DEVELOPMENT OF LINK-CAPACITY FUNCTIONS FOR ISTANBUL FREEWAYS

by

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B.S., Civil Engineering, Boğaziçi University, 2004

Submitted to the Institute for Graduate Studies in
Science and Engineering in partial fulfillment of
the requirements for the degree of
Master of Science

Graduate Programs in Civil Engineering
Boğaziçi University
2007

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FUNCTIONS FOR
ISTANBUL FREEWAYS

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DATE OF APPROVAL: 28.06.2007

ACKNOWLEDGEMENTS

First and foremost, I would like to thank my thesis advisor, Prof. Dr. Gökmen Ergün, for his guidance throughout my study, but also for his tolerance and his belief in me. Furthermore, I would like to thank my thesis examining committee Asst Prof. Dr. Ilgın Yaşar and Assoc. Prof. Dr. Murat Çelik for their valuable comments and recommendations.

I would like to thank Istanbul Greater Municipality Metropolitan Planning & Urban Design Center, Assoc. Prof. Dr. Murat Çelik and Berna Çalışkan for providing me with the data used in this study.

I would like to thank my friends Tarcan Karaoğlu and Yaman Özakın for their helpfulness, advice and valuable time.

Finally, I would like to express my gratitude to my family, especially my grandfather and grandmother for their love, encouragement, patience, trust and support that made everything possible. I dedicate this dissertation to my family.

ABSTRACT

DEVELOPMENT OF LINK-CAPACITY FUNCTIONS FOR ISTANBUL FREEWAYS

The relationships between traffic flow and travel time are called link-capacity functions and are fundamental inputs for capacity restrained trip assignment methods. In order to estimate realistic results for travel demand, trip assignment is to be handled carefully and for this purpose link-capacity functions are necessary. Unfortunately, there is not an ultimate link-capacity function being valid for any road; instead there are many different types of functions with different parameters. These parameters show dependency to the links, vehicle and driver characteristics of that specific region. Istanbul, being the most crowded city of Turkey has its own specific parameters, too. This study aimed to calibrate suitable link-capacity functions and the estimates of their parameters for Istanbul. This was achieved by first investigating the basic relationships between traffic stream parameters and after finding out the characteristic inputs for the functions, such as free flow speed, and capacity. The two dimensional traditional approach to the relationships between these traffic parameters and the estimation of them is followed by a three dimensional newer approach to a small part of the existing data in order to see its relevance, experimentally. The results of this study may lead the transportation planners of Istanbul to convenient forecast analysis.

ÖZET

ISTANBUL OTOYOLLARI İÇİN LİNK-KAPASİTE FONKSİYONLARININ GELİŞTİRİLMESİ

Trafik akımı ve seyahat süresi arasındaki ilişkiler link kapasite fonskiyonları olarak tanımlanmıştır ve bu fonksiyonlar kapasite sınırlamalı seyahat atamalarında temel girdileri oluştururlar. Gerçekçi seyahat tahminleri için seyahat atamalarının dikkatle ele alınması gerekmektedir ve bu amaç için de link kapasite fonksiyonlarına ihtiyaç vardır. Ne yazık ki var olan her yol için geçerli, nihai bir link kapasite fonksiyonu mevcut değildir, onun yerine, farklı değişkenleri keşfedilmeyi bekleyen bir çok link kapasite fonksiyonu vardır. Bu değişkenler bahsi geçen yol kesitine, araç ve sürücü özelliklerine bağımlıdırlar. Türkiye'nin en kalabalık şehri olan Istanbul'un da kendine has parametreleri vardır. Bu çalışmanın amacı Istanbul için uygun link kapasite fonksiyonlarını ve onların değişkenlerini tespit etmektir. Bunun için önce trafik akım parametreleri arasındaki ilişkiler incelenip, fonksiyonlara girdi olarak kullanılacak serbest akım hızı ve kapasite gibi değerler tespit edilmiştir. Bu trafik parametreleri arasındaki ilişkilerin iki boyutlu geleneksel yaklaşımınından ve bunun sonucunda bulunan değişkenlerin ardından, üç boyutlu yeni bir yaklaşım da mevcut verilerin bir kısmına uyumunu gözlemlemek için sadece deneysel olarak uygulanmıştır. Bu çalışmanın sonuçlarının Istanbul'lu ulaştırma planlamacılarını doğru tahmin analizlerine yönlendirmesini umarım.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS / ABBREVIATIONS

a, b, c	coefficients
C	Capacity, veh/hour or veh/hour/lane
C_p	Practical capacity of a link, veh/hour or veh/hour/lane
C_s	Steady state capacity of a link, veh/hour or veh/hour/lane
d	Distance or length of the link
D	Density, veh/km or veh/km/lane
E_R	Passenger car equivalent for recreational vehicles
E_T	Passenger car equivalent for trucks or buses
F_1	Maximum flow at which free flow conditions prevail
F_2	Capacity flow
f_{HV}	Heavy vehicle adjustment factor
f_p	Driver population factor
k	Density, veh/km
L_v	Average length of the vehicle, m
L_d	Length of the detector, m
N	Number of lanes
O	Occupancy, %
P_R	Proportion of recreational vehicles
P_T	Proportion of trucks or buses
q, Q	Rate of flow, veh/hour or veh/hour/lane
S	Space mean speed, km/hour
S_0	Free flow speed, km/hour
S_1	Speed at capacity flow F_2
T	Travel time per unit distance at flow Q, sec/km
T_0	Travel time per unit distance at zero flow, sec/km
T_A	Travel time at practical capacity, sec/km
T_B	Travel time at steady state capacity, sec/km
u	Space mean speed, km/hour

v	Rate of flow, veh/hour or veh/hour/lane
V	Hourly volume, vph
V_{10}	Volume during the peak 10 min. of the peak hour, veh/10 min.
V_{15}	Volume during the peak 10 min. of the peak hour, veh/10 min.
V_p	15-min. passenger car equivalent flow rate, pcphpl
X	State variable
Y, Z	Control variables

α, β, γ, δ	Parameters of the link-capacity functions

D_{jam}	Jam Density
df	Degree of freedom
FFS	Free flow speed
km	Kilometer
min	minute
pc	Passenger car
pcphpl	Passenger car per hour per lane
PHF	Peak hour factor
RTMS	Remote traffic microwave sensor
sec	second
SPSS	Statistical package for social sciences
Std	Standard
TT	Travel time
Veh	Vehicle
vph	Vehicle per hour

1. INTRODUCTION

1.1. Problem Definition

People's need for transportation stems from the interaction among social and economical activities dispersed in space. The variety and interaction of these activities result in many determinants of transportation needs. The reasons people need to travel are endless; it may range from the unavoidable need for food to any voluntary recreational activity. Under these circumstances, it will be obvious that there will be many trip makers and trips, which may result in traffic and if the situation worsen, in traffic congestion.

Traffic congestion, with its harmful effects to efficiency, safety and environment is increasing every single day all over the world. It affects the nominal capacity of the available infrastructure, which leads to a cycle of further congestion increase. In fact, the traffic throughput measured in congested highways is usually below the nominal capacity (Kanafani, 1983). Istanbul, being one of the most crowded cities in the world with 1,5 million passenger cars in 2004 (Turkish Statistical Institute, 2004) has also this kind of problems, like not being able to provide an efficient transportation system to its inhabitants. The inefficiency of the transportation system can easily be observed through with the crowded networks during peak hours.

In transportation systems, there are non-monetary aspects of supply that are as important as the price charged by the operator. Travel time in many types of transportation is the most important attribute of supply. According to a study for Istanbul (Ergün and Şahin, 2006), the delays were calculated to be 73.9 hour/person/year. But, for transportation there are also additional factors besides the time spent in travel. Among these, the loss of value of goods while in transit, the costs for extra fuel consumption and the cost of the inconvenience and discomfort of traveling under congested conditions can be mentioned. For this reason, it is common in transportation analysis to formulate a generalized cost in order to reflect all these factors (Kanafani, 1983).

Transportation demand analysis plays an important role by providing a framework for estimating the needs for transportation, and for forecasting the volumes of traffic that will use present and planned transportation facilities. This forecasting is essential for the design of additional transportation facilities and for the evaluation of their economic feasibility. The four main steps of the classical transportation demand forecasting model are trip generation, trip distribution, modal split and trip assignment (Papacostas and Prevedouros, 1993). Trip generation is the part where the total number of trips are generated and attracted to each zone within the network. Trip distribution allocates these trips to particular destinations. Modal split is the section involving the choice of mode and the final step deals with the assignment of the trips by each mode to their corresponding networks using the link-capacity functions. The assignment is done according to the travel times calculated with these functions.

The critical challenge of traffic engineering is to plan and design for a medium that is not easily predictable. It involves both physical constraints of the environment and complex behavioral characteristics of human beings (Roess et al., 2004). Even if people behave in a rational manner and do so consistently, there will always be certain perturbations that cause behavior not to repeat itself exactly when the decision process is repeated. Either due to the inability of the modeling schemes to sufficiently represent all causes of these perturbations, or because the behavioral process itself is not very well defined, even in the mind of the decision maker, the models of demand contain random elements. The incorporation of stochastic analysis into demand modeling and the introduction of probability and statistics as tools for the analysis have certainly advanced the state of the art (Kanafani, 1983).

Another problem for the traffic engineers is that the need for transportation is manifested in the form of traffic volume; however the volume of traffic by itself may be a misleading indication of the true need for transportation because it represents a need that is tempered by the availability of transportation services. Clearly, the traffic volume in a congested transportation facility that is operating at capacity can not be taken as a manifestation of the true need for transportation, for it does not include additional traffic that might flow into the facility if additional capacity were available to carry it. If the addition of capacity leads to an increase in traffic volume, then obviously the potential traffic is greater than the traffic volume originally observed.

Much of what determines the attributes of transportation supply is a result of user rather than supplier behavior. Many of the important aspects of transportation level of service that directly affect the evolution of traffic flows depend on how travelers use the available transportation systems and cannot therefore be considered as supply attributes. As an example; in urban passenger transportation, travel time is determined largely by the traveler's choice of route. In rural highway transportation, the travel time and vehicle operating cost incurred by an automobile traveler depend primarily on speed which, within limits, is largely at the discretion of the driver (Kanafani, 1983).

Determining these driver dependent variables is a challenge for transportation engineer, but it is quite necessary for transportation demand analysis. In order to estimate the travel demand, a fundamental part of it, namely, traffic assignment has to be handled diligently. To assign trips to the network realistically enough, some parameters such as free flow speed, travel time and capacity should be determined first. This study aims to estimate link-capacity functions that explain the relationship between these parameters.

With the data obtained from the traffic sensors placed by the Istanbul Municipality, first the basic relationships have been explored at some selected sections of the freeway network. Second, the characteristics of the freeway link i. e., capacity, jam density, free flow speed, and free flow travel time have been estimated, which are necessary for the determination of parameters of link-capacity functions.

According to the Summary Statistics on Transportation in 2004 (Turkish Statistical Institute, 2004), the circulation and transportation on motorways forms 15 per cent of the total roads (See Table 1.1) and considering that the total motorway length of Istanbul is about 230 km. which is more than 10 per cent of the total length in Turkey, 1892 km. Hence, the important role of freeways among all roads and the value of any study regarding the freeways can be understood easily.

Table 1.1 Circulation and Transportation on Motorways (Turkish Statistical Institute, 2004)

	vehicle/km	ton/km	passenger/km
motorways	7764 (13%)	23735 (15%)	25979 (15%)
total	57767	156853	174312

in millions

1.2. Goals and Objectives

The parameters of link-capacity functions are sensitive to the properties of the region; therefore they may show variance at different cities. The major goal of this research is to determine these parameters for the freeways of Istanbul, and to understand the nature of the traffic flow characteristics and their interaction with each other. With this understanding and through the use of the developed functions it is hoped to make better estimates for new facilities or for the expansion of the existing ones. To reach these goals this research is focused on the objectives listed below:

- To search the literature about the link-capacity functions used for the traffic assignment.
- To study the relationships of traffic stream parameters in two and three dimensions.
- To calibrate link-capacity functions to be used for the assignment models by Istanbul Greater Municipality Metropolitan Planning & Urban Design Center.

2. LITERATURE REVIEW

2.1. Traffic Stream Characteristics

Traffic streams are formed by individual drivers and their interaction with each other and with the physical elements of the roadway. Since there is always a variation between different drivers, vehicles and roads, it is expected that the traffic stream do not behave in the same manner which makes traffic flow a complex phenomenon (Roess et al.). Understanding traffic flow characteristics in order to predict the dynamics of these characteristics under given demand, supply and control conditions is an essential requirement for the planning, design and operation of a transportation system. However, it is highly unlikely that traffic science will ever produce a complete theory on the motion of individual cars. But, the last five decades have provided some incomplete, neither deductive nor inductive, but rather intermediate theories, such as basic mathematical model structures (Papageorgiou et al., 2006).

There are three main approaches to the understanding and quantification of traffic flow. Macroscopic approach describes traffic stream as a whole while microscopic approach considers each individual vehicle separately. The third one is human-factor approach, which tries to define the mechanism by which an individual driver locates himself with reference to other vehicles and to the highway system. The three principal macroscopic parameters that describe a traffic stream are volume or rate of flow, speed, and density or occupancy.

2.1.1. Speed

Speed is defined as rate of motion in distance per unit time. Since there is a broad distribution of individual speeds in a traffic stream, an average traffic travel speed is considered. There are two ways in which an average speed for a traffic stream can be computed: Time mean speed is the arithmetic mean of the measured speeds of all vehicles passing a fixed point during a given interval of time. Space mean speed is the average speed of all vehicles occupying a given section of highway over some specified time period. Space mean speed will be used in the remaining of this study, since it is the one, used in the fundamental relationship mentioned in Equation 2.1.

2.1.2. Volume or Rate of Flow

Volume and rate of flow are two different measures. Traffic volume is the actual number of vehicles observed to be passing a point during a given time interval. The rate of flow represents the number of vehicles passing a point during a specified time interval less than 1 hour, but expressed as an equivalent hourly rate (Roess et al., 2004). For one specific hour of volume count, rate of flow might be, and is most of the time, different from the volume values as can be seen in the example presented in Table 2.1. The volume counts for this example hour comes out to be 4200 vehicles, while rate of flow during 17:30-17:45, for example, is found out as 4800 vehicle/hour.

Table 2.1. Illustration of volumes and rates of flow (Roess et al., 2004)

Time Period	Volume (veh)	Rate of flow (veh/hour)
17:00-17:15	1000	4000
17:15-17:30	1100	4400
17:30-17:45	1200	***4800***
17:45-18:00	900	3600
17:00-18:00	***4200***	

2.1.3. Density

Density is defined as the number of vehicles occupying a given length of lane or roadway, generally expressed as vehicles per kilometer (veh/km). Direct measurement of density can be obtained by aerial photography; therefore, it is difficult to measure this parameter from the field. Hence, it is calculated from equation (2.1), which is known as the fundamental relationship in traffic engineering (Roess et al., 2004).

$$v = S \times D \tag{2.1}$$

where,

v= Rate of flow (veh/hour or veh/hour/lane),
S= Space mean speed (km/hour),
D= Density (veh/km or veh/km/lane).

2.1.4. Occupancy

An easier parameter to measure is the occupancy. Lane occupancy is defined as the ratio of the sum of vehicle lengths to the length of the road section. It can also be described as the ratio of the time that vehicles are present at a detection station in a traffic lane compared to the sampling time. In most cases, the detector is actuated as soon as the front bumper crosses the detector area and remains on until the rear bumper leaves the detector area. There is a relationship between occupancy and density, if the detector is a magnetic loop detector with a linear vision of sight, which is given by Equation 2.2 below.

$$D = \frac{1000 \times O}{L_v + L_d} \tag{2.2}$$

where,

D = Density (veh/km/lane),
O = Occupancy (%),
L_v= Average length of the vehicle (m.),
L_d= Length of the detector (m.).

The relationships of these characteristics are summarized in Figure 2.1 (Transportation Research Board, 2000). In this approach, the relationships between speed vs. flow rate and flow rate vs. density are assumed to be quadratic, while the speed vs. density relationship is linear. The first derivative of the quadratic relationship between speed and flow rate gives maximum flow rate, in other words, capacity. With that information the speed at capacity can be found. The first derivative of the quadratic relationship between flow rate and density gives also the capacity value. One of the roots of this equation determines the jam density, the value of density where no flow is observed (Roess et al.).

