

AN EXPERIMENTAL ANALYSIS OF TWO-FLUID
TRAFFIC MODEL PARAMETER SENSITIVITY

by

Lev A. Malakhoff

Thesis submitted to the Faculty of the
Virginia Polytechnic Institute and State University
in partial fulfillment of the requirements for the degree of
Master of Science
in
Civil Engineering

APPROVED:

Siamak A. Ardekani, Chairman

John W. Dickey

Donald R. Drew

March 6, 1986
Blacksburg, Virginia

AN EXPERIMENTAL ANALYSIS OF TWO-FLUID TRAFFIC MODEL PARAMETER SENSITIVITY

by

Lev A. Malakhoff

Siamak A. Ardekani, Chairman

Civil Engineering

(ABSTRACT)

The Two-Fluid traffic model is a tool used to measure the quality of service in city traffic networks. The model has been calibrated for the central business district of Roanoke, Virginia and the results have been compared to those from various other cities around the world.

In addition, the sensitivity of the Two-Fluid model parameters has been studied for different unit trip lengths used in calibration for the Roanoke CBD. The three unit trip lengths tested were 1-mile, 2-mile, and 3-mile. Statistical analyses led to the conclusion that the model parameters are not significantly different for different trip lengths.

Furthermore, parameter sensitivity was also established for three types of driver behavior, specifically, "aggressive," "normal," and "conservative." The two-fluid trends showed that the two-fluid curve corresponding to the aggressive driver fell below those of the normal and conservative drivers for off-peak traffic conditions but began to approach the normal trend during peak period. On the other hand, the conservative trend continuously maintained higher trip time values for any given stop time, as compared to the other two trends. This implies that aggressive driving behavior can decrease travel times during off-peak conditions, but has little effect during peak periods, where surrounding traffic does not allow excessive speed or lane changing.

ACKNOWLEDGEMENTS

Support for this research came from many sources, especially useful were the thoughtful comments and ideas provided by Dr. Siamak Ardekani.

The data collection was made possible by the willing services of Rao Kadari, Matt Radinovic, and Dave Parker and the students of the Transportation Division at Virginia Tech.

Most importantly, thanks go to the author's parents for their intellectual, emotional, and financial support.

TABLE OF CONTENTS

1.0	Introduction	1
2.0	Theory	4
3.0	Data Collection and Analysis	9
3.1	Study Area	9
3.2	Data Collection	11
3.2.1	Observational Range	14
3.2.2	Chase Car Technique	16
3.3	Calculations and Results	17
3.3.1	Data Analysis	17
3.3.1.1	Output	19
3.3.2	Statistical Analyses	20
3.4	Comparative Results	23
4.0	Effects of Trip Length	27
4.1	Hypothesis Testing	31
4.1.1	Student's t-Distribution	33
4.1.2	Sample Calculations	34
4.2	Results	36
5.0	Driver Behavior	38
5.1	Behavior Types	38

5.2	Random Routing Technique	41
5.2.1	Observational Details	43
5.2.2	Turning Probabilities	45
5.3	Experimental Observations	45
5.4	Results	46
6.0	Conclusions and Discussions	55
Appendix A.	CBD Map of Roanoke, Virginia	58
Appendix B.	Fortran Listing	59
Appendix C.	Fortran Output	61
Appendix D.	MINITAB Sample Regression Output	63
Appendix E.	Random Routing Navigation Guide	65
References		66
Vita		67

LIST OF ILLUSTRATIONS

Figure 1. Family Two-Fluid Curves	7
Figure 2. Sample Data Sheet	12
Figure 3. Time Periods of Data Collection	15
Figure 4. Sample Data Entry	18
Figure 5. Two-Fluid Model Parameters for Various Cities	23
Figure 6. Two-Fluid Traffic Trends for Various Cities	24
Figure 7. Two-Fluid Traffic Trend for Roanoke, Virginia	25
Figure 8. Parameter Values under Various Trip Lengths	28
Figure 9. Two-Fluid Trend for 2-mile Trips	29
Figure 10. Two-Fluid Trend for 3-mile Trips	30
Figure 11. Coefficients and Standard Deviations of Various Trip Lengths	33
Figure 12. Levels of Significance for Coefficients . .	36
Figure 13. Example of Random Turning Assignment . . .	43
Figure 14. Observed Turning Ratios	45
Figure 15. Two-Fluid Parameters for Various Behavior Types	47
Figure 16. Coefficients and Standard Deviations . . .	48
Figure 17. Levels of Significance for Coefficients . .	49
Figure 18. Two-Fluid Trend for Aggressive Behavior . .	50
Figure 19. Two-Fluid Trend for Normal Behavior	51
Figure 20. Two-Fluid Trend for Conservative Behavior .	52
Figure 21. Two-Fluid Trends for Test and Control Behaviors	53

1.0 INTRODUCTION

Transportation, a means to an end, seems to have created as many problems as it has solved. More specifically, the mere presence of vehicles in those rivers of goods and humanity we call "traffic" has led a few of us to conclude that staying at home is not such a bad idea. It is hard to assign blame for existing traffic conditions, since they are usually a product of policy, technology, and geometric constraints, but when somebody has to answer for an inadequate or an unsafe facility, it is often the traffic engineer.

Social pressures have brought about a relatively new discipline, that of traffic science. Traffic science has addressed a wide variety of questions within the past thirty years, especially those problems dealing with traffic processes. Subjects have ranged from 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, and to studies of network traffic fuel consumption and environmental impacts.

As part of a continuing effort to help characterize the quality of service in non-freeway urban street networks, this research will address several objectives. The first

objective is 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.

Another objective of this research will be to calibrate the Two-Fluid model for an existing traffic network, specifically the central business district of Roanoke, Virginia. This calibration makes it possible to compare the quality of service of the Roanoke network to those of other cities, which is accomplished by comparing the Two-Fluid model parameters.

The sensitivity of the model parameters will be addressed in two sections. First of all, the effect of using various trip lengths will be analyzed in relation to the model parameters. The reason this subject is being addressed is that Two-Fluid model data are usually collected for a particular unit trip length. Most of the previous studies using this model relied on a unit trip length of one-mile. This research will include a comparison of the parameters as determined by the one-mile trips with those determined by two-mile and three-mile trips.

The second analysis of the sensitivity of Two-Fluid model parameters will address the effects of various driver behaviors. Two test behaviors, aggressive and conservative, will be used in order to determine the model parameters. These test parameters will then be compared to those found

by "normal behavior," which reflects the results found by using what is called the Chase-Car technique. The Chase-Car technique is a practice where the test vehicle recreates the travel time history of a followed vehicle using the network.

2.0 THEORY

The Two-Fluid Model of Town Traffic, initially proposed by Herman and Prigogine {Ref. 1}, has grown from the earlier Kinetic Theory of Multilane Traffic {Ref. 1}. In the Kinetic Theory, at sufficiently high vehicular concentrations a transition to the collective flow regime was achieved at which the velocity distribution for cars split into two parts - one corresponding to moving vehicles and the other to vehicles that are stopped. The collective flow regime refers to the region in which the concentration is high enough so that the desires of individual drivers are restrained by the vehicles around them.

By building on the Kinetic Theory, the traffic on non-highway routes, particularly city traffic, can be analogized as a two fluid traffic matrix - one fluid composed of moving vehicles and the other of vehicles that are stopped as a result of congestion, traffic control devices, obstructions resulting from construction, accidents, etc., but not parked vehicles.

From the Kinetic Theory, it is assumed that the average running speed, V_r , depends on the fraction of vehicles that are moving, F_r , in the following manner:

$$V_r = V F_r^{-1} = V_m F_r^n = V_m(1-F_s)^n \quad (1)$$

where V_m is a model parameter representing the average maximum running speed in the network, V is the average overall speed of the traffic, F_s is the average fraction of vehicles stopped in the network, and n is a model parameter. It is important that the following relationships are noted:

$$F_r + F_s = 1, \quad (2)$$

$$V_m = 1/T_m, \quad (3)$$

$$V_r = 1/T_r, \quad (4)$$

$$\text{and } V = 1/T. \quad (5)$$

where T_m is a parameter representing the average minimum trip time per unit distance, T_r is the running time per unit distance, and T is the trip time per unit distance. If we consider T_s to be the stopped time per unit distance, it would follow that:

$$T = T_s + T_r \quad (6)$$

A second assumption of the Two-Fluid Theory hypothesizes that the fraction of time a particular vehicle is stopped (T_s/T) is equal to the fraction of total vehicles using the network during the same time period that are stopped, F_s . This is known as the "ergodic" assumption and in the Two-Fluid Model takes the following form:

$$F_s = (T_s/T) \quad (7)$$

Experiments have been performed which show that the above model assumptions (equations 1 and 7) are approximately valid {Ref. 2}. From these assumptions, we are lead to the following relation between trip time T, and running time Tr:

$$Tr = T_m \frac{1}{n+1} T^{\frac{n}{n+1}} \quad (8)$$

which in turn yields this equivalent result:

$$Ts = T - T_m \frac{1}{n+1} T^{\frac{n}{n+1}} \quad (8a)$$

The Two-Fluid model represented by equation 8a yields a curvilinear relation between T and Ts as shown in Figure 1 for Tm values of 1.5 and 3.0 minutes per mile and for n values of 1,2, and 3. Data examined from various cities around the world appear to fit the model which predicts a slightly convex upward curve for T versus Ts. However, the curvature is so slight in these cases that the two-fluid model curve may be approximated as a line by performing a linear regression of T against Ts.

The "quality of traffic service" refers to the ability of a particular street system to carry a certain demand. A system of higher "quality" in general will show lower T, Ts, and Tr values for the same demand on a lower quality system. Likewise, the parameters Tm and n are useful in representing the quality of traffic service in a network. This can be shown by rewriting equation 8a as:

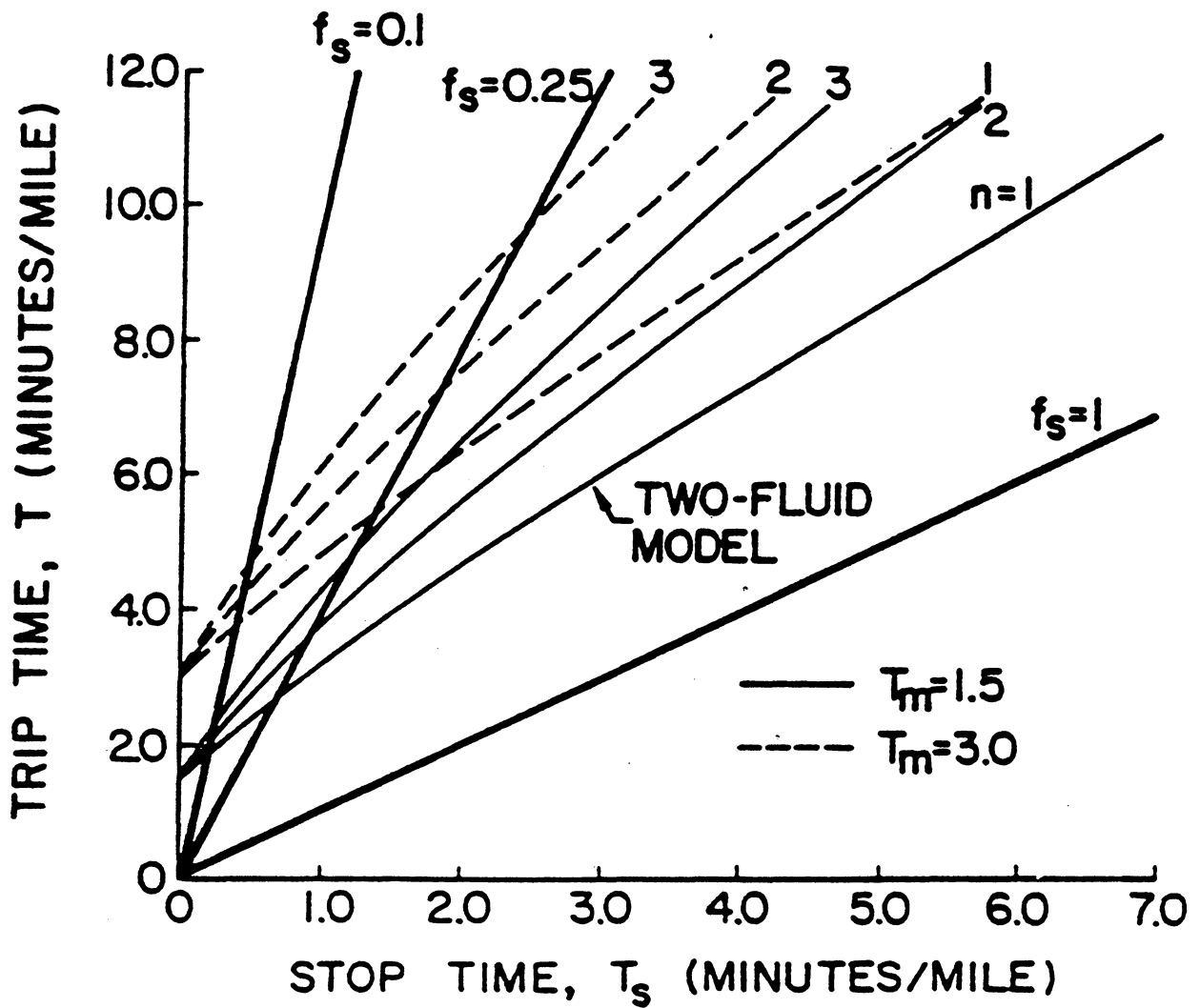


Figure 1. Family Two-Fluid Curves: T_m Values of 1.5 and 3 minutes/mile and n Values of 1, 2, 3, and 4

$$F_s = 1 - (T_m/T)^{\frac{1}{n+1}} \quad (9)$$

$$\text{or} \quad T = T_m(1-F_s)^{-(n+1)} \quad (10)$$

recalling that $T_s/T = F_s$.

