146.8	7.6	13.21		0.170	151.2	5.9	12.40

NOTE: Also tabulated for each pair of frames are the perceived fraction of vehicles stopped, the sum of apparent speeds of stopped vehicles which are greater than the cutoff speed and the sum of speeds of the running vehicles which are less than or equal to the cutoff speed.

CONCENTRATION MEASUREMENTS

The determination of the vehicular concentration for each frame involved counting the vehicles (excluding the parked cars) and measuring the total lane-miles of streets in the part of the study network which appeared on each frame. For each frame, the ratio of the number of vehicles to the total lane-miles would then be the vehicular concentration in vehicles per lane-mile. The average concentration for a pair of frames was obtained by dividing the total number of vehicles in both frames by the sum of the total lane-miles covered by the two frames.

The total lane-miles of streets in the portion of the study network covered by each of the two flight paths (strips A and B) were obtained by dividing the street network into 19 east-west links each covering the entire width of a strip and 162 south-north links for strip A and 198 south-north links for strip B. The south-north links did not include the intersection areas so that the intersection areas would not be included twice in determination of the total lane-miles. The length of each of the 398 links was then measured using 1" = 200' maps of the Austin CBD. The number of lanes for each link was determined using the aerial photographs and was also verified through site inspection. For each of the two flight strips a matrix of link lengths in lane-feet was then established. Knowing the inclusive street boundaries of a photographic frame, we were then able to use the matrix of link lengths to determine the corresponding lane-miles of streets encompassed by those boundaries.

FLOW MEASUREMENTS

The aerial photographs in Austin were taken during four periods and in two strips, each strip composed of about 30 exposures two to three seconds apart. A meaningful estimate of the vehicular flow, i.e. the number of

vehicles passing a fiducial line per hour, cannot be obtained for time periods as short as the duration of a strip of aerial photographs (60 to 90 seconds). Therefore, an estimate of the average flow in the network must be based on ground observations made during relatively longer time periods.

To determine the average flow in the Austin study network, 29 observers were assigned to various locations in the network (Fig 5.9). The observations were made simultaneously with the taking of the aerial photographs and the ergodic experiments on November 12, 1982. Each observer was instructed to make five-minute counts over two one-hour periods of all the motor vehicles (including motorcycles) passing the observation point. The observation periods were from 3:30 to 4:30 p.m. and from 4:45 to 5:45 p.m.

The assignment of observers to various locations in the network was based on the following procedure. The streets in the study network were first classified into seven categories. The categories were based on the street type and the dominant land use, as shown in Table 5.6. The number of observers assigned to streets of each category was made proportional to the fraction of total lane-miles that belonged to each respective category. Within each category observers were assigned to random links. Observations were made at mid-block locations rather than at intersections. Observers assigned to two-way streets made vehicle counts at both directions but were not asked to separate the counts for each travel direction.

The flows reported by observers assigned to each street category were averaged to yield a mean flow measurement for that category (Table 5.6). The mean flow in the network for each observation period was then calculated as the arithmetic mean of flows for the seven street categories weighted

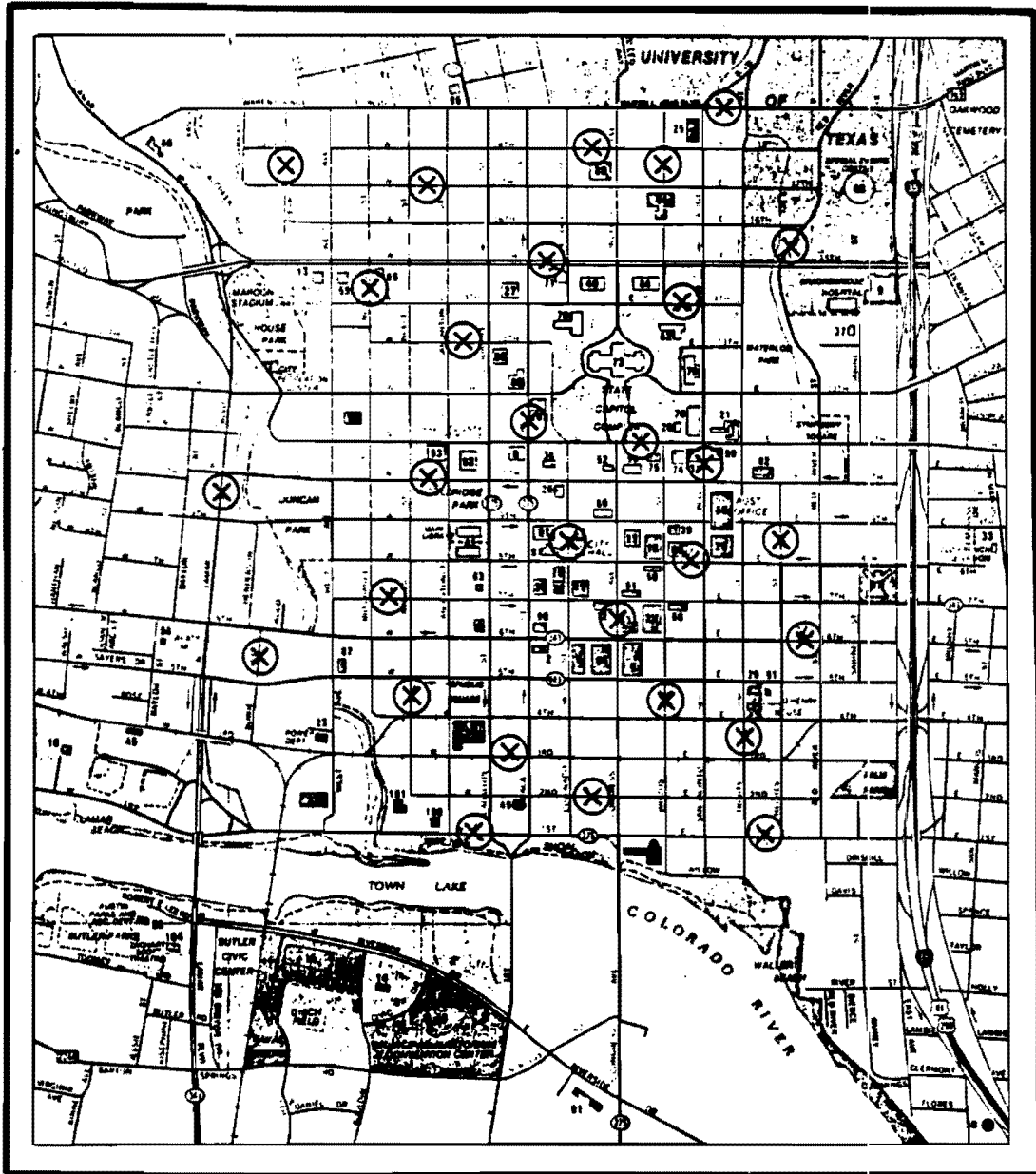


Figure 5.9. The Austin CBD aerial photographic study area. Five-minute volume counts were taken over two one-hour periods at the 29 locations marked with X in order to determine the average flow in the network.

TABLE 5.6. STREET CATEGORIES IN THE AUSTIN CBD AND THEIR RESPECTIVE MEAN FLOW VALUES DURING TWO PERIODS ON FRIDAY AFTERNOON, NOVEMBER 12, 1982

Street Type	Dominant Lane-Use	Code	Lane-Miles Percent	Flow Veh/(lane hr.) 3:45 - 4:30 p.m.	Flow Veh/(lane hr.) 4:45 - 5:45 p.m.
Collector-Distributor	Commercial	1-2	33.9	132.3	164.7
	Government	1-3	2.6	274.3	286.5
	Industrial	1-4	4.1	59.9	62.4
	Residential	1-5	6.5	167.3	199.0
Residential	Industrial	2-4	1.8	84.0	83.5
	Residential	2-5	8.0	33.8	40.4
Arterial	Commercial	3-2	42.2	355.7	365.8
Mean Flow in the Network				220	238

according to the percent of lane-miles encompassed by each category (Table 5.6).

The reduction procedure described in this chapter resulted in direct observations of average speed, average concentration, and fraction of vehicles stopped for each pair of frames. In addition, estimates of average speed and fraction of vehicles stopped were obtained through the ergodic experiments. Ground observations also yielded estimates of the average flow in the study network. In the following chapter the analysis of these observations and the results obtained are presented.

CHAPTER 6. AERIAL PHOTOGRAPHIC RESULTS

The use of aerial photographic traffic surveys as a means of measuring network-wide traffic variables such as speed, flow, and fraction of vehicles stopped is by no means feasible on a routine basis. Enormous resources are required to obtain the photographs, to establish minimal controls for their proper reduction, and to carry out their reduction and analysis. What is feasible, however, is to perform aerial photographic surveys in order to search for ways of estimating the above-stated pertinent variables using ground observations. The main objective of our aerial photographic studies of urban street networks has indeed been to explore any relations that may exist among the pertinent traffic variables such as speed, flow, concentration, and fraction of vehicles stopped. Such relations would then allow one to estimate the above-mentioned pertinent traffic variables through network-wide ground observations, estimates which in turn can be used as measures of the productivity and the quality of traffic service in urban street networks.

Over the years, various efforts regarding the aerial traffic surveys of urban street networks have been made. These undertakings have been mostly descriptions of available methodologies for such studies. Only a few of the works have been observational, out of which a handful have included simultaneous ground observations. Methodologically, Peleg, Stoch, and Etrog [Ref 57] have presented a rather thorough discussion of equipment, flight planning and reduction procedures suitable for aerial traffic surveys of urban networks. Ashwood and Inglis [Ref 59], Vanmans [Ref 60], and Smith [Ref 61] have also discussed the procedural aspects of network-wide aerial traffic surveys.

Due to the tediousness and expensive nature of network-wide aerial photographic surveys of urban traffic only a few observational studies, each very limited in scope, have been undertaken in the field of traffic. Godfrey [Ref 52] has investigated the relation between average speed and concentration using ground observations of speed and simultaneous concentration measurements from aerial photographs. In the work reported here we have also examined such relations but on a much larger scale including direct measurements of speed, concentration, and fraction of vehicles stopped from aerial photographs as well as simultaneous estimates of speed, fraction of vehicles stopped, and vehicular flow from ground experiments and observations.

Other aerial observational work on the urban network traffic surveys include a study conducted by Baker and Owens [Ref 62] in Glasgow to assess methods of coordinating the traffic signals in a network. Their report [Ref 62] is a testimony to the problematic nature of aerial photography as a routine means of conducting traffic surveys. In a simultaneous ground and air speed observations William and Gordon [Ref 63] have discussed the accuracy of aerial photography to determine speed through time-lapse aerial photographic surveys along with simultaneous ground experiments to determine the speed of buses in the traffic stream. Garner and Uren [Ref 64] have also undertaken a study of speeds of vehicles in the central network of Leeds, using a reduction procedure conceptually similar to the one employed in our studies, as described in Chapter 5.

EXPERIMENTAL AND OBSERVATIONAL DETAILS

The scope of the studies reported in this chapter have been to make network-wide aerial measurements of speed, concentration, and fraction of vehicles stopped and correlate them with simultaneous ground estimates of

speed, flow, and fraction of vehicles stopped in order to establish relations among these variables. Such relations may then be used to characterize the quality of traffic service in urban street networks. The study has included four south-north fly-overs made over the Austin central network described in the previous chapter. The four fly-overs, each consisting of two strips with ~ 5 percent lateral overlap, have been made in four periods starting at 8:00 a.m., 12:00 noon, 3:30 p.m., and 5:00 p.m. and each lasting 2.5 to three minutes. During the latter two periods, i.e. from $\sim 3:30-4:30$ p.m. and $\sim 4:45-5:45$ p.m., ten vehicles were also circulating in the area and recording, for one-mile trips, their total trip time and the absolute times of each stop and the subsequent resumption of motion. In addition 29 observers were assigned to mid-block locations in the study area to make five-minute counts of vehicles passing their observation points over the two one-hour periods.

The analysis of the time-lapse aerial photographs resulted in direct observations of average speed, \bar{v} , average concentration, \bar{k} , and fraction of vehicles stopped, f_s , for each pair of frames analyzed as well as for each of the two strips and the four time periods. An attempt was made to observationally verify theoretically postulated relations among the variables f_s , \bar{v} , and \bar{k} .

The ground observations during the two afternoon periods, on the other hand, provided estimates of the average speed, the fraction of vehicles stopped, and the average vehicular flow in the network during their respective study periods, i.e. 3:30-4:30 p.m. and 4:45-5:45 p.m. These observations enabled us to compare the ground estimates of \bar{v} and f_s to those obtained from the air and to explore relations among network-wide averages of speed, concentration, and flow.

AERIAL VERSUS GROUND OBSERVATIONS

The ergodic assumption of the Two-Fluid model postulates that over a sufficiently long period of time the fraction of time each vehicle is stopped in a network is identical to the average fraction of vehicles stopped in that network during the same time period. Five ergodic experiments performed in the Austin CBD using five to ten vehicles circulating in the network for about one hour showed that, for each experiment, the fractions of time stopped reported by the test vehicles were narrowly distributed around their average value which itself was identical to the average fraction of the test vehicles stopped for each respective experiment (see Fig 3.1).

In order to fully verify the Two-Fluid model ergodic assumption, it was necessary to determine whether or not the average fraction of test vehicles stopped was a reasonable estimate of the average fraction of the population of vehicles stopped. During the last two ergodic experiments in the Austin CBD, simultaneous time-lapse aerial photographs of the network were also obtained to investigate the above question. From these photographs the average fraction of the population of vehicles stopped for each of the two experiments were determined using the reduction techniques described in the previous chapter.

The aerial photographs corresponding to the two ground experiments were taken from 3:34:38 to 3:41:43 p.m. and from 4:55:52 to 5:02:16 p.m.. The average f_s for the population of vehicles during these two periods were determined from the aerial photographs to be 0.214 and 0.246, respectively. The averages of f_s from the ground experiments were 0.236 and 0.260 for the same general periods, respectively. These results are shown in Table 6.1.

As shown in Table 6.1, the fractions of vehicles stopped estimated by the vehicles involved in the ergodic experiments are in rather close

TABLE 6.1. COMPARISONS OF THE ESTIMATES OF THE FRACTION OF VEHICLES STOPPED AND THE AVERAGE SPEED AS OBTAINED FROM THE AERIAL OBSERVATIONS AND THE GROUND EXPERIMENTS

Experiment	Fraction of Vehicles Stopped			Average Speed (mph)		
	Air	Ground*	% Deviation	Air	Ground*	% Deviation
3-PM	0.214	0.236	10.3	15.32	14.63	4.5
5-PM	0.246	0.260	5.7	13.79	12.82	7.0

* The ground values correspond to the durations of aerial observations and not to the entire durations of the two ergodic experiments reported in Table 3.1.

agreement with the f_s for the population of vehicles determined from the aerial photographs with a maximum deviation of ten percent. Combining the above results with those of the ergodic experiments, it can be concluded that the fraction of time stopped for any test vehicle in the ergodic experiment, $(T_s/T)_i$, is a close estimate of the average fraction of time stopped averaged over all the test vehicles, $\langle T_s/T \rangle_t$. Furthermore, during the same time period and over the same vehicles $\langle T_s/T \rangle_t$ is identically equal to the average fraction of test vehicles stopped, $\langle f_s \rangle_t$, which itself closely estimates the average fraction of vehicles stopped of the population of vehicles, $\langle f_s \rangle_p$, namely,

$$(T_s/T)_i \approx \langle T_s/T \rangle_t \equiv \langle f_s \rangle_t \approx \langle f_s \rangle_p . \quad (6.1)$$

Hence, the ergodic assumption of the Two-Fluid model appears to be a reasonable assumption based on both the ground experiments and the aerial photographic observations.

The aerial and ground observational results can also be compared in terms of the observed average speeds. The aerial photographic observations during the 3:00 p.m. and the 5:00 p.m. periods yielded average speeds of 15.32 mph and 13.79 mph, respectively. The corresponding averages for the ground observations were 14.63 mph and 12.82 mph, respectively. As shown in Table 6.1, the deviations of the ground average speeds from the aerial average speeds were 4.5 percent and 7.0 percent for the 3:00 p.m. and 5:00 p.m. periods, respectively. These results suggest that as few as ten vehicles circulating in an area can provide relatively reliable estimates of the average speed over the entire population of vehicles.

RELATIONS BETWEEN FRACTION OF VEHICLES STOPPED AND CONCENTRATION

In comparing two networks with the same Two-Fluid trends, i.e. the same values of T_m and n , a given fraction of vehicles stopped would result in the same values of T and T_s , thus implying the same quality of traffic service. However, comparing these two hypothetical networks under the same f_s is not necessarily equivalent to comparing these networks under the same concentration k since the relation between f_s and k would likely not be the same for different networks. Hence, a functional relation between f_s and k is required to complement the Two-Fluid picture and allow the comparison of networks under similar traffic loadings.