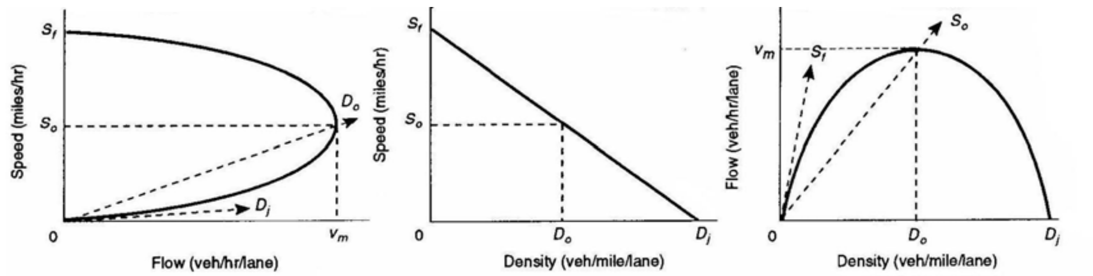

Figure 2.1. Relationships between traffic stream characteristics, traditional approach (Transportation Research Board, 2000)

Speed, volume, and density are the macroscopic parameters that describe the traffic stream; on the other hand, microscopic parameters include speed of individual vehicles, headway, and spacing, which are not the concern of this study. Some basic definitions of these parameters are given in the following sections. For further information, refer to Roess et al., 2004.

2.1.5. Headway

Headway is the time interval between successive vehicles as they pass the observer's location one after another.

2.1.6. Spacing

Spacing is defined as the distance between successive vehicles, measured from some common reference point, such as front bumper or front wheels.

2.2. Traffic Assignment

Traffic assignment problem is the distribution of traffic in a network considering the demand between origin destination pairs and the transport supply of the network. Assignment methods try to model the distribution of traffic in a network according to a set of constraints, like capacity, time and cost (Hall, 2007). Traffic assignment model is the last step in sequential demand forecasting (See Figure 2.2). The major components of this sequence are:

1. The decision to travel for a given purpose (generation)
2. The choice of destination (distribution)
3. The choice of travel mode (modal choice)
4. The choice of route or path (assignment)

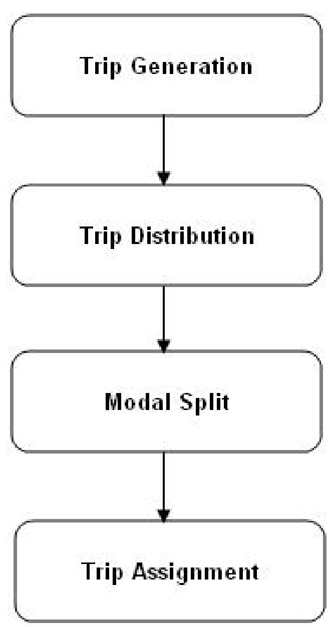

Figure 2.2. Flowchart of sequential demand forecasting models

Traffic assignment estimates the expected flows that a freeway link may experience in order to anticipate potential capacity problems and to plan accordingly. This will allow the traffic engineers to make decisions related to the geometric design and capacity of proposed freeways (Papacostas et al.). Here are the primary and secondary objectives of traffic assignment (Ortuzar et al.).

1. Primary objectives:
   - To get aggregate network measures, e.g. total freeway flows
   - To estimate the travel costs or times zone-to-zone for a specified demand
   - To obtain reasonable link flows and to identify heavily congested links.
2. Secondary objectives:
   - To estimate the routes used between origins destinations
   - To analyze which origin destination pairs use a particular link
   - To obtain turning movements for the design of future junctions.

It is possible to categorize traffic assignment models in two ways. First, they may be categorized as either all-or-nothing or multipath models, depending on whether they allocate the interzonal demand on a single path or on multiple paths. Second, they are identified either as free-assignment or capacity-restrained models, depending on whether they account for the effect of congestion or not. This categorization is summarized in Table 2.2 (Ortuzar and Willumsen, 2004). This study looks after the parameters which will be necessary in the capacity restrained traffic assignment models; therefore, the following section covers only this kind of assignment.

Table 2.2. Classification for traffic assignment (Ortuzar and Willumsen, 2004)

		Stochastic effects?	
		NO	YES
Capacity restrained?	NO	All-or-nothing	Pure stochastic
	YES	Wardrop's equilibrium	Stochastic user equilibrium

2.2.1. Capacity-Restrained Traffic Assignment

“Under equilibrium conditions traffic arranges itself in congested networks in such a way that no individual trip maker can reduce his path costs by switching routes.” (Wardrop, 1952)

"Under social equilibrium conditions traffic should be arranged in congested networks in such a way that the average or total travel cost is minimized."(Wardrop, 1952)

These are the two principles of John Glen Wardrop (Ortuzar and Willumsen, 2004). They have same concepts as in the "game theory", but in transportation networks there are many players, which make the analysis more difficult.

With the increasing flow levels to capacity, the average speed of vehicles decreases from the free flow speed to the speed at capacity. Beyond this point, the internal friction between the vehicles becomes severe, which makes the traffic conditions worse, to the level of service E or even F, and severe shock waves and slow-moving platoons develop.

As a rule of thumb of the free traffic assignment procedures, the trips are assigned to the minimum paths computed on the basis of travel costs or travel times, but if the assigned flow levels are close to or higher than the capacity of that link, the link travel times would be higher than the estimated ones. As a result the minimum paths computed prior to trip assignment may not be the minimum paths after trip assignment. The simplest way to overcome this problem is to make successive assignments with fixed link costs and to compute the travel times again between the iterations using link-capacity functions. This is referred to as the "incremental assignment technique".

2.3. Review of Link-Capacity Functions

Links are homogeneous stretches of basic freeway sections, which are outside of the influence area of ramps or weaving areas as shown in Figure 2.4. Weaving areas are the sections of the freeway, where two or more vehicle flows must cross each other's path. They occur usually when a merging area is followed by a diverging area, while ramp junctions are points at which on and off ramps join the freeway section (Transportation Research Board, 2000). Usually at weaving and ramp areas, there is some disorder since the drivers might get disturbed because of the interaction. Therefore, instead of these sections, homogeneous links are preferable for the proper analysis.

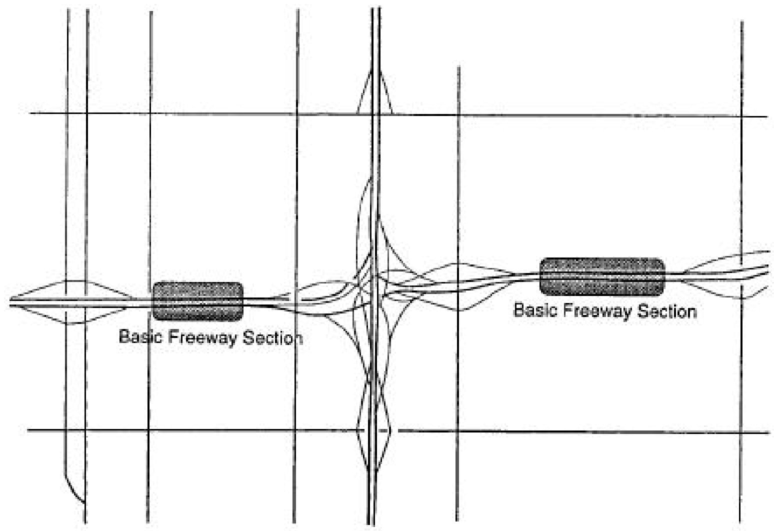

Figure 2.3. Example of basic freeway sections (Transportation Research Board, 2000)

Link-capacity functions, also known as volume delay curves, can be described as the relationship between link flow and link impedance. They represent the relationship between the travel time or cost and the flow rate. Travel time might remain the same up to a point of flow rate, namely capacity, but beyond that point travel times show rapid exponential increase, which should be explained by some mathematical functions but there is very little agreement between researchers on the type of the link-capacity function which may be suitable for any particular network. Instead, there is always some new link-capacity functions proposed for the traffic assignment procedure.

The following items are the desirable properties of mathematical functions in order to be classified as a link-capacity function which will be used for traffic assignment (Ortuzar and Willumsen, 2004):

- The modeled travel times should be realistic enough.
- The function should be non-decreasing and monotone; increasing flow should not reduce travel times, where at low flow levels the travel times are expected to be constant.
- The function should be continuous and differentiable.

- The function should allow the region with overloaded flow without generating infinite travel times.
- In order to transfer the relationship from one context to another, the use of engineering parameters like free-flow speed and capacity, are desirable.

Some proposed functions from the early 1960's until today were reviewed in this part. For the rest of this section the following notation will be used:

$Q =$ Flow on a link (veh/hour or veh/hour/lane)
$T =$ Travel time per unit distance at flow Q (sec/km)
$T_0 =$ Travel time per unit distance at zero flow (sec/km)
$C_p =$ Practical capacity[1] of a link (veh/hour or veh/hour/lane)
$C_s =$ Steady state capacity of a link (veh/hour or veh/hour/lane)
$\alpha, \beta, \gamma, \delta, ... =$ Parameters of the Link-Capacity functions

One of the earliest link-capacity functions used in an assignment procedure was the one proposed by Irwin, Dodd and Von Cube in 1961. The function is formed by two different line segments (Branston, 1976).

$$T = T_A + \alpha(Q - C_p) \ \ , \text{ for } Q < C_p , \tag{2.3}$$

$$T = T_A + \beta(Q - C_p) \ \ , \text{for } Q \geq C_p , \tag{2.4}$$

where,

$$T_A = T_0 + \alpha C_p . \tag{2.5}$$

The graph of this function consists of 2 parts as shown in Figure 2.4, first one lasts until the practical capacity, from that point on the second part takes over with a different slope and it passes through the steady state capacity into the overload region.

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[1]: Practical capacity corresponds to service volume for a level of service C in the current practice.

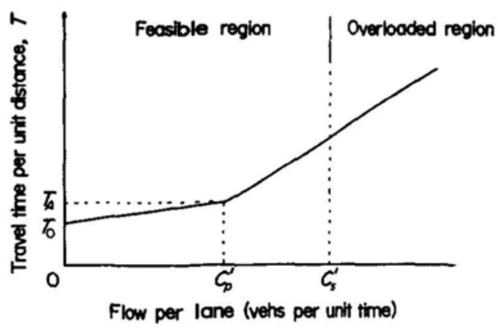

Figure 2.4. Two segment function proposed by Irwin et al. (Branston, 1976)

This function is updated by Irwin and Von Cube in 1962 to the following function:

$$T = T_A + \alpha(Q - C_p) \quad \text{, for } Q < C_p \text{,} \tag{2.6}$$

$$T = T_A + \beta(Q - C_p) \quad \text{, for } C_p \le Q \le C_s \text{,} \tag{2.7}$$

$$T = T_B + \gamma(Q - C_s) \quad \text{, for } Q \ge C_s \text{,} \tag{2.8}$$

where,

$$T_A = T_0 + \alpha C_p \text{,} \tag{2.9}$$

$$T_B = T_A + \beta(C_s - C_p) \text{.} \tag{2.10}$$

In addition to the previous one, this function has a particular line segment for the overload region, which takes over at steady state capacity (See Figure 2.5).

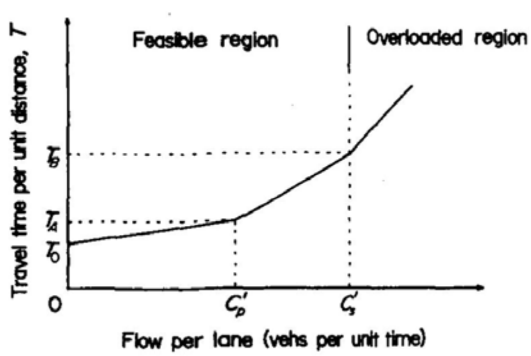

Figure 2.5. Three segment function proposed by Irwin et al. (Branston, 1976)

The main problem in these functions is to find out the location of the practical capacity when no data is available. Therefore, curvilinear link-capacity functions are preferred. Smock (1962) proposed one of the earliest curvilinear link-capacity functions, which was an exponential curve, whose roots were outlined by Smulick (1961) (Branston, 1976).

$$T = T_0 \exp\left[Q/C_s\right]. \tag{2.11}$$

Soltman (1965) used a different curvilinear function:

$$T = T_0 2^{Q/C_p}, \tag{2.12}$$

where,

$$Q/C_p \leq 2. \tag{2.13}$$

A general formula was proposed by Overgaard (1967), given by Formula 2.14:

$$T = T_0 \alpha^{\left(Q/C_p\right)^{\beta}}. \tag{2.14}$$

Mosher (1963) suggested two candidates to be link-capacity functions. The first type, a logarithmic formula was as given in Formula 2.15:

$$T = T_0 \ln(\alpha) - \ln(\alpha - Q), \tag{2.15}$$

where,

$$Q \leq \alpha. \tag{2.16}$$

The second type, a hyperbolic formula was as given in Formula 2.17:

$$T = \beta - \frac{\alpha(T_0 - \beta)}{Q - \alpha}, \tag{2.17}$$

where,

$$Q \leq \alpha \,. \tag{2.18}$$

These two functions were inappropiate for the traffic assignment procedures because they generate infinite travel times at the early stages of the assignment. Since the flows at the early stages can be very high, α should be high enough to provide the conditons given in Equations 2.16 & 2.18 and a high α value gives travel time values approaching to infinity. These infinite travel times can not be used in the computer iterations. For that reason, the functions were revised. The new logarithmic function had the following form:

$$T = T_0 + \beta \ln\left[\alpha\right] - \beta \ln\left[\alpha - Q\right] \text{ , for } Q \leq C_s \,, \tag{2.19}$$

$$T = T_c + sQ \qquad \text{ , for } Q > C_s \,, \tag{2.20}$$

where,

$$T_c = T_0 + \beta \ln\left[\alpha\right] - \beta \ln\left[\alpha - C_s\right], \tag{2.21}$$

$$s = \beta / \left(\alpha - C_s\right), \tag{2.22}$$

$$\alpha > C_s \,. \tag{2.23}$$

And the new hyperbolic function was of the form:

$$T = \beta - \frac{\alpha(T_0 - \beta)}{Q - \alpha}, \text{ for } Q \leq C_s \,, \tag{2.24}$$

$$T = T_c + sq \qquad \text{ , for } Q > C_s \,, \tag{2.25}$$

where,

$$T_c = \beta - \frac{\alpha(T_0 - \beta)}{C_s - \alpha}, \tag{2.26}$$

$$s = \alpha(T_0 - \beta) \big/ \left(C_s - \sigma\right)^2 , \tag{2.27}$$

$$\alpha > C_s ,\ T_0 > \beta . \tag{2.28}$$

One of the most popular link-capacity functions is the one that was developed by the Bureau of Public Roads in 1964 (Branston, 1967). It's most general version is:

$$T = T_0 \left[1 + \alpha \left(Q/C_p \right)^{\beta} \right]. \tag{2.29}$$

The suggested values for alpha and beta by the Bureau of Public Roads engineers are 0,15 and 4, respectively.

Steenbrink (1974) proposed the same function but used the steady state capacity instead:

$$T = T_0 \left[1 + \alpha \left(Q/C_s \right)^{\beta} \right]. \tag{2.30}$$

And, for the parameters alpha and beta, he recommended 2,62 and 5, respectively, which are quite different from the Bureau of Public Roads suggestions.

The Traffic Research Corporation (1966) proposed the following detailed function:

$$T = \alpha + \beta\left(Q - \gamma\right) + \sqrt{\left[\beta^2 \left(Q - \gamma\right)^2 + \delta \right]} . \tag{2.31}$$

Branston (1976) suggested that the approximation of the simpler BPR function (Equation 2.29) may be used instead of this detailed function (Equation 2.31).