By looking analytically at equation 10, or at the graphical results of Figure 1, one can conclude that for a given fraction of vehicles stopped, $F_s = T_s/T$, the network with the larger value of T_m or n is the one with higher trip time resulting in a lower average speed. It has been shown that in an urban system the mean speed is directly related to the average delay and fuel utilization {Ref. 3}; therefore, a lower average speed indicates a poorer quality of traffic service. The importance of the Two-Fluid model is in predicting the magnitude of the changes expected in the average speed and stop time per unit distance from one network to another for a given change in fraction of vehicles stopped or a given change in vehicular concentration.

3.0 DATA COLLECTION AND ANALYSIS

In order to calibrate the two-fluid model for a particular city; that is to say, to determine the parameters T_m and n ; it is necessary to define the boundaries of the study area, as well as the criteria for recording the actual data. Both of these topics are discussed in the following sections, followed by the calculations and results of the model calibration. Also included in this chapter are tables and figures showing the relationships between the model parameters of various cities around the world.

3.1 STUDY AREA

For the purpose of this project, the central business district (CBD) of Roanoke, Virginia was selected to be the area of study. Historically, Roanoke was first chartered as the Town of Big Lick in 1874 with a population of less than 500. In 1884, this rapidly growing railroad town became the City of Roanoke and now boasts a population of more than 100,000 people. Roanoke is approximately 185 miles southwest of Washington, D.C., and happens to be the largest Virginia city west of Richmond.

The city of Roanoke was an ideal choice for a major traffic study due to a number of reasons. First of all, its

nearness to the University made feasible several day trips for data collection. The downtown area is fairly homogeneous and reasonably well defined. For the purpose of this study, an area of approximately 6000'x7000' (about 1.5 square miles) was used. Referring to the Roanoke map in Appendix A, the boundaries include Orange Avenue and Highland Avenue on the north and south, respectively, and Williamson Road and 10th Street on the east and west, respectively.

Within these boundaries are the Civic Center, city hall, two hospitals, a bus station, right-of-way for the Norfolk and Western Railroad, several churches, the municipal complex, several banks, a television station, and a farmers market. Directly adjacent to the CBD are entrance and exit ramps for two exits of Interstate 581. Vehicles entering the CBD network from Interstate 581 are regulated by traffic signals at Williamson Road.

The traffic network itself consists mostly of unsignalized intersections and two-way two-lane streets. Campbell Avenue, running east through the network, is a one-way street that is synchronized in progression. This is of considerable advantage to the users of this route, however, the extensive green periods on Campbell may be penalizing users of cross streets, particularly during offpeak periods, by forcing extensive red periods. Most of the streets east of the municipal building are one-way and signalized.

It appears that most of the vehicles using the network are passenger cars, while there is also a significant minority of local buses. Most of the users appear to be either entering or leaving the network; occasionally a taxi may make an intra-city trip.

3.2 DATA COLLECTION

For the analysis of the quality of service of the Roanoke CBD, almost six and a quarter hours of traffic data were collected. The data were taken on 5 April and on 6 June 1985 in approximately three-hour blocks. The vehicle supplied by the University for the research was a 1983 Plymouth Reliant equipped with automatic transmission and a 2.2 liter engine.

On the 5 April collection period, the author recorded the data while Dr. S.A. Ardekani controlled the vehicle in the manner of the "chase car technique" as described in one of the following sections. On 6 June, the author controlled the vehicle while Mr. V.R. Kadari recorded the data.

Since one of the advantages of the Two-Fluid Theory is that it can indicate the quality of service with a mere pair of parameters - T_m and n , it is therefore useful that data be collected in such a manner to facilitate the calculations. For this study, data was taken in increments of one-mile trips. Referring to the example data sheet in Figure 2, the

DRIVER: ARDEKANI (S.A.)

OBSERVER: MALAKHOFF (L.A.)

STARTING MILEAGE: 71.0

STARTING TIME: 15:11:15

TIME OF STOP HRS:MIN:SECS	TIME OF START HRS:MIN:SECS	COMMENTS
11:25	11:38	
11:54	12:05	
12:56	13:03	
13:42	14:38	
15:04	15:22	
16:08	16:24	
16:30	16:45	
17:03	17:25	
17:41	17:43	
17:59	18:00	
18:12	18:51	

END OF TRIP:

TIME: 15:19:06

MILEAGE READING 72.0

Figure 2. Sample Data Sheet

reader should note that the end of trip mileage reading (72.0), and the starting mileage (71.0), are exactly 1.0 mile apart. On the way from Blacksburg to Roanoke, the odometer of the test vehicle was calibrated between mileposts No. 120 and No. 133. From this we found the odometer to be rather accurate.

Occasionally, the driver, who is responsible for notifying the data recorder at the end of each trip, may be distracted by the driving task and consequently, the trip must be recorded as something somewhat larger than 1.0 mile. This does not directly affect the results, which are normalized to become 1.0 mile trips, and will be seen in the chapter of conclusions and results.

From inspection of the data sheet, we see that there are two columns of entries; a number of stop times to an equal number of start times. It is the task of the collector to record each of these times to the minute and second using a digital watch. Since one-mile trips rarely ever take an hour to make, the hour entry is only made at the beginning and ending time of each trip. To record the stop times, the observer records the approximate instant of the absolute cessation of motion; i.e., when the wheels have locked without skidding, as perceived by the observer to be the case. Observations were made continuously - the ending time of one trip became the starting time of the next one. The

following section details the time periods in which these data were collected.

3.2.1 OBSERVATIONAL RANGE

To calibrate the Two-Fluid Model as accurately as possible, it is desirable to have as wide a range in the values of both T and T_s . During off-peak hours such as late night or early morning hours, we would expect values of stopped time per unit distance, T_s , to approach 0.0 and values of total trip time, T , to approach the minimum travel time per unit distance, T_m . This would give us values close to the intercept on a plot of T versus T_s . To find values far away from the intercept it is necessary to collect data during the peak hours, such as the morning and evening rush hours, and the noon rush hour, all of which should be studied during week days. Studies have shown that the noon hour shows the highest hourly traffic demand {Ref. 4}, and Fridays as having the highest daily demand of a given week day {Ref. 5}.

The data for the Roanoke CBD was collected under a variety of traffic conditions with a total of eighty one-mile trips. On 5 April, three approximately one-hour blocks of data were collected. The time periods during which these trips were collected included the medium demand of mid-afternoon, the high demand of evening rush hour, and the

low demand of early night. For the 6 June collection, data was observed in two approximately one-and-a-half-hour blocks for the high demand lunch hour traffic, and the medium demand of mid-afternoon. These different time periods yield a wide distribution of data which is desirable for modeling purposes, and are categorized in Figure 3. The times given are to be considered accurate only for the data collection, and reflect Eastern Daylight Time.

Period	Time	No. of Trips	Avg. T	Avg. Ts
Late Morning	11:23:01 to 12:02:51	10	3.826	0.913
Noon	12:02:51 to 12:36:45	7	4.843	1.386
Early Afternoon	13:28:11 to 15:17:15	23	4.593	1.318
Late Afternoon	15:11:15 to 16:19:16	14	4.735	1.532
Evening	16:57:29 to 17:56:25	12	4.963	1.522
Night	18:57:29 to 20:00:36	14	4.127	0.983

Figure 3. Time Periods of Data Collection

3.2.2 CHASE CAR TECHNIQUE

It is desirable that the data collection procedure be as unbiased as possible, and one of the methods being used to maintain objectivity is what is called the "Chase Car Technique" as described by Chang and Herman {Ref. 6}. The data gathered by this method involves the use of the test vehicle to follow random cars in the streets in such a manner that the speed-time history of the followed car could be reproduced.

The various cars followed in the field were selected randomly from the population of cars within the study area. The selection was done in the following manner: as the test vehicle entered the area, the driver began following the first vehicle encountered. This car was followed until its trip terminated. A trip termination refers to a followed vehicle either leaving the study area or stopping to park. This first trip is slightly biased since it depends on when and where the test vehicle enters the study area, and therefore, this "follow" is not considered in the network analysis.

The next vehicle selected was the first vehicle to originate a trip from the previous terminating location. If a new trip was not being initiated from that location, the next vehicle to drive by was followed. After the second followed vehicle is found, the operator must wait until the

odometer registers to the nearest tenth of a mile. For the test data sheet shown in Figure 2, the first one-mile trip began at a last-two-digit reading of 71.0 miles.

In the transition between following one car and the picking up of the next, data collection is not terminated; i.e., the driver continues to drive until another vehicle comes along. Occasionally, the followed driver may be driving in an unsafe or illegal manner, or may exhibit suspicion of the test vehicle. If any of these cases are apparent to the test team, that vehicle will be dropped and another picked up to be chased.

3.3 CALCULATIONS AND RESULTS

From the output of the data collection described in the previous chapter, the parameters T_m and n were calculated for the Roanoke CBD using statistical methods and logarithmic relations. The following sections demonstrate the operations and statistical analyses needed to facilitate the calculations.

3.3.1 DATA ANALYSIS

The raw data listing each mile's trip times; including every stop, start, beginning, and ending time; is fed into a data file using the mainframe IBM system at Virginia Tech.

As an example, the data for the first one-mile trip was entered as shown in Figure 4.

```
1.0
11.15
11.25
11.38
11.54
12.05
12.56
13.03
13.42
14.38
15.04
15.22
16.08
16.24
16.30
16.45
17.03
17.25
17.41
17.43
17.59
18.00
18.12
18.51
19.06
0.0
```

Figure 4. Sample Data Entry

Note that the first entry reads "1.0." This will indicate to the fortran program {Ref. 4} that it should begin analyzing a 1.0 mile trip. The following entry, "11.15," is the time at which the trip begins. Each pair of entries following this are stopping and starting times, respectively, with the exception of the last pair. The next to last entry, "19.06," is the time at which the one-mile trip was completed. The last entry, "0.0," alerts the program to

conclude the analysis of this particular trip and to prepare to analyze the next trip. At the completion of all the trips, the entry "-1.0" is used in place of 0.0 to indicate the end of the data. An interactive version of this program is also available.

The above time entries indicate only the real-time minutes and seconds of a particular event. For example, the entry "11.25" in the third line tells the program that at eleven minutes and twenty-five seconds after the hour, the vehicle came to a full stop. The program is able to interpret the entry this way, and not as a decimal entry. For a listing of this program, the reader should refer to Appendix B.

3.3.1.1 Output

A fortran program was generated to calculate the total trip time, T , for each mile by subtracting the beginning trip time from the ending trip time. Likewise, the running time for each mile, Tr , is calculated by summing up the differences in each pair of entries beginning with the starting time. The program also lists the stopped time per mile, Ts ; the logarithms (to the base 10), of the trip time and running time, $\log(T)$ and $\log(Tr)$, respectively; the fraction of stopped time, Fs ; the average trip speed in miles per hour, V ; the number of stops per trip, Ns ; and the number of stops per mile, $Ns/Mile$. Referring to the sample output

in Appendix C, it is relevant to note that the last two columns, NS and NS/MILE, occasionally have differing values. This occurs as a result of trip lengths other than one mile.