A simple functional dependence of concentration on the fraction of vehicles stopped has been postulated by Herman and Prigogine [Ref 21]. The relation is of the form

$$f_s = (k/k_m)^p \quad (6.2)$$

where k_m is the average maximum concentration at which the traffic jams in the system, and the parameter p is, in some sense, a measure of the quality of traffic service of the street network as it determines the fraction of vehicles stopped for a given partial concentration, k/k_m . Since k/k_m is smaller than 1, then for a given k/k_m the higher p the smaller f_s and the better the quality of traffic service.

Figure 6.1 is a theoretical plot of fraction of vehicles stopped versus concentration for values of p in the range 0.10 to 2.0 in 0.10 increments. Superimposed on this plot is the data from the Austin CBD for the 71 frames analyzed. To plot the data in Fig 6.1, a maximum (jam) concentration of $k_m = 100$ vehicles/lane-mile is used. If we were to assume that each vehicle on the average occupied 30 feet of lane length, then a physical upper bound of

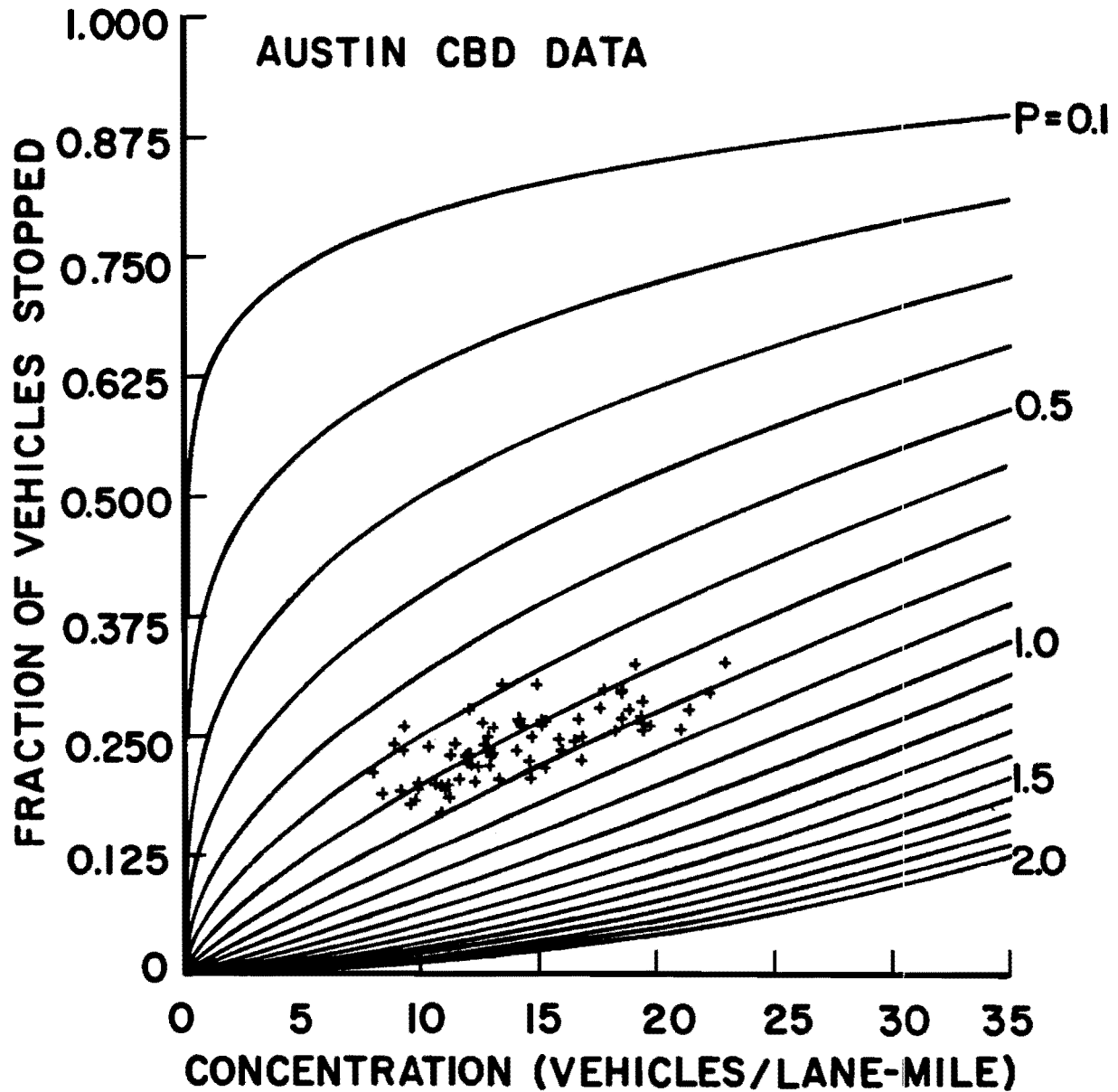


Figure 6.1. Theoretical plot of fraction of vehicles stopped, f_s , versus concentration, k , according to the relation $f_s = (k/k_m)^p$ for $k_m = 100$ vehicles per lane-mile and p values in the range 0.1 to 2.0 in 0.1 increments. Also shown are the aerial photographic data from the Austin CBD for which $p = 0.720$.

176 vehicles/lane-mile is determined for k_m . In reality, however, due to instability of flow in vehicular platoons such as those discussed in the car-following theory [Refs 65 and 66] the jam concentration is expected to be reached at $k_m < 176$ vehicles/lane-mile. A recent study [Ref 67] of the relation between f_s and k/k_m using the NETSIM [Ref 68] simulation model revealed that a run with an intended k of 150 vehicles/lane-mile showed extreme congestion levels with prevailing spillbacks at most intersections which effectively delayed the complete loading of the vehicles onto the network. In determining the values of the parameters in the relation between f_s and k we have used a $k_m = 100$ vehicles/lane-mile.

The data in Fig 6.1 was also aggregated into eight points each representing a flight strip and a time period. The aggregation was also made for concentration intervals of one vehicle/lane-mile starting from $k = 8$ vehicles/lane-mile. The parameter $p = \ln(f_s)/\ln(k/k_m)$ was determined through regressing $\ln(f_s)$ against $\ln(k/k_m)$ and forcing the regression through the origin. In this manner, p values of 0.720, 0.719 and 0.720 were obtained for the unaggregated data (71 points), the data aggregated over each strip and time period (8 points), and the data aggregated over $\Delta k = 1$ vehicle/mile-lane (17 points), respectively. Using a $p = 0.720$ for the Austin CBD we may then write

$$f_s = (k/k_m)^{0.720} \quad (6.3)$$

The postulated relation in Eqs 6.2 or 6.3 implies that as the concentration approaches zero, the fraction of vehicles stopped will also approach zero. However, observations of trip time and stop time taken in the Austin CBD under very light traffic concentrations of early morning hours indicate that a non-zero lower bound, $(T/T)_s \min = 0.189$, exists for the

fraction of vehicles stopped in Austin. This non-zero minimum fraction of time or vehicles stopped exists because traffic control devices continue to function even when vehicular concentrations approach zero. Consequently, we have postulated a new relation between f_s and k to meet the lower boundary condition that $f_s \neq 0$ when $k \rightarrow 0$, namely,

$$f_s = f_{s,\min} + (1 - f_{s,\min}) (k/k_m)^\pi \quad (6.4)$$

where $f_{s,\min}$ is the average minimum fraction of vehicles stopped and π is the new Two-Fluid parameter replacing p as a measure of the quality of service in urban street networks. Figure 6.2 shows the family of f_s versus k/k_m curves for $f_{s,\min} = 0.15$ and π values in the range 0.1 to 2.0 in 0.1 increments. Note that in the new formulation (Eq 6.4) as $k \rightarrow k_m$, $f_s \rightarrow 1$ and as $k \rightarrow 0$, $f_s \rightarrow f_{s,\min}$. Williams, et al [Ref 69] in a series of NETSIM simulation runs for a five node by five node grid network of 40 two-way street links have also obtained a family of four f_s versus k curves with a general resemblance to those of Fig 6.2 for $\pi < 1$. These simulation runs have been performed for concentration levels ranging from about 8 to 60 vehicles per lane-mile. The four curves correspond to four different levels of interactions (activity levels) among the vehicles in the network. It is also interesting to note that when the Two-Fluid model parameters T_m and n were determined separately for each activity level, similar values of T_m but different values of n were resulted for the data under different activity levels [Ref 69].

The model of Eq 6.4 has been calibrated for the aerial photographic data obtained for the Austin CBD. This has been achieved through performing a non-linear regression to the Austin data of the form

$$f_s = a + b (k/k_m)^\pi \quad (6.5)$$

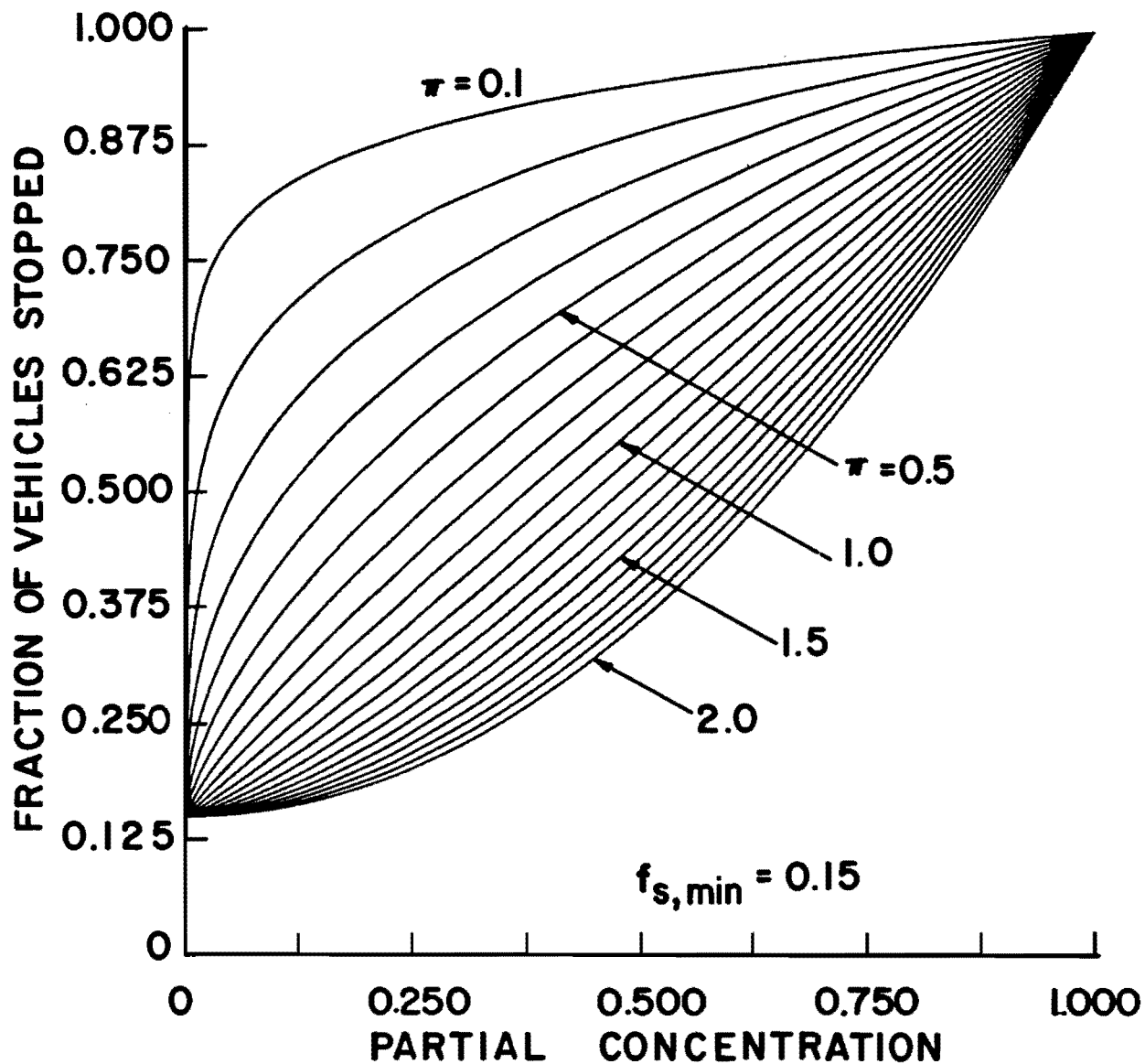


Figure 6.2. Family of fraction of vehicles stopped, f_s , versus partial concentration, k/k_m , according to Eq (6.4) for parameters $f_{s,\min} = 0.15$ and π values in the range 0.1 to 2.0 in 0.1 increments.

subject to the condition $a + b = 1$. For the 71 unaggregated data points the above constraint ($a + b = 1$) was met at $\pi = 1.177$ with $a = 0.153$ (Fig 6.3). The data was then aggregated for intervals in k of 1 vehicle/lane-mile starting at 8 vehicles/lane-mile. For this case the constraint $a + b = 1$ was satisfied at $\pi = 1.276$ and $a = 0.168$ (Fig 6.4). Finally when the aggregation was made over each strip and each time period, resulting in eight pairs of data points, $\pi = 1.216$ and $a = 0.161$. Figure 6.5 shows these eight data points along with a family of f_s, k curves for π values in the range of 0.1 to 2.0. For all the above three cases, the value of R^2 for the model in which $a + b = 1$ was very close to the maximum R^2 which was achieved for an unconstrained non-linear regression to the three respective data sets. The results of these calibration efforts are presented in Table 6.2.

While the parameter values remain rather stiff, the best fit is obtained when the data is aggregated for each flight strip and time period. Therefore, adopting this method of aggregation we can write for the Austin CBD that

$$f_s = 0.161 + 0.839 (k/k_m)^{1.216} \quad (6.6)$$

In the above relation as $k \rightarrow 0$, $f_s \rightarrow 0.161$. This implies an $f_{s,\min} = 0.161$ in the Austin CBD. Note that observationally $(T_s/T)_{\min} = 0.189$ in this network yielding rather close agreement with the above approach.

Aerial photographs of the Dallas CBD also resulted in a set of 20 f_s, k unaggregated points obtained over five time periods. Calibration of Eq (6.5) for the data results in $\pi = 0.950$ and $f_{s,\min} = 0.176$ for the Dallas CBD with $R^2 = 0.790$. These parameters for the Dallas CBD are to be compared with $\pi = 1.216$ and $f_{s,\min} = 0.161$ for the Austin CBD. The f_s, k data for the

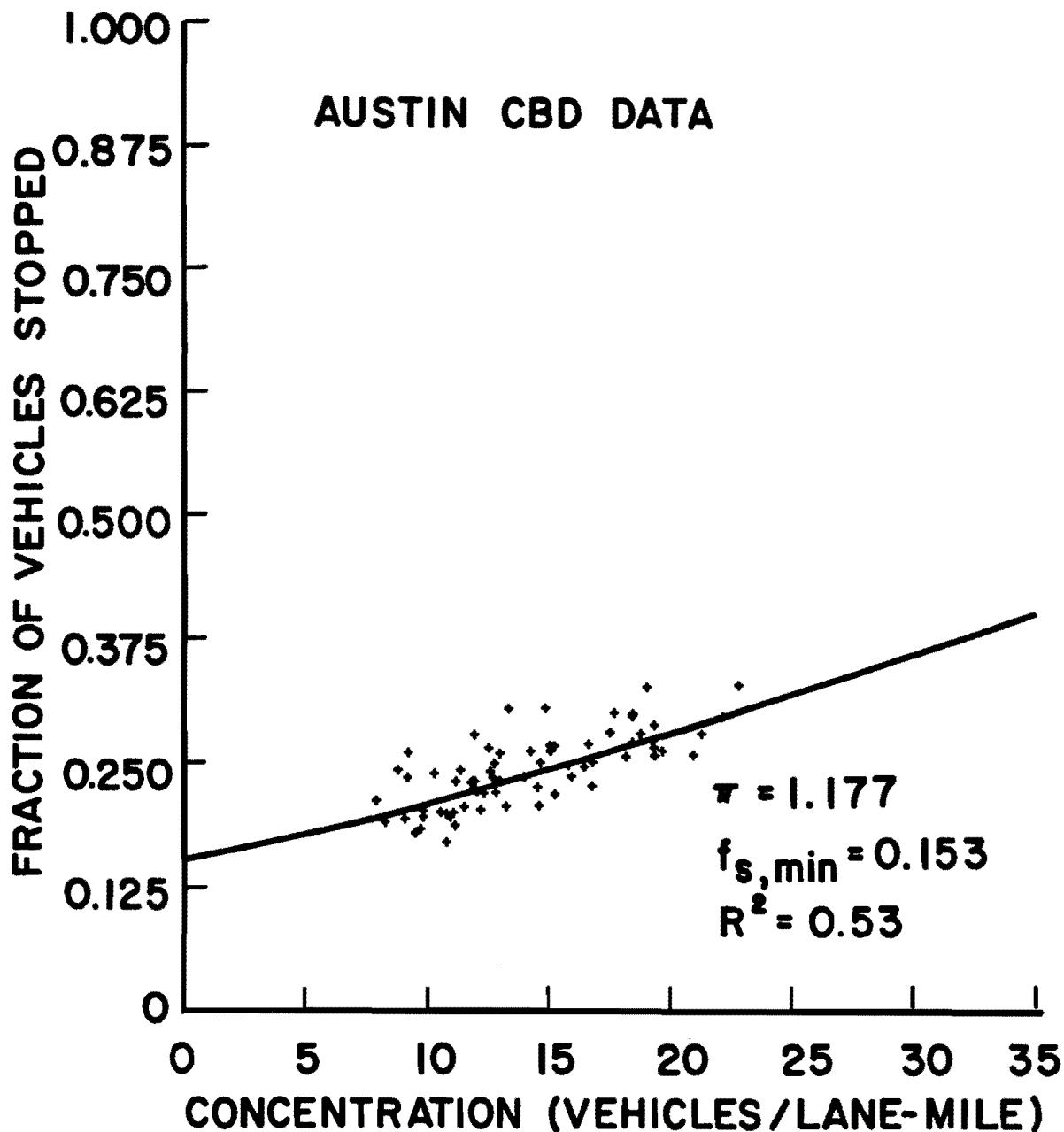


Figure 6.3. Plot of fraction of vehicles stopped versus concentration for the 71 unaggregated Austin CBD data points shown. The solid curve corresponds to Eq (6.4) calibrated for the data shown using $k_m = 100$ vehicles per lane-mile. The resulting parameters for Austin CBD are $\pi = 1.177$ and $f_{s,\min} = 0.153$ with $R^2 = 0.53$.