The Department of Transport (1985) in the UK had produced different functions for urban, sub-urban and inter-urban roads, as given below:

$$T = \begin{cases} \dfrac{d}{S_0} & ,\text{for } V < F_1, \\ \dfrac{d}{S_0 + SS_{01}F_1 - SS_{01}V} & ,\text{for } F_1 \le V \le F_2, \\ \dfrac{d}{S_1} + \dfrac{(V/F_2 - 1)}{8} & ,\text{for } V > F_2, \end{cases} \tag{2.32}$$

where,

$$SS_{01} = \frac{S_0 - S_1}{F_1 - F_2}. \tag{2.33}$$

For the parameters S_0, S_1, F_1, and F_2 there are some typical values as shown in Table 2.3 (Ortuzar and Willumsen, 2004).

Table 2.3. Typical speed-flow curve coefficients for functions proposed by Department of Transport (Ortuzar and Willumsen, 2004)

Type	S_0 km/h	S_1 km/h	F_1 pcu/h/lane	F_2 pcu/h/lane
Single 2 lane, rural	63	55	400	1400
Dual 2 lane, rural	79	70	1600	2400
Single 2 lane, urban,outer area	45	25	500	1000

2.4. Catastrophe Theory

Catastrophe theory is a mathematical model, which was created in 1972 by Rene Thom (Acha-Daza and Hall, 1993). Like some other mathematical functions it relates its properties to practical problems, in this case the relations of traffic flow parameters. Catastrophe theory has the ability to produce sudden changes in one variable where a gradual change has been observed in the other two variables. These sudden changes are called “catastrophes”. Traffic flow models based on traditional traffic flow theory such as

Greenshields linear model, Underwood exponential model, Drew model and May model are continual traffic flow models in two dimensions. But these models have sometimes difficulties to explain the occurrence of discontinuities in some of the collected data. These occur when operations proceed to and from the congested regime. Catastrophe traffic models can account for these discontinuities from three-dimension. Of the seven elementary catastrophes presented by Thom, two have been applied to traffic flow: the fold and the cusp catastrophe. From these two catastrophes, cusp catastrophe (See Figure 2.6) has been selected for representing traffic operations by Navin (1986) and Hall and others (1987).

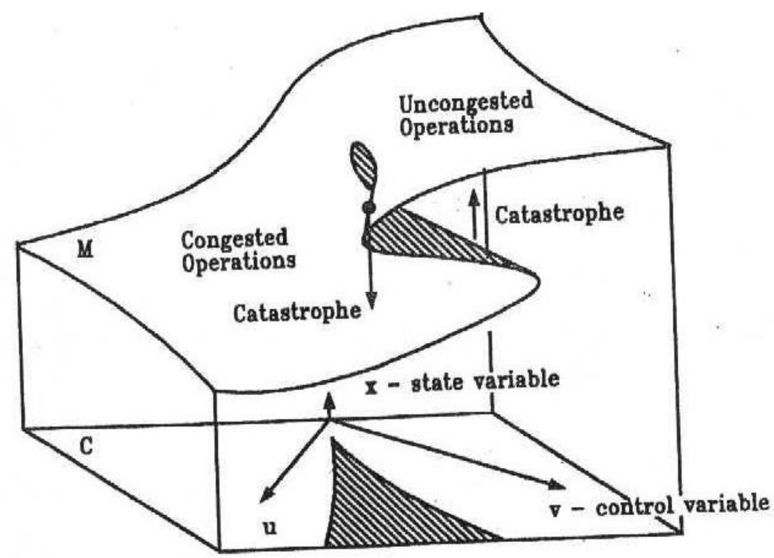

Figure 2.6. The cusp catastrophe (Navin, 1985)

A cusp catastrophe surface resembles a partially folded piece of paper. The projection of the folded portion onto a horizontal plane produces a cusp shaped figure. The surface for the model is described in terms of one state and two control variables. The state variable is the one which exhibits the sudden change while the control variables exhibit gradual changes. The cusp catastrophe model possesses five properties which must be satisfied for the model to be considered applicable. These five properties are catastrophes, bimodality, hysteresis, divergence and inaccessibility as shown in Figure 2.7.

2.4.1. Catastrophes

A catastrophe is a sudden change in a parameter when gradual changes were previously experienced and were expected to continue. Speed is the parameter, where these sudden changes occur. Therefore, it should be selected for state variable, while occupancy and volume remain as control variables.

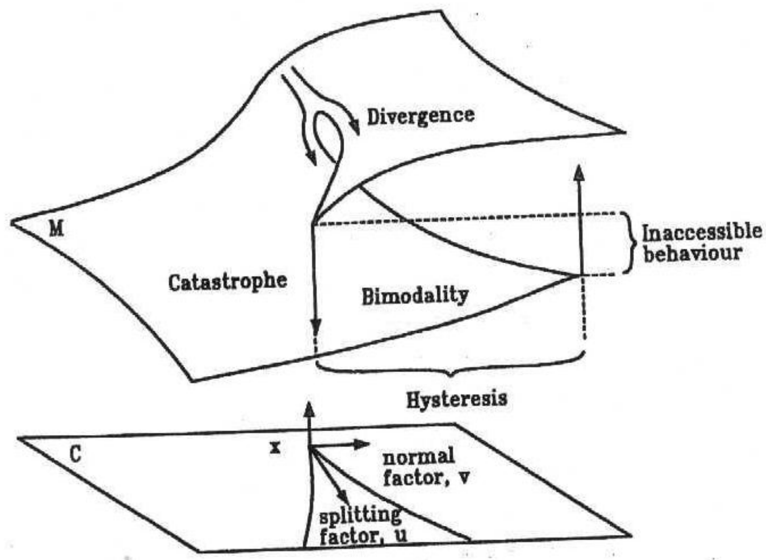

Figure 2.7. Five characteristic properties of the cusp catastrophe (Navin, 1985)

2.4.2. Bimodality

Bimodality means that for certain values of the control variables there may be two values for the state variable. Since speed is selected for the state variable, for any particular set of flow and occupancy values there might be two speed values, one in the congested and the other one in the uncongested region. Lack of bimodal behavior lead the researchers in transportation area to use a Maxwell convention (See Figure 2.8) rather than a perfect delay convention (See Figure 2.9) (Forbes et al., 1986). Maxwell convention has a sudden perpendicular fall surface while perfect delay convention experiences overlapping.

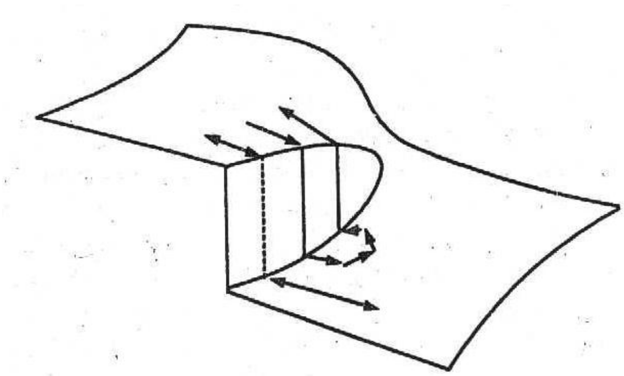

Figure 2.8. Maxwell convention (Forbes and Hall, 1989)

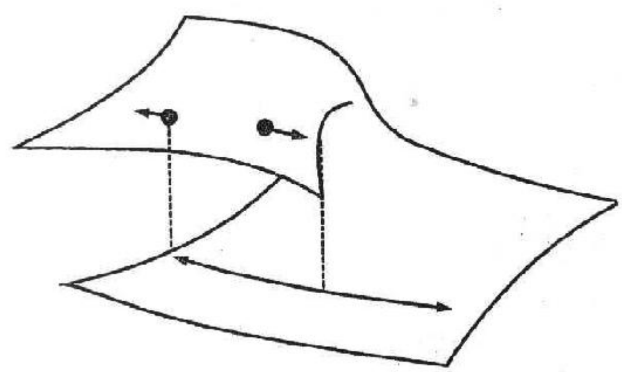

Figure 2.9. Perfect delay convention (Forbes and Hall, 1989)

2.4.3. Hysteresis

Hysteresis means that the sudden changes of the state variable from the upper sheet to the lower one on a catastrophe surface do not necessarily occur at the same values of the control variables as do changes from the lower to upper. In other words the paths for going from upper to lower sheet and from lower to upper sheet are not necessarily the same (See Figure 2.7).

2.4.4. Divergence

Divergence means that a small change in the control variables may result in a great change of the value of state variable (See Figure 2.7).

2.4.5. Inaccessibility

Inaccessibility means that the middle part of the fold represents values for the state variable that are mathematically inaccessible and in reality do not occur or are unstable and change quickly to stable values on the upper or lower sheets (See Figure 2.7).

According to Hall's view the basic catastrophe models are as follows:

$$W(X) = aX^4 + bX^2Y + cXZ \text{ ,} \tag{2.34}$$

which has a critical surface defined by,

$$4aX^3 + 2bXY + cZ \text{ ,} \tag{2.35}$$

where,

X = state variable,
Y, Z = control variables,
a, b, c = coefficients.

This model will be tested on the data for four days obtained from RTMS 12.

3. RESEARCH METHODOLOGY

The proposed methodology for the thesis research was basically summarized in the flow chart shown in Figure 3.1.

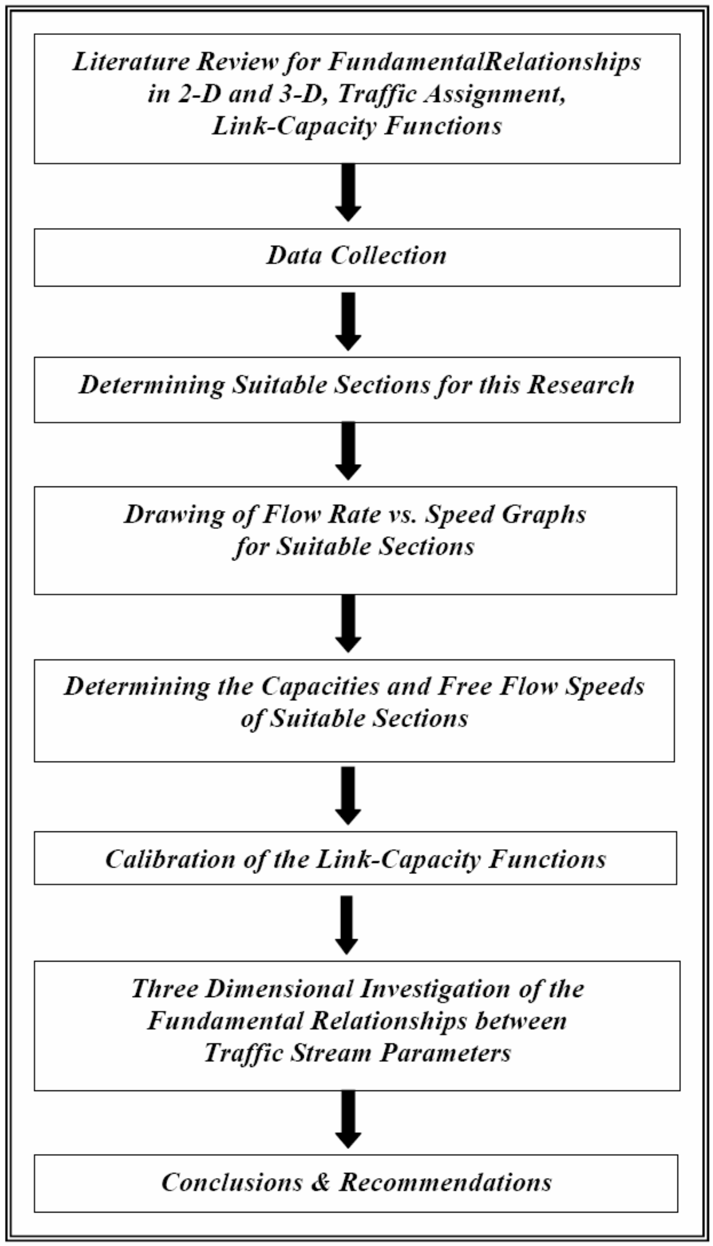

Figure 3.1. Flow chart for the research methodology

To begin with, searching the literature for link-capacity functions was the first step of the methodology. Fundamental relationships between traffic stream parameters in two and three dimensions and capacity constrained traffic assignment models were also investigated.

After that, the data were obtained from Istanbul Metropolitan Planning Center consisted of volumes, speeds and occupancy values for 69 different sections at different roads in Istanbul for four months, namely May, June, October and November 2006. From this data, volumes were converted to flow rates using the corresponding factors given in the Highway Capacity Manual (Transportation Research Board, 2000) and with the related average speeds, the flow rates vs. speed data were plotted for each individual sections. Because not all of these sections had appropriately positioned sensors and data, only some of them have been used. The criteria used for selecting the sections included the followings:

1. Sections should not be within the influence areas of weaving sections and interchange ramps. Weaving areas are the sections of the freeway, where two or more vehicle flows must cross each other's path. They occur usually when a merging area is followed by a diverging area, while ramp junctions are points at which on and off ramps join the freeway section. Usually at weaving and ramp areas, there is some disorder since the drivers might get disturbed because of the interaction. Therefore, instead of these sections, homogeneous links are preferable for the proper analysis.

2. Sections should have a wide-ranging data starting from free flow up to the congested flow conditions so that the full relationships between the traffic parameters could be observed.

3. The hours during which heavy vehicles were allowed, were dropped out on TEM freeway because at these hours truck percentages reached very high proportions and the traffic parameters were mainly determined by their characteristics.

With these criteria, out of 69 different sections three were selected for E-5 and four sections for TEM freeway. After determining suitable sections, quadratic curves were fitted to the flow rate vs. speed plots by means of a statistical program (SPSS, 2006), assuming the relationship between them was quadratic (See Figure 2.1). Using these quadratic equations, capacities and free flow speeds were determined for each section. In order to determine free flow speeds, the roots were determined and for capacities the derivatives were calculated. After individual calculations, two groups were created according to their free flow speeds. The flow rates vs. speed data for each section within the same group were aggregated and the procedure up to here was repeated for this aggregated data. Capacities were found for each group. From the speed values, corresponding travel times were determined using the Formula 5.4. In order to determine the travel times at zero flow rate, averages of travel times were taken for flow rates less than 1000 pc/hour/lane. Suggested link-capacity functions, as mentioned in the literature review, were tested and the BPR Model (Equation 2.29), with steady state capacity, as it was suggested by Steenbrink (1974) (Equation 2.30) was used for the analysis. Its parameters were calibrated by means of a statistical program (SPSS, 2006) using nonlinear regression analysis. This model was selected for its simplicity and functionality. It is probably the most common formula of this type (Ortuzar and Willumsen, 2004) and has the properties that a link-capacity function should have in order to be used for traffic assignment models.

After the calibration, three dimensional relationships of the traffic stream characteristics were investigated. The catastrophe theory was also tried for finding the three-dimensional relationships among the traffic stream parameters. The equation of the critical surface for this theory was determined for one section and its validation was checked out.

Finally, some conclusions and recommendations were made for further research.

4. DATA COLLECTION

In order to examine the existing traffic situation across Istanbul, the Istanbul Municipality had placed RTMS (Remote Traffic Microwave Sensor) detectors for monitoring the streets and collecting traffic data (Figure 4.1).