3.3.2 STATISTICAL ANALYSES

In order to calculate the parameters T_m and n , it is necessary to use statistical methods applied to actual field data. This is done by using linear regression to find the relationships between T , T_s , and T_r . Due to the large amount of data to be analyzed, it is convenient to use package programs available on the Virginia Tech mainframe. One such statistical program is Minitab {Ref. 7} which can perform linear regression, plot variables, and calculate the coefficient of correlation, R^2 .

To find the parameters T_m and n , it is necessary to perform a linear regression of $\log(T_r)$ against $\log(T)$. The regression was performed using the data shown in Appendix C and the MINITAB statistical package which yielded the output shown in Appendix D. The regression results are shown in equation 11 with a coefficient of correlation, R^2 , of 78.9 percent.

$$\log(T_r) = 0.109 + 0.616 \log(T). \quad (11)$$

To be able to use the above equations in finding T_m and n , it is important to understand the relationships of the

coefficients. Since the two-fluid model predicts a linear relation between the logarithms of the running time and the trip time, both per unit distance, the following relation can be made by taking the logarithms of both sides of equation 8, namely,

$$\log(\text{Tr}) = \frac{1}{n+1} \log(\text{Tm}) + \frac{n}{n+1} \log(\text{T}). \quad (12)$$

On the other hand, a linear regression of $\log(\text{Tr})$ against $\log(\text{T})$ would obtain this general relation:

$$\log(\text{Tr}) = A + B \log(\text{T}) \quad (13)$$

where A and B are the regression parameters.

By looking at equation 11a calculated by Minitab, and the theoretical equation 13, we see for the eighty 1-mile trips in the Roanoke CBD that the coefficient A has a value of 0.109 and B has a value of 0.616. To obtain the parameters Tm and n, we combine equations 12 and 13 into the following forms:

$$n = B/(1-B) \quad (14)$$

$$\log(\text{Tm}) = A/(1-B) \quad (15)$$

or,

$$\text{Tm} = 10^{\frac{A}{1-B}}. \quad (15a)$$

Using the above values of A and B in equations 14 and 15a, the parameters T_m and n for the Roanoke CBD are calculated to be:

$$T_m = 1.92 \text{ minutes/mile}$$

$$n = 1.60$$

In the same manner, parameter values of T_m and n have been determined for four cities around the world for which data was collected in 1983 and 1984 {Ref. 8}. These include the downtown networks of Matamoros, Mexico; Brussels, Belgium; Houston, Texas; and Milwaukee, Wisconsin. The results from these cities, including those from Roanoke, are summarized in Figure 5 and presented graphically in Figure 6. Included in Figure 5 for these five cities are the two-fluid model parameters T_m and n as well as several other useful indicators such as correlation coefficients and linear representations of the two-fluid model. The linear representations are given by the intercept, A, and the slope, B, where trip time, T, and stop time, T_s , are the ordinate and the abscissa, respectively.

Figure 7 includes a plot of the Roanoke CBD data from the eighty 1-mile trips and the respective two-fluid trend for the Roanoke CBD as determined from the data shown. Also listed are the values of T_m , n , and R^2 for the data. Referring to Figures 5 and 6, it can be seen that the Roanoke CBD traffic network exhibits neither high nor low T_m and n values when compared to the networks of these four other cities.

CITY	No. of Trips	Linear Rep.			2-Fluid Parameters		
		A	B	R^2	T_m	n	R^2
Brussels	90	2.13	1.80	0.87	1.26	2.76	0.92
Houston	159	3.07	1.30	0.90	2.70	0.80	0.63
Matamoros	88	4.23	1.62	0.82	2.98	2.10	0.88
Milwaukee	24	1.81	1.61	0.84	1.58	1.41	0.81
Roanoke	80	2.61	1.49	0.84	1.92	1.60	0.79

Figure 5. Two-Fluid Model Parameters for Various Cities

3.4 COMPARATIVE RESULTS

Since a particular two-fluid model parameter for a given city cannot objectively be characterized as "good" or "poor" without some previously determined criteria, the only measure of the network quality of service is a comparison of one city's parameters to those of another. In other words, if both the T_m and n values for one city are lower than those of another, it can be said that this first city enjoys a higher quality of service in its traffic network than the second city, for all given stop times. In the case of the Roanoke CBD, comparisons to each of the four previously mentioned cities are given below.

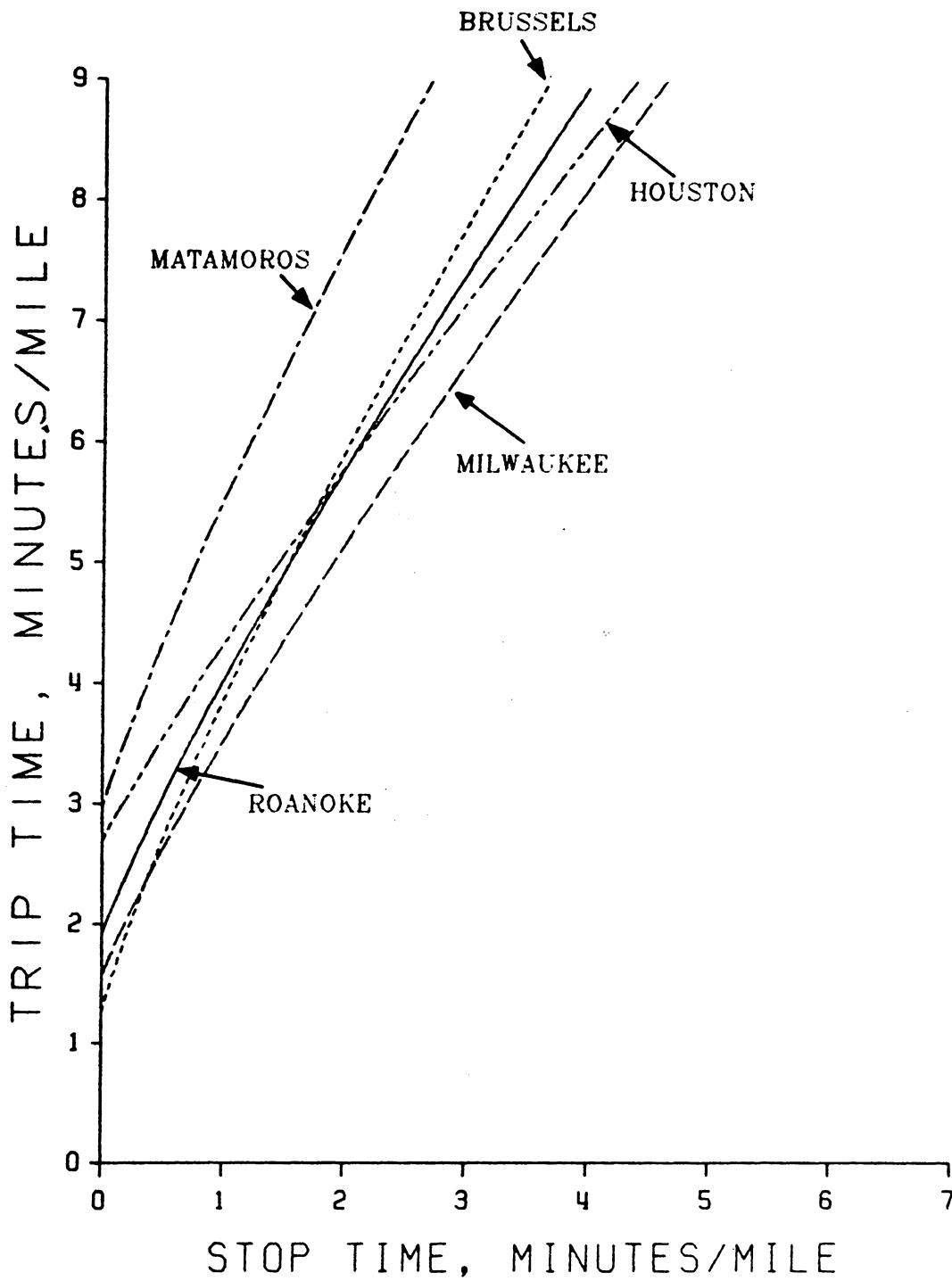


Figure 6. Two-Fluid Traffic Trends for Various Cities

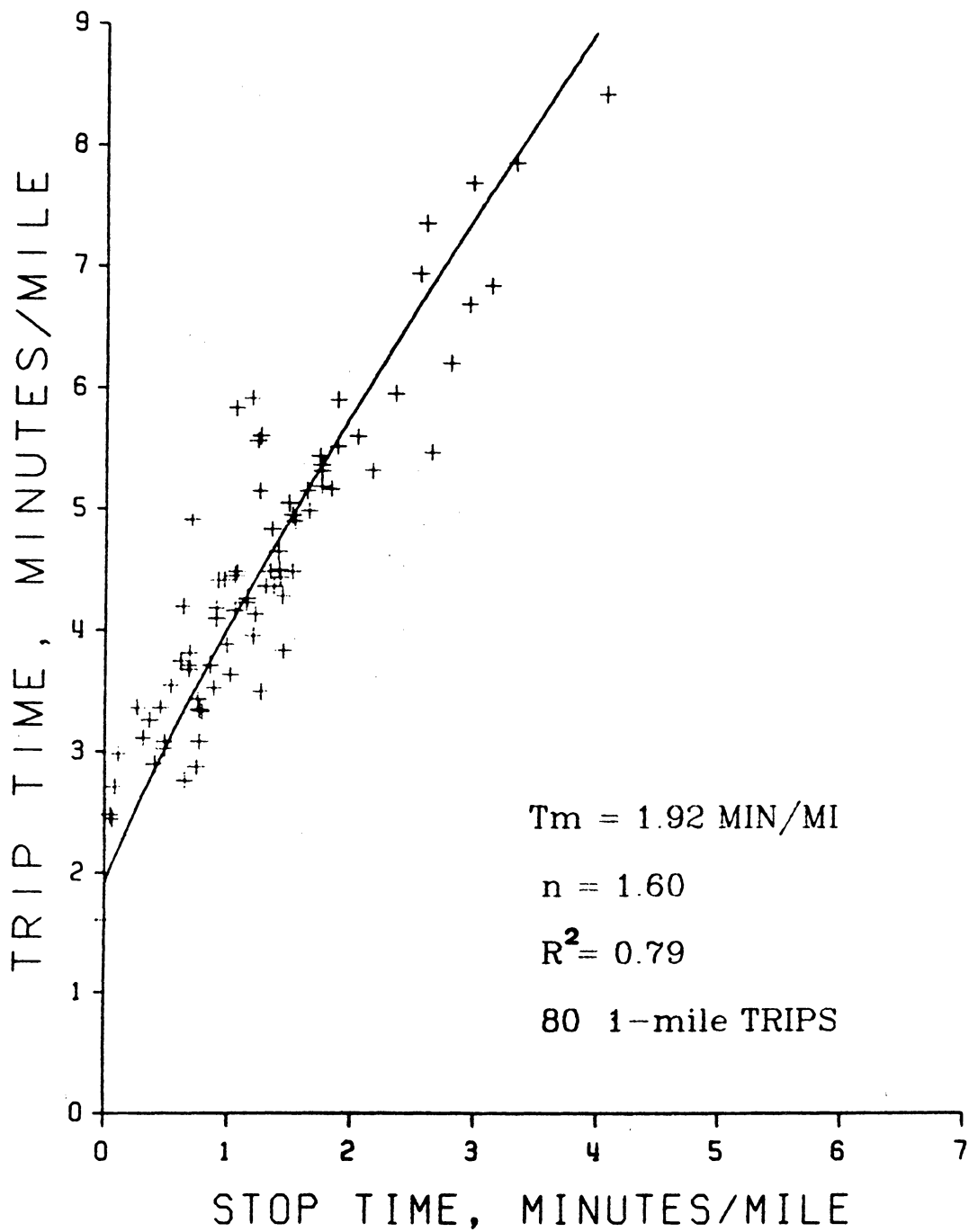


Figure 7. Two-Fluid Traffic Trend for Roanoke, Virginia

The best and worst qualities of service are found in the networks of Milwaukee and Matamoros, respectively. The Milwaukee network, with its lower T_m and n values, will enable an average driver to maintain a lower total trip time for any given stop time when compared to the Roanoke network. Conversely, the Matamoros network requires a driver to spend a higher trip time for any given stop time when compared to Roanoke.