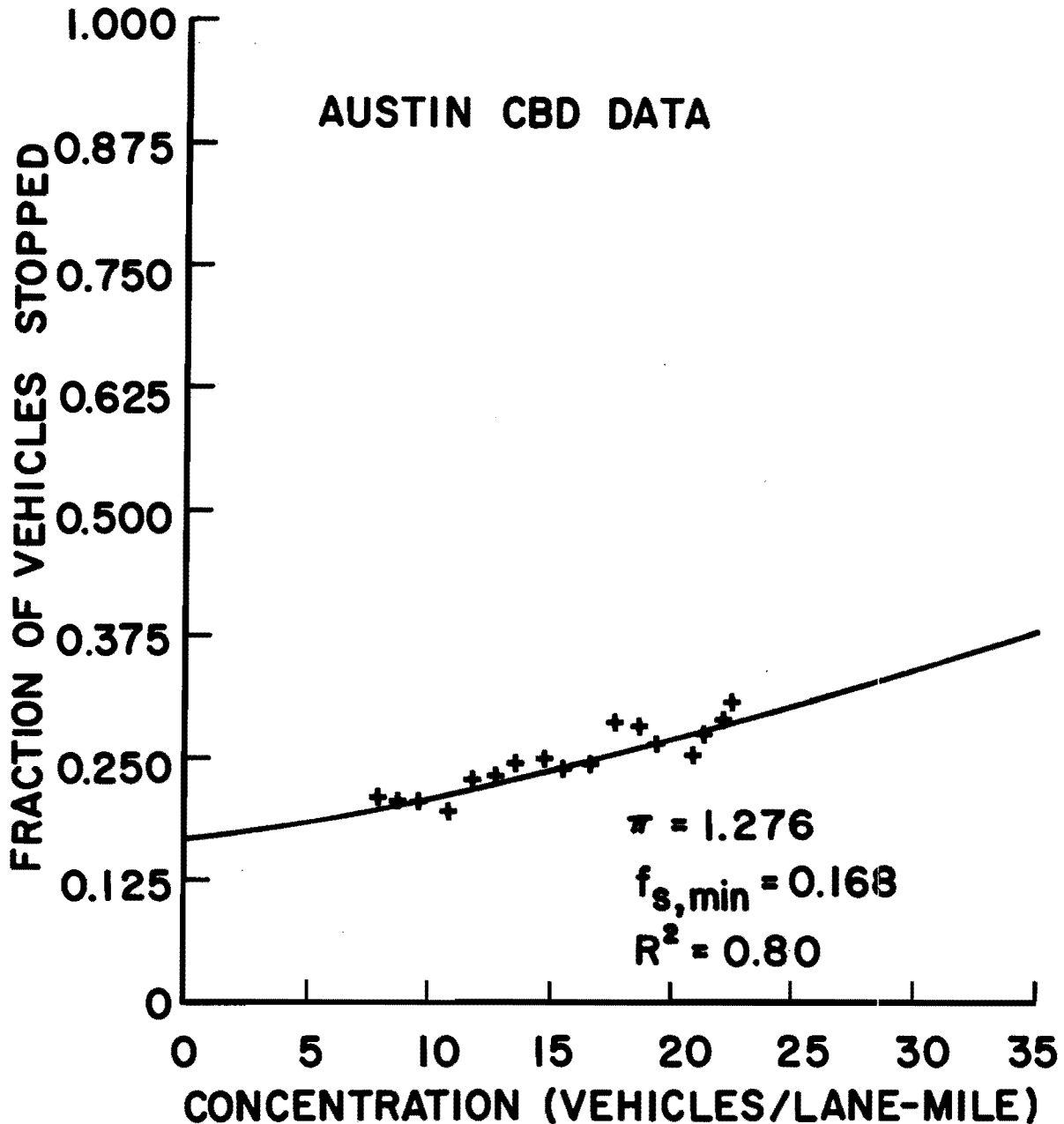


Figure 6.4. Plot of fraction of vehicles stopped versus concentration for 17 data points in Austin CBD aggregated for concentration intervals of one vehicle per lane-mile. The solid curve corresponds to Eq (6.4) calibrated for the data shown using $k_m = 100$ vehicle per lane-mile. The resulting parameters for the Austin CBD are $\pi = 1.276$ and $f_{s,\min} = 0.168$ with $R^2 = 0.80$.

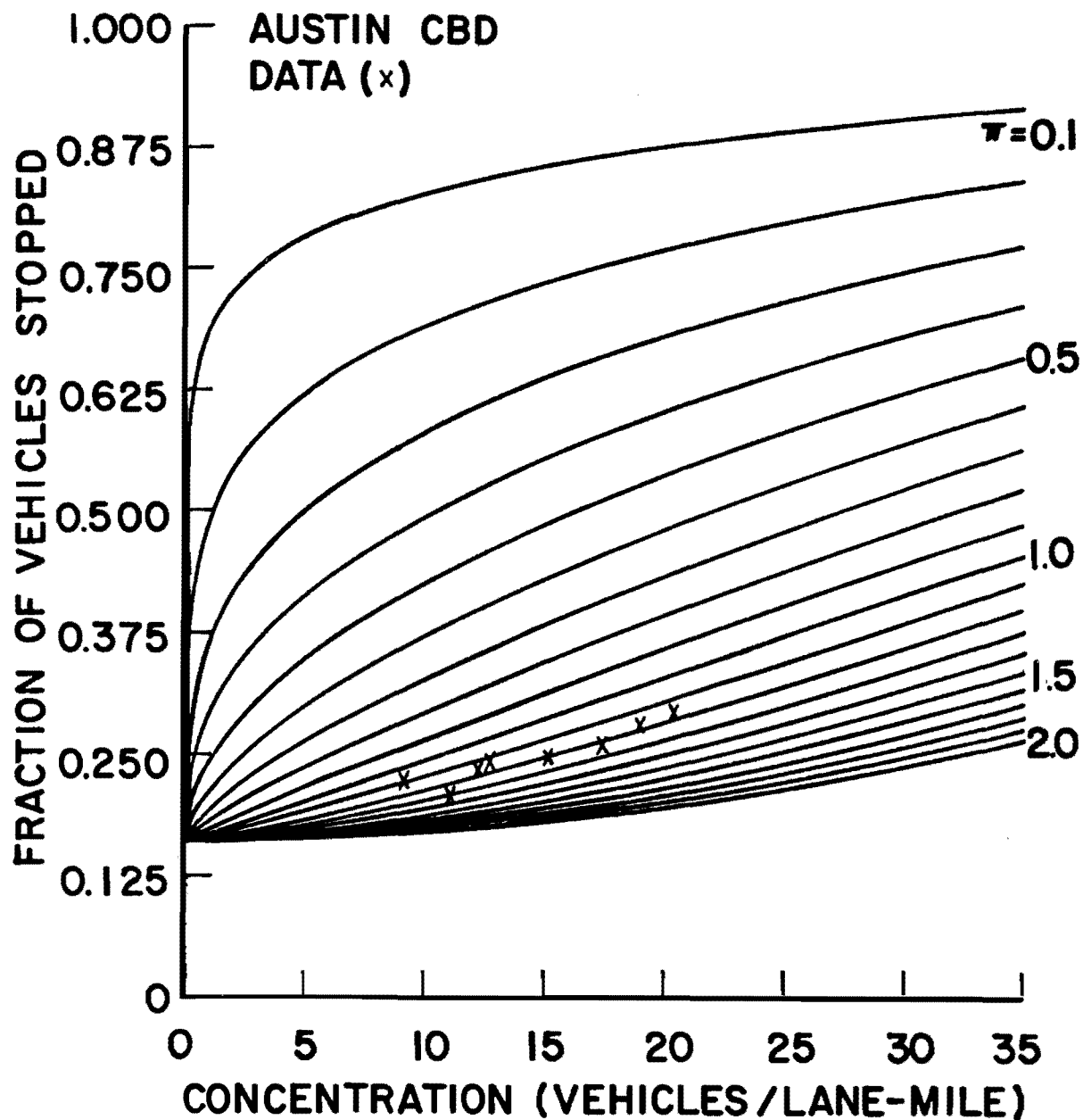


Figure 6.5. Family of fraction of vehicles stopped, f_s , versus concentration, k , according to Eq (6.4) for parameters $f_{s,\min} = 0.161$, $k_m = 100$ vehicles per lane-mile and π values in the range 0.1 to 2.0 in 0.1 increments. Also shown are the Austin CBD data points aggregated over each of the two flight strips and four observation periods. Calibration of Eq (6.4) for the data shown results in $\pi = 1.216$ and $f_{s,\min} = 0.161$ with $R^2 = 0.90$.

TABLE 6.2. RESULTS OF THE CALIBRATION OF THE MODEL $f_s = a + (1 - a)(k/k_m)^\pi$ FOR THE AUSTIN CBD

Type of Data	$f_s = a + (1 - a)(k/k_m)^\pi$			
	π	a	R ²	R ² _{max}
Unaggregated (71 points)	1.177	0.153	0.527	0.528
Aggregated over $\Delta k = 1$ (17 points)	1.276	0.168	0.802	0.804
Aggregated over each strip and time period (8 points)	1.216	0.161	0.902	0.919

NOTE: The model was calibrated using unaggregated data as well as two sets of aggregated data. The aggregation was once made over concentration intervals of 1 vehicle/lane-mile and the second time over each flight strip and time period. The maximum possible R² for an unconstrained regression is also shown. A $k_m = 100$ vehicles/lane-mile has been used.

Dallas CBD are plotted in Fig 6.6 along with a family of curves for $f_{s,\min} = 0.176$ and π values in the range of 0.1 to 2.0. While $f_{s,\min}$ in Austin is slightly greater than in Dallas (8 percent greater) the value of π in Austin is greater than the Dallas value by 28 percent, implying an overall better quality of traffic service in the Austin CBD. These findings are found to be consistent with our earlier results [Ref 29] of a preliminary examination of the relation between f_s and k in the two cities.

Having established a relation between f_s and k , each point along a Two-Fluid trend can now be associated with a fraction of vehicles stopped as well as with a demand in terms of the vehicular concentration. This is shown for the Austin CBD2 trend in Figure 6.7. For example, a (T, T_s) point of (6.0, 2.1) minutes/mile on the Austin CBD2 Two-Fluid trend is associated with $f_s = T_s/T = 0.35$, and for this point a vehicular concentration can be calculated using Eqs 6.4 and 6.6 namely,

$$k = k_m \left(\frac{f_s - f_{s,\min}}{1 - f_{s,\min}} \right)^{\frac{1}{\pi}} \quad (6.7a)$$

or

$$k = 100 \left(\frac{0.35 - 0.161}{1 - 0.161} \right)^{\frac{1}{1.216}} = 29 \text{ vehicles/lane-mile} \quad (6.7b)$$

Note that for the Dallas CBD ($T_m = 1.79$ minutes/mile, $n = 1.62$) the same value of $f_s = 0.35$ corresponds to $(T, T_s) = (5.5, 1.9)$ minutes/mile and $k = 19.5$ vehicles/lane-mile. As can be seen, while under the same value of f_s in the two networks the T, T_s values are not significantly different, the Austin CBD accommodates 1.5 times greater vehicular concentration, signifying a better quality of traffic service.

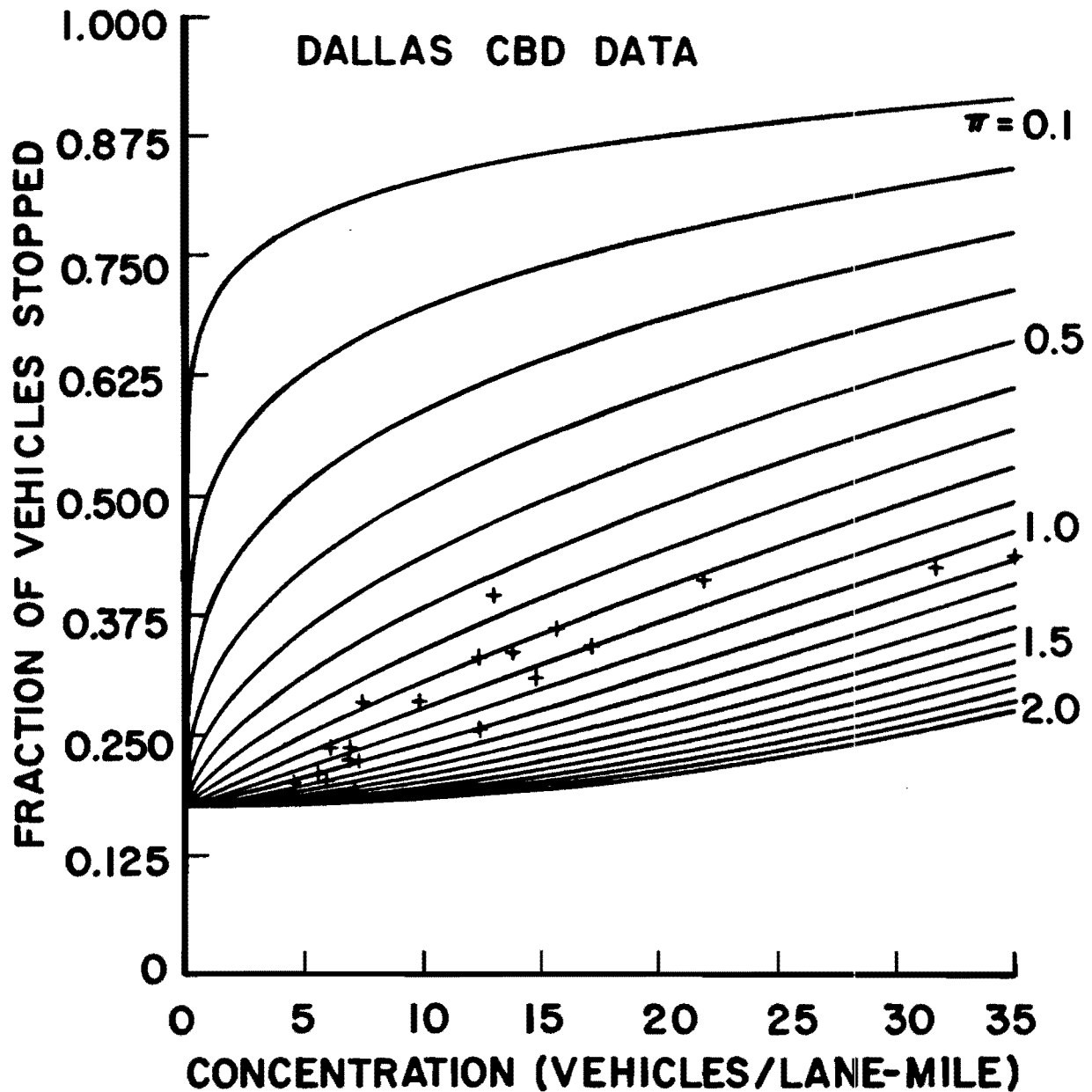


Figure 6.6. Family of fraction of vehicles stopped, f_s , versus concentration, k , according to Eq (6.4) for parameters $f_{s,\min} = 0.176$, $k_m = 100$ vehicles per lane-mile and π values in the range 0.1 to 2.0 in 0.1 increments. Also shown are the Dallas CBD 20 aerial photographic data points for which $f_{s,\min} = 0.176$ and $\pi = 0.950$ with $R^2 = 0.79$.