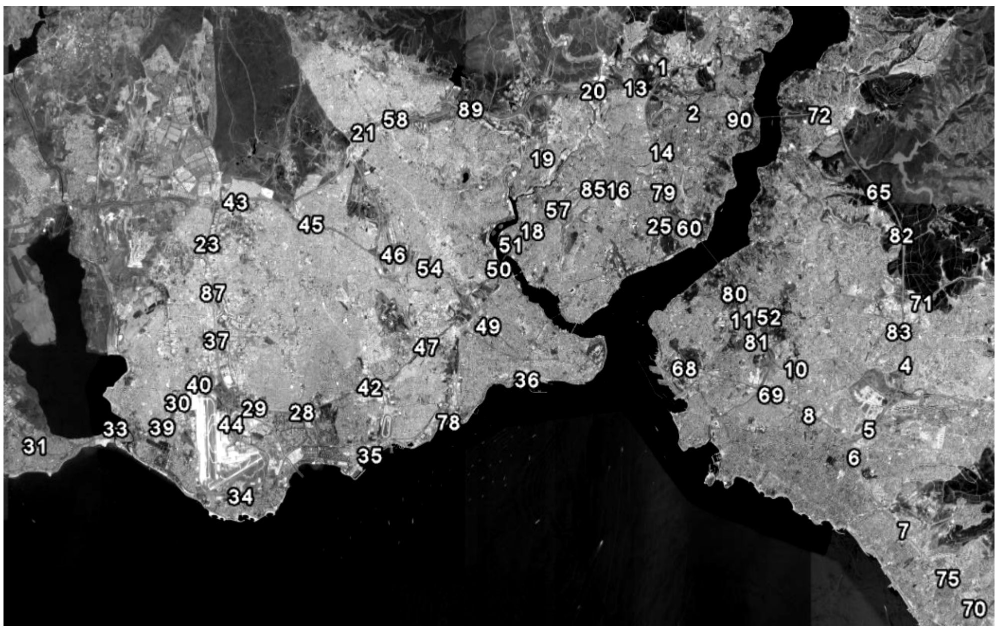

Figure 4.1. Locations of RTMS devices

This data included number of heavy vehicles, number of all vehicles, speed averages for the time interval and occupancy values for the time interval. As a property of the detector the data was collected for each lane separately and for 8 lanes (See Table 4.1). If the section had less than 8 lanes then the detector assigned zero values for the non-existing lanes. For speed, it assigned "240" for the missing values (See Figure 4.2). The complete data contained 4 months (May, June, October, and November 2006). During May and June, RTMS devices sent messages at each 5 minutes, whereas for October and November this interval was 2 minutes. For consistency October and November data was used for the analysis. For the three dimensional investigation a randomly chosen data from May was used.

Table 4.1. Explanation of the RTMS message fields

Data collected by the RTMS device from the field	
MsgTime	Time interval between successive messages
RtmsNo	ID information of the RTMS device
VL1	Total long vehicle number using lane 1 in the related time interval
VL2	Total long vehicle number using lane 2 in the related time interval
VL3	Total long vehicle number using lane 3 in the related time interval
VL4	Total long vehicle number using lane 4 in the related time interval
VL5	Total long vehicle number using lane 5 in the related time interval
VL6	Total long vehicle number using lane 6 in the related time interval
VL7	Total long vehicle number using lane 7 in the related time interval
VL8	Total long vehicle number using lane 8 in the related time interval
V1	Total vehicle number using lane 1 in the related time interval
V2	Total vehicle number using lane 2 in the related time interval
V3	Total vehicle number using lane 3 in the related time interval
V4	Total vehicle number using lane 4 in the related time interval
V5	Total vehicle number using lane 5 in the related time interval
V6	Total vehicle number using lane 6 in the related time interval
V7	Total vehicle number using lane 7 in the related time interval
V8	Total vehicle number using lane 8 in the related time interval
O1	Occupancy value of the lane 1 in the related time interval
O2	Occupancy value of the lane 2 in the related time interval
O3	Occupancy value of the lane 3 in the related time interval
O4	Occupancy value of the lane 4 in the related time interval
O5	Occupancy value of the lane 5 in the related time interval
O6	Occupancy value of the lane 6 in the related time interval
O7	Occupancy value of the lane 7 in the related time interval
O8	Occupancy value of the lane 8 in the related time interval
S1	Average speed for the lane 1 in the related time interval
S2	Average speed for the lane 2 in the related time interval
S3	Average speed for the lane 3 in the related time interval
S4	Average speed for the lane 4 in the related time interval
S5	Average speed for the lane 5 in the related time interval
S6	Average speed for the lane 6 in the related time interval
S7	Average speed for the lane 7 in the related time interval
S8	Average speed for the lane 8 in the related time interval

msgtime	RtmsNo	VL1	VL2	VL3	VL4	VL5	VL6	VL7	VL8	V1	V2	V3	V4	V5	V6	V7	V8	S1	S2	S3	S4	S5	S6	S7	S8	o1	o2	o3	o4	o5	o6	o7	o8
2006-11-26 12:16:18.000	12	3	3	3	0	0	0	0	0	45	41	52	0	0	0	0	0	21	19	34	240	240	240	240	240	38	46	28	0	0	0	0	0
2006-11-26 12:00:18.000	12	2	0	2	0	0	0	0	0	51	38	52	0	0	0	0	0	19	14	24	240	240	240	240	240	38	53	38	0	0	0	0	0
2006-11-26 12:02:18.000	12	2	2	1	0	0	0	0	0	61	52	77	0	0	0	0	0	23	21	47	240	240	240	240	240	35	38	18	0	0	0	0	0
2006-11-26 12:04:18.000	12	2	1	1	0	0	0	0	0	50	42	55	0	0	0	0	0	23	17	28	240	240	240	240	240	34	41	33	0	0	0	0	0
2006-11-26 12:06:18.000	12	4	1	4	0	0	0	0	0	56	55	58	0	0	0	0	0	38	34	34	240	240	240	240	240	23	29	26	0	0	0	0	0
2006-11-26 12:08:18.000	12	1	3	4	0	0	0	0	0	47	43	59	0	0	0	0	0	20	15	29	240	240	240	240	240	37	47	28	0	0	0	0	0
2006-11-26 12:10:18.000	12	0	0	1	0	0	0	0	0	54	59	70	0	0	0	0	0	33	30	40	240	240	240	240	240	23	30	18	0	0	0	0	0
2006-11-26 12:30:18.000	12	3	2	2	0	0	0	0	0	45	43	60	0	0	0	0	0	17	13	31	240	240	240	240	240	35	53	30	0	0	0	0	0
2006-11-26 12:14:18.000	12	0	1	2	0	0	0	0	0	62	58	74	0	0	0	0	0	37	42	53	240	240	240	240	240	24	25	18	0	0	0	0	0
2006-11-26 11:52:19.000	12	2	2	4	0	0	0	0	0	56	53	61	0	0	0	0	0	26	22	32	240	240	240	240	240	30	39	32	0	0	0	0	0
2006-11-26 12:18:19.000	12	2	2	0	0	0	0	0	0	59	54	76	0	0	0	0	0	29	26	44	240	240	240	240	240	29	33	19	0	0	0	0	0
2006-11-26 12:20:18.000	12	0	1	3	0	0	0	0	0	51	37	54	0	0	0	0	0	18	12	25	240	240	240	240	240	39	54	42	0	0	0	0	0
2006-11-26 12:22:18.000	12	1	2	1	0	0	0	0	0	53	50	55	0	0	0	0	0	22	18	24	240	240	240	240	240	34	42	35	0	0	0	0	0
2006-11-26 12:24:19.000	12	3	2	2	0	0	0	0	0	45	40	52	0	0	0	0	0	20	16	22	240	240	240	240	240	34	45	40	0	0	0	0	0
2006-11-26 12:26:18.000	12	1	4	5	0	0	0	0	0	56	55	66	0	0	0	0	0	36	33	45	240	240	240	240	240	23	27	18	0	0	0	0	0
2006-11-26 11:20:18.000	12	0	2	0	0	0	0	0	0	28	60	51	0	0	0	0	0	67	93	122	240	240	240	240	240	6	12	6	0	0	0	0	0
2006-11-26 12:12:18.000	12	2	4	6	0	0	0	0	0	58	45	56	0	0	0	0	0	27	20	28	240	240	240	240	240	30	40	29	0	0	0	0	0
2006-11-26 11:38:18.000	12	1	5	1	0	0	0	0	0	39	67	69	0	0	0	0	0	66	67	75	240	240	240	240	240	11	17	9	0	0	0	0	0
2006-11-26 09:00:18.000	12	0	1	0	0	0	0	0	0	16	49	43	0	0	0	0	0	73	101	138	240	240	240	240	240	3	9	5	0	0	0	0	0
2006-11-26 11:24:19.000	12	0	3	0	0	0	0	0	0	32	63	62	0	0	0	0	0	66	86	117	240	240	240	240	240	6	13	7	0	0	0	0	0
2006-11-26 11:26:18.000	12	0	1	0	0	0	0	0	0	24	54	62	0	0	0	0	0	67	89	122	240	240	240	240	240	5	9	7	0	0	0	0	0
2006-11-26 11:28:18.000	12	1	0	0	0	0	0	0	0	44	80	73	0	0	0	0	0	60	79	113	240	240	240	240	240	11	17	9	0	0	0	0	0
2006-11-26 11:30:18.000	12	0	1	0	0	0	0	0	0	29	69	67	0	0	0	0	0	66	85	109	240	240	240	240	240	6	14	8	0	0	0	0	0
2006-11-26 11:32:18.000	12	0	2	0	0	0	0	0	0	33	60	59	0	0	0	0	0	68	81	125	240	240	240	240	240	7	13	7	0	0	0	0	0
2006-11-26 11:58:18.000	12	3	2	5	0	0	0	0	0	41	34	46	0	0	0	0	0	13	11	24	240	240	240	240	240	47	53	35	0	0	0	0	0
2006-11-26 11:36:18.000	12	2	0	0	0	0	0	0	0	37	68	58	0	0	0	0	0	70	77	104	240	240	240	240	240	9	15	7	0	0	0	0	0
2006-11-26 11:54:18.000	12	2	2	2	0	0	0	0	0	63	62	68	0	0	0	0	0	32	38	46	240	240	240	240	240	28	28	16	0	0	0	0	0
2006-11-26 11:40:19.000	12	1	1	2	0	0	0	0	0	30	55	54	0	0	0	0	0	56	80	102	240	240	240	240	240	10	14	6	0	0	0	0	0
2006-11-26 11:42:18.000	12	4	3	4	0	0	0	0	0	48	58	60	0	0	0	0	0	55	53	73	240	240	240	240	240	16	20	12	0	0	0	0	0
2006-11-26 11:44:18.000	12	6	3	2	0	0	0	0	0	43	38	54	0	0	0	0	0	20	17	33	240	240	240	240	240	38	41	30	0	0	0	0	0
2006-11-26 11:46:19.000	12	0	1	3	0	0	0	0	0	34	26	51	0	0	0	0	0	11	7	26	240	240	240	240	240	41	69	32	0	0	0	0	0
2006-11-26 11:48:18.000	12	4	2	0	0	0	0	0	0	38	23	37	0	0	0	0	0	16	4	15	240	240	240	240	240	33	70	41	0	0	0	0	0
2006-11-26 11:50:18.000	12	2	0	1	0	0	0	0	0	53	52	73	0	0	0	0	0	27	23	37	240	240	240	240	240	29	37	23	0	0	0	0	0
2006-11-26 12:32:18.000	12	2	3	3	0	0	0	0	0	45	30	50	0	0	0	0	0	16	8	18	240	240	240	240	240	37	67	46	0	0	0	0	0
2006-11-26 11:34:19.000	12	1	2	1	0	0	0	0	0	40	57	58	0	0	0	0	0	63	75	113	240	240	240	240	240	10	12	7	0	0	0	0	0

Figure 4.2. Sample format of the data

4.1. Features of Remote Traffic Microwave Sensor

This section explains some of the technical properties of this traffic sensor. The RTMS radar is a traffic sensor which detects presence of vehicles and measures traffic parameters like volume, occupancy, speed in multiple independent lanes (for this study maximum of 4 lanes in one direction) up to 60m. The RTMS devices are usually mounted on existing poles as shown in Figure 4.3 in side-fired configuration. It is easy to install and remove, and is fully programmable to support a variety of applications. In its forward-looking configuration it is mounted on overhead sign structures to monitor a specific lane.

Figure 4.3. Side-fired RTMS (www.rtms-by-eis.com)

The beam of the RTMS "paints" a long elliptical footprint on the road surface. Any non-background targets will reflect the signal back to the device where the targets are detected and their range measured. The RTMS range measurement resolution of 2 meters divides the footprint ellipse into 32 range-slices (See Figure 4.4).

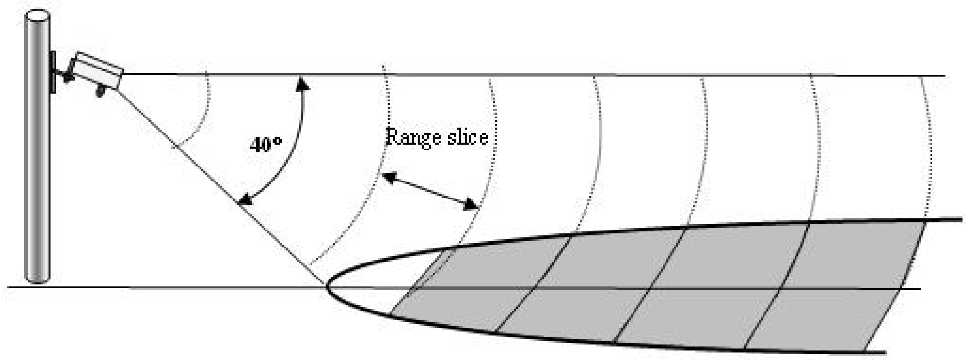

Figure 4.4. Range-slices of RTMS (www.rtms-by-eis.com)

The long microwave wavelength and the range-measurement capability make the RTMS immune to all weather effects and to most occlusion situations allowing vehicles hidden behind other vehicles to be detected, as shown in the Figure 4.5.

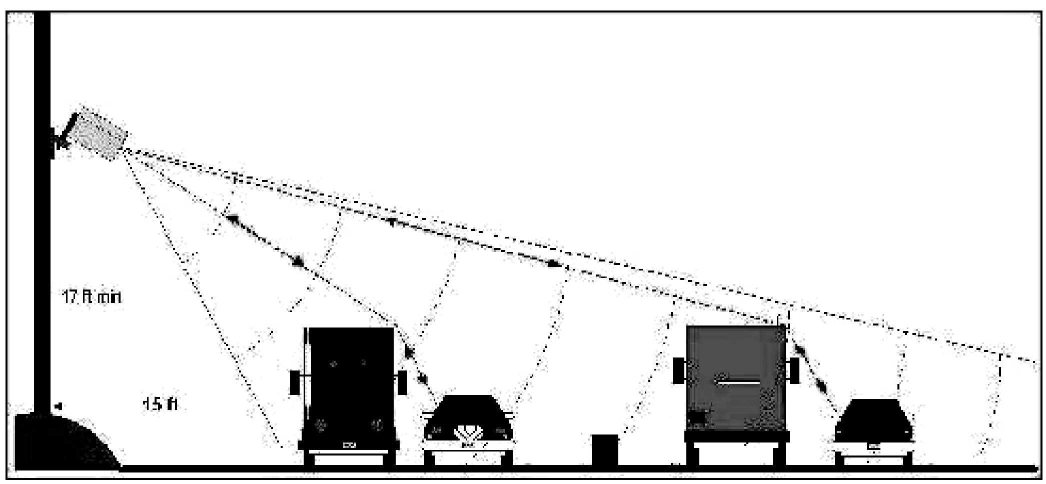

Figure 4.5. RTMS principles of operation (www.rtms-by-eis.com)

The RTMS internal microcomputer controls in real-time 8 opto-isolator relays corresponding to the detection zones. Relay contacts are closed when a target is present within the respective detection zone. The contact-pairs can be connected directly to traffic controllers. In addition, short-term statistical data on each zone are accumulated and transmitted by the RTMS via its serial port. Typically, every 30 to 300 seconds a message containing the volume, occupancy and average speed data in each detection zone is transmitted (www.rtms-by-eis.com). For this study, the messages were transmitted at each 120 or 300 seconds.

4.2. Preliminary Investigation of the Data

4.2.1. Weekdays

Hourly changes of the data within 24 hours were analyzed for two months period, October and November 2006. For this analysis, hourly flow rate, speed and occupancy averages of weekdays were taken. The graphs were drawn for some sections in order to show the characteristics of the data.

RTMS 12 was located at the entrance of the Boğaziçi Bridge on the Asian Side of Istanbul. The flow rate vs. time graph of this data was shown in Figure 4.6. Flow rate values between 06:00 and 17:00 were close to the lane capacities but after 17:00, a sudden drop was observed. This was not because the demand was reducing, but instead because of the decrease in lane capacity. The one lane, given to the reverse traffic flow was the main reason of this problem.