As an example of how congestion can radically change the quality of service, the T_m value for Brussels is substantially lower than that of Roanoke, while its n value almost twice as large. The intersection of the curves of these two cities occurs at $T_s=1.5$ and $T=4.9$ minutes/mile. This point is an estimate of the fraction of vehicles stopped, F_s ; where

$$F_s = T_s/T = 1.5/4.9 = 0.31$$

which is to say, for traffic levels which exceed an F_s of 31 percent, Roanoke will yield lower travel times, and hence, a higher quality of service, than Brussels.

4.0 EFFECTS OF TRIP LENGTH

The Two-Fluid theory presents the parameters T_m and n as characterizers of the quality of traffic service. The values of these parameters are determined by using a set of micro-trip data collected under various traffic conditions. In light of the above, a sensitivity analysis of the magnitudes of T_m and n as influenced by the length of the micro-trips is necessary. This issue has not been systematically addressed before, although studies have been conducted in other cities using one mile, two mile, as well as one or two kilometer trip lengths {Ref. 8}. An objective of this research has been to assess, through a series of field observations, the influence of trip length on the Two-Fluid Theory parameters for the same network. Ideally, one expects that the parameter values would not be significantly influenced by reasonable selections of trip length.

The data collected in Roanoke are made up of eighty 1-mile trips. Although these were not all consecutive trips (since data was collected in one-hour blocks), most were able to be collated into two and three mile actual trips. The selection of 2-miles and 3-miles as trip lengths, of course, is as arbitrary as using a 1-mile trip length, but provides

insight into whether or not a trip length two or three times greater than the "standard" will affect the results.

After aggregating the data into 2-mile and 3-mile trip lengths, it is necessary to perform the same manipulations that were used for the 1-mile trips. These manipulations are detailed in the chapter of "Calculations and Results." From the calculations, the values of T_m , n , and R^2 for both the 2-mile and 3-mile trips are listed in Figure 8 to be compared with those found for the 1-mile trips.

Trip Length (Minutes/Mile)			
Unit	T_m	n	R^2
1-Mile	1.92	1.60	0.79
2-Mile	2.12	1.30	0.81
3-Mile	1.96	1.51	0.84

Figure 8. Parameter Values under Various Trip Lengths

The curvilinear relationships between trip time and stop time for the 2-mile and 3-mile trips are shown in Figures 9 and 10. Note that both curves closely resemble each other as well as that of the 1-mile trips as shown in Figure 7.

From a casual glance at Figure 8, it may appear that the parameter values for the different unit trip lengths are not as close to each other as may be expected. This is most noticeable in the difference between the n value for the 2-mile trips of 1.30 and that of the 1-mile trips of 1.60; a

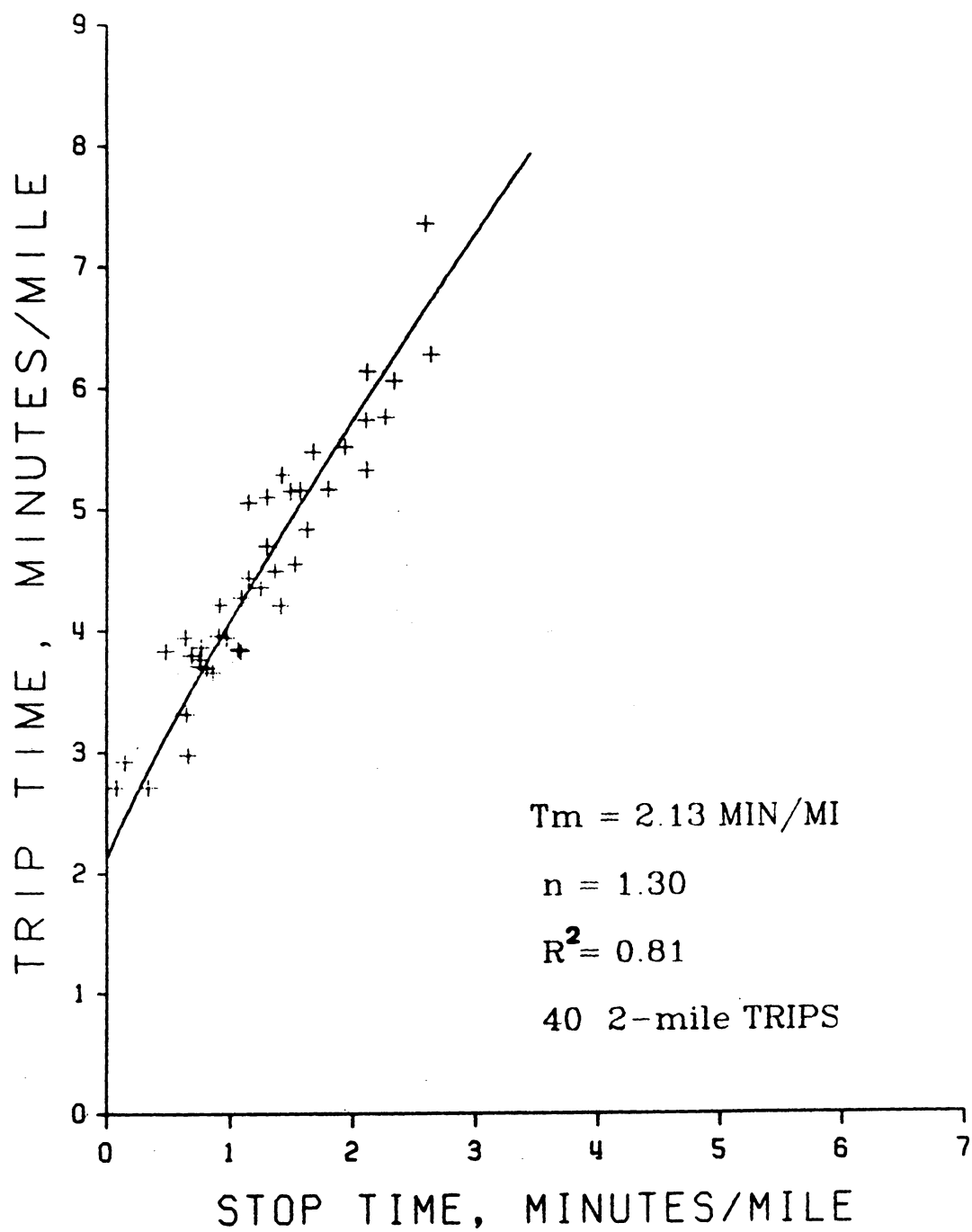


Figure 9. Two-Fluid Trend for 2-mile Trips

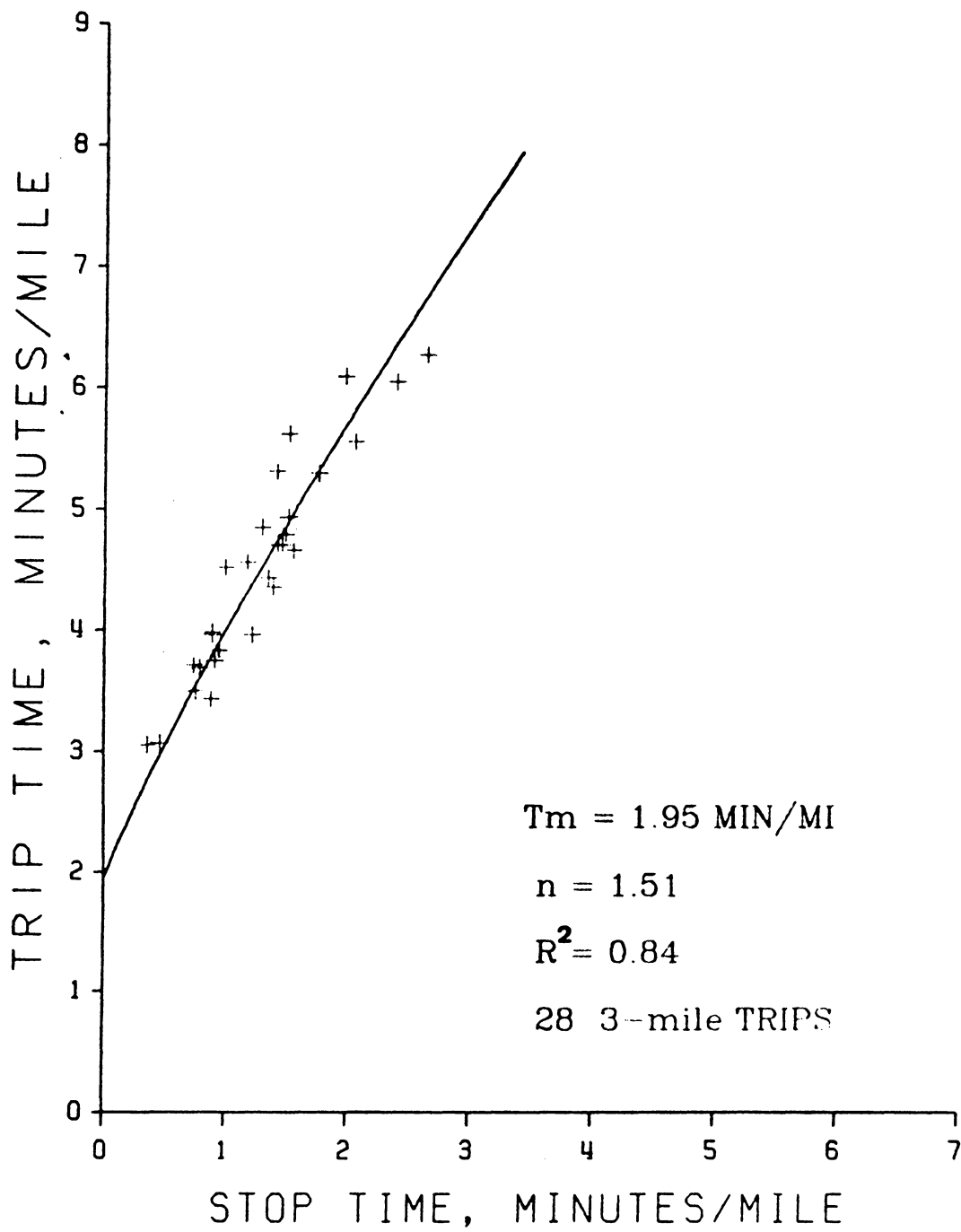


Figure 10. Two-Fluid Trend for 3-mile Trips

reduction of about 19 percent. This raises the question of whether or not the results from the 2-mile and 3-mile trips are statistically significantly different from those found using the 1-mile trip lengths.

The following section presents the results of formal statistical tests on the differences manifested in Figure 8.

4.1 HYPOTHESIS TESTING

In order to investigate whether or not T_m and n differ statistically from one trip length to another, it is convenient to compare parameters A and B from which T_m and n are computed. Recalling the equations used to calculate T_m and n ,

$$n = B/(1-B) \quad (14)$$

and

$$T_m = 10^{\frac{A}{1-B}}, \quad (15a)$$

it should be noted that T_m and n are functions of the regression coefficients, A and B . The following hypotheses are made in regard to parameter sensitivity of the two-fluid model.

$$H_0: A_i = A_1$$

$$B_i = B_1$$

$$H_1: A_i \neq A_1$$

$$B_i \neq B_1$$

where A_1 and B_1 are parameters calculated for the 1-mile trips (see equation 13), and A_i and B_i are parameters calculated for unit-trips other than the standard of 1-mile. The symbols H_0 and H_1 are used to indicate two possibilities, acceptance or rejection, respectively. Of course, there are other permutations of relations between A_i , B_i , A_1 , and B_1 ; e.g., $A_1 = A_i$ with $B_1 \neq B_i$. For now the analysis will be restricted to the above two conditions. To test these hypotheses, the values of A and B for both 2-mile and 3-mile trips will be compared against those for 1-mile trips; then, the 3-mile trip parameters will be compared against the 2-mile trips.

As described in chapter 2, a linear relation can be found between the logarithms of the running time, Tr , and the trip time, T . Using statistical software, least squares regression algorithms compute the intercept, A , and the slope, B , for each set of data, as well as the standard deviations of these coefficients. In this case, the coefficient A can be thought of as logarithm of the minimum running time per unit length. It is necessary when comparing two A values that the standard deviation of each be considered. The values of A , B , their respective standard deviations, and the number of trips (data points) for 1-mile, 2-mile, and 3-mile unit trip lengths are listed in Figure 11.