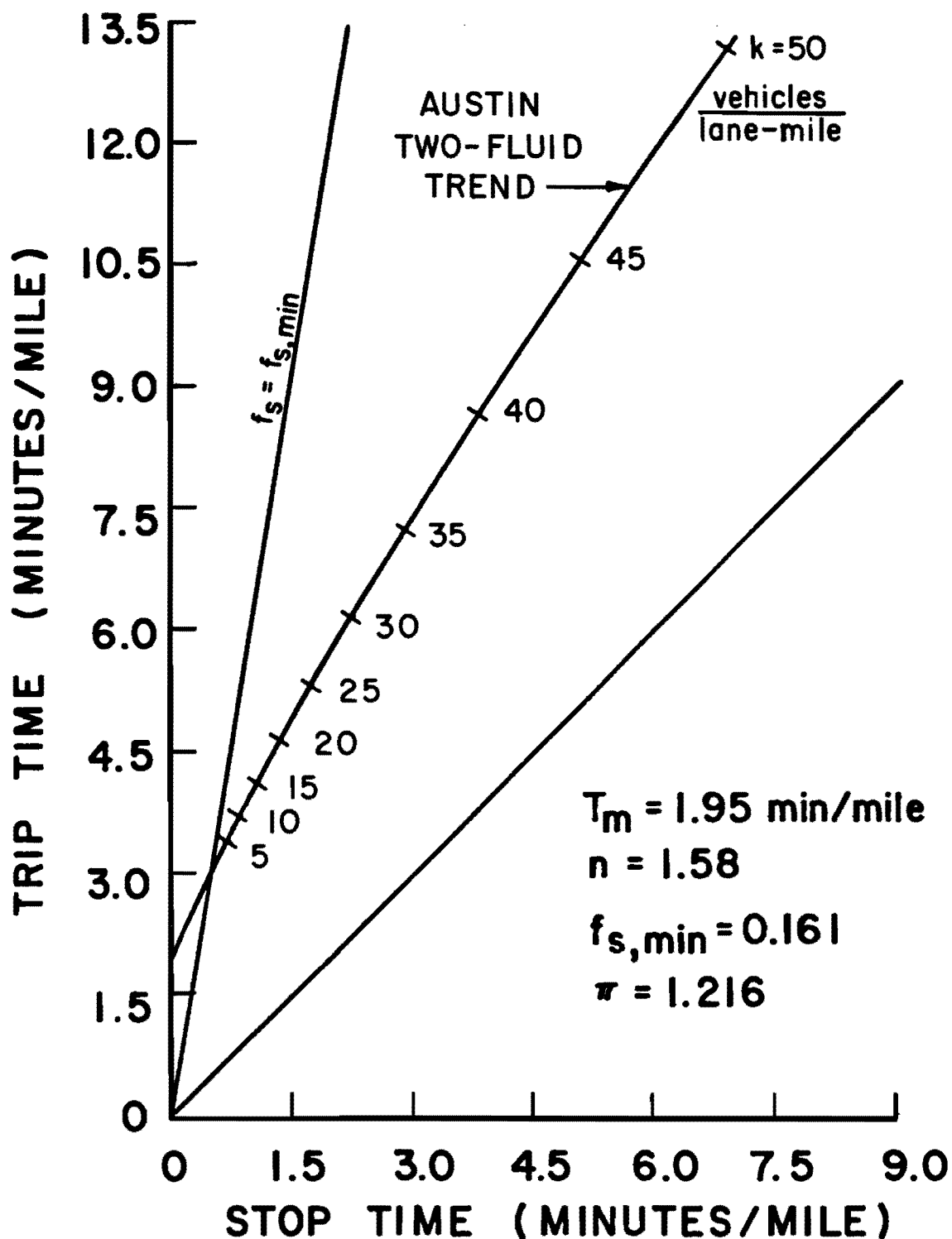


Figure 6.7. The Two-Fluid trend for the Austin CBD2 network ($T_m = 1.95$ minutes/mile, $n = 1.58$). Various points along this trend are associated with levels of concentration according to the fraction of vehicles stopped versus concentration relation for the Austin CBD2 network (Eq 6.6) with parameters $f_{s,min} = 0.161$ and $\pi = 1.161$. Also shown are lines of $f_{s,min}$ associated with $k = 0$ and $f_s = 1$ associated with $k = k_m$.

In the context of the Two-Fluid model, we have compared the quality of traffic service of various street networks under the same fraction of vehicles stopped. However, the same fraction of vehicles stopped in different networks is not necessarily equivalent to the same vehicular demand as shown above for Dallas and Austin. Nonetheless, a comparison under similar traffic loading conditions is possible once the parameters $f_{s,\min}$ and π of Eq 6.4 are determined for the networks of interest.

RELATION BETWEEN SPEED AND CONCENTRATION

Equation 6.4 when combined with the Two-Fluid assumption, $v = v_m (1 - f_s)^{n+1}$, implies a relation between the average speed and concentration, namely,

$$v = v_m \left[1 - f_{s,\min} - (1 - f_{s,\min}) (k/k_m)^\pi \right]^{n+1} \quad (6.8a)$$

or

$$v = v_m (1 - f_{s,\min})^{n+1} \left[1 - (k/k_m)^\pi \right]^{n+1} . \quad (6.8b)$$

Figure 6.8 shows a family of v/v_m versus k/k_m curves based on Eq 6.8b for values in the range of 0.1 to 2.0 in 0.1 intervals. Williams, et al [Ref 69] in their NETSIM simulation runs have also obtained a family of v versus k curves resembling the curves of Fig 6.8 for $\pi < 1$. In constructing Fig 6.8, values of $v_m = 30.77$ mph ($T_m = 1.95$ minutes/mile), $n = 1.58$ and $f_{s,\min} = 0.161$ have been used. These values correspond to the Austin CBD network in which aerial photographic data is available. As can be seen in Fig 6.8, in comparing the performance of various networks, those with the larger values of π are associated with a greater average speed for a given vehicular concentration. Hence, the greater the value of π , the better the quality of traffic service.

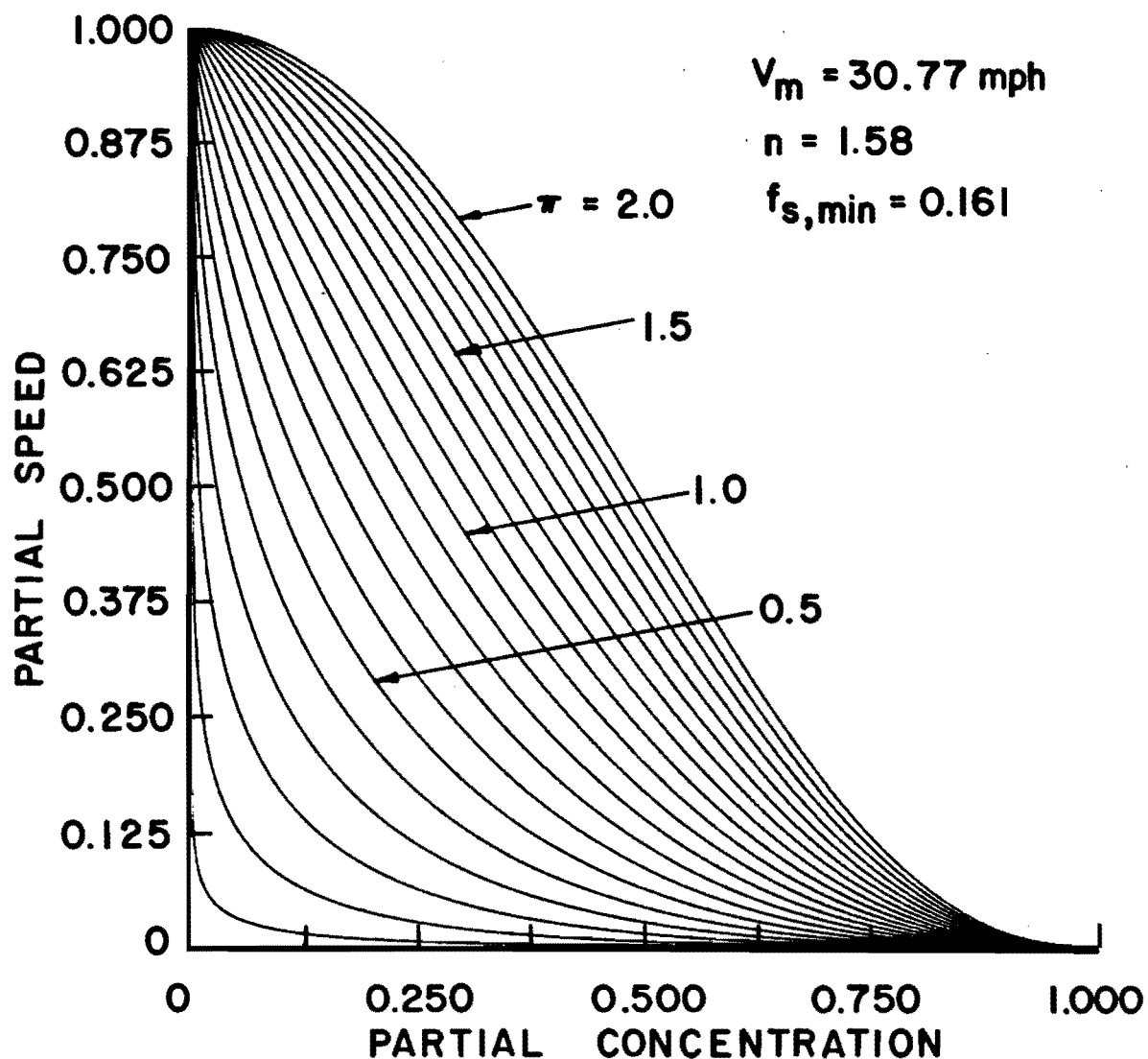


Figure 6.8. Family of partial speed, v/v_m , versus partial concentration, k/k_m , for the Austin CBD2 network ($v_m = 30.77 \text{ mph}$, $n = 1.58$, $f_{s,\min} = 0.161$) for π values in the range 0.1 to 2.0 in 0.1 increments.

Using the aerial photographic data for the Austin CBD, the parameters $f_{s,\min}$ and π in Eq 6.8b have been determined. Again, three sets of data are used: the 71 unaggregated data points, the 17 data points aggregated over Δk intervals of 1 vehicle/lane-mile, and the 8 data points aggregated over four time periods and two flight strips. As shown in Table 6.3, the best fit ($R^2 = 0.790$) is obtained using the last set of data resulting in a $\pi = 1.239$ and a $f_{s,\min} = 0.181$. These values of π and $f_{s,\min}$ are to be compared to $\pi = 1.216$ and $f_{s,\min} = 0.161$ (Table 6.2) obtained through the regression of f_s against (k/k_m) using the same data set. This corresponds to a two percent difference in π and an 11 percent difference in $f_{s,\min}$. Hence, for $\pi = 1.239$ and $f_{s,\min} = 0.181$, the following relation between average speed and concentration is obtained for the Austin CBD,

$$v = (18.38 \text{ mph}) \left[1 - (k/k_m)^{1.239} \right]^{2.58} \quad (6.9)$$

Figure 6.9 shows a graphical representation of Eq 6.9 as a solid curve. Also shown in Fig 6.9 are the data used in determining the parameters π and $f_{s,\min}$ used in Eq 6.9. The dashed curve in Fig 6.9 represents the speed-concentration relation inferred from the relation between fraction of vehicles stopped and concentration (Eq 6.6), i.e. $\pi = 1.216$ and $f_{s,\min} = 0.161$. While the differences between the solid and the dashed curves are not highly significant, evaluation of the values of π and $f_{s,\min}$ through a regression of f_s against k is preferable since it does not involve assumptions about values of v_m and n . The latter point is further apparent through a comparison of the R^2 obtained in regressing f_s against k ($R^2 = 0.90$) versus that in regressing v against k ($R^2 = 0.79$). Therefore, in our discussions hereafter we will use $\pi = 1.216$ and $f_{s,\min} = 0.161$ for the

TABLE 6.3. RESULTS OF THE CALIBRATION OF THE MODEL

$$v = v_m (1 - f_{s,\min})^{n+1} [1 - (k/k_m)^\pi]^{n+1}$$

FOR THE AUSTIN CBD

Type of Data	$v = v_m (1 - f_{s,\min})^{n+1} [1 - (k/k_m)^\pi]^{n+1}$			
	π	$f_{s,\min}$	R^2	R^2_{\max}
Unaggregated (71 points)	1.272	0.185	0.320	0.321
Aggregated over $\Delta k = 1$ (17 points)	1.341	0.197	0.724	0.724
Aggregated over each strip and time period (8 points)	1.239	0.181	0.790	0.808

NOTE: The model was calibrated using unaggregated data as well as two sets of aggregated data. The data aggregation was once made over concentration intervals of 1 vehicle/lane-mile and the second time over each flight strip and time period. The regressions of v against $[1 - (k/k_m)^\pi]^{n+1}$ were made subject to a zero intercept constraint. The maximum R^2 for an unconstrained regression is also known. A $k_m = 100$ vehicles/lane-mile, $v_m = 30.77$ mph, and $n = 1.58$ have been used.

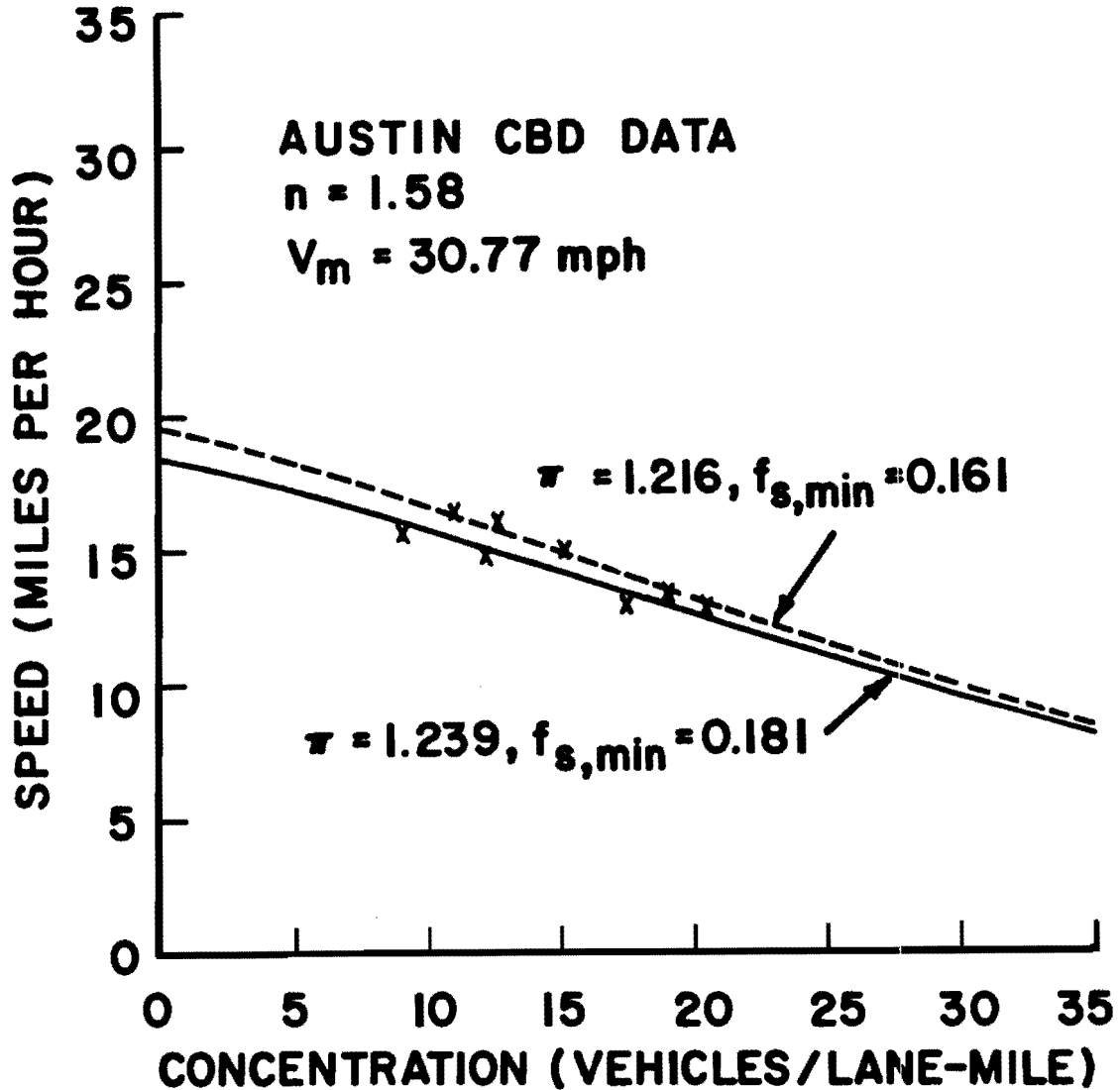


Figure 6.9. Speed versus concentration for the Austin CBD aerial photographic data shown. The solid curve is a regression fit to the data shown resulting in parameters $\pi = 1.239$ and $f_{s,\min} = 0.181$ (see Eq 6.9) with $R^2 = 0.79$. The dashed curve represents the speed-concentration relation inferred from the relation between fraction of vehicles stopped and concentration (Eq 6.6) with parameters $\pi = 1.216$ and $f_{s,\min} = 0.161$.