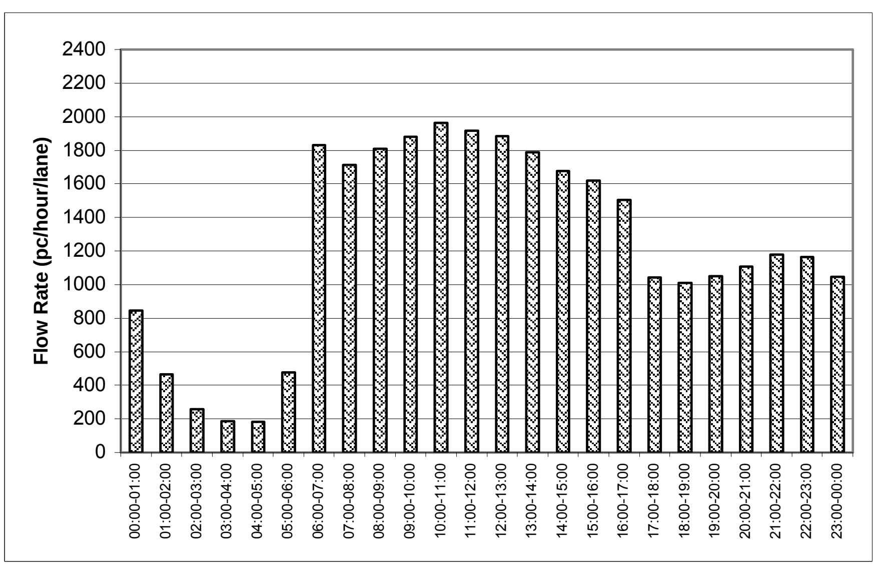

Figure 4. 6. Flow rate vs. time graph of RTMS # 12 for weekdays

During the same hours, speed levels were significantly low (See Figure 4.7) and the occupancy values were high (See Figure 4.8). The speed levels of morning peak hours were almost three times greater than the evening peak hours. Even during noon, the speed levels were not close to the free flow speeds. Occupancy vs. time graph had inversely related graph with the speed. During the night peak hours, occupancy level raised up to 50 per cent.

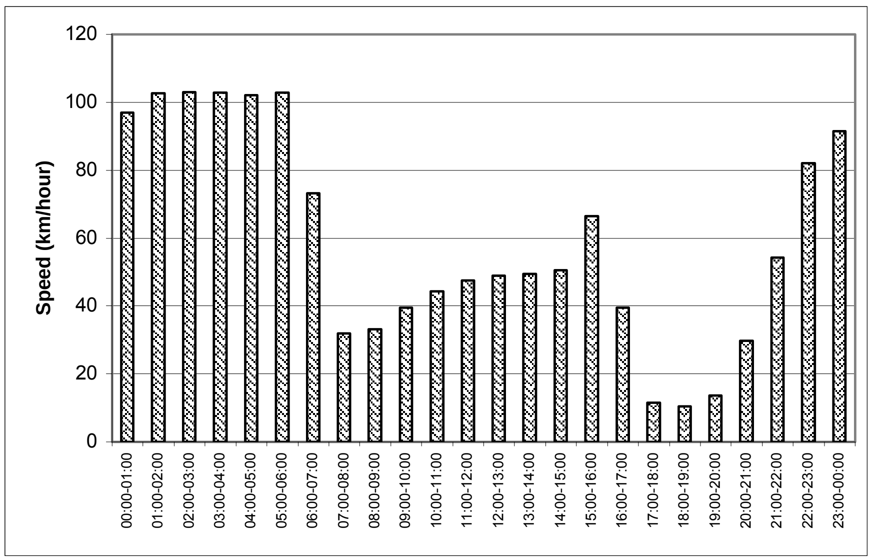

Figure 4.7. Speed vs. time graph of RTMS # 12 for weekdays

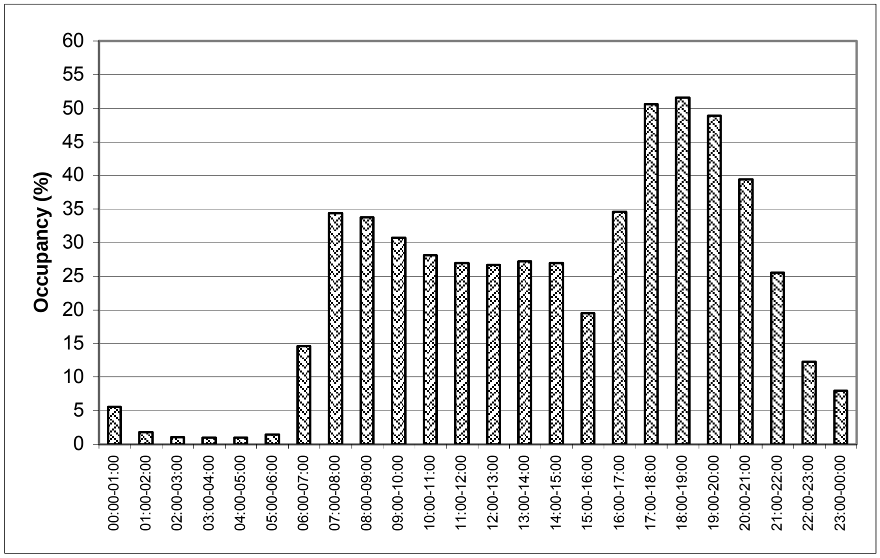

Figure 4.8. Occupancy vs. time graph of RTMS # 12 for weekdays

RTMS 60 was positioned at the European Side of the Boğaziçi Bridge. There was no variance among the flow rate values after 07:00 during the day (See Figure 4.9). The lowest flow rate values are the same with the data of RTMS 12.

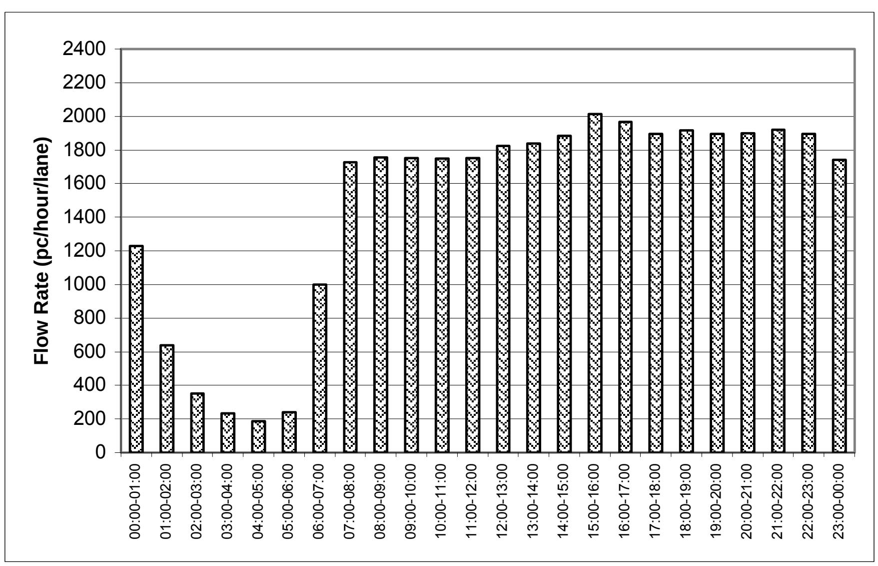

Figure 4. 9. Flow rate vs. time graph of RTMS # 60 for weekdays

Morning and evening peak hours had higher speed values than the data of RTMS 12 (See Figure 4.10). The evening peak hour speed levels were twice the morning levels and they were three times higher than the evening peak hour speed values of RTMS 12. Occupancy levels were reached to their peak during evening peak hours (See Figure 4.11), but these are not as high as the occupancy levels of RTMS 12.

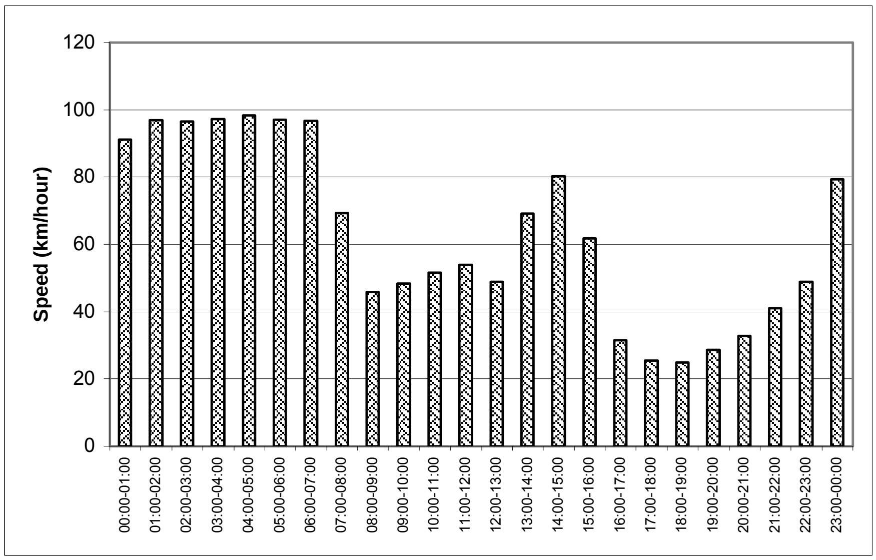

Figure 4. 10. Speed vs. time graph of RTMS # 60 for weekdays

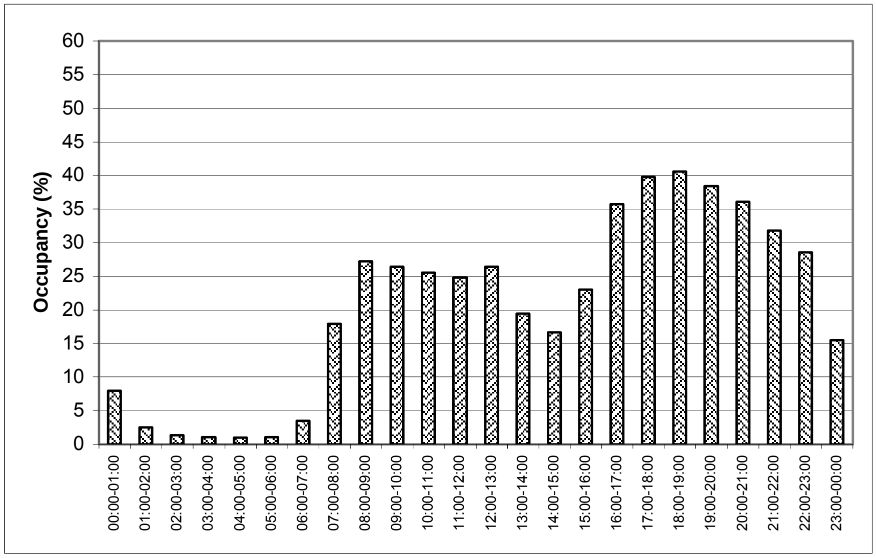

Figure 4. 11. Occupancy vs. time graph of RTMS # 60 for weekdays

4.2.2. Weekends

The speed variations during the weekends were analysized for RTMS 12 and RTMS 60 and shown in Figure 4.12 and 4.13, respectively. These graphs were formed up using speed averages within four continous days. They included averages of two months data, October and November 2006. The differences between the speed levels during weekdays and weekends peak hours were shown with dark color. Especially, on saturdays after midnight the speed values were quite low, since the travel demand for recreational activities was quite high during these hours.