Unit Trip Length	Number of Trips	A	S.D. _A	B	S.D. _B
1-mile	80	0.109	0.0234	0.616	0.0360
2-mile	41	0.142	0.0288	0.566	0.0442
3-mile	28	0.116	0.0338	0.602	0.0518

Figure 11. Coefficients and Standard Deviations of Various Trip Lengths

4.1.1 STUDENT'S T-DISTRIBUTION

In order to determine whether the A and B values of one unit trip length differ significantly from those of another, the Two Sample t-Test is employed. The method yields a t^* value as follows:

$$t^* = \frac{P_1 - P_i}{S_p} \quad (16)$$

where: t^* is a limiting value within the t-distribution of the two samples,

P_1 is the parameter value (A or B) for the 1-mile trips,

P_i is the parameter value for the i-mile trip considered,

S_p is a weighted average standard deviation (pooled standard deviation) taking into account the degrees of freedom and the standard deviations of both the 1-mile and the i-mile trips.

To calculate the pooled standard deviation, S_p , the following equation is used.

$$S_p^2 = \frac{(N_1-1)S_1^2 + (N_i-1)S_i^2}{N_1 + N_i - 2} \quad (17)$$

where: N_1 = the number of data points from the 1-mile trips,
 N_i = no. of data points of i-mile trip being considered,
 S_1 = standard deviation of the coefficient from the
1-mile trips,
and
 S_i = standard deviation of i-mile trip coefficient being
considered.

4.1.2 SAMPLE CALCULATIONS

This section is devoted to a comparison between the A value of the 1-mile trips and that found for the 2-mile trips to see if they are statistically different from each other. These same calculations were performed in the comparison of 3-mile trip parameters to those of the 1-mile trips and to the 2-mile trips. B values were compared in the same manner.

In order to calculate S_p to compare A_1 to A_2 , the following values need to be substituted into equation 17.

N_1 = 80 trips (data points)

N_2 = 41 trips

S_1 = 0.0234 (standard deviation of A_1)

S_2 = 0.0288 (standard deviation of A_2)

This yields an S_p value of 0.0253. This value in turn is needed to calculate t^* along with the following values to be substituted into equation 16;

$$A_1 = 0.109$$

$$A_2 = 0.142$$

which results in a t^* -value of 1.312. By including the total number of individual data points from the two samples, the degree of freedom is calculated as:

$$df = N_1 + N_2 - 2 \quad (18)$$

in this case,

$$df = 80 + 41 - 2 = 119.$$

For the given degree of freedom and t^* -value, statistical tables based on the t-distribution were accessed in order to find the Level of Significance (LOS). In this example, the hypothesis that $A_1 = A_2$ is accepted at an 81 percent confidence level. This is to say that a rejection of $H_0 : A_1 = A_2$ will run a 19 percent chance of rejecting a correct hypothesis. Commonly acceptable LOS's run from 10 or even as low as 5 percent, which leads to 10 or only 5 percent chances of rejecting a valid hypothesis.

	A1	A2	A3
A1	100%	19%	79%
A2	19%	100%	40%
A3	79%	40%	100%

[Figure 12a - Comparison of the coefficient A]

	B1	B2	B3
B1	100%	21%	74%
B2	21%	100%	45%
B3	74%	45%	100%

[Figure 12b - Comparison of the coefficient B]

Figure 12. Levels of Significance for Coefficients: Calculated from 1,2 and 3 Mile Trips

According to statistical theory, it may be concluded that a large LOS means that A_1 and A_i are virtually the same. Using this concept, an LOS of 100% would mean that A_1 and A_i are identical; this is shown in Figure 12a where the intersection of a column and row, both titled A_1 shows an LOS of 100%.

4.2 RESULTS

Using the methodology described in the sample calculations, the Level of Significance was derived for each

possible relationship of the coefficients A and B. In Figures 12a and 12b these relations are demonstrated.

Given an understanding of the Level of Significance concept, the values listed in Figure 12 are crucial in validating the previous hypotheses. In this case we found an LOS of 19 percent for the comparison of the A parameters of the one-mile and the two-mile trip lengths. This LOS value is the lowest found in the comparisons, and according to the analysis, is the most crucial. If one is willing to accept the null hypothesis ($H_0: A_1=A_2$) at the 10 percent level, then the null hypothesis is accepted at anything greater than 10 percent.

The lowest LOS as determined being 19 percent, which is substantially larger than the commonly accepted minimum of 10 percent, it is therefore evident that the values of A_1 and A_2 are statistically not different. Likewise, for the comparisons of one-mile with three-mile trips, the respective parameters are also statistically not different. It follows that since parameters of the one-mile with respect to the two-mile and the one-mile with respect to the three-mile trips are statistically the same, then the parameters of the two-mile with respect to the three-mile trips should also be identical. This last comparison, was not performed as it was felt to be self-evident in light of the results.

5.0 DRIVER BEHAVIOR

Another aspect of the Two-Fluid model calibration not adequately addressed in the past is the effect of driver behavior on the model parameters. The Chase Car technique, described in chapter 3, has been used in the Roanoke data collection in order to minimize the effect of driver behavior on the parameter values by simulating the behavior of an "average" driver through sampling a variety of drivers. However, the question still remains as to what extent the data from "extreme" drivers (aggressive or conservative) could affect the values of T_m and n .

5.1 BEHAVIOR TYPES

Attempts were made to simulate three driver behaviors, a normal, an aggressive, and a conservative behavior. The chase-car data was characterized to represent a normal driver. In this case, "normal" is synonymous with "average" since a chase-car driver will sample a variety of driver behaviors over the course of each observation period and as such, the resulting datum point can be considered to represent an average user of the network.

A conservative behavior, on the other hand, was demonstrated by taking on some characteristics that are generally associated with a conservative driver, namely,

- Low acceleration and deceleration rates.
- Minimal lane changing
- Delayed starting at intersections.
- Excessive defensive driving.
- Slowing when coming to a "stale" green light.
- Stopping on amber when possible.
- Rejecting many gaps at stop signs.

An Aggressive driver was in turn characterized, among others, by the following behavioral patterns:

- High accelerations and decelerations.
- Breaking away and trying to stay ahead of the platoon.
- Frequent lane changing.
- Maintaining as high a speed as is legal/rational.
- Accepting short gaps at stop signs.
- Crossing during late amber/early red indications.

In order to take on these characteristic driver behaviors, it is necessary that the test vehicle use a method of route selection other than that determined by the chase-car technique. The chase-car route selection is

essentially unbiased, since the test vehicle driver follows a variety of other drivers. It is the interest of this chapter, however, to examine the sensitivity of the two-fluid parameters if, for example, the followed vehicles consisted mostly of "aggressive" or mostly of "conservative" drivers.

As an example, if the driver of the test vehicle, when offered the choice of several vehicles to follow, chooses what he perceives to be the more aggressively driven target vehicle, then such a behavior may be reflected in the data, and consequently, in the parameters. Likewise is the case for observations mainly consisting of conservatively driven vehicles.

In order to examine the effects of these two driver behavior types on the two-fluid parameters, test vehicles were driven by operators instructed to drive in the manner outlined above, both "aggressively" and "conservatively" as well as one test vehicle that was driven "normally." The "normal" driver, in this case, simply used the chase-car technique, while all three vehicles had observers collecting data simultaneously under the same boundaries. The aggressive and conservative drivers did not utilize the chase-car technique (since driver perception is very difficult to control), and instead had to rely on a random routing technique described in the next section.

5.2 RANDOM ROUTING TECHNIQUE

In developing the Random Routing Technique, several simplifying assumptions were made in order to produce a method that yielded unbiased route selection, while at the same time gave results pertaining to those found from an aggressive or conservative driver.

The first step in understanding the Random Routing Technique, is to conceptualize the study area network not as a grouping of streets and intersections, but rather as a matrix of links and nodes, with a node (intersection) being a "decision point." In realistic terms, a decision point would be the point at which a driver must decide whether to continue driving straight, or to make a turn, be that right or left. These decision points do not include turns into parking areas, driveways, or prohibited turning direction at intersections where such restrictions exist.

Using this conceptualization of the study network, the conclusion was drawn that a "random" route could be generated if a test driver is informed before coming to each decision point as to which of the three maneuvers to execute, i.e. to go straight, turn left, or turn right. The decision must be made at random, while at the same time be representative of the movements made by the network vehicular population. This is important because the number of turns made per unit length may have a definite effect on the number of stops and total

stopped time per unit length. In other words, the random route should contain approximately the same number of left turns, right turns, and straight maneuvers per unit length as an average vehicle for a given network.

In order to determine the numbers of the various turning maneuvers of an "average" vehicle in the network, it was assumed that the percentage for each maneuver made by a number of vehicles followed in the chase-car mode would yield representative numbers of turns per unit length as obtained under random route selections. It is necessary, therefore, to follow and observe these "target" vehicles throughout the network, recording each movement at each decision point that they encounter. The target vehicle is selected at random as in the Chase Car technique, and followed until it leaves the network, parks, or shows signs of suspicion or awareness of the test vehicle.

This procedure is repeated continuously, following a large number of vehicles in order to collect an even larger number of turning data for a travel history of an "average" network user. After these data are collected, the percentage left turns, right turns, and straight maneuvers are calculated for an any given decision point.

Random Routing "navigational guides are then prepared through the use of random number generating programs. Most of these programs generate a decimal number between 0 and 1. Once the percentage of left turns has been determined for the

network, say 20%, then a generation of a random number between 0.00 and 0.20 will assign a predetermined left turn to the decision point under consideration. An example of the use of these random numbers is shown in Figure 13.

Type of Turn	Percentage	Random Number Range	Assignment
Left	20 %	0.000 - 0.200	L
Right	30 %	0.201 - 0.500	R
Straight	50 %	0.501 - 1.000	S

Figure 13. Example of Random Turning Assignment

For convenience, the randomly generated numbers should be assigned letter values depending on which range they fall into. These assignments can be hard copied and used in the test vehicles in the form of a navigations guide, eliminating the need for random generating equipment to be transported, operated, and the results interpreted during the data collection. An example of the random turning assignments navigational guides can be seen in Appendix E.

5.2.1 OBSERVATIONAL DETAILS

The area used to collect data for the Random Routing Technique was a sub-area of the Roanoke CBD defined earlier. The boundaries of this sub-area include Norfolk and Elm Avenues on the north and south, respectively; and Williamson

Road and 5th Street on the east and west, respectively. All of these links were considered to be inclusive boundaries for this part of the study.

On 15 October 1985, while the two-fluid data were being collected in the Roanoke CBD from 16:30 to 17:30, an additional observer was assigned to keep count of the left/right/straight maneuvers of the chased vehicles. The traffic conditions in the Roanoke CBD were varying in intensity, since data were collected during the medium demand of pre-rush hour as well as the higher demand of the evening rush hour. This led to a situation where trip time/stop time data were collected continuously, but turning data could only be collected when the actions of a target vehicle could be observed; which usually is the case during off-peak data collection.

On the other hand, when target vehicles are abundant, as during peak periods, it is possible to observe the turning maneuvers of more than one vehicle. This usually occurs at the sub-study area boundaries; when a current target vehicle exits the area, its last turn is recorded while shortly previous to this, a new target vehicle was selected and its turning history was being recorded concurrently.

5.2.2 TURNING PROBABILITIES

The ratios of turning maneuvers obtained from the one-hour observation period are presented in Figure 13. Similar data have also been collected aboard the chase-car test vehicle during the course of the driver behavior experiment and do not show any substantial deviation from the table values.

Type of Turn	Number Observed	Percentage
Left	22	18.9%
Right	23	18.9%
Straight	77	63.1%
Totals	122	100.0%

Figure 14. Observed Turning Ratios

The above turning ratios were then incorporated into in a random number generating program to produce the random routing navigations sheet as shown in Appendix E.

5.3 EXPERIMENTAL OBSERVATIONS

Once the probabilities of particular turns were found from the 15 October experiment, a new experiment was designed in order to gather data which is generated by the three categories of drivers. For each of the test behaviors -

aggressive and conservative - data were collected along streets as indicated by the random routing navigation guide sheet. It is important to note that every turn indicated on the guide sheet could not be executed, since there are numerous intersections in the Roanoke CBD where an indicated turn is restricted; also, the selected boundaries deterred a number of turns as indicated by the guide sheet.