Austin CBD as obtained from a regression of f_s against k for the data aggregated over two strips and four time periods.

SPEED, FLOW, AND CONCENTRATION RELATIONS

The time-lapse aerial photographs of the Austin CBD were taken over two strips ~ 4500 feet wide and ~ 3 miles long. The photographs were taken on November 12, 1982 from 3:35 to 3:45 p.m. and from 4:55 to 5:05 p.m. During these periods 29 observers were assigned to seven categories of streets. The number of observers assigned to streets of each category was proportional to the fraction of total lane-miles contained in each category. Weighted mean flows were determined based on the four ten-minute ground flow observations corresponding to the aerial photographic observation periods. The weighted mean values were computed according to the procedure outlined in the section on flow measurements in Chapter 5.

The aerial and ground observations of speeds, concentrations and flows during the two ten-minute periods and along the two strips result in four independent observations of speed, v , concentration, k , and flow, q , as shown in Table 6.4. A regression of \bar{q} against the product of \bar{k} and \bar{v} using the data of Table 6.4 yields the relation

$$\bar{q} = \beta \bar{k} \bar{v} \quad (6.10)$$

where $\beta = 1.023$. The regression is, of course, forced through the origin to meet the boundary conditions that as \bar{k} approaches zero, so will the mean flow \bar{q} .

A hypothesis may then be tested to see whether $H_0: \beta = 1$ or $H_1: \beta \neq 1$. For this purpose a t -statistic with three degrees of freedom is calculated for β as follows:

TABLE 6.4. INDEPENDENT OBSERVATIONS OF THE MEAN SPEED, CONCENTRATION, AND FLOW IN THE AUSTIN CBD OVER TWO PHOTOGRAPHIC STRIPS AND TWO 10-MINUTE PERIODS

Strip	Period	Speed, \bar{v} (mph)	Concentration, \bar{k} (vehicles/lane-mile)	Flow, \bar{q} (vehicles/lane-hour)
1	3:35 - 3:45 P.M.	14.54	12.1	196
1	4:55 - 5:05 P.M.	12.64	17.3	280
2	3:35 - 3:45 P.M.	16.18	10.9	140
2	4:55 - 5:05 P.M.	14.73	15.0	190

$$t = \frac{\beta_{\text{estimate}} - \beta_{\text{Ho}}}{s(\beta)} = \frac{1.023 - 1}{0.116} = 0.198 \quad (6.11)$$

where $s(\beta)$ is the standard deviation of the estimated β . Since $t = 0.198$ corresponds to a level of significance greater than 85 percent, then if we reject the null hypothesis ($H_0 : \beta = 1$), there is a high probability (85 percent) that a correct hypothesis has been rejected. In other words, the very small value of t ($= 0.198$) indicates that the difference between β and its hypothesized value of 1 is statistically not significant.

The above analysis enables us to postulate that in urban street networks the relation among the average speed, flow, and concentration is similar to the relation among these variables along a single roadway, namely

$$\bar{q} = \bar{k} \bar{v} \quad (6.12)$$

Equation 6.12 can be combined with Eq 6.8b to give

$$q = kv_m (1 - f_{s,\min})^{n+1} \left[1 - (k/k_m)^\pi \right]^{n+1} \quad (6.13a)$$

or

$$q = Kq_m (1 - f_{s,\min})^{n+1} (1 - K^\pi)^{n+1} \quad (6.13b)$$

where $K = k/k_m$ is the partial concentration and $q_m = k_m v_m$. Note that $q_m = k_m v_m$ is not physically possible since when the concentration approaches k_m , the speed will approach zero and not v_m .

Equation 6.13b provides a relation between the average flow and concentration. Figure 6.10 shows a family of q versus k curves for the fixed values of v_m and n for the Austin CBD and π values in the range 0.1 to 2.0.

The maximum flow in a network may then be estimated by solving the equation

$\frac{dq}{dK} = 0$ for K and q , as follows:

$$\frac{dq}{dK} = q_m (1 - f_{s,\min})^{n+1} \left[(1 - K^\pi)^{n+1} - \pi(n+1)K^\pi (1 - K^\pi)^n \right] = 0 \quad (6.14)$$

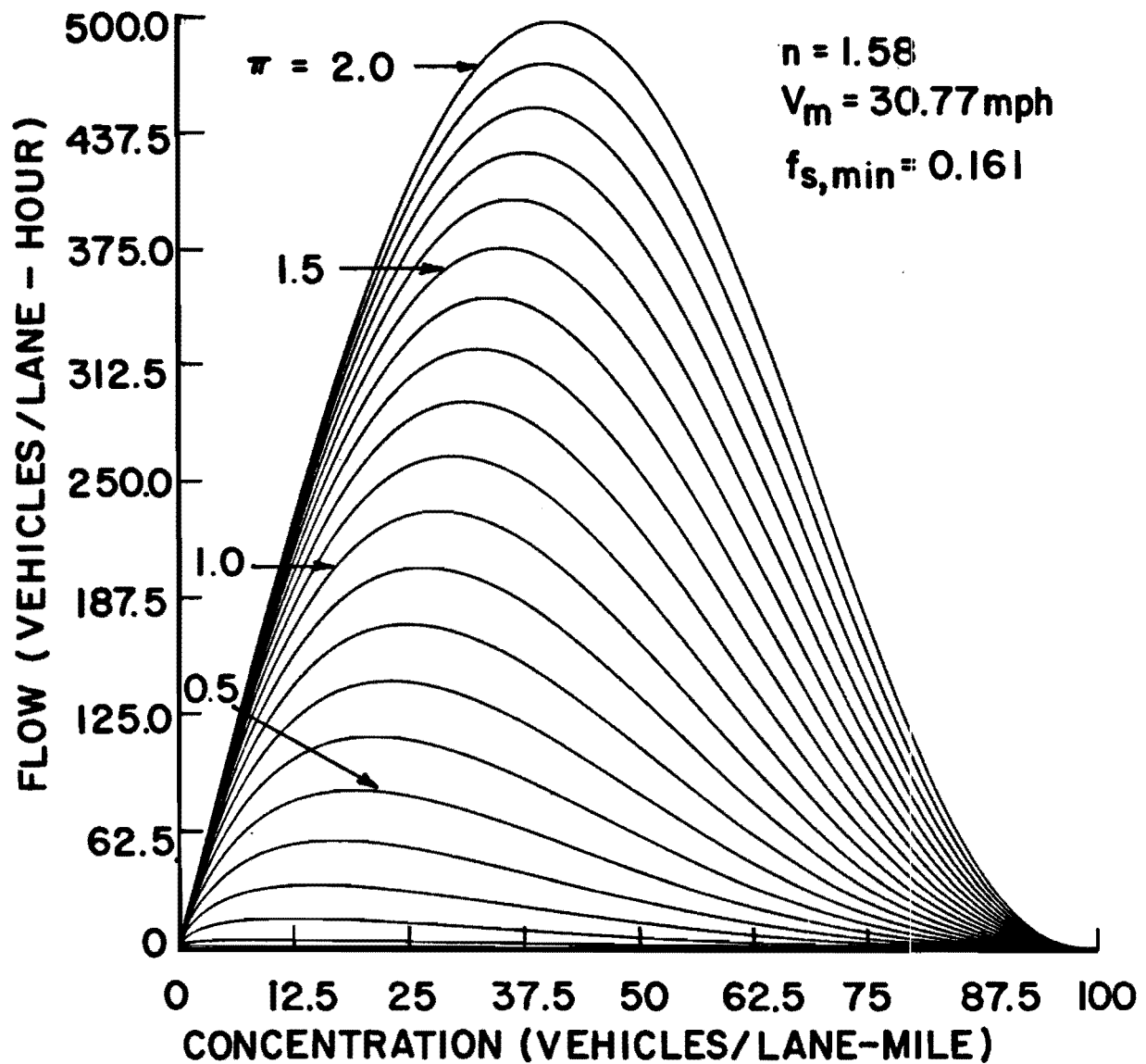


Figure 6.10. Family of flow versus concentration curves (Eq 6.13b) for the fixed values of v_m, n and $f_{s,\min}$ of the Austin CBD2, $k_m = 100$ vehicles per lane-mile, and π values in the range 0.1 to 2.0 in 0.1 increments.

From the above equation we obtain the partial concentration for which q is maximum, i.e.,

$$K(q_{\max}) = [1 + \pi(n + 1)]^{-\frac{1}{\pi}} \quad (6.15)$$

Combining Eqs 6.13b and 6.15 we can then write

$$q_{\max} = q_m (1 - f_{s,\min})^{n+1} \frac{[\pi(n + 1)]^{n+1}}{[1 + \pi(n + 1)]^{n+1 + 1/\pi}} \quad (6.16)$$

Figure 6.11 shows a family of speed versus concentration curves based on Eq 6.8b and using the values of v_m , n , and $f_{s,\min}$ for the Austin CBD (30.77 mph, 1.58, and 0.161 respectively) and π values in the range of 0.1 to 2.0. Also shown in Fig 6.11 are the speed-concentration data from the Austin aerial photographs and the locus of the maximum flow, q_{\max} of Eq 6.16 for v_m , n and $f_{s,\min}$ values for the Austin CBD at different values of π . As discussed earlier a non-linear regression through the data shown in Fig 6.11 yields $\pi = 1.216$ and $f_{s,\min} = 0.161$ which implies a maximum flow of $q_{\max} = 298$ vehicles/lane-hour in the Austin CBD ($T_m = 1.95$, $n = 1.58$) occurring at $k(q_{\max}) = 31.1$ vehicles/lane-mile and $v(q_{\max}) = 8.36$ mph corresponding to a maximum flow of 224 vehicles/lane-hour are obtained. Once again note that although the CBD networks of Austin and Dallas have similar T_m and n values, the Austin CBD with a higher value of π yields a greater q_{\max} and thus a better quality of traffic service. This point is further illustrated in Fig 6.11 in which the locus of q_{\max} ascends in the direction of increasing π values.

The data in Table 6.4 also enables us to examine a network performance relation known as the α -relation, formulated by Zahavi in 1972 [Ref 15]. Zahavi has postulated that for a given urban street network

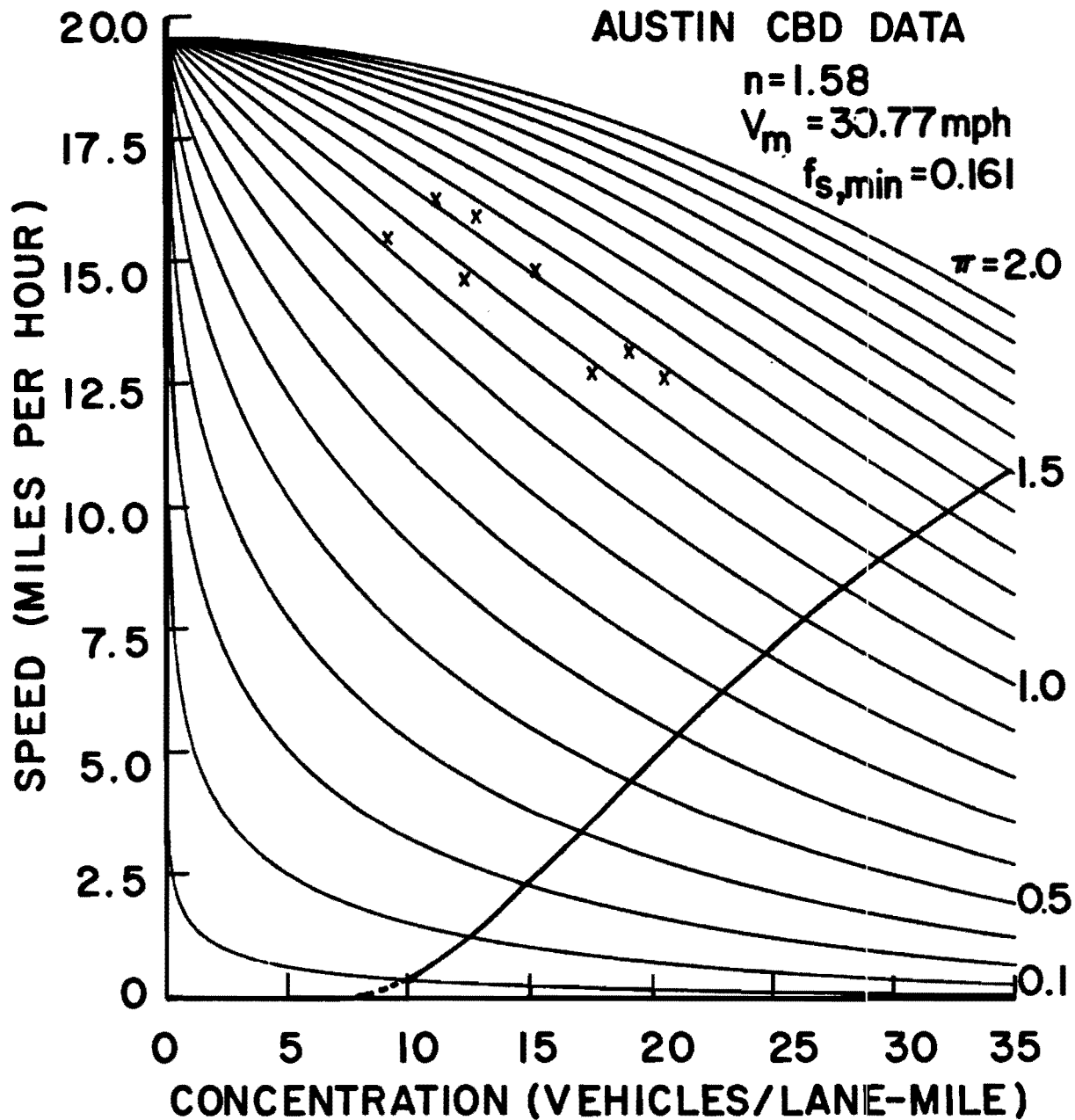


Figure 6.11. Family of speed versus concentration curves (Eq 6.8b) for the fixed values of v_m, n and $f_{s,\min}$ of the Austin CBD2, $k_m = 100$ vehicles per lane-mile, and π values in the range of 0.1 to 2.0 in 0.1 increments. Also shown are the aggregated aerial photographic data in Austin as well as the curve representing the locus of the maximum flow, q_{\max} , according to Eq (6.16) for the Austin CBD2 parameters. Note that for the Austin parameters $q_{\max} = 298$ vehicles per lane-hour corresponding to $k = 31.1$ vehicles per lane-mile and $v = 9.58$ mph.

$$qv = kv^2 = \alpha = \text{constant} \quad (6.17)$$

where q , v , and k are the average vehicular flow, speed, and concentration, respectively. According to Zahavi the parameter α is supposed to be a measure of the quality of traffic service in the network. Zahavi argues that α can be thought of as the "kinetic energy" of the traffic through the resemblance of kv^2 to $1/2 mv^2$, where m is the physical mass [Ref 70]; and that it represents the interaction of flow and speed which is related to the ability of a roadway network to hold and pass through it a certain amount of traffic "kinetic energy". Hence he suggested that the higher α , the better the quality of traffic service.

The data in Table 6.4 yield two values of α for strip 1, namely $\alpha = 2850$ and 3539 vehicle-miles/(lane hour²). They also provide two α values for strip 2, i.e. $\alpha = 2265$ and 2799 vehicle-miles(lane hour²). These results are tabulated in Table 6.5. The deviation of α within each strip is ~ 21 percent of their respective means. In addition, the deviation of the average values of α for each of the two strips from the overall mean is 23 percent. The above results indicate that the variation of α within each strip is as great as the variation across strips.

The data in Tables 6.4 and 6.5 also indicate a high positive correlation between α ($= qv$) and the concentration k , i.e. ($R^2 = 0.79$). Such a significant correlation level is contradictory to the hypothesis that the quantity α ($= qv$) is constant. Buckley and Wardrop [Ref 16] have also shown that α is highly correlated with the space mean speed which is itself highly correlated with concentration.