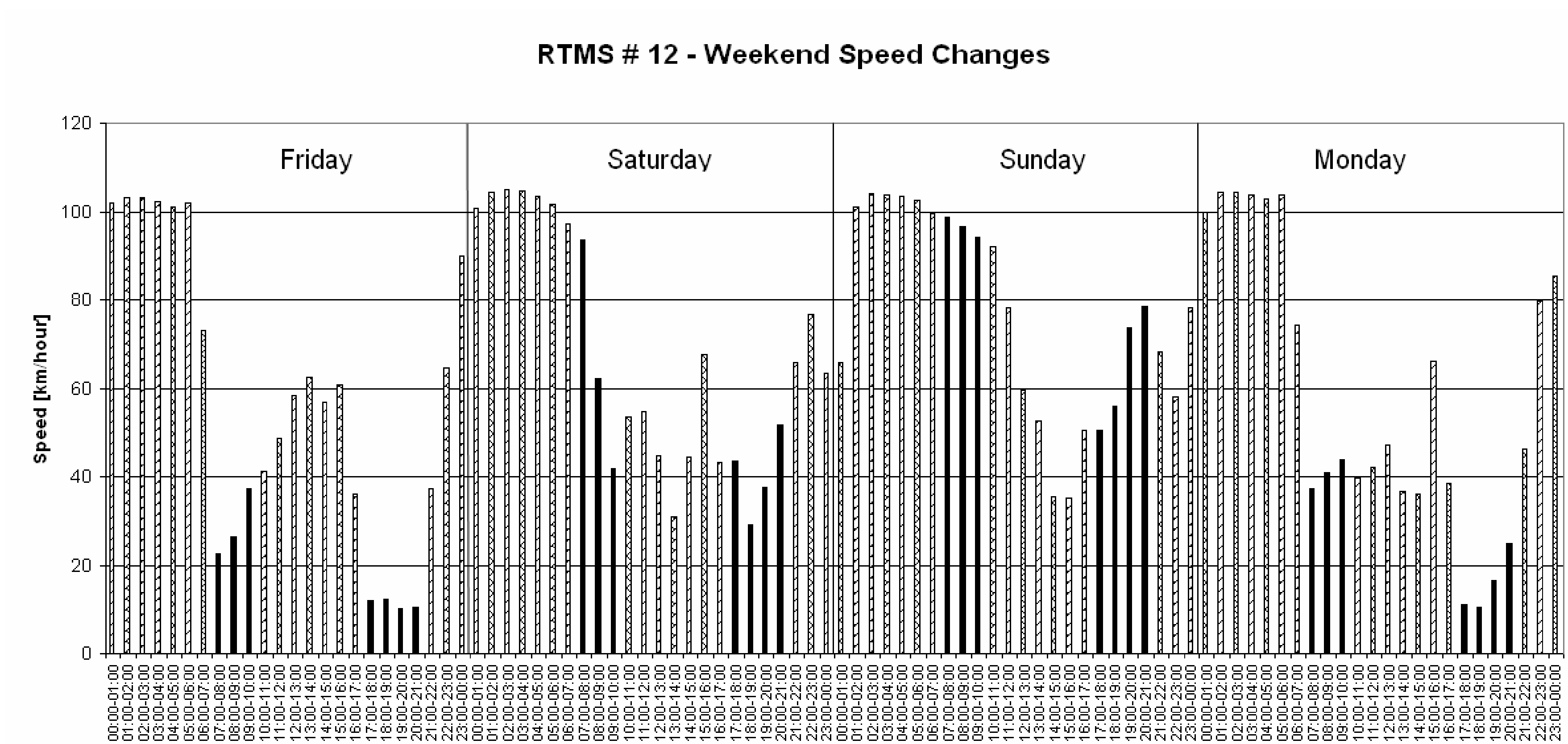

Figure 4.12. Changes in Speed values during weekends for RTMS # 12

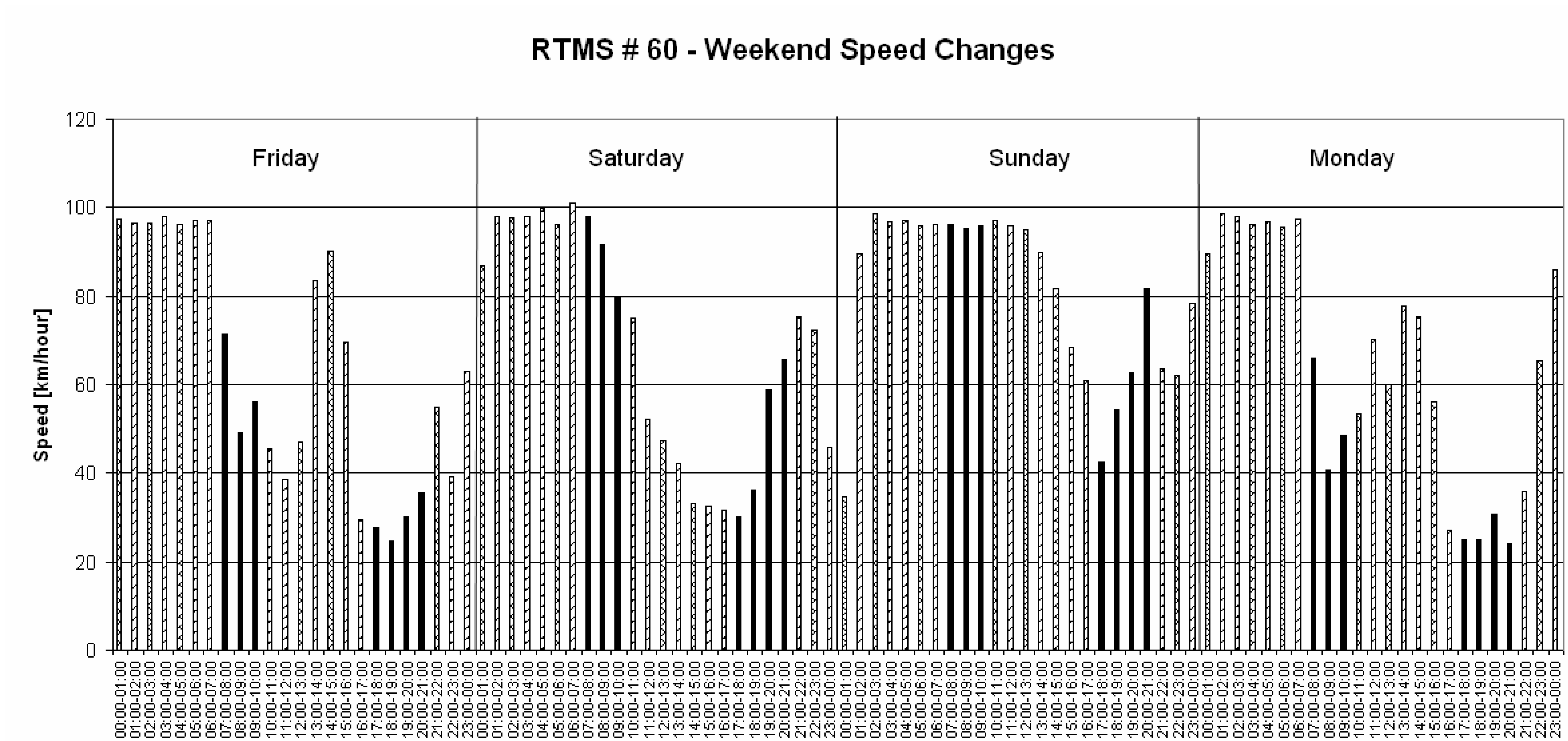

Figure 4.13. Changes in Speed values during weekends for RTMS # 60

5. DEVELOPMENT OF LINK-CAPACITY FUNCTIONS

5.1. Determination of Flow Rates and Average Speeds

As a first step the collected raw data was processed by finding the flow rates and average speeds. The hourly flow rate of a section must represent the effects of heavy vehicles, the temporal variation of traffic flow within an hour, and also the characteristics of the driver population. Therefore the hourly volume counts that were collected from the field were processed by heavy vehicle adjustment factor, peak-hour adjustment factors and driver population factor. The equivalent passenger-car flow rates were obtained by the following formula given in Highway Capacity Manual (Transportation Research Board, 2000).

$$V_p = \frac{V}{PHF \times N \times f_{HV} \times f_p} \tag{5.1}$$

where,

V_p = 15-min. passenger-car equivalent flow rate (pcphpl)
V = hourly volume (vph)
PHF = peak-hour factor
N = number of lanes
f_{HV} = heavy-vehicle adjustment factor
f_p = driver population factor

The peak hour factor was determined with the following formula:

$$PHF = V/\left(6 \times V_{10}\right), \tag{5.2}$$

where,

PHF = peak-hour factor,
V = hourly volume (vph),
V_{10} = volume during the peak 10 min. of the peak hour (veh/10 min.).

The heavy vehicle factor was found by the formula suggested by Highway Capacity Manual (Transportation Research Board, 2000), which is:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}, \tag{5.3}$$

where,

E_T, E_R = passenger-car equivalents for trucks or buses and recreational vehicles (RV) in the traffic stream, respectively,
P_T, P_R = proportion of trucks or buses and RVs in the traffic stream,
f_{HV} = heavy-vehicle adjustment factor.

Within this formula the percentage of recreational vehicles were taken zero since there was no information about that. The passenger-car equivalent for trucks and buses was taken 1,5 assuming level terrain as suggested by Highway Capacity Manual in Table 5.1 (Transportation Research Board, 2000).

Table 5.1. Passenger car equivalents on freeways (Transportation Research Board, 2000)

	Type of Terrain		
Category	Level	Rolling	Mountainous
ET for trucks and buses	1,5	3,0	6,0
ER for recreational vehicles	1,2	2,0	4,0

Since there was no data available to calculate the driver population factor, its default value was taken into consideration, which is 1.

For the average speed determination, the data was aggregated into hours and for each hour the average speed was calculated by just taking the mean of the speeds of all vehicles, including heavy vehicles during that hour.

5.2. Selection of the Sections

After converting the volume to flow rate, and finding the corresponding speeds, the appropriate sections were selected. For this selection the positions of the links were considered using satellite view by Google Earth and out of 69 different devices (See Figure 4.1) only those that were appropriate were selected, using the criteria explained below.

The first criteria was that the sections should be outside of the influence area of ramps or weaving areas. Usually at weaving and ramp areas, there is some disorder since the drivers might get disturbed because of the interaction. Therefore, instead of these sections, homogeneous links are preferable for the proper analysis.

Also, it was important that the data explains all kind of traffic which varies from free flow traffic to congested, stop-and-go traffic. Almost all of the traffic flow models try to explain the behavior of traffic variables over the full range of operation. While trying to create numerical models with specific parameter values, the data collected have to cover the full range of operation from free flow to stop-and-go traffic. If this is not established, then the curve fitting may become a serious problem. Therefore, first flow rate vs. speed averages for the corresponding hours were plotted for every section, in order to check if they satisfied this criterion. Since the freeways of Istanbul have various traffic flows ranging from uncongested to stop-and-go traffic for a particular section, the data explained all the different parts of the curve for all the sections. The peak hours generate data for the congested part, while the off peaks create free flow traffic. Also the variations during the days add important data points to the graphs.

Another important consideration was that the data achieved during the hours with heavy vehicle permission on TEM freeway was dropped and not used for the analysis. All these criteria were taken into account, while looking for appropriately positioned sensors.

The data from the improperly calibrated detectors were not taken into account. For instance, some preferred locations came out to be with missing data. Therefore, out of 69 RTMS devices, only seven sections were selected for the purpose of this research. The exact locations of the selected RTMS devices were shown in Figure 5.1.

The summary of the locations and directions of the selected RTMS detectors were as shown in Table 5.2.

Figure 5.1. Locations of the selected RTMS devices

Table 5.2. Summary of locations of the selected RTMS detectors.

			Traffic Flow	
RTMS #	***Road***	***Location***	***From***	***To***
12	E-5	Beylerbeyi	Altunizade	Boğaziçi Bridge
20	TEM	Arıcılar	Mahmutbey	Fatih Sultan Mehmet Bridge
60	E-5	Boğaziçi Bridge Europe Entrance	Topkapı	Boğaziçi Bridge
61	TEM	Kavacık	Ümraniye	Fatih Sultan Mehmet Bridge
65	TEM	Kavacık	Fatih Sultan Mehmet Bridge	Kozyatağı
72	TEM	Kavacık	Fatih Sultan Mehmet Bridge	Ankara
86	E-5	Okmeydanı	Mecidiyeköy	Topkapı

5.3. Investigation of the Section Data

For each selected section, first the flow rate vs. speed graphs were drawn. In Figure 5.2 the plot for RTMS 86 was presented.

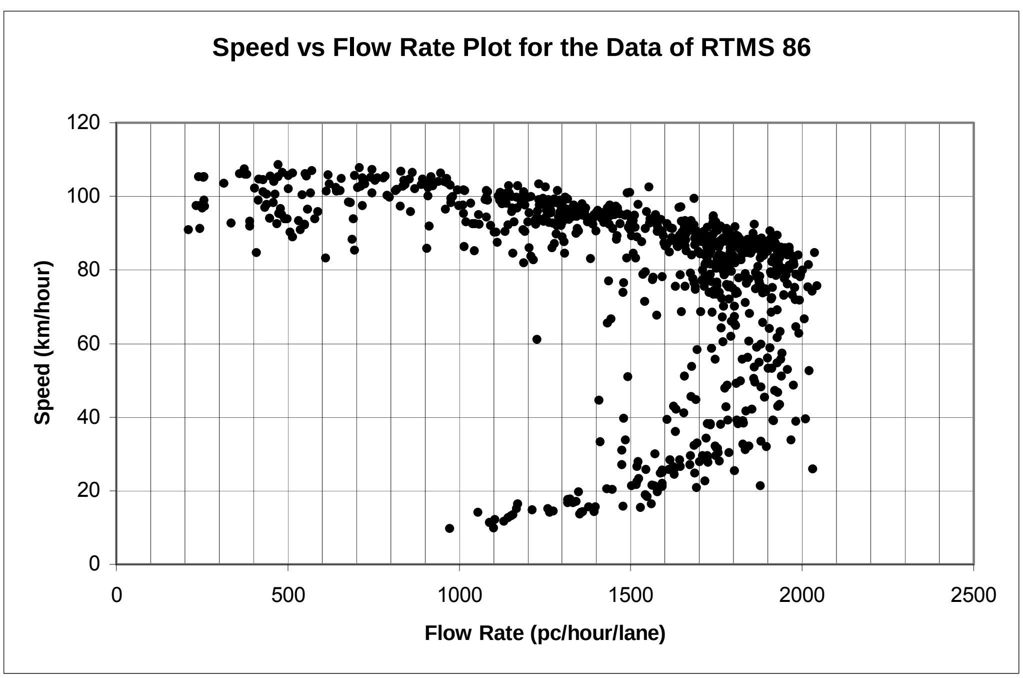

Figure 5.2. Speed vs. flow rate plot for the data of RTMS 86

Occupancy data was not used for the analysis, nor was it tried to be transferred into density since this process is impossible if the detector on the field is not a magnetic loop detector. As discussed in the previous section RTMS is not a magnetic loop detector. Figure 5.3 shows the elliptical footprint used by RTMS. The width of this elliptical area will be different for each lane and depending on the distance between the RTMS device and the road it will vary. Therefore the detector length given in the Formula 2.2, could not be determined accurately, which made the transformation of density from occupancy impossible. Therefore occupancy data was not used for this research.

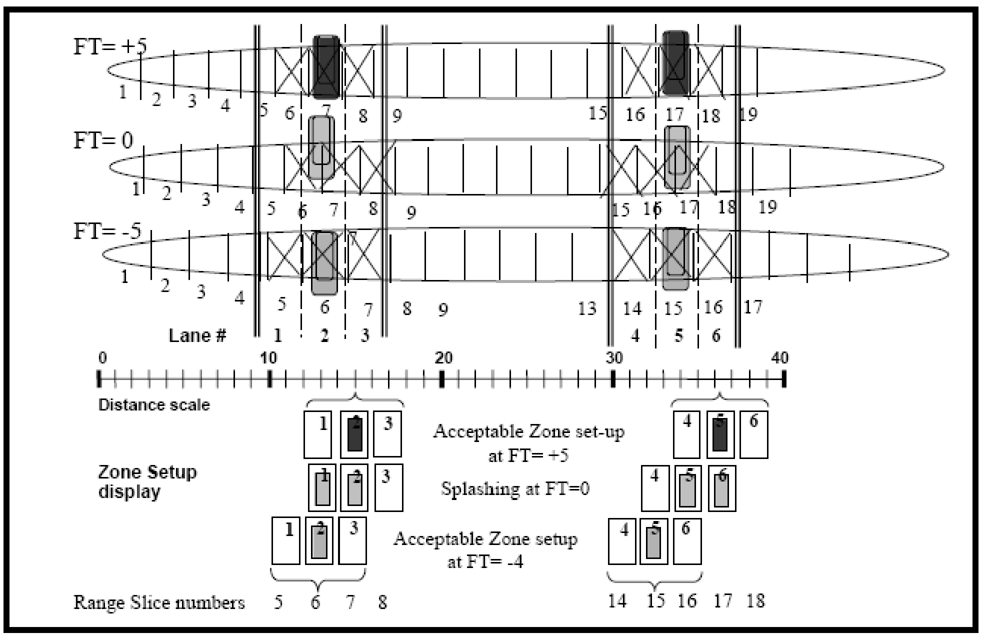

Figure 5.3. Elliptical footprint used by RTMS

To be an example, flow rate vs. occupancy and speed vs. occupancy graphs of the data of RTMS 86 were shown in Figure 5.4 and 5.5, respectively.

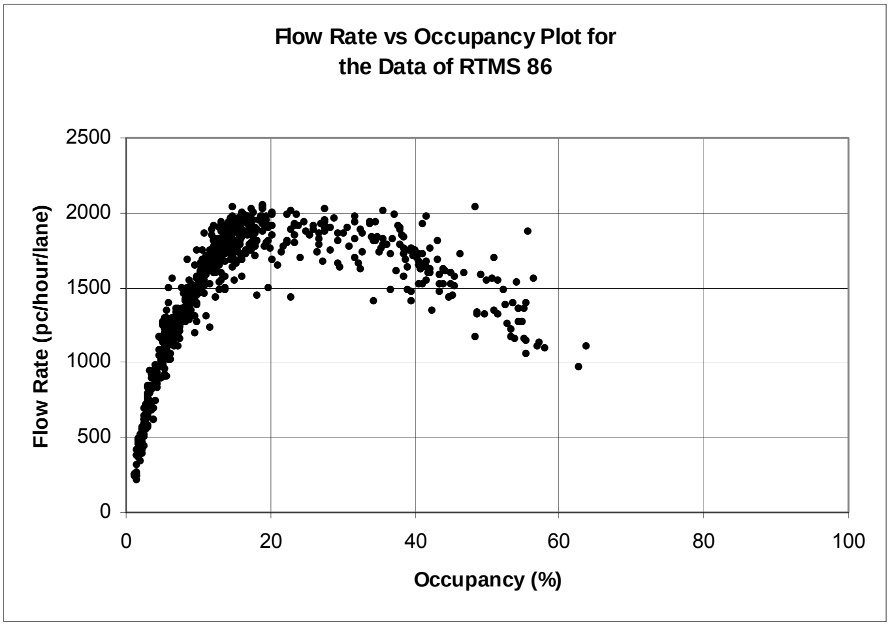

Figure 5.4. Flow Rate vs. occupancy plot for the data of RTMS 86

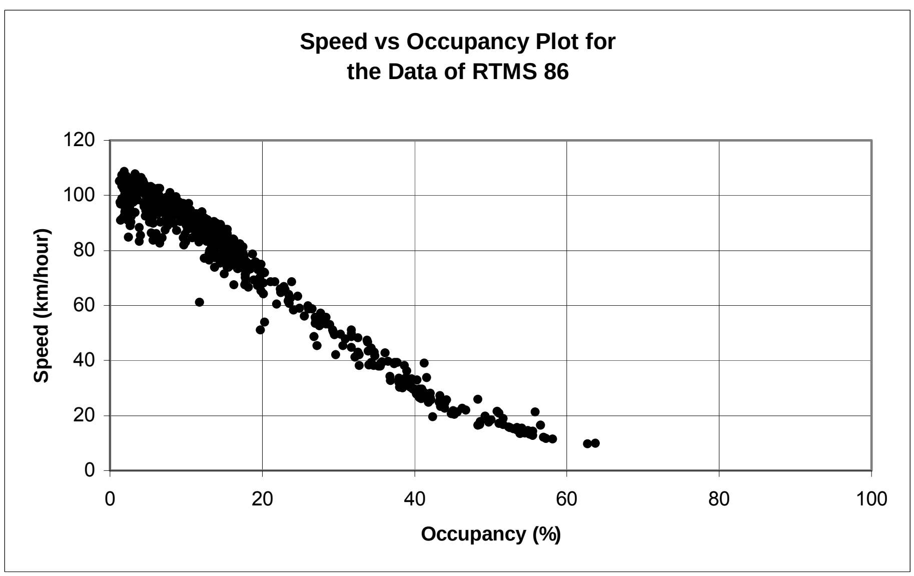

Figure 5.5. Speed vs. occupancy plot for the data of RTMS 86

By means of a statistical program SPSS (2006), quadratic curves were fitted to the related data points (See Figure 5.6). The plots for the other sections were given in Appendix A.