When an indicated turn is prohibited, or not executed because of boundary proximity, the navigator responded by scanning down the guide sheet to the next implementable turn decision. This practice led to the elimination of a number of turn decisions; this probably would not affect the bias of the data, since those turn decisions that are remaining on the navigation sheet are not a function of those previously eliminated, but were determined by the network characteristics, which is what is essentially the topic of study.

5.4 RESULTS

Using methods of calculations as outlined in chapters 3 and 4, the parameters T_m and n were computed for the two test behaviors, aggressive and conservative, as well as for the control "normal" behavior. All these values are for the smaller study area as described at the beginning of this chapter. Referring to Figure 15, it is important to note that

the values of the coefficient of correlation, R^2 , are fairly low, especially for the test behaviors.

Behavior Type	No. of Trips	Tm	n	R^2
Aggressive	56	1.97	0.98	0.48
Normal	45	2.68	0.71	0.51
Conservative	47	2.90	0.87	0.49

Figure 15. Two-Fluid Parameters for Various Behavior Types

These low indicators of correlation can be partly attributed to the boundary conditions, the lack of acute congestion, and the artificial constraints imposed by the behavioral instructions. These instructions led the aggressive to maintain a low running time, T_r , regardless of whatever stop time, T_s , that he may experience. Such a constraint weakens the correlation between T and T_r [and $\log(T)$ and $\log(T_r)$]. Conversely, the conservative driver was instructed to elongate his T_r , even if his T_s was very low, as was the case during off-peak data collection, and likewise produced a low coefficient of correlation. The two-fluid trends can be seen in Figures 18, 19, and 20 for the aggressive, normal, and conservative driver behaviors, respectively.

The coefficients of regression, A and B , as described in chapter 4, were found for each driver behavior trend.

These values, with their respective standard deviations, are reported in Figure 16.

Driver Behavior	A	S.D. _A	B	S.D. _B
Aggressive	0.149	0.0453	0.495	0.0697
Normal	0.250	0.0450	0.415	0.0627
Conservative	0.248	0.0528	0.464	0.0708

Figure 16. Coefficients and Standard Deviations: Various Driver Behaviors

This information was then used to calculate the pooled standard deviation, S_p , and the t^* -value for each combination of behavior comparisons. Then, to relate each behavior type to each of the other two, the t^* -value was used to find the Levels of Significance, LOS, which are outlined in Figure 17.

	Aggr.	Norm.	Cons.
Aggressive	100%	3%	5%
Normal	3%	100%	99%
Conservative	5%	99%	100%

[Figure 17a - Comparison of the coefficient A]

	Aggr.	Norm.	Cons.
Aggressive	100%	23%	66%
Normal	23%	100%	47%
Conservative	66%	47%	100%

[Figure 17b - Comparison of the coefficient B]

Figure 17. Levels of Significance for Coefficients: For Aggressive, Normal, and Conservative Driver Behaviors

Comparing the two-fluid trends, as in Figures 15 and 20, the normal behavior curve remains within the two test behavior curves throughout the range of data found in Roanoke. The slopes, however, are not equal, although the conservative curve is approximately parallel to the aggressive curve. Referring to Figure 20, the intercept, T_m , of the normal curve is close to that of the conservative curve, while the aggressive intercept is much lower. The aggressive T_m (minimum travel time) is artificially determined not by any chase-car data, but by the geometric