According to Zahavi's α parameter concept (Eq 6.17), a relation between the average concentration and speed is implied, namely,

TABLE 6.5. THE α -PARAMETER VALUES FOR THE TWO PHOTOGRAPHIC STRIPS OVER THE AUSTIN CBD DURING TWO OBSERVATION PERIODS

Strip	Period	$\alpha = \bar{q} \bar{v}$ (vehicle-miles/lane/hour ²)	Concentration, \bar{k} (vehicles/lane-mile)
1	3:35 - 3:45 P.M.	2850	12.1
1	4:55 - 5:05 P.M.	3539	17.3
2	3:35 - 3:45 P.M.	2265	10.9
2	4:55 - 5:05 P.M.	2799	15.0

NOTE: Also given are the concentration values. The correlation coefficient between α and k is +0.89.

$$\bar{v} = \left(\frac{\alpha}{k} \right)^{\frac{1}{2}} . \quad (6.18)$$

A regression of v against $(k)^{-1/2}$, forced through the origin, for the Austin CBD data yields $\alpha = 2794$ vehicle-miles/(lane hour²). Equation 6.18 with $\alpha = 2794$ is plotted in Fig 6.12. In addition, Fig 6.12 shows the curve based on the Two-Fluid v, k relation of Eq (6.8b) calibrated for the Austin CBD data which is also shown in the figure. The corresponding parameters are $\pi = 1.239$ and $f_{s,\min} = 0.181$, with $R^2 = 0.79$. In Fig 6.12, despite the relatively short range of the data it appears that the Two-Fluid trend of Eq (6.8b) is more reasonable than Zahavi's α -relation (Eq 6.18) since as $k \rightarrow 0$ the speed must approach the finite value $v(k = 0)$. The point shown in asterisk near the intercept in Fig 6.12 is a speed observation made for the very low vehicular concentration at midnight and early morning hours in the Austin CBD network. Note that while this point has not been used in establishing the two trends in Fig 6.12, it nevertheless lies very close to the intercept of the trend of Eq (6.8b). On the other hand, at very high concentration levels the average speed must approach zero at a faster rate than implied by the trend of Eq (6.18) because of the existence of a finite jam concentration. A physical limit to the level of concentration is reached at 176 vehicles/lane-mile, assuming that each vehicle occupies an average of 30 feet of the lane length. Operationally, however, because of instability in flow of vehicles in a platoon at very high concentrations, a jam concentration would probably occur at concentration values lower than 176 vehicles/lane-mile.

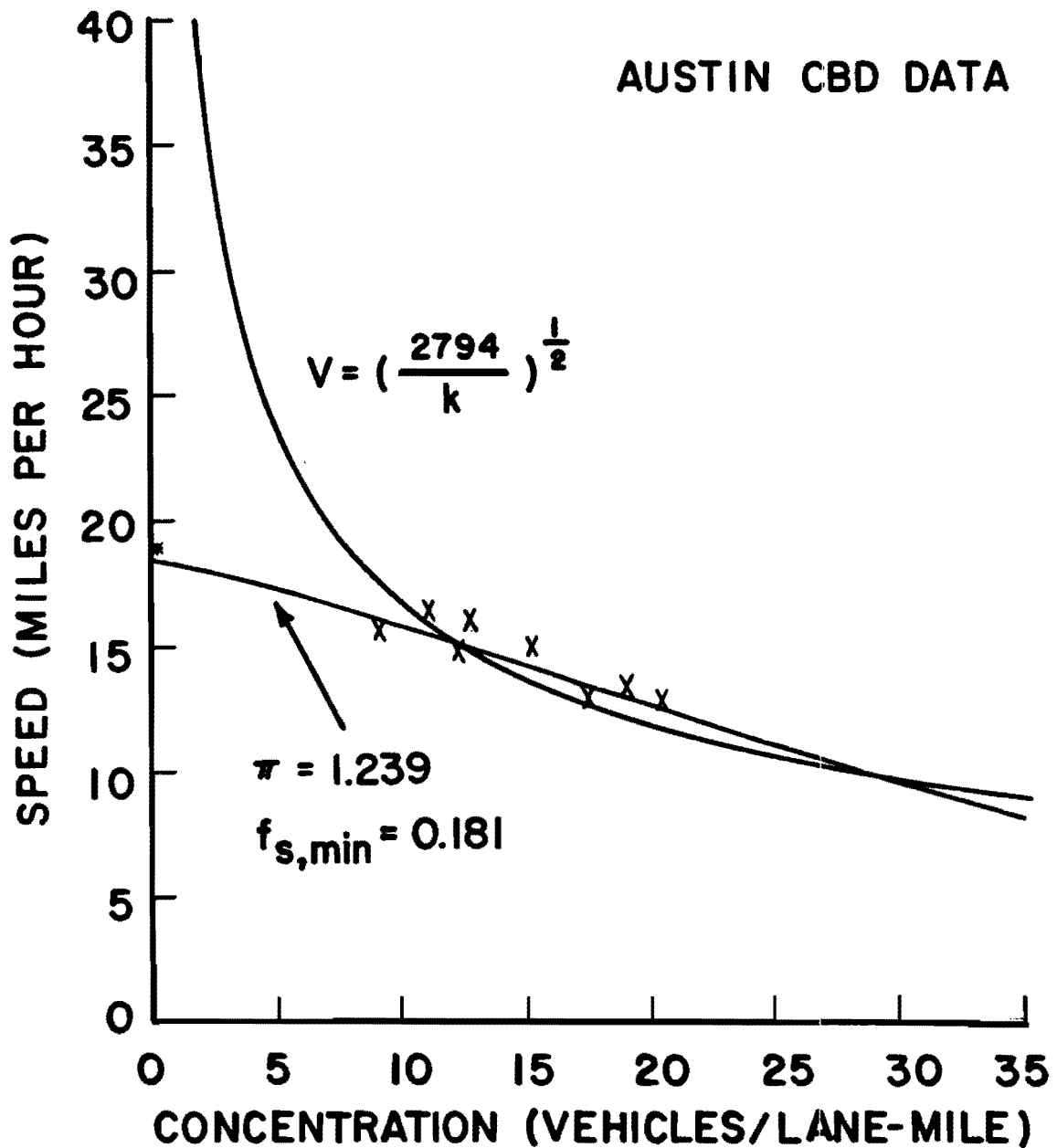


Figure 6.12. Speed versus concentration curves according to Zahavi's α -concept (Eq 6.18) and the Two-Fluid relation (Eq 6.8b) both calibrated for the Austin aerial photographic data shown with X. The point shown as asterisks near $k = 0$ is a speed observation made in the Austin CBD during midnight and early morning hours. Note that while this point has not been used in calibrating either of the two trends, it lies very close to the intercept of the Two-Fluid trend of Eq (6.8b).

SUMMARY

In assessing the quality of traffic service in a street network, it is helpful to study the relations among network-wide variables such as the averages of speed, flow, concentration, and fraction of vehicles stopped. Such relations allow comparisons of the level of performance of networks under similar traffic loading conditions, and in addition provide a measure of the productivity of the network through variables such as the vehicular flow. In this context, it is also important to note the rate at which the average speed in a network decreases if the fraction of vehicles stopped increases with increases in the concentration. Such rates are themselves measures of the quality of traffic service in an urban street network.

Data from time-lapse aerial photographic observations in Austin and Dallas have been analyzed and used to establish relations among averages of speed, concentration, fraction of vehicles stopped, and flow. These relations have involved two parameters, namely $f_{s,\min}$ which is an estimate of the average minimum fraction of vehicles stopped in a network and π which is a parameter. We have shown that the parameters $f_{s,\min}$ and π can be used as measures of the quality of traffic service. As an example the CBD networks of Dallas and Austin are compared through the values of T_m , n , π , and $f_{s,\min}$.

In determining the values of parameters π and $f_{s,\min}$ for the Austin CBD, similar results are obtained using speed versus concentration data or fraction of vehicles stopped versus concentration data. For each data set, however, the values of π and $f_{s,\min}$ depend on an assumption regarding the value of the maximum or jam concentration in the network, k_m . A value of $k_m = 100$ vehicles/lane-mile has been used in our analysis. It is not clear whether this parameter itself is a function of the quality of traffic service

or its value is constant for all networks. We have assumed the latter in our analysis. In the case of speed versus concentration data, values of the Two-Fluid model parameters T_m and n are also necessary for the determination of $f_{s,\min}$ and π .

On-the-ground flow observations made simultaneously with the aerial photographs have also been used in establishing relations among the averages of flow, speed and concentration. Our limited data suggests that like arterial traffic, the traffic flow in a street network may also be described by $\bar{q} = \bar{k}\bar{v}$ conditioned upon the proper definitions of the averages involved in the relation. Finally, the independent observations of flow, speed, and concentration suggest that the Zahavi's α parameter, namely $\alpha = qv$ is not constant over our study network. Rather it is found that α is highly correlated ($R^2 = 0.79$) with the vehicular concentration in the network and consequently is not particularly constant for a given network.

CHAPTER 7. SUMMARY AND DISCUSSION

Some years ago a Two-Fluid model of town traffic was developed by Herman and Prigogine [Ref 21] as an outgrowth of their Kinetic Theory of Multilane Traffic [Ref 31]. The work reported in this dissertation is an attempt to validate, expand and apply the Two-Fluid theory as a means of characterizing and rank ordering the quality of traffic service in urban street networks. A summary and discussion of the results including possible extensions of the work are presented below.

SUMMARY OF RESULTS

Experiments and observations have been performed to validate the two basic assumptions of the Two-Fluid model, namely, that (1) the average running speed of the traffic in a street network is proportional to the n th power of the fraction of the vehicles that are moving, and (2) the fractional stop time of a test vehicle circulating in a network is equal to the average fraction of vehicles stopped, i.e. $T_s/T = f_s$, during the same period. A series of four ergodic experiments were carried out in Austin during a two-year period. The results have shown that for any given number of vehicles the value of $\langle T_s/T \rangle$ is identical to $\langle f_s \rangle$. Moreover, with various numbers of test vehicles, each employing the chase-car technique of passively following 'random vehicles', we find that the mean fraction of time stopped for each test vehicle closely estimates the sample mean taken over all the test vehicles, and as such is an estimator for the population mean $\langle T_s/T \rangle_p$. The overall results show that the fractions of the time stopped for the test vehicles were narrowly distributed with a coefficient of variation of only 11 percent.

This then shows, for a given test vehicle, that $(T_s/T)_i \approx \langle T_s/T \rangle_t$ and therefore $(T_s/T)_i \approx \langle f_s \rangle_t$. Simultaneous aerial photographic observations have, on the other hand, shown that $\langle f \rangle_{s\ t} \approx \langle f \rangle_{s\ p}$. The results have also been shown to verify the first assumption, namely, $v_r = v_m f_r^n$.

A discussion is given of how T, T_s data and the parameters T_m and n of the Two-Fluid model are obtained and used to compare the quality of traffic service in a number of Texas cities, namely, Austin, Dallas, Houston, Lubbock, and San Antonio as well as Albuquerque, and Matamoros, Mexico. Comparisons are made with other cities around the world. In examining the data from the cities studied and comparing the Two-Fluid model for those cities, it was noted that (1) there existed relations between T, T_s, T_r and the number of stops per unit distance, N_s , and (2) these relations had significantly different parameter values for those cities with different values of T_m and n . In general it is shown that for a given increase in the fraction of vehicles stopped, that system has the better quality of traffic service which has smaller values of the parameters T_m and n and therefore the smaller associated increase in T_s as well as in T_r . This method of ranking the quality of traffic service can be employed using a single vehicle and collecting data for several hours over a few days, and therefore provides a rather simple and efficient engineering technique.

Before and after studies were also conducted in the CBD networks of Dallas, San Antonio, and Lubbock to determine whether the Two-Fluid methodology is sufficiently sensitive to detect major modifications in network control strategies, street patterns, etc. In the before/after studies in Dallas and San Antonio the Two-Fluid model parameters did not change significantly. However, the distributions of the (T, T_s) data along

the Two-Fluid trend for the before/after cases in Dallas and San Antonio were significantly different.

On the other hand, the before/after studies in Lubbock resulted in considerable changes in the parameters T_m and n . The Lubbock after study indicated an 11 percent increase in T_m and a 27 percent decrease in n compared to the before case, thus implying a worsening of the quality of service during off-peak periods and an improvement during peak periods. The before/after studies in the three cities have shown that Two-Fluid representation can be used as a means of assessing the impact of major changes in urban traffic networks. In addition, comparisons of the Two-Fluid trends in cities with highly different network characteristics have shed light on the specific network features that may most strongly affect the quality of traffic service and thus the Two-Fluid model parameters.

The effect of the vehicular mode used in the collection of data on the Two-Fluid model parameters has been studied. Data have been obtained from buses and a passenger car on the same University of Texas shuttle bus fixed routes. The application of the Two-Fluid model to these data has indicated that the parameter n is significantly greater when buses rather than the test passenger cars were used to sample the traffic on the same routes. It then appears that different vehicular types may have different 'perceptions' of the quality of traffic service in the same street network, i.e. they interact in a different manner with the overall population of vehicles.

Time lapse aerial photographs of the Austin CBD network have provided direct observations of averages of speed, concentration, and fraction of vehicles stopped. Simultaneous ground experiments and observations have resulted in estimates of average speed, fraction of vehicles stopped as well as average flow in the study network. In addition, aerial photographs taken

over the Dallas CBD have been used to determine the fraction of vehicles stopped, f_s , at different concentration levels, k . The resulting data have been used to establish relations among the fraction of vehicles stopped, concentration, speed, and flow as well as to examine the accuracy of the ground estimates of speed and fraction of vehicles stopped. The relation between f_s and k , or between v and k has given rise to two additional Two-Fluid model parameters, namely π and $f_{s,\min}$. In determining these parameters for Austin and Dallas a value of 100 vehicles per lane-mile has been assumed for the maximum concentration, k_m . The quality of traffic service in the Dallas and Austin CBD networks has been compared through the parameters π , $f_{s,\min}$, T_m and n . In general it has been shown for a given increase in demand, i.e. an increase in the fractional concentration k/k_m , that the system which has the larger value of the parameter π and the smaller values of the parameters T_m , n , and $f_{s,\min}$ has the better quality of traffic service.

LIMITATIONS AND EXTENSIONS

The Two-Fluid characterization of traffic in various cities around the world as well as the before/after studies have been helpful in identifying the physical features of an urban network which have the greatest influence on the quality of traffic service and thus on the Two-Fluid model parameters. From the standpoint of traffic engineering it is of utmost interest not only to characterize the quality of service in a network but also to be able to implement modifications which improve the service. It is from this vantage point that further work related to the sensitivity of the model parameters to changes in various network features is of special importance. Such studies, of course, would not be conclusive unless they include careful documentation of the changes involved.

Furthermore, calibration of the Two-Fluid model for a given street network using passenger cars as well as buses has begun to indicate that the values of the model parameters may vary depending on the type of vehicle used in data collection. It would be interesting to expand the scope of this preliminary investigation to include a wider range of test vehicles such as trucks, buses, vans, mini cars, and possibly motorcycles to study the interaction of these various vehicles with the population as a whole.

In the course of ground observations made to calibrate the Two-Fluid model the chase-car technique has been employed. This technique has been used in order to sample the network streets according to the manner and frequency of their utilization by the users of the network. A question not addressed in this body of work is the effect of driver behavior on the values of the parameters T_m and n . In the high trip time - stop time traffic regime, the behavior and desire of drivers are strongly influenced and dictated by the collective behavior of the surrounding traffic. However, in the low T, T_s regime individual drivers have greater control over their running speed. Therefore it is possible that in a given street network, data collected by a conservative versus an aggressive driver may result in significantly different values of T_m and n . The influence of driver behavior on the Two-Fluid formulation presents itself as a limitation requiring further investigation.

The chase-car technique has been used as a means of "randomizing" the routes taken by the test vehicle based on whose data the model is calibrated. Aside from the above-stated driver behavioral questions, another aspect is whether or not the Two-Fluid methodology can be applied to fixed rather than "random" routes. Preliminary studies bus routes in Austin have yielded favorable results regarding the application of the model to fixed routes.

This aspect of the work, however, remains a limitation of the methodology and warrants further studies. Along the same lines, it is also important to point out another limitation of the methodology regarding the impact of the selected network street boundaries on the outcome of the studies involving application of the chase-car technique. For example, if all the street boundaries consist of major arterials, as has been the case in most of our studies, then the test vehicles spend a significant fraction of the study duration circulating in the boundary streets. The effect on the model parameters must be the subject of further investigation.

Aerial photographic studies have also been used to establish a relation between fraction of vehicles stopped and concentration as well as relations among network-wide averages of speed, concentration and flow. Such relations allow comparisons of the quality of traffic service in various networks under similar traffic loading conditions. In addition, these relations provide a measure of the relative productivity of a network under a given demand level.