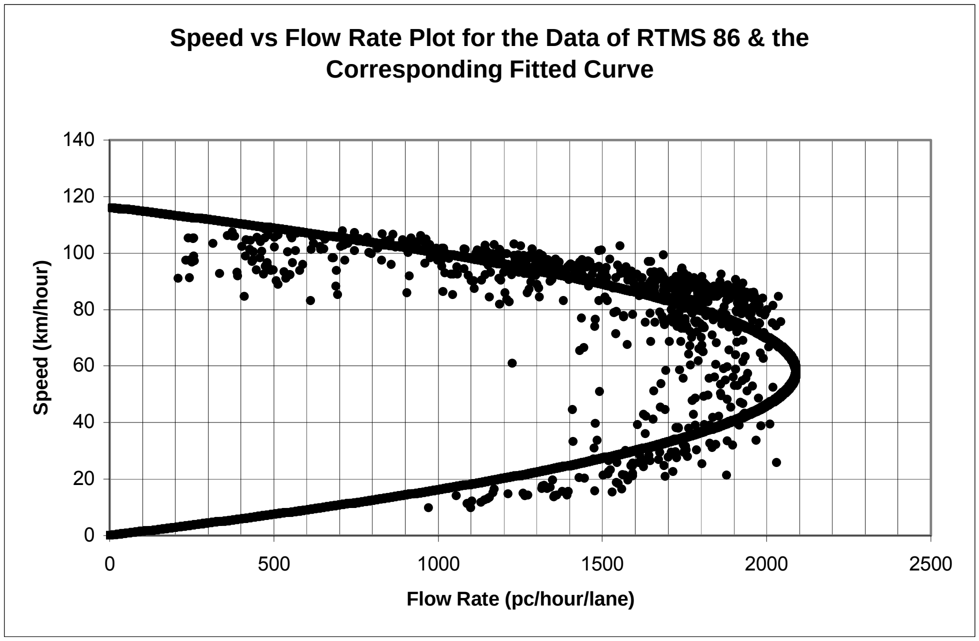

Figure 5.6. The data points and the fitted curve for RTMS 86

From these quadratic curves, capacity, free flow speed, and jam density were found for each selected section as summarized in Table 5.3. The first derivatives of the quadratic equations were set equal to zero. This way the speeds at the capacity values for each section were determined. Setting these speed values back into the equations gave the capacity value. Free flow speeds (FFS) were determined by finding the root of these equations. In order to specify jam densities (Djam), SxD was written instead of V. After setting "S=0", the jam densities were determined.

Table 5.3. Quadratic equations and results for the selected sections

RTMS#	Equation	Capacity	Djam	FFS	R^2
12	$V=81,076S-0,730S^2$	2248	81	111	0,916
20	$V=78,632S-0,806S^2$	1921	79	98	0,907
60	$V=80,133S-0,733S^2$	2190	80	109	0,926
61	$V=72,311S-0,645S^2$	2027	72	112	0,939
65	$V=107,626S-1,168S^2$	2479	108	92	0,924
72	$V=68,857S-0,563S^2$	2105	69	122	0,930
86	$V=71,994S-0,620S^2$	2090	72	116	0,962

The high R^2 values indicate for good model estimations. Some critical points to mention here were that there exist two distinct groups, namely RTMS # 20 and RTMS # 65 were close with respect to their free flow speeds. And the remaining 5 RTMS devices make another group. It will be wise at that point not to group the devices with respect to the road, E-5 or TEM, they were monitoring, since the road characteristics vary greatly from section to section. Actually RTMS devices 12, 60 and 86 were situated on E-5 and 20, 61, 65 and 72 were situated on TEM. Since the speed variance on the same road was considerably large, the grouping was done according to the free flow speeds and capacities.

The capacity of RTMS 65, which was determined with the quadratic curve (see Figure 5.7) looked higher than it was. This was because this section did not had too many data points in the congested region (See Figure 5.8).

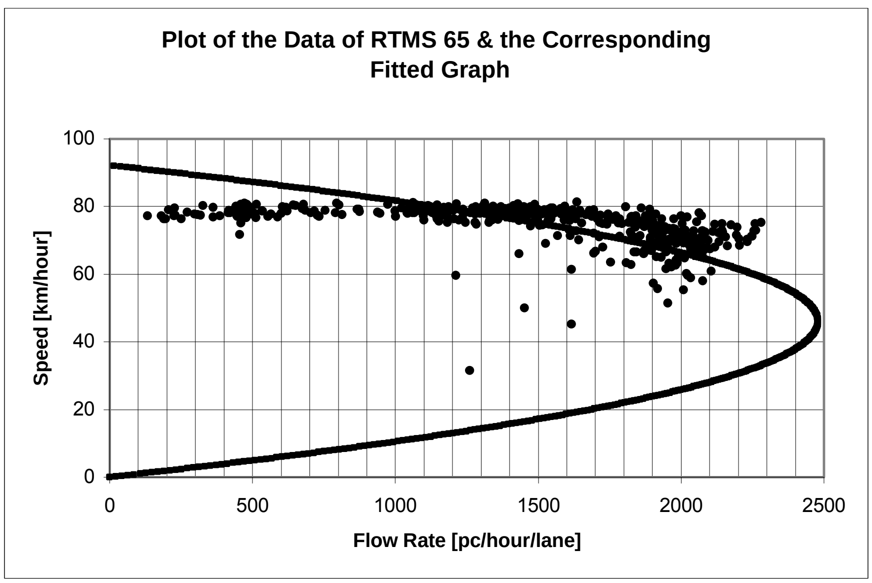

Figure 5.7. The data points and the fitted curve for RTMS 65

5.4. Calibration of Link-capacity Functions

Using the relationship between speed and travel time, as shown in Equation 5.4., the travel times were calculated.

$$TT = \frac{3600}{S}, \tag{5.4}$$

where,

TT = Travel time (sec/km),
S = Speed (km/hour).

The related travel times were calculated and the travel time vs. flow rate graphs were constructed for each quadratic equation in Table 5.3., as summarized in Figure 5.8. One important aspect here is that all the graphs were drawn up to their individual capacity levels, since travel time increases dramatically from that point on.

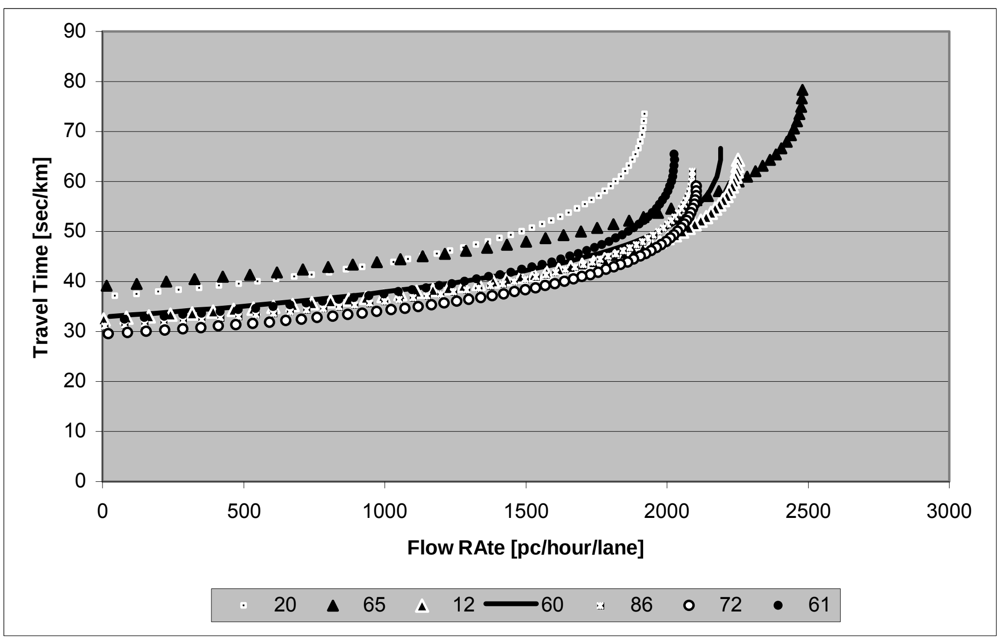

Figure 5.8. Travel time vs. flow rate curves for the selected sections

The curves result in two distinct groups in terms of the free flow speeds, namely group A, which contains data from RTMS devices 20 and 65 and group B with all the remaining devices. The free flow speeds within the groups show quite similar properties. At this point the decision was made that the research was not continued on TEM and E-5 separately, but on two different groups of free flow speeds. The capacity value of RTMS 65 approach 2500 pc/hour/lane but this was because this section did not had any data points related to congested section. With the grouping procedure this problem was solved.

After grouping of the data the same procedure was repeated. The equations of the grouped data, capacities, jam densities and free flow speeds were presented in Table 5.4. These equations were fitted by means of a statistical program, SPSS (2006).

Table 5.4. Quadratic equations and results for the grouped data

Group	RTMS#	Equation	Capacity	Djam	FFS	R^2
A	20 & 65	$V=82,60S-0,848S^2$	2011	83	97	0,910
B	12&60&61&72&86	$V=76,904S-0,68S^2$	2174	77	113	0,924

The travel time vs. flow rate curves of the grouped data were drawn in Figure 5.9.

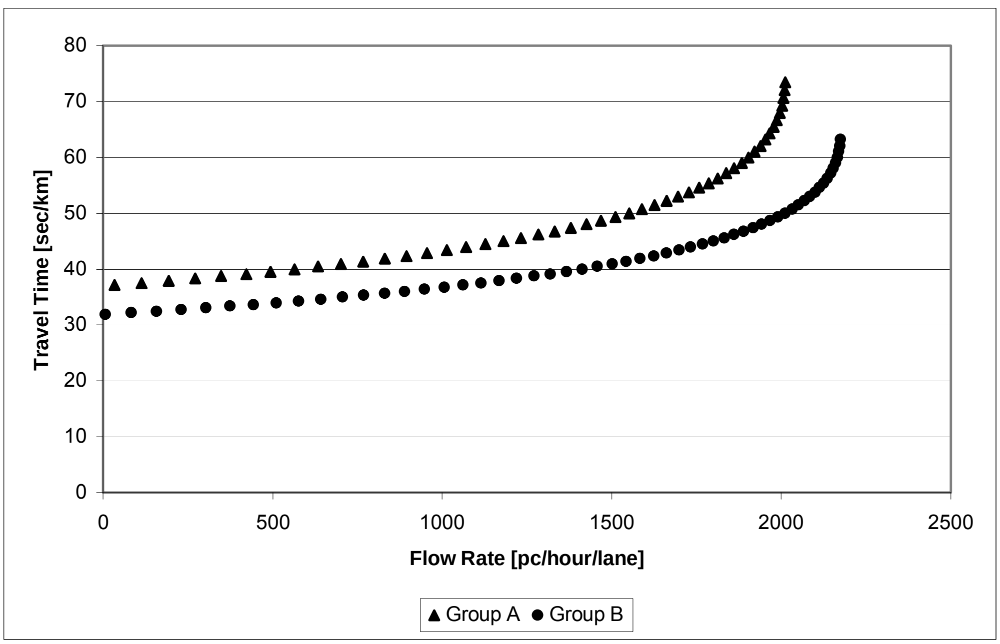

Figure 5.9. Travel time vs. flow rate curves for the grouped data

According to some empirical work dealing with the speed-flow relationship by Banks (1989, 1990), Hall and Hall (1990), Chin and May (1991), Wemple, et al. (1991), Agyemang-Duah and Hall (1991) and Ringert and Urbanik (1993), speeds remain nearly constant even at high flow rates (Hall, 2007). Therefore, in order to decide for the free flow speed and the corresponding travel times, the flow rates less than 1000 passenger car/hour/lane were used and the corresponding average speeds were found. The free flow speed for group A came out to be 78,43 km/hour and for group B it was 99,12 km/hour.

For group A the travel times for this speed came out as 36,32 sec/km while for group B it was 45,90 sec/km.

The travel time values were also taken up to the level of capacity, from that point up the travel times might be unstable and therefore ignored. With this new data set, the modified version of Bureau of Public Roads model, which is given in Formula 5.5, was calibrated. The capacity value in this formula is referred to as “ultimate capacity”, in similar studies.

$$T = T_0\left[1+\alpha(V/C)^{\beta}\right] \tag{5.5}$$

where,

T = Travel time at flow rate V (sec/km),
T_0 =Travel time at zero flow rate(sec/km),
α, β =Calibration parameters,
V =Flow rate(passengercar/hour/lane),
C =Capacity of the link(passengercar/hour/lane).

Other link-capacity functions, as mentioned in the literature review, were tried initially to be fit to the data. The three segment linear function, which was an improved version of the two segment function, had difficulties to explain the data in the overloaded region (See Figure 2.5).

The revised versions of the logarithmic and hyperbolic functions (Equations 2.19 & 2.20, 2.24 & 2.25, respectively) led to negative values of parameter α, which made the application impossible, since taking the logarithm of a negative number is mathematically impossible.

The iterations of parameter estimation for the function suggested by the Traffic Research Corporation (1966) (Equation 2.31) did not converge, probably because the starting values for its parameters were not known, to this author.

Therefore, the BPR Model (Equation 2.29), with steady state capacity, as it was suggested by Steenbrink (1974) (Equation 2.30) was used for the analysis. The nonlinear regression analysis was carried out by the help of SPSS (2006) while for the initial start-up values of α and β, 0,15 and 4 were entered respectively. These were the values suggested by BPR. The other calibration values were as given in Table 5.5.

Table 5.5. Estimated parameters for the calibration process

	Group A	**Group B**
Capacity (pc/hour/lane)	2011	2174
TTo (sec/km)	45,90	36,32
Free Flow Speed (km/hour)	78,43	99,12
TT @ Cap. (sec/km)	73,47	63,16
α (start value)	0,15	0,15
β (start value)	4,00	4,00

The model was calibrated using the "Nonlinear Regression" routine of SPSS. The values of α and β for Group A were determined as 0,158 and 3,843 respectively (See Table 5.6) and the related ANOVA table is given in Table 5.7. R^2 was calculated as 0,248, which was low but since this was a non-linear fitting the output did not include statistics, such as F-statistic in linear regression, to test the statistical significance of the relationship.

Table 5.6. Calibrated values of α and β for Group A

			95% Confidence Interval	
Parameter	Estimate	Std. Error	Lower Bound	Upper Bound
alfa	,158	,006	,145	,170
beta	3,843	,377	3,103	4,584

Table 5.7. ANOVA Table of calibration for Group A

Source	Sum of Squares	df	Mean Squares	R2
Regression	2221986	2	1110993	0,248
Residual	23809	916	25,993	
Uncorrected Total	2245795	918		
Corrected Total	31674	917		

The estimations of α and β for Group B were 0,397 and 3,333, respectively (See Figure 5.8) and the related ANOVA table is given in Table 5.9. In this case R^2 came up to be 0,431, which was better than the one for Group A locations.