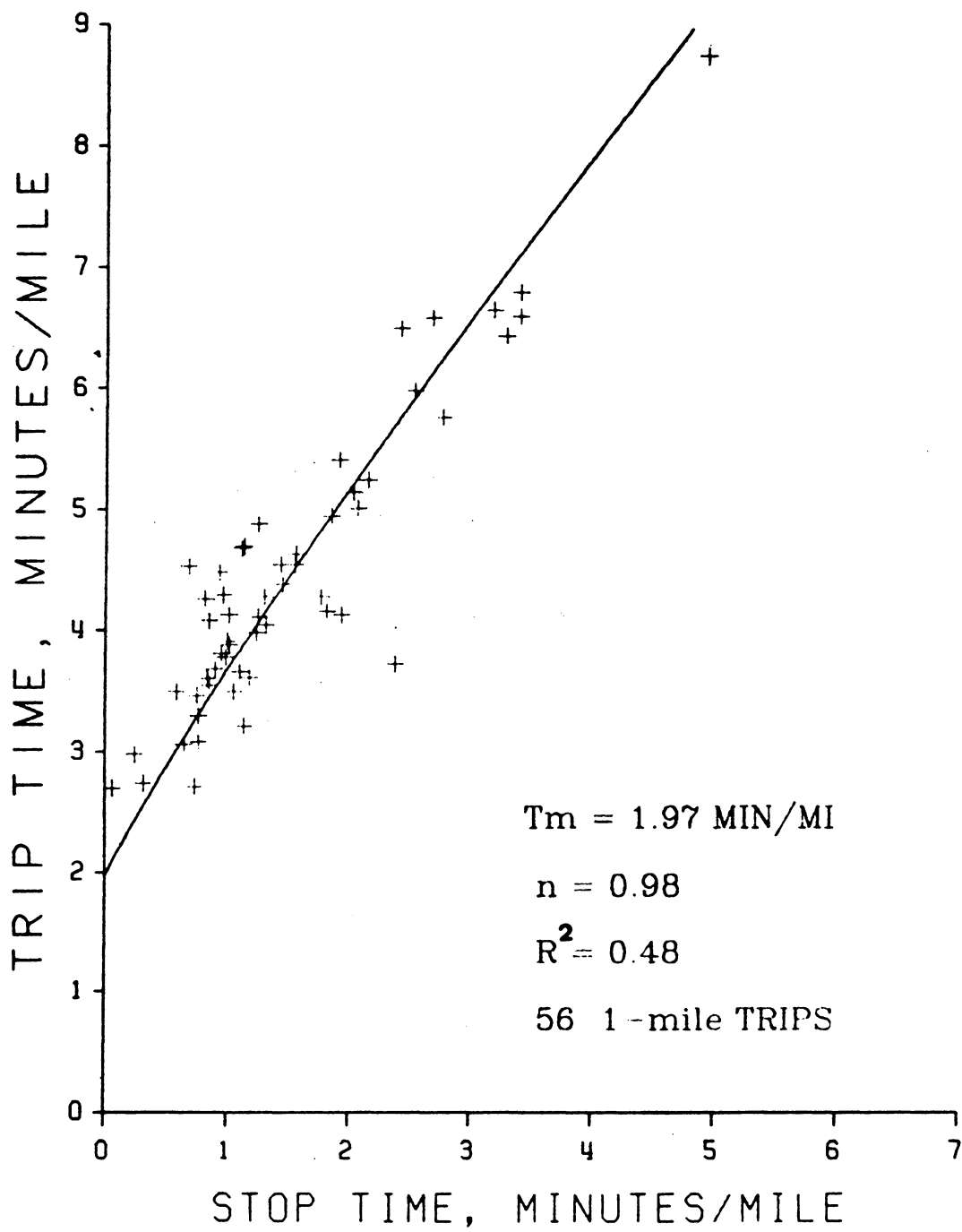


Figure 18. Two-Fluid Trend for Aggressive Behavior

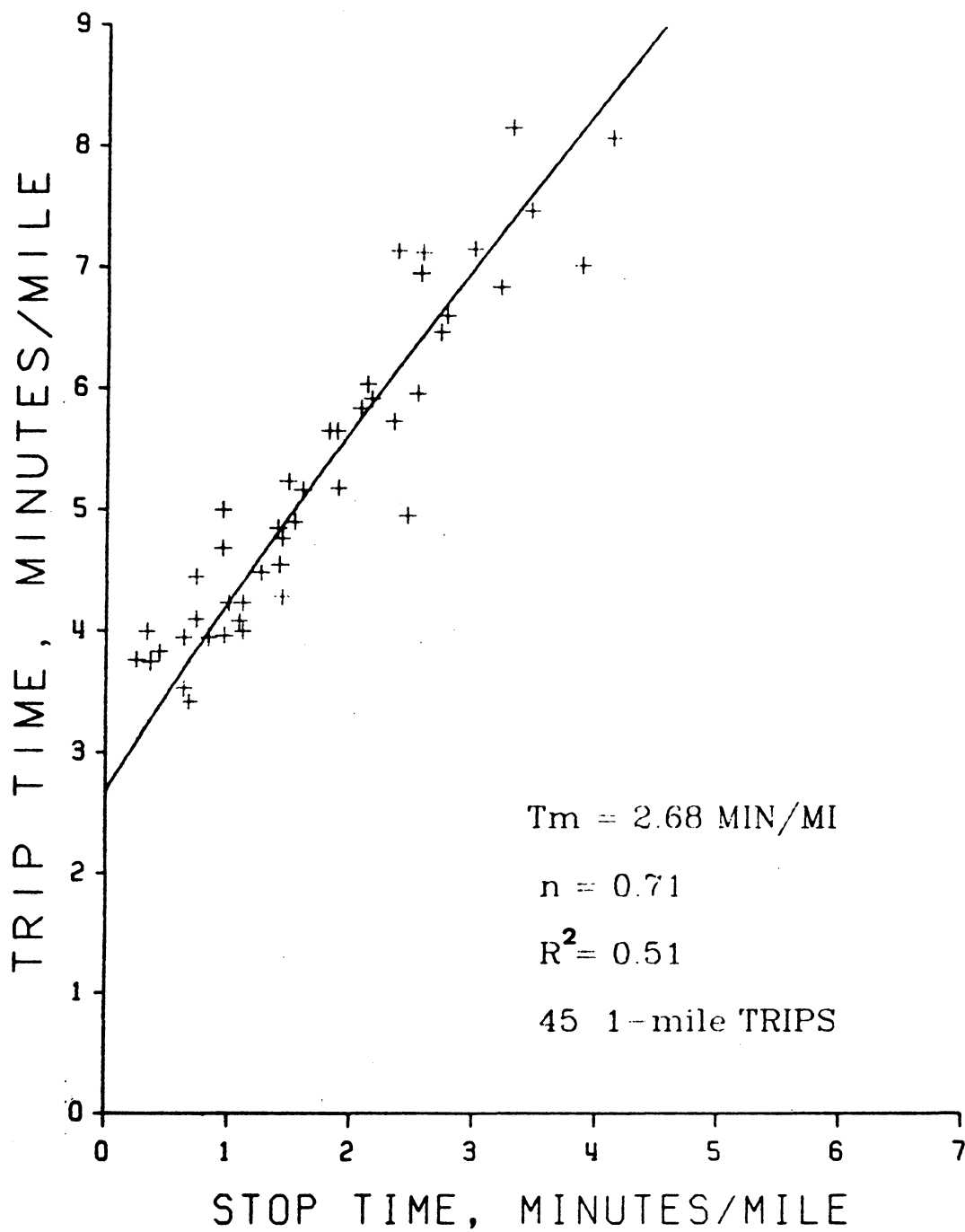


Figure 19. Two-Fluid Trend for Normal Behavior

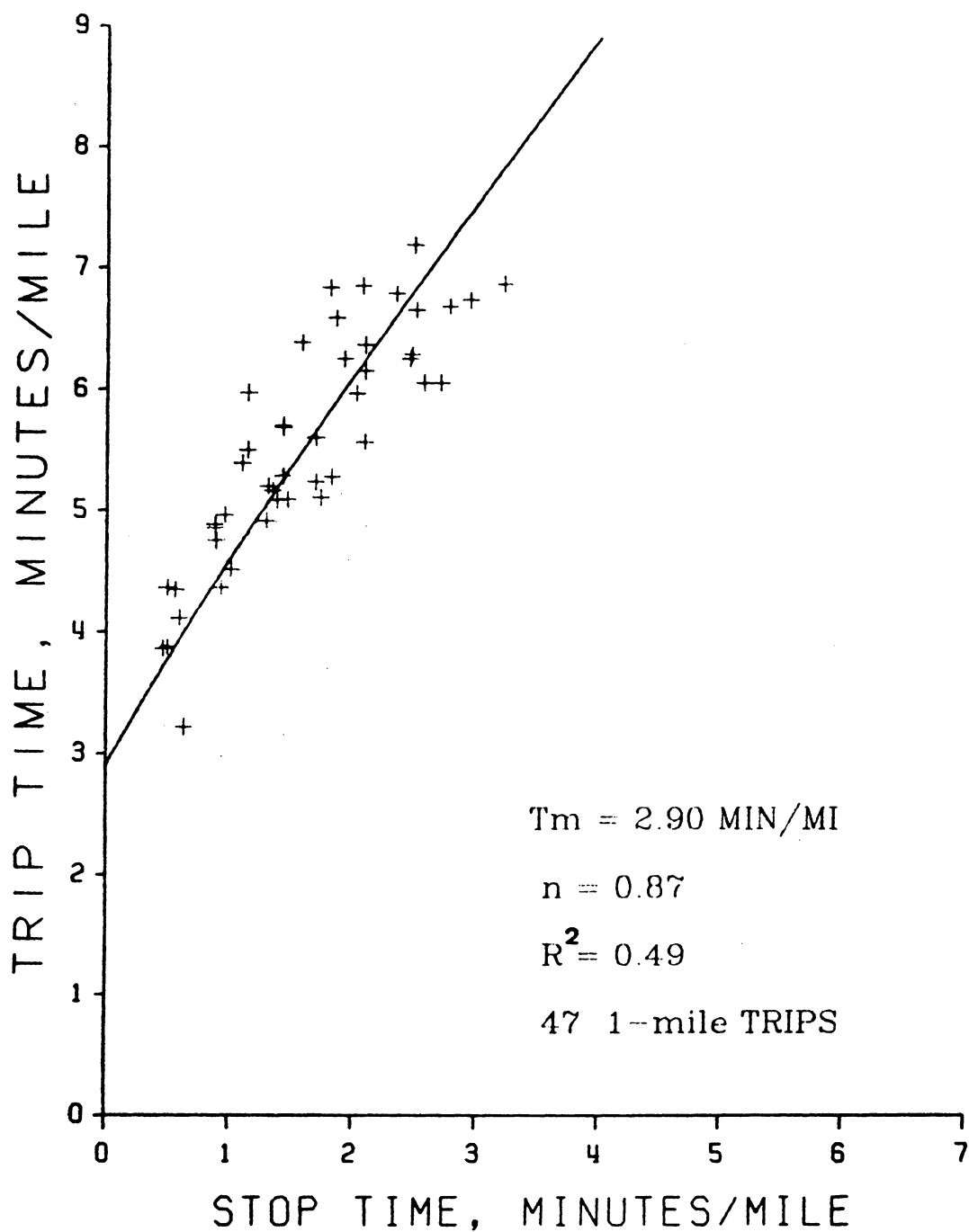


Figure 20. Two-Fluid Trend for Conservative Behavior

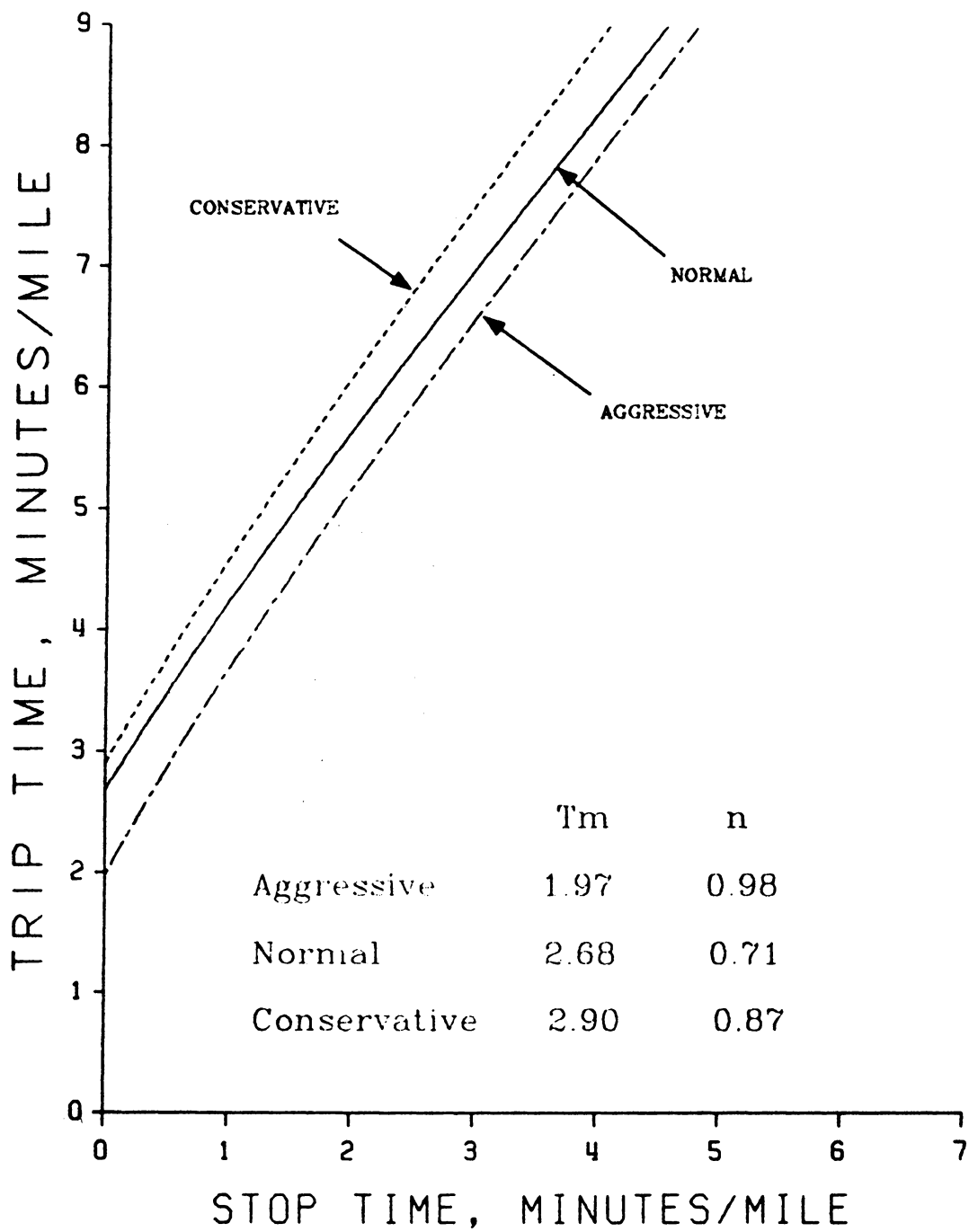


Figure 21. Two-Fluid Trends for Test and Control Behaviors

conditions that limit the aggressive driver's maximum speed. The aggressive n causes the curve to approach the normal curve at high trip times since the slope is mainly influenced by the value of n . This implies that at high traffic volumes, the aggressive driver is forced to drive "normally."

On the other hand, the conservative curve begins and retains higher trip times for any given stop time than the other two curves. Although the conservative T_m is close to the normal T_m , the higher conservative n value implies that a conservative driver will always maintain a lower speed than the surrounding traffic.

The results shown in Figure 17 are markedly different from those determined for the trip length comparisons in Figure 12. Mainly, a couple of the LOS values in Figure 17 are significantly small, especially the LOS used to compare the coefficient A between the aggressive and the normal behavior data which was found to be less than 3 percent. This result helps justify the conclusion that the differing artificial constraints on the driver behaviors tends to decrease the correlation between and among them, and can have a significant effect on the Two-Fluid model parameters.

6.0 CONCLUSIONS AND DISCUSSIONS

The use of the Two-Fluid model as a tool to measure the quality of service in city traffic networks has proven to be fairly effective. This conclusion is drawn from the results in chapter 3, where the model is calibrated for the central business district of Roanoke, Virginia. The model was found to explain over 79 percent of the variation in the data when regressing the logarithm of running time versus the logarithm of trip time.

Calibrating the model for Roanoke produced the Two-Fluid parameters minimum travel time, T_m , and an exponential coefficient affecting slope, n . These values, $T_m = 1.92$ and $n = 1.60$, are characteristic parameters of the city in question, and are the basis of comparison between the qualities of service of two different cities. Using the parameters T_m and n , Two-Fluid trends were drawn for various cities around the world, which relate the stop time to trip time for any given fraction of vehicles stopped, F_s . For any given F_s , the quality of service of Roanoke was determined to be better than that of Matamoros, Mexico, and poorer in comparison to Milwaukee, Wisconsin. At values of F_s greater than 31 percent, the quality of service in Roanoke was found to be better than that of Brussels, Belgium, and poorer for any F_s less than 31 percent.

After calibrating the model using the Chase-Car technique (a method that unbiasedly reproduces that travel-time histories of users of the traffic network), the interest of the research led to an analysis of the sensitivity of the Two-Fluid model parameters. The first aspect to be analyzed were the effects of unit trip length on the model calibration. The three unit trip lengths tested were 1-mile, 2-mile, and 3-mile. The model parameters T_m and n were computed and statistical tests were performed on their respective coefficients of regression. This led to the conclusion that the model parameters are statistically not different for a high Level of Significance.

Another aspect of parameter sensitivity were the effects of driver behavior. Two general types that were tested include "aggressive" and "conservative" which were compared to "normal" behavior data as determined by the Chase-Car technique. From the two-fluid trends the aggressive data showed a low minimum travel time, T_m , while the T_m values found for the normal and conservative behaviors were relatively close. The aggressive curve was found to converge with the normal, while the conservative behavior curve continuously maintained a higher trip time for any given stop time. This implies that an aggressive driver can enjoy low travel times during off-peak conditions, but is forced to accept "normal" travel times during peak flows. On the other hand, conservative drivers maintain higher travel times and

stop times than the surrounding traffic regardless of traffic conditions for a given level of vehicular demand.

APPENDIX A. CBD MAP OF ROANOKE, VIRGINIA



APPENDIX B. FORTRAN LISTING

```
$JOB
C   TO RUN THIS PROGRAM ISSUE:      WATFIV DPROP1
C   THE OUTPUT GOES TO A FILE CALLED: DPROP1 LISTING
C   PROGRAM DPROP(INPUT,OUTPUT,TAPE1=INPUT,TAPE2=OUTPUT)
      DIMENSION TT(2999),TSI(2999),ZFS(2999),ZV(2999),ZNS(2999)
      DIMENSION ZNSP(2999)
      DIMENSION ZTS(2999),ZTR(2999),
-      ZT(2999),ZTL(2999),ZTRL(2999)
      IX=0
1000  IX=IX+1
      RUNTOT=0.
      READ(5,*)D
      I=0
      2  I=I+1
      READ(5,5)TT(I)
      5  FORMAT(F6.2)
      IF(TT(I).GT.0.)GOTO2
      K=I-1
      DO 11 J=1,K
11     TT(J)=(TT(J)-FLOAT(INT(TT(J))))*100.0+
      1  (FLOAT(INT(TT(J)))*60.0)
      TOT=TT(K)-TT(1)
      DO 12 J=1,K,2
      L=J+1
      RUNINC=TT(L)-TT(J)
      RUNTOT=RUNTOT+RUNINC
      TSI(J)=0.
      TSI(L)=TT(L+1)-TT(L)
12     STOPTO=TOT-RUNTOT
      T=TOT/(60.*D)
      TS=STOPTO/(60.*D)
      TR=RUNTOT/(60.*D)
      TL=ALOG10(ABS(T))
      TRL=ALOG10(ABS(TR))
      FS=(TS/T)*100.
      V=(1./T)*60.
      ZT(IX)=T
      ZTS(IX)=TS
      ZTR(IX)=TR
      ZTL(IX)=TL
      ZTRL(IX)=TRL
      ZFS(IX)=FS
      ZV(IX)=V
      NS=INT(FLOAT(K-2)/2.)
      ZNS(IX)=NS
```

```

      ZNSP(IX)=ZNS(IX)/D
      IF(TT(I).LT.0.)GOTO3
      GO TO 1000
3  WRITE(6,22)
22  FORMAT(//4X,'TS',3X,'TR',8X,'T',
-3X,'LOG(T)',1X,'LOG(TR)',3X
*, 'FS',3X,'V',7X,'NS',3X,'NS/MILE')
      DO20 J=1,IX
      WRITE(6,21)ZTS(J),ZTR(J),ZT(J),ZTL(J),
-ZTRL(J),ZFS(J),ZV(J),ZNS(J),
-ZNSP(J)
21  FORMAT(9(1X,F6.3))
20  CONTINUE
      STOP
      END
$ENTRY