The relation between fraction of vehicles stopped, f_s , and concentration, k , includes the parameters $f_{s,\min}$, π and k_m . The determination of these parameters requires direct observations of f_s and k . While direct observations of f_s and k involve photography from some high vantage point, it is possible to estimate the value of these variables through ground observations of averages of speed, flow and fraction of time stopped. Efficient ways of estimating these variables through ground observations is another aspect of the present work which requires further study.

Moreover, in determining the parameters $f_{s,\min}$, π and k_m it is necessary to make an assumption regarding the value of one of the three parameters. In our analysis we have chosen an average maximum concentration

$k_m = 100$ vehicles per lane-mile for both the Austin and Dallas CBD networks. The maximum or jam concentration, k_m , is defined as the concentration level at which the traffic jams in the system as a result of instabilities. The value of k_m in a given network is, of course, subject to fluctuations and is not expected nor is implied to be a constant. It is for this reason that we speak, in a loose sense, of an average maximum concentration. The values of $f_{s,min}$ and π , however, depend on the specific value chosen for the average k_m . Furthermore, it is reasonable to assume that k_m itself is a network-specific parameter. Despite all the above difficulties, it must be noted that high correlation coefficients are obtained for the non-linear fit to the aerial photographic v versus k and f_s versus k data in Austin and Dallas when a $k_m = 100$ vehicles per lane-mile is employed.

The work reported here has been pursued with the conviction and hope that as a result of collective effects among the large number of pertinent traffic variables associated with an urban network various simple relations may be found that make it feasible to assess efficiently and simply the quality of traffic service in city street networks. In addition to attempting to characterize traffic in various metropolitan areas, we have also been searching for ways to evaluate relative 'quality of traffic service', 'traffic quality', and possibly 'quality of traffic system'. The term 'quality of traffic' might be reserved more to take into account the human behavioral aspects and the perceptions of the driver users of the system, an important aspect which has not been addressed within the scope of this work. The term 'quality of traffic system' then might be used to incorporate the attributes of both the quality of traffic service and the quality of traffic.

The overall results of the present research have indicated that it is, indeed, feasible to characterize, with rather minimal resources, the quality of traffic service in an urban traffic network according to the Two-Fluid model. Having done so, then for a given level of vehicular concentration, one can estimate the averages of quantities such as speed, flow, fraction of time and vehicles stopped, etc., in the network of interest. From the perspective of both short-term and long-term planning, the transportation analyst is then in a position, through the Two-Fluid methodology, to predict the resulting traffic conditions associated with a given demand in a network. Furthermore, having calibrated the Two-Fluid model for different zones of a city network or having established a contour map of the Two-Fluid parameter over a city network, difficult as it may be to accomplish, it is then possible to utilize the results as a tool for making assignments of trips destined from one city zone to another along various routes connecting the respective zones. Thus, the Two-Fluid model and the fuel consumption model together constitutes a relatively simple and a straight forward means of not only addressing specific traffic engineering questions such as the impact of changes in traffic control strategies, etc., but also addressing broader issues in urban planning and transportation economics in urban street networks.

REFERENCES

1. R. Herman. "Theoretical and Experimental Studies in Vehicular Traffic," Proceedings of the Third Conference of the Australian Road Research Board, 3, 25 (1966).
2. R. Herman, "Remarks on Traffic Flow Theories and the Characterization of Traffic in Cities," Self Organization and Dissipative Structures, (Editors: Schieve, W.C. and Allen, P.M.), pp. 260-284, University of Texas Press (1982).
3. R. Herman, "Remarks on Research in Transportation," in Proceedings of the Eight Symposium on Transportation and Traffic Theory, V.F. Hurdle, E. Haller and G.N. Steuart (eds.), University of Toronto Press, Toronto, 1983.
4. Highway Research Board, Committee on Highway Capacity, "Highway Capacity Manual," U.s. Department of Commerce, Bureau of Public Roads, Washington, D.C. (1950); also see the revised version of Highway Capacity Manual, Special Report No. 87, Highway Research Board, National Academy of Sciences, National Research Council, Publication 1328, 1965.
5. B.D. Greenshields, D. Shapiro and E.L. Ericksen, "Traffic Performance at Urban Street Intersections," Yale Bureau of Highway Traffic, Technical Report No. 1, New Haven, Connecticut (1947).
6. J.G. Wardrop, "The Capacity of Roads," Operational Research Quarterly, Vol. 5, No. 1, pp. 14-24, March (1954).
7. J.G. Wardrop and J.T. Duff, "Factors Affecting Road Capacity," International Study Week in Traffic Engineering, Stresa, 1-5, October, 1956.
8. J.G. Wardrop, "Traffic Capacity of Town Streets," Roads and Road Construction, Vol. 30, No. 350, pp. 39-42, and Vol. 30, No. 351, pp. 68-71 (1953).
9. District Planning Section, District 15, Texas State Department of Highways and Public Transportation, "San Antonio-Bexar County Long Range Transportation Plan - Basic Elements," Report No. 12-A, July 24, 1979.
10. T.P. Hutchinson, "Urban Traffic Speeds, - II: Relation of the Parameters of Two Simpler Models to Size of the City and Time of Day," Transportation Science, Vol. 8, No. 1, pp. 50-57 (1974); also see D.M. Branston, "Urban Traffic Speeds - I: A Comparison of Proposed Expressions Relating Journey Speed to Distance from a Town Center," Transportation Science, Vol. 8, No. 1, pp. 35-49 (1974).

11. R.J. Vaughan, A. Loannou, and R. Phylactou, "Traffic Characteristics as a Function of Distance to Town Center," *Traffic Engineering and Control*, Vol. 14, No. 5, pp. 224-227 (1972).
12. R.J. Smeed, "The Effect of the Design of Road Networks on the Intensity of Traffic Movement in Different Parts of a Town with Special Reference to the Effects of Ring Roads," *Construction Industry Research Information Association, Technical Note 17*, London (1971).
13. D.E. Blumenfeld, "Effects of Road System Design on Congestion and Journey Times in Cities," Ph.D. Thesis, University College London (1972).
14. Y. Zahavi, "Traffic Performance Evaluation of Road Networks by the α - Relationship," part 1, *Traffic Engineering and Control*, Vol. 14, No. 5, pp. 228-231 (1972).
15. Y. Zahavi, "Traffic Performance Evaluation of Road Networks by the α - Relationship," Part 2, *Traffic Engineering and Control*, Vol. 14, No. 6, pp. 292-293 (1972).
16. D.J. Buckley and J.G. Wardrop, "Some General Properties of a Traffic Network," *Australian Road Research*, Vol. 10, No. 1, pp. 21-31, (1980).
17. M.-F. Chang and R. Herman, "An Attempt to Characterize Traffic in Metropolitan Areas," *Transportation Science*, Vol. 12, No. 1, pp. 58-79 (1978).
18. R. Herman, R.G. Role and M.W. Jackson, "Fuel Economy and Exhaust Emissions Under Two Conditions of Traffic Smoothness," *Passenger Car Meeting*, Troy Hilton, Troy, Michigan, June 5-9, 1978, SAE Paper No. 780614.
19. T.M. Johnson, D.L. Formenti, R.F. Gray and W.C. Peterson, "Measurement of Motor Vehicle Operation Pertinent to Fuel Economy," presented at the SAE Annual Engineering Congress and Exposition, SAE Paper No. 750003, Detroit, February 24-28, 1975.
20. M.-F. Chang and R. Herman, "Trip Time Versus Stop Time and Fuel Consumption Characteristics in Cities," *Transportation Science*, Vol. 15, No. 3, pp. 183-209 (1981).
21. R. Herman and I. Prigogine, "A Two-Fluid Approach to Town Traffic," *Science*, Vol. 204, pp. 148-151 (1979).
22. L. Evans, R. Herman and T.N. Lam, "Multivariate Analysis of Traffic Factors Related to Fuel Consumption in Urban Driving," *Transportation Science*, Vol. 10, No. 2, pp. 205-215 (1976).
23. L. Evans, R. Herman and T.N. Lam, "Gasoline Consumption in Urban Traffic," *Society of Automotive Engineers*, SAE Paper 760048, 1976.
24. M.-F. Chang, L. Evans, R. Herman, and P. Wasielewski, "The Influence of Vehicle Characteristics, Driver Behavior, and Ambient Temperature on

- Gasoline Consumption in Urban Traffic," Transportation Research Record No. p. 599, pp. 25-30 (1976). The title was unfortunately shortened by editors to "Gasoline Consumption in Urban Traffic" and should not be confused with an entirely different paper with the same title published as SAE Paper No. 760048 (Reference 23).
25. C.A. Amann, W.H. Haverdink, and M.B. Young, "Fuel Consumption in the Passenger Car System," General Motors Research Laboratories, Research Publication GMR-1632 (1975).
 26. E. Pelensky, W.R. Blunden and R.D. Munro, "Operating Costs of Cars in Urban Areas," in Proceedings, Fourth Conference of the Australian Road Research Board, Vol. 4, Part 1, pp. 475-504 (1968).
 27. P.F. Overall, "The Effect of Road and Traffic Conditions on Fuel Consumption," RRL Report LR 226, Road Research Laboratory, Crowthorne, England (1968).
 28. G.J. Roth, "The Economic Benefits to be Obtained by Road Improvements, with Special Reference to Vehicle Operation Costs," Department of Scientific and Industrial Research, Road Research Laboratory, Research Note No. RN/3426/GJR, Harmondsworth (1959) (unpublished).
 29. R. Herman and S. Ardekani, "Characterizing Traffic Conditions in Urban Areas," *Transportation Science*, Vol. 18, No. 2, 1984.
 30. S. Ardekani, V. Torres Verdín and R. Herman, "El Modelo Bifluido y La Calidad del Tránsito en La Ciudad de México," *Revista Ingeniería Civil*, Colegio de Ingenieros Civiles de México, Submitted in March 1984 (In Press).
 31. R. Herman and S. Ardekani, "The Influence of Stops on Vehicle Fuel Consumption in Urban Traffic," *Transportation Science*, Vol. 19, No. 1, 1985 (In Press).
 32. I. Prigogine and R. Herman, "Kinetic Theory of Vehicular Traffic," American Elsevier, New York (1971). [See also R. Herman and T. Lam, "On the Mean Speed in the 'Boltzman-Like' Traffic Theory: Analytical Derivation," *Transportation Science*, Vol. 5, pp. 314-327 (1971); "----: A Numerical Method," *Transportation Science*, Vol. 5, pp. 418-429, (1971); and L.C. Edie, R. Herman and T.N. Lam, "Observed Multilane Speed Distribution and the Kinetic Theory of Vehicular Traffic," *Transportation Science*, Vol. 14, pp. 55-76 (1980).]
 33. S. Ardekani, "Quality of Traffic Service in Urban Street Networks," M.S. Thesis, University of Texas at Austin (1981).
 34. S. Ardekani and R. Herman, "Quality of Traffic Service," Research Report 304-1, Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, June 1982.
 35. R.R.M. Johnston, R.S. Trayford, and M.J. Woolridge, "Fuel Economy in Peak Hour Travel," *SAE Australasia*, Vol. 37, No. 2, pp. 53-59, Paper No. 16, March-April 1977.

36. J.H. Kent, G.H. Allen and G. Rule, "A Driving Cycle for Sydney," *Transportation Research*, Vol. 12, No. 3, pp. 147-152 (1978).
37. J.L. Deneubourg, private communication.
38. General Motors Corporation, "San Antonio System Expands Country-Wide with Good Service, Sharp Marketing and New Equipment," *Metro*, Vol. 80, No. 3, pp. 27-30, May-June 1984.
39. M. Fager, "Environmental Traffic Management in Stockholm," *ITE Journal*, Vol. 54, No. 7, July 1984.
40. T.F. Golob, E.T. Canty and R.L. Gustafson, "Classification of Metropolitan Areas for the Study of New Systems of Arterial Transportation," Research Publication GMR-1225, General Motors Research Laboratories, Warren, Michigan (1972).
41. R. Marino, "Freeway Inventory Geometric Bottleneck Congestion," First Interim Report, California Transportation Agency, Department of Public Works, Division of Highways - District 7, February 1970.
42. P.K. Munjal, Y.S. Hsu, R.L. Lawrence, "Analysis and Validation of Lane-Drop Effects on Multi-lane Freeways," *Transportation Research*, Vol. 5, No. 4, pp. 257-266, December 1971.
43. B.C. Goodwin and R.L. Lawrence, "Investigation of Lane Drops," *Highway Research Record* 388, pp. 45-61 (1972).
44. R.G. Biggs and M.J. Misleh, "I.P.E. 408 U.S. 59 (South West Fwy) in Houston, Control 27-13, Study Results Freeway Surveillance and Control," Texas Highway Department, Houston, Texas, October 1971.
45. J.H. Buhr, D.R. Drew, J.A. Wattleworth, and T.G. Williams, "A Nationwide Study of Freeway Merging Operations," *Highway Research Record* 202, pp. 76-122 (1967).
46. R.T. Johnson, and L. Newman, "East Los Angeles Interchange Operation Study," *Highway Research Record* 244, pp. 27-46 (1968).
47. J.I. Taylor, "Photogrammetric Determinations of Traffic Flow Parameters," Ph.D. dissertation, Ohio State University (1965).
48. J. Treiterer and J.I. Taylor, "Traffic Flow Investigations by Photogrammetric Techniques," *Highway Research Record* 142, pp. 1-12 (1966).
49. J.C. Yu and J. Lee, "Internal Energy of Traffic Flows," *Highway Research Record* 456, pp. 40-49 (1973).
50. P.K. Munjal and Y.S. Hsu, "Experimental Validation of Lane-Changing Hypotheses from Aerial Data," *Highway Research Record* 456, pp. 8-19 (1973).

51. L. Breiman, R. Lawrence, D. Goodwin, and B. Bailey, "The Statistical Properties of Freeway Traffic," *Transportation Research*, Vol. 11, No. 4, pp. 221-228, August 1977.
52. J.W. Godfrey, "The Mechanism of a Road Network," *Traffic Engineering and Control*, Vol. 11, No. 7, pp. 323-327, November 1969.
53. O. Desforges, "Une Methode D'Enquete Origine-Destination par Photographies Aeriennes," Monograph, Institute of Transport Research, France, September 1976.
54. K. Ruhm, "Traffic Data Collection and Analysis by Photogrammetric Method," *Traffic Engineering and Control*, Vol. 13, No. 8, pp. 337-0341, December 1971.
55. J. Holroyd and D. Owens, "Measuring the Effectiveness of Area Traffic Control Systems," TRRL Report LR420, Transport and Road Research Laboratory, Crowthorne, England (1971).
56. R.C. Brinker and P.R. Wolf, Elementary Surveying, Chapter 25, Sixth Edition, New York: T.Y. Crowell Company, 1977.
57. M. Peleg, L. Stoch, and U. Etrog, "Urban Traffic Studies from Aerial Photographs," *Transportation*, Vol. 2, No. 4, pp. 373-390, December 1973.
58. R.E. Davis, F.S. Foote, J.M. Anderson and E.M. Mikhail, Surveying Theory and Practice, Chapter 16, Sixth Edition, New York, McGraw-Hill Inc., 1981.
59. J.E. Ashwood and P.F. Inglis, "Time-Lapse Aerial Photography: Application to Traffic Surveys," *Traffic Engineering and Control*, Vol. 16, No. 1, pp. 19-21, January 1975.
60. M. Vanmans, "Aerial Monitoring of Road Traffic," *Transport Environment Circulation*, No. 19, November 1976.
61. K. Smith, "Analiza Gradskog Prometa iz Podataka Aerosnimaka," *Geodetskih List, Savez Geodetskih Inzenjera i Geometara*, Vol. 31, No. 416, Zagreb, Yugoslavia (1977).
62. R.T. Baker and D. Owens, "The Use of Aerial Photography for Assessing Traffic Control Systems in Glasgow," TRRL Monograph 93UC, Transport and Road Research Laboratory, Crowthorne, England (1974).
63. A. William and D.R. Gordon, "Monitoring Area Control," *Traffic Engineering and Control*, Vol. 16, No. 9, pp. 379-382, September 1975.
64. J.B. Garner and J. Uren, "A Digital System for Urban Traffic Flow Data," *Traffic Engineering and Control*, Vol. 19, No. 8/9, pp. 389-391, August 1978.