```

APPENDIX C. FORTRAN OUTPUT

TS	TR	T	LOG(T)	LOG(TR)	FS	V	NS	NS/MILE
3.333	4.517	7.850	0.895	0.655	42.463	7.643	11.000	11.000
0.917	3.500	4.417	0.645	0.544	20.754	13.585	10.000	10.000
1.017	2.617	3.633	0.560	0.418	27.982	16.514	4.000	4.000
0.617	3.133	3.750	0.574	0.496	16.445	16.000	1.000	1.000
2.983	4.700	7.683	0.886	0.672	38.829	7.809	8.000	8.000
0.742	2.136	2.879	0.459	0.330	25.790	20.842	1.000	0.909
1.267	2.233	3.500	0.544	0.349	36.190	17.143	1.000	1.000
0.067	2.383	2.450	0.389	0.377	2.722	24.490	2.000	2.000
1.300	3.067	4.367	0.640	0.487	29.771	13.740	9.000	9.000
0.900	3.283	4.183	0.622	0.516	21.514	14.343	4.000	4.000
0.848	2.864	3.712	0.570	0.457	22.857	16.163	7.000	6.364
2.167	3.153	5.319	0.726	0.499	40.731	11.279	8.000	6.667
1.217	2.917	4.133	0.616	0.465	29.436	14.516	4.000	4.000
4.067	4.350	8.417	0.925	0.638	48.316	7.129	11.000	11.000
1.409	3.091	4.500	0.653	0.490	31.313	13.333	5.000	4.545
1.333	3.150	4.483	0.652	0.498	29.740	13.383	6.000	6.000
1.733	3.700	5.433	0.735	0.568	31.902	11.043	5.000	5.000
2.950	3.733	6.683	0.825	0.572	44.139	8.978	11.000	11.000
2.350	3.600	5.950	0.775	0.556	39.495	10.084	7.000	7.000
1.883	3.633	5.517	0.742	0.560	34.139	10.876	8.000	8.000
0.450	2.917	3.367	0.527	0.465	13.366	17.822	4.000	4.000
2.550	4.383	6.933	0.841	0.642	36.779	8.654	8.000	8.000
1.258	4.348	5.606	0.749	0.638	22.432	10.703	12.000	10.909
1.050	3.400	4.450	0.648	0.531	23.596	13.483	4.000	4.000
0.767	2.317	3.083	0.489	0.365	24.864	19.459	2.000	2.000
0.533	3.017	3.550	0.550	0.480	15.023	16.901	3.000	3.000
1.050	3.117	4.167	0.620	0.494	25.201	14.400	5.000	5.000
0.879	2.652	3.530	0.548	0.423	24.893	16.996	4.000	3.636
1.061	4.773	5.833	0.766	0.679	18.182	10.286	8.000	7.273
0.408	2.492	2.900	0.462	0.396	14.081	20.690	8.000	4.000
0.267	3.100	3.367	0.527	0.491	7.920	17.822	6.000	6.000
0.050	2.433	2.483	0.395	0.386	2.014	24.161	2.000	2.000
0.967	3.450	4.417	0.645	0.538	21.887	13.585	3.000	3.000
1.650	3.333	4.983	0.698	0.523	33.110	12.040	9.000	9.000
0.900	3.200	4.100	0.613	0.505	21.951	14.634	4.000	4.000
0.679	3.000	3.679	0.566	0.477	18.467	16.307	7.000	5.385
1.150	3.117	4.267	0.630	0.494	26.953	14.063	5.000	5.000
0.367	2.900	3.267	0.514	0.462	11.224	18.367	5.000	5.000
1.197	2.758	3.955	0.597	0.441	30.269	15.172	4.000	3.636
3.133	3.700	6.833	0.835	0.568	45.854	8.780	9.000	9.000
0.485	2.545	3.030	0.481	0.406	16.000	19.800	4.000	3.636
1.517	3.433	4.950	0.695	0.536	30.640	12.121	7.000	7.000
0.767	2.583	3.350	0.525	0.412	22.886	17.910	2.000	2.000
1.750	3.617	5.367	0.730	0.558	32.608	11.180	7.000	7.000
1.141	3.090	4.231	0.626	0.490	26.970	14.182	7.000	5.385
0.633	3.567	4.200	0.623	0.552	15.080	14.286	6.000	6.000
1.367	3.000	4.367	0.640	0.477	31.298	13.740	5.000	5.000

0.783	2.550	3.333	0.523	0.407	23.500	18.000	3.000	3.000
0.683	3.133	3.817	0.582	0.496	17.904	15.721	7.000	7.000
0.000	1.617	1.617	0.209	0.209	0.000	37.113	0.000	0.000
0.317	2.800	3.117	0.494	0.447	10.160	19.251	2.000	2.000
1.067	3.417	4.483	0.652	0.534	23.792	13.383	6.000	6.000
2.650	2.817	5.467	0.738	0.450	48.476	10.976	6.000	6.000
1.233	4.333	5.567	0.746	0.637	22.157	10.778	6.000	6.000
0.483	2.600	3.083	0.489	0.415	15.676	19.459	4.000	4.000
1.350	3.483	4.833	0.684	0.542	27.931	12.414	8.000	8.000
2.600	4.750	7.350	0.866	0.677	35.375	8.163	10.000	10.000
0.700	4.217	4.917	0.692	0.625	14.237	12.203	4.000	4.000
2.042	3.556	5.597	0.748	0.551	36.477	10.720	11.000	9.167
0.983	2.900	3.883	0.589	0.462	25.322	15.451	4.000	4.000
0.750	2.683	3.433	0.536	0.429	21.845	17.476	3.000	3.000
1.444	2.389	3.833	0.584	0.378	37.681	15.652	5.000	4.167
1.400	3.250	4.650	0.667	0.512	30.108	12.903	6.000	6.000
1.533	3.367	4.900	0.690	0.527	31.292	12.245	6.000	6.000
0.650	2.117	2.767	0.442	0.326	23.494	21.687	2.000	2.000
1.833	3.333	5.167	0.713	0.523	35.484	11.613	10.000	7.692
1.250	3.900	5.150	0.712	0.591	24.272	11.650	10.000	10.000
1.750	3.433	5.183	0.715	0.536	33.762	11.576	6.000	6.000
1.517	2.967	4.483	0.652	0.472	33.829	13.383	5.000	5.000
2.800	3.400	6.200	0.792	0.531	45.161	9.677	7.000	7.000
1.750	3.567	5.317	0.726	0.552	32.915	11.285	4.000	4.000
1.633	3.517	5.150	0.712	0.546	31.715	11.651	6.000	6.000
0.683	3.033	3.717	0.570	0.482	18.387	16.143	7.000	7.000
0.117	2.867	2.983	0.475	0.457	3.911	20.112	5.000	5.000
1.417	3.017	4.433	0.647	0.480	31.954	13.534	4.000	4.000
1.883	4.017	5.900	0.771	0.604	31.922	10.169	7.000	7.000
1.483	3.567	5.050	0.703	0.552	29.374	11.881	6.000	6.000
1.183	4.733	5.917	0.772	0.675	20.000	10.141	10.000	10.000
1.433	2.850	4.283	0.632	0.455	33.463	14.008	4.000	4.000
0.083	2.633	2.717	0.434	0.421	3.067	22.086	3.000	3.000

APPENDIX D. MINITAB SAMPLE REGRESSION OUTPUT

MTB > REGRESS C3 1 C1

THE REGRESSION EQUATION IS

$C3 = 2.61 + 1.49 C1$

COLUMN	COEFFICIENT	ST. DEV. OF COEF.	T-RATIO = COEF/S.D.
	2.6104	0.1111	23.50
C1	1.48712	0.07324	20.30

S = 0.5303

R-SQUARED = 84.1 PERCENT

R-SQUARED = 83.9 PERCENT, ADJUSTED FOR D.F.

ANALYSIS OF VARIANCE

DUE TO	DF	SS	MS=SS/DF
REGRESSION	1	115.92	115.92
RESIDUAL	78	21.93	0.28
TOTAL	79	137.85	

ROW	C1	Y C3	PRED. Y VALUE	ST.DEV. PRED. Y	RESIDUAL	ST.RES.
1	3.33	7.8500	7.5669	0.1614	0.2831	0.56 X
14	4.07	8.4170	8.6585	0.2124	-0.2415	-0.50 X
23	1.26	5.6060	4.4812	0.0593	1.1248	2.13R
29	1.06	5.8330	4.1882	0.0615	1.6448	3.12R
40	3.13	6.8330	7.2695	0.1479	-0.4365	-0.86 X
53	2.65	5.4670	6.5512	0.1164	-1.0842	-2.10R
54	1.23	5.5670	4.4440	0.0594	1.1230	2.13R
58	0.70	4.9170	3.6513	0.0730	1.2657	2.41R
78	1.18	5.9170	4.3696	0.0597	1.5474	2.94R

R DENOTES AN OBS. WITH A LARGE ST. RES.

X DENOTES AN OBS. WHOSE X VALUE GIVES IT LARGE INFLUENCE.

DURBIN-WATSON STATISTIC = 2.06

MTB > REGRESS C5 1 C4

THE REGRESSION EQUATION IS
C5 = 0.109 + 0.616 C4

COLUMN	COEFFICIENT	ST. DEV. OF COEF.	T-RATIO = COEF/S.D.
	0.10891	0.02339	4.66
C4	0.61583	0.03603	17.09

S = 0.04119

R-SQUARED = 78.9 PERCENT

R-SQUARED = 78.7 PERCENT, ADJUSTED FOR D.F.

ANALYSIS OF VARIANCE

DUE TO	DF	SS	MS=SS/DF
REGRESSION	1	0.49577	0.49577
RESIDUAL	78	0.13236	0.00170
TOTAL	79	0.62813	

ROW	C4	Y C5	PRED. Y VALUE	ST.DEV. PRED. Y	RESIDUAL	ST.RES.
7	0.544	0.34900	0.44392	0.00569	-0.09492	-2.33R
14	0.925	0.63800	0.67855	0.01136	-0.04055	-1.02 X
29	0.766	0.67900	0.58064	0.00655	0.09836	2.42R
50	0.209	0.20900	0.23762	0.01608	-0.02862	-0.75 X
53	0.738	0.45000	0.56339	0.00588	-0.11339	-2.78R
58	0.692	0.62500	0.53506	0.00502	0.08994	2.20R
62	0.584	0.37800	0.46855	0.00498	-0.09055	-2.21R
78	0.772	0.67500	0.58433	0.00671	0.09067	2.23R

R DENOTES AN OBS. WITH A LARGE ST. RES.

X DENOTES AN OBS. WHOSE X VALUE GIVES IT LARGE INFLUENCE.

DURBIN-WATSON STATISTIC = 2.08

MTB > STOP

MINITAB * STATISTICS DEPT * PENN STATE UNIV.

APPENDIX E. RANDOM ROUTING NAVIGATION GUIDE

[illegible]

REFERENCES

1. Herman, R. and I. Prigogine, "A Two-Fluid Approach to Town Traffic," Science, Vol. 204, pp. 148-151, 1979.
2. Herman, R. and S. Ardekani, "Characterizing Traffic Conditions in Urban Areas," Transportation Science, Vol. 18, No. 2, pp. 101-140, 1984.
3. Chang, M.-F. and R. Herman, "Trip Time Versus Stop Time and Fuel Consumption Characteristics in Cities," Transportation Science, Vol. 15, No. 3, pp. 183-209, 1981.
4. Herman, R., and S. Ardekani, "Characterizing the Quality of Traffic Service in Urban Street Networks," Center for Transportation Research Report No. 304-2F, University of Texas at Austin, November 1984.
5. Pignataro, L., Traffic Engineering: Theory and Practice, Prentice Hall, Inc., Englewood Cliffs, New Jersey, 1973.
6. Chang, M.-F. and R. Herman, "An Attempt to Characterize Traffic in Metropolitan Areas," Transportation Science, Vol. 12, No. 1, pp. 58-79, 1978.
7. Ryan, T., B. Joiner, and B. Ryan, MINITAB Student Handbook, Duxbury Press, North Scituate, Massachusetts, 1976.
8. Ardekani, S. and R. Herman, "A Comparison of the Quality of Traffic in Downtown Networks of Various Cities Around the World," Traffic Engineering and Control, Vol. 26, No. 12, 1985.
9. Herman, R., T. Lam, and I. Prigogine, "Kinetic Theory of Vehicular Traffic: Comparison with Data," Transportation Science, Vol. 6, No. 4, pp.440-452, 1972.
10. Automobile Fuel Consumption in Actual Traffic Conditions, Organisation for Economic Co-operation and Development, 1982.

**The vita has been removed from
the scanned document**