65. D.C. Gazis, R. Herman, and R. Potts, "Car-Following Theory of Steady State Traffic Flow," *Operations Research*, Vol. 7, No. 4, pp. 499-505 (1959).
66. D.C. Gazis, R. Herman, and R.W. Rothery, "Nonlinear Follow-the-Leader Models of Traffic Flow," *Operations Research*, Vol. 9, No. 4, pp. 545-567 (1961).
67. H. Mahmassani, J.C. Williams, and R. Herman, "Investigation of Network-Level Traffic flow Relationships: Some Simulation Results," presented to the 63rd Transportation Research Board Meeting, Washington, D.C. (1984).
68. "Traffic Network Analysis with NETSIM - A User Guide," Implementation Package FHWA-IP-80-3, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C. (1980).
69. J.C. Williams, H.S. Mahmassani and R. Herman, "Analysis of Traffic Network Flow Relations and Two-Fluid Model Parameter Sensitivity," Presented to the 64th Transportation Research Board Meeting, Washington, D.C. (1985).
70. D.R. Drew and C.J. Keese, "Freeway Level-of-Service as Influenced by Volume and Capacity Characteristics," Highway Research Record No. 99, Highway Research Board, Washington, D.C. (1965).

APPENDIX A. NETWORK BOUNDARIES

Austin CBD1 (~1.5 x ~3.0 miles)

Inclusive Boundaries:

38th Street on the north
1st Street on the south
Red River Street on the east
Lamar Boulevard on the west

Austin CBD2 (~1.25 x ~1.5 miles)

Inclusive Boundaries:

Martin Luther King Boulevard on the north
1st Street on the south
Red River Street on the east
Lamar Boulevard on the west

Dallas CBD1

Consisted of three one-mile wide corridors centered along Elm, Ross and Hall Streets. The Elm and Ross corridors were each three miles long, both starting from Stemmons Freeway and stretching eastward and northeastward, respectively. The Hall corridor was 4.5 miles long, beginning at Wycliff Street and stretching south-eastward.

Dallas CBD2 (~1.5 x ~1.5 miles)

Inclusive Boundaries:

Hall Street on the northeast
Thornton Freeway on the southeast (exclusive)
Woodall Rodgers on the northwest
Houston Street on the southwest

Houston CBD (~1 x ~2 miles)

Inclusive Boundaries:

Franklin Street on the northeast
Westheimer on the southeast
Smith Street on the northwest
Chenevert Street on the southwest

Houston Non-CBD (~2 x ~3.5 miles)

Inclusive Boundaries:

Westheimer on the north
Bellaire Boulevard on the south
West Loop Freeway frontage road on the east
Hillcroft Avenue on the west

Lubbock CBD (~1 x ~2 miles)

Inclusive Boundaries:

4th Street on the north
19th Street on the south
Avenue A on the east
University Avenue on the west

San Antonio CBD (~2.5 x ~2.5 miles)

Exclusive Boundaries:

IH-35 on the north
Durango Street on the south (inclusive)
IH-37 on the east
IH-10 on the west

San Antonio Non-CBD1 (~2.5 x ~2.5 miles)

Inclusive Boundaries:

Culebra Avenue on the north
Ceralvo Street on the south
IH-10 frontage road on the east
N.W. 24th Street and Cupples Road on the west

San Antonio Non-CBD2 (~1.5 x ~2.5 miles)

Inclusive Boundaries:

Hildebrand Avenue on the north
Culebra Avenue and Cypress Street on the south
North Expressway (Hwy 81) on the east (exclusive)
N. Elemendorf Street on the west

Albuquerque CBD (~0.75 x ~0.75 miles)

Inclusive Boundaries:

Lomas Boulevard on the north
Coal Avenue on the south
Broadway Boulevard on the east
10th Street on the west

Matamoros (State of Tamaulipas, Mexico) CBD (~1.25 x ~1.25 miles)

Inclusive Boundaries:

Hidalgo, Iturbide and Constitucion Avenues on the north
Rio Bravo and Canales Streets on the south
1st Street on the east
21st Street on the west

APPENDIX B. VEHICLE MATCHING COMPUTER PROGRAM




```

C
C THE INPUT DATA FILE MUST INCLUDE THE VEHICLES ON THE MASTER FRAME
C WITH EACH LINE REPRESENTING ONE VEHICLE. EACH LINE MUST CONTAIN
C THE VEHICLE SIZE, COLOR AND TRAVEL DIRECTION CODES FOLLOWED BY
C THE STATUS CODE (OPTIONAL), THE FRAME SERIAL NUMBER AND THE
C X- AND Y-COORDINATES. THE LIST OF VEHICLES ON THE MASTER FRAME
C MUST THEN BE FOLLOWED BY A LIST OF VEHICLES ON THE CONJUGATE
C FRAME, AGAIN WITH EACH LINE REPRESENTING ONE VEHICLE. NOTE THAT
C THE LAST LINE OF THE INPUT FILE MUST BE A BLANK LINE.
C
C
C      DO2 I=1,1700
C      READ(5,1)C1(I),C2(I),C3(I),C4(I),C5(I),X(I),Y(I)
C 1  FORMAT(I1,4I,14,4X,1E,0,4X,1E,0)
C      IF(INTR(X(I)),1E,0)GOTO3
C 2  NO=1
C 3  NN=0
C      DO 6 I=1,NO
C      F(I)=0.
C
C
C      DISTINGUISHES VEHICLES ON THE MASTER FRAME FROM THOSE ON THE
C      CONJUGATE FRAME.
C
C
C      IF(C5(I).NE.C5(1))GOTO7
C      LL=1
C
C
C      XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
C
C      MATCHING PROCESS BEGINS
C
C      XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
C
C
C      DO5 J=I+1,NO
C
C      ENSURES THAT THE VEHICLE TO BE PATCHED (KEY VEHICLE) IS PROPERLY
C      CODED.
C
C      IF(C1(I).EQ.0)GOTO6
C
C      ENSURES THAT THE KEY VEHICLE AND ITS POTENTIAL PATCH ARE NOT ON
C      THE SAME FRAME.
C
C      IF(C5(I).EQ.C5(J))GOTO5
C
C      ENSURES THAT THE SIZE AND COLOR OF THE KEY AND ITS POTENTIAL MATCH
C      ARE THE SAME.
C
C      IF(C1(I).EQ.C1(J).AND.C2(I).EQ.C2(J))GOTO4
C
C
C      THE FOLLOWING THREE STATEMENTS ENSURE THAT THE SIZES OF THE KEY
C      VEHICLE AND ITS POTENTIAL MATCH ARE CONSISTENT FROM ONE FRAME TO
C      THE NEXT; WITH PROVISIONS MADE TO TAKE INTO ACCOUNT THE OPERATORS'
C      ERRORS IN JUDGING THE VEHICLE SIZE INCONSISTENTLY DUE TO PARRALAX,
C      HILLIF DISPLACEMENT, ETC.
C

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```

C
C      IF ((C1(I).EQ.3.AND.C1(J).EQ.5).OR.(C1(I).EQ.5.AND.C1(J).EQ.3)).AN
-D.C2(I).EQ.C2(J))GOTO4
C      IF ((C1(I).EQ.2.AND.C1(J).EQ.3).OR.(C1(I).EQ.3.AND.C1(J).EQ.2)).AN
-D.C2(I).EQ.C2(J))GOTO4
C      IF ((C1(I).EQ.7.AND.C1(J).EQ.8).OR.(C1(I).EQ.8.AND.C1(J).EQ.7)).AN
-D.C2(I).EQ.C2(J))GOTO4
C      GOTO5
C
C      MATCHES THE TRAVEL DIRECTIONS.
C
C      4 IF(C3(I).NE.C3(J))GOTO5
C      A=(X(J)-X(I))
C      B=(Y(J)-Y(I))
C
C      C
C      C      ALLOWS UP TO 500 MICRONS OF DISCREPANCY IN THE COORDINATES THAT ARE
C      C      NOT SUPPOSED TO CHANGE DEPENDING ON THE CODED TRAVEL DIRECTION.
C      C      FOR EXAMPLE, IF A VEHICLE IS TRAVELLING IN DIRECTION 3, THEN ITS
C      C      Y-COORDINATE FROM ONE FRAME TO THE NEXT MUST NOT BE VERY DIFFERENT
C      C
C      IF(C3(I).EQ.1.AND.C3(J).EQ.1.AND.ABS(A).GT.500.)GOTO5
C      IF(C3(I).EQ.3.AND.C3(J).EQ.3.AND.ABS(B).GT.500.)GOTO5
C      IF(C3(I).EQ.5.AND.C3(J).EQ.5.AND.ABS(A).GT.500.)GOTO5
C      IF(C3(I).EQ.7.AND.C3(J).EQ.7.AND.ABS(B).GT.500.)GOTO5
C      IF(C3(I).EQ.1.AND.C3(J).EQ.1.AND.H.GT.500.)GOTO5
C      IF(C3(I).EQ.3.AND.C3(J).EQ.3.AND.A.LT.-500.)GOTO5
C      IF(C3(I).EQ.5.AND.C3(J).EQ.5.AND.B.LT.-500.)GOTO5
C      IF(C3(I).EQ.7.AND.C3(J).EQ.7.AND.A.GT.500.)GOTO5
C      D(I)=SQRT(ABS((X(I)-X(J))**2+(Y(I)-Y(J))**2))
C
C      C      SETS A SPEED UPPER BOUND OF ABOUT 55 MPH BY NOT CONSIDERING
C      C      POTENTIAL MATCHES WITH A TRAVELLED PHOTO DISTANCE GREATER THAN
C      C      10,000 MICRONS IN ABOUT 2.5 SECONDS OF ELAPSED TIME BETWEEN TWO
C      C      SUCCESSIVE FRAMES.
C      C
C      IF(D(I).GT.10000.)GOTO4
C      F(I)=F(I)+1.
C      5 CONTINUE
C      6 CONTINUE
C
C      C
C      C      IN THE FOLLOWING DO-LOOP THE VEHICLES IN THE MASTER FRAME ARE
C      C      RE-ORDERED SO THAT THOSE WITH ONLY ONE POTENTIAL MATCH ARE PLACED
C      C      FIRST IN THE FILE. THIS IS DONE IN PREPARATION FOR A SECOND
C      C      MATCHING ITERATION.
C      C
C      7 DOB I=1,LL
C      IF(INT(F(I)).NE.1)GOTO6
C      NN=NN+1
C      Z1=C1(IN)
C      Z2=C2(IN)
C      Z3=C3(IN)
C      Z4=C4(IN)
C      Z5=C5(IN)
C      ZX=X(IN)
C      ZY=Y(IN)
C      C1(NN)=C1(I)
C      C2(NN)=C2(I)
C      C3(NN)=C3(I)

```



```

C(CCN)=C(I)
C5(NN)=C5(I)
X(NN)=X(I)
Y(NN)=Y(I)
C1(I)=Z1
C2(I)=Z2
C3(I)=Z3
C4(I)=Z4
C5(I)=Z5
X(I)=ZX
Y(I)=ZY
C CONTINUE
  
```

C
C
C
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C
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XXXXXXXXXX,XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
  
```

SECOND MATCHING ITERATION

```

XXXXXXXXXX\XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
  
```

THE MATCHING PROCEDURE IS REPEATED, THIS TIME USING THE RE-ORDERED LIST OF KEY VEHICLES. IN THIS MATCHING ITERATION, HOWEVER, THE CONJUGATE VEHICLES THAT HAVE BEEN FOUND TO BE A POTENTIAL MATCH FOR ONLY ONE KEY VEHICLE ARE NOT CONSIDERED AS A MATCH FOR ANY OTHER KEY VEHICLE.

```

R016 I=1,LL
F(I)=0.
R012 J=LL+1,NC
IF (C1(I).EQ.0)GOTO16
IF (C1(I).EQ.C1(J).AND.C2(I).EQ.C2(J))GOTO9
IF ((C1(I).EQ.3.AND.C1(J).EQ.5).OR.(C1(I).EQ.5.AND.C1(J).EQ.3)).AND.C2(I).EQ.C2(J))GOTO9
IF ((C1(I).EQ.2.AND.C1(J).EQ.3).OR.(C1(I).EQ.3.AND.C1(J).EQ.2)).AND.C2(I).EQ.C2(J))GOTO9
IF ((C1(I).EQ.7.AND.C1(J).EQ.8).OR.(C1(I).EQ.8.AND.C1(J).EQ.7)).AND.C2(I).EQ.C2(J))GOTO9
GOTO12
* IF (C3(I).NE.C3(J))GOTO12
A=(X(J)-X(I))
B=(Y(J)-Y(I))
IF (C3(I).EQ.1.AND.C3(J).EQ.1.AND.ABS(A).GT.500.)GOTO12
IF (C3(I).EQ.3.AND.C3(J).EQ.3.AND.ABS(B).GT.500.)GOTO12
IF (C3(I).EQ.5.AND.C3(J).EQ.5.AND.ABS(A).GT.500.)GOTO12
IF (C3(I).EQ.7.AND.C3(J).EQ.7.AND.ABS(B).GT.500.)GOTO12
IF (C3(I).EQ.1.AND.C3(J).EQ.1.AND.B.GT.500.)GOTO12
IF (C3(I).EQ.3.AND.C3(J).EQ.3.AND.A.LT.-500.)GOTO12
IF (C3(I).EQ.5.AND.C3(J).EQ.5.AND.B.LT.-500.)GOTO12
IF (C3(I).EQ.7.AND.C3(J).EQ.7.AND.A.GT.500.)GOTO12
U(I)=SQRT (ABS(C1(I)-X(J))**2+(Y(I)-Y(J))**2)
IF (U(I).GT.1000.)GOTO12
F(I)=F(I)+1.
NUM=INT(F(I))
DD(I,NUM)=U(I)
  
```

C
C
C

UNLESS NO MATCH IS FOUND, THE NUMBER OF POTENTIAL MATCHES AND THE

```

C      IDENTIFICATION OF KEY VEH AND THE MATCHED VEHICLES ARE PRINTED
C      HERE ALONG WITH THE DISTANCE TRAVELLED FOR EACH POTENTIAL MATCH.
C
C      WRITE(6,10)F(I),C1(I),C2(I),C3(I),C4(I),C5(I),X(I),Y(I),C1(J),C2(J)
C      -),C3(J),C4(J),C5(J),X(J),Y(J),D(I)
10  FORMAT(//1X,'NO. OF POTENTIAL VEHICLES FOUND = ',F3.0,/1X,
C      -4I,14,4X,F7.0,4X,F7.0,/1X,4I,14,4X,F7.0,4X,
C      -F7.0,/1X,'DISTANCE TRAVELLED = ',F9.0)
C      IF(NUM.NE.1)GOTO11
C      JJ=J
C      Z=C1(J)
C      C1(J)=0
11  IF(NUM.LE.1)GOTO12
C      C1(J)=Z
12  CONTINUE
C      IF(F(I).GT.0.)GOTO14
C
C      PRINTS THE VEHICLES WITHOUT A MATCH AND THEIR POSITION COORDINATES
C
C      WRITE(6,13)C1(I),C2(I),C3(I),C4(I),C5(I),X(I),Y(I)
13  FORMAT(////1X,4I,14,4X,F7.0,4X,F7.0,4X,'MATCHING VEHICLE NOT FOU
C      -ND')
C      DO15 K=LL+1,NO
C      IF(C1(K).EQ.0)GOTO15
C      IGAM=C3(I)-C3(K)
C      IGAMA=ABS(IGAM)
C      IF(IGAMA.EQ.3.OR.IGAMA.EQ.4.OR.IGAMA.EQ.5)GOTO15
C      D(I)=SQRT(ABS((X(I)-X(K))**2+(Y(I)-Y(K))**2))
C      IF(D(I).GT.10000.)GOTO15
C
C      PRINTS ALL THE VEHICLES ON THE CONJUGATE FRAME THAT ARE LOCATED
C      WITHIN A RADIUS OF 10,000 MICRONS FROM THE VEHICLE FOR WHICH NO
C      MATCH HAS BEEN FOUND. FOR MANUAL MATCHING PURPOSES THE POSITION
C      COORDINATES OF ALL SUCH VEHICLES ARE ALSO PRINTED.
C
C      WRITE(6,14)C1(M),C2(M),C3(M),C4(M),C5(M),X(M),Y(M),D(I)
14  FORMAT(1X,4I,14,4X,F7.0,4X,F7.0,4X,'DIST. FROM KEY VEHICLE = '
C      -F9.0)
15  CONTINUE
16  CONTINUE
C      WRITE(6,17)
17  FORMAT(////1X,'KEY VEH.',4X,'X-COORD.',2X,'Y-COORD.',3X,'MICRONS
C      -TRAVELLED')
C      DO19 I=1,LL
C      IF(INT(F(I)).NE.1)GOTO19
C
C      PRINTS A LIST OF ALL VEHICLES WITH ONE POTENTIAL MATCH ONLY, ALONG
C      WITH THEIR POSITION COORDINATES AND THE CORRESPONDING TRAVELLED
C      DISTANCES.
C
C      WRITE(6,18)C1(I),C2(I),C3(I),C4(I),C5(I),X(I),Y(I),DD(I,1)
18  FORMAT(1X,4I,14,4X,F7.0,4X,F7.0,4X,F6.0)
19  CONTINUE
C      STOP
C      END

```