Table 5.8. Calibrated values of α and β for Group B

Parameter	Estimate	Std. Error	95% Confidence Interval	
			Lower Bound	Upper Bound
alfa	,397	,010	,377	,416
beta	3,333	,129	3,081	3,586

Table 5.9. ANOVA table of calibration for Group B

Source	Sum of Squares	df	Mean Squares	R2
Regression	4151129	2	2075564	0,431
Residual	54552	2542	21,46	
Uncorrected Total	4205680	2544		
Corrected Total	95805	2543		

These relations were drawn as shown in Figure 5.10. The curves are parallel while the flow rate values are low. They converge with the increasing values of flow rates. The dark lines represent the relationship up to capacity value and the dotted lines represent the flow rates higher than the capacity for each group. Models established with the model calibration are as follows:

Group A: Free Flow Speed $\approx$80 km/hour and capasity=2011 pc/hour/lane

$$T = 45,90\left[1 + 0,158(V/C)^{3,843}\right] \tag{5.6}$$

Grup B: Free Flow Speed $\approx$ 100 km/hour and capasity=2174 pc/hour/lane

$$T = 36,32\left[1 + 0,397(V/C)^{3,333}\right] \tag{5.7}$$

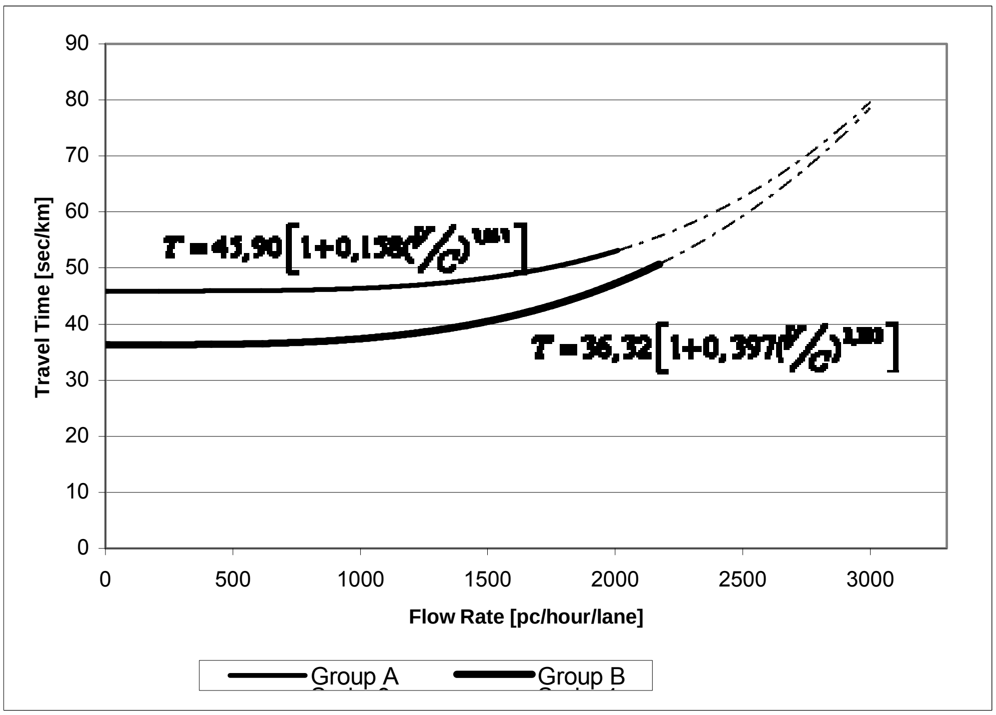

Figure 5.10. Travel time vs. flow rate curves for the calibrated models

Using Formulas 5.6 and 5.7, the relationship between speed and flow rate can also be calculated. The formulas for speed were as given in Formula 5.8 & 5.9. With these formulas the curves given in Figure 5.11 were drawn.

Group A: Free Flow Speed $\approx$ 80 km/hour and capacity =2011 pc/hour/lane

$$S = \frac{3600}{45,90\left[1 + 0,158(V/C)^{3,843}\right]} \tag{5.8}$$

Group B: Free Flow Speed $\approx$ 100 km/hour and capacity =2174 pc/hour/lane

$$S = \frac{3600}{36,32\left[1+0,397(V/C)^{3,333}\right]} \quad (5.9)$$

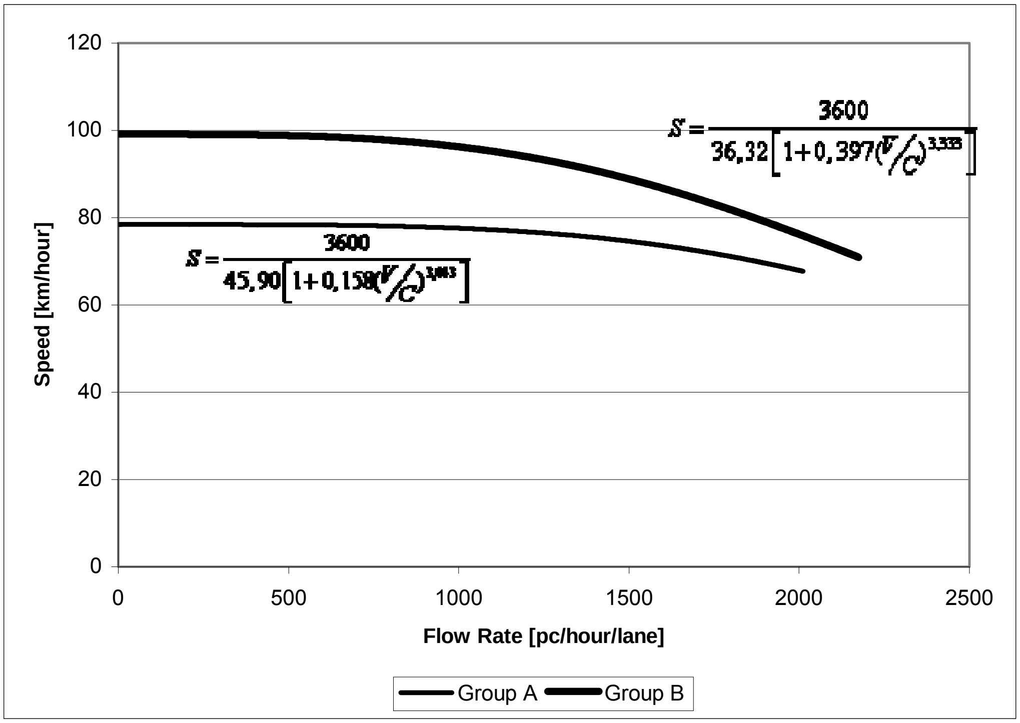

Figure 5.11. Speed vs. flow rate curves for the calibrated models

These final graphs are quite similar to the ones in the Highway Capacity Manual (Figure 5.12). The main difference between those two was that the graphs in the manual are more insensitive to flow changes, the speeds do not change up to a flow rate level of 1300 pc/hour/lane, while in this study they started to change between 500-1000 pc/hour/lane. The reason for that was the curves in the Highway Capacity Manual do not represent any theoretical equation, but instead represent a generalization of empirical results and were mostly hand-fitted. On the other hand the graphs of this research were developed by fitting mathematical models to the data.

Speed-Flow Relationship

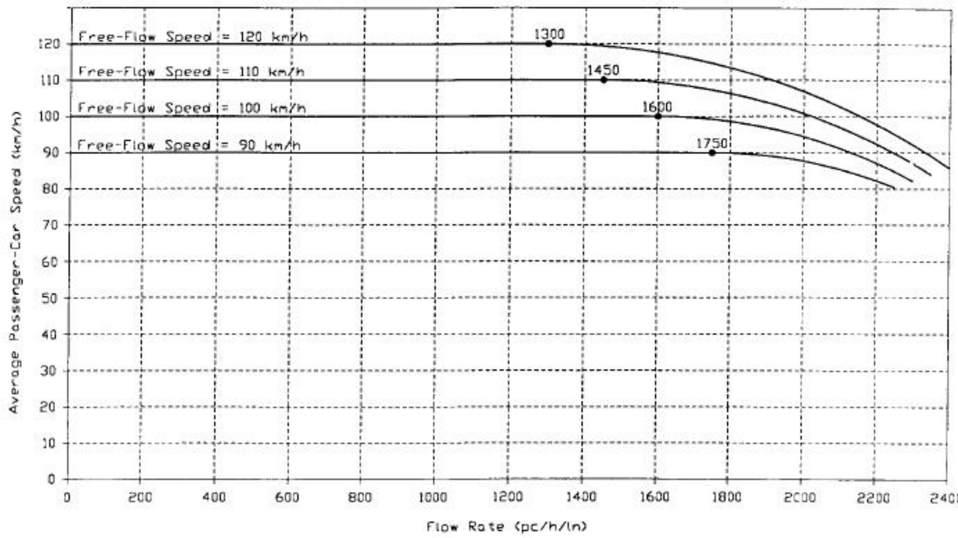

Figure 5.12. Speed vs. Flow rate relationships for basic freeway sections in the Highway Capacity Manual (Transportation Research Board, 2000)

5.5. The Need for a Three-Dimensional Model

In a paper of Hall, et al.(1992) a different model was suggested (See Figure 5.13), which is a jump stone between the traditional quadratic curves and the new ones in the Highway Capacity Manual (Hall, 2007). This curve and also the curves in the Highway Capacity Manual (Transportation Research Board, 2000) as shown in Figure 5.12 do not represent any theoretical equation, but instead they represent a generalization of empirical results. In that fundamental respect, the most recent research differs considerably from the earlier work

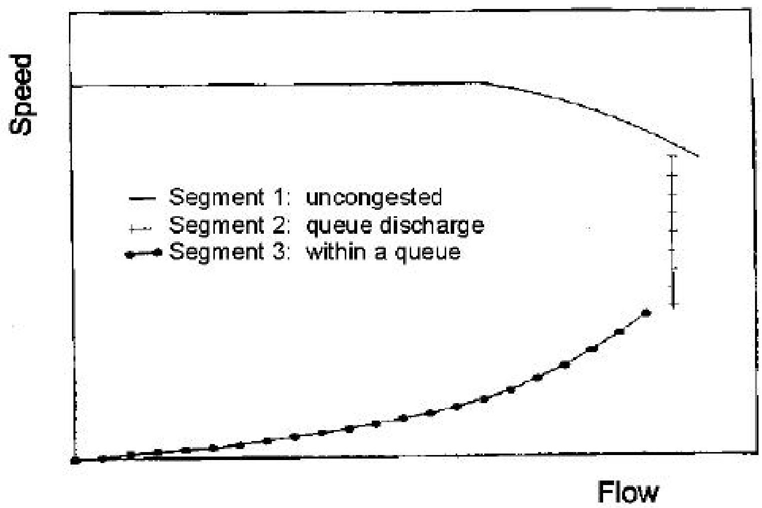

Figure 5.13. Generalized shape of speed flow curve proposed by Hall, Hurdle, and Banks (Hall et al. 1992)

The empirical observations rarely fit exactly with the relationship $v = S \times D$, especially when the observations are taken during congested conditions (Hall, 2007). Hence, focusing on the two-dimensional relationships will not often provide even implicitly a valid three-dimensional relationship, which makes the need for a three dimensional analysis desirable. For a three dimensional approach a mathematical model named as catastrophe theory was found out to be quite appropriate. In the following sections the characteristics of this theory were summarized and a preliminary testing of the model was performed.

5.6. Application of Catastrophe Theory to the Data

For the application of catastrophe theory four sample days data, 23-26 May 2006, one of the RTMS devices, namely RTMS # 12 was chosen. The main reason for choosing this one is that it had a critical position on the Anatolian side of the Bosphorus Bridge entry (See Figure 4.1).

A catastrophe is a sudden change in a parameter when gradual changes were previously experienced and were expected to continue. The sudden changes in speed, flow and occupancy were represented in Figure 5.14, 5.15, and 5.16, respectively. From these three variables, speed was selected as the state variable (Hall, 1986). As it can be seen from the graphs the changes in the speed was more rapid if all the graphs were considered.

The graphs contain the data of four days. The variation of the graphs can be categorized under four groups; the circle starts with night and goes on with morning period, noon and evening, respectively. Especially, the changes from night to morning and from evening to night were dramatic. There were sudden jumps during those sections all the time. But the transitions from morning to noon some times experience those rapid changes and some times not.

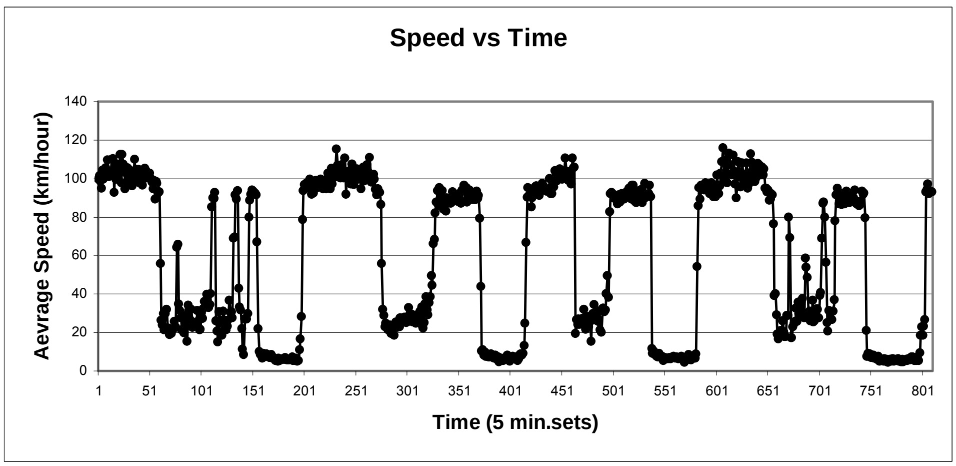

Figure 5.14. Speed vs. time graph for four day data of RTMS 12

The transitions of the flow data from night to morning and from evening to night were not as catastrophic as the speed data (See Figure 5.15), nor were the occupancy data (See Figure 5.16). There were some data points during these transitions.

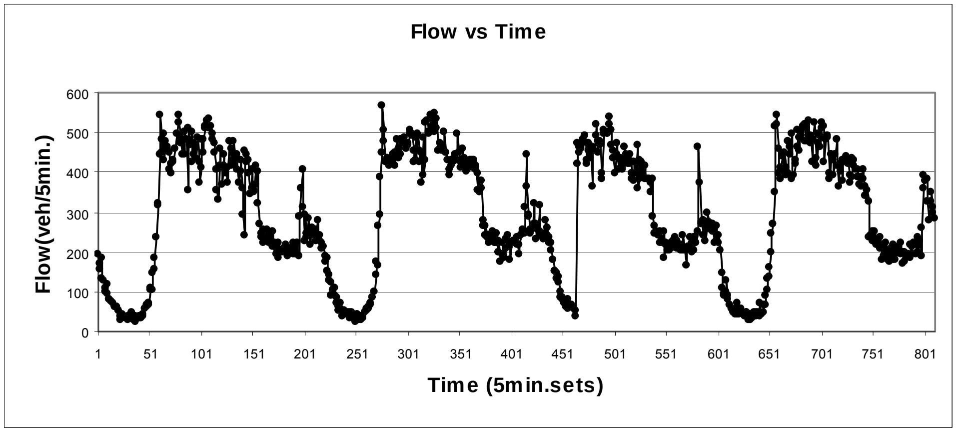

Figure 5.15. Flow vs. time graph for four day data of RTMS 12

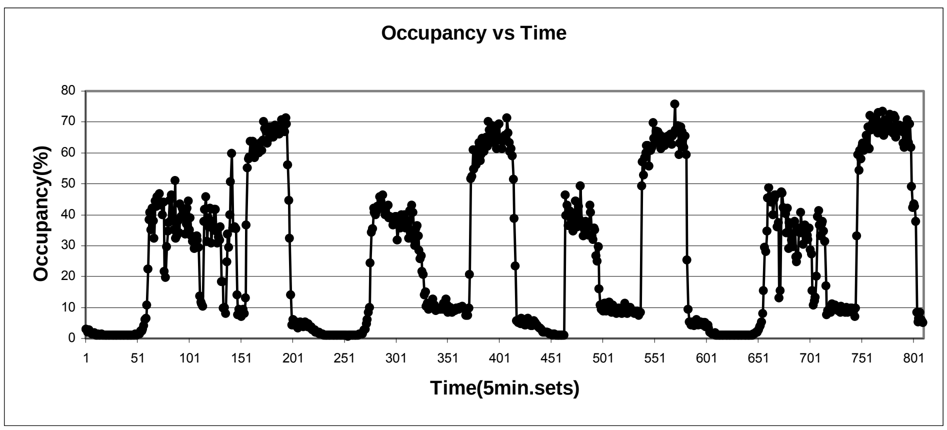

Figure 5.16. Occupancy vs. time graph for four day data of RTMS 12

The data for four days contains 809 trios of speed-volume-occupancy values. These data points were plotted three dimensionally (See Figure 5.17). At this plot it was obvious that there is a sudden drop at the speed values as in the catastrophe theory. Also it was seen that this data was more appropriate to get explained with a Maxwell convention rather than perfect delay convention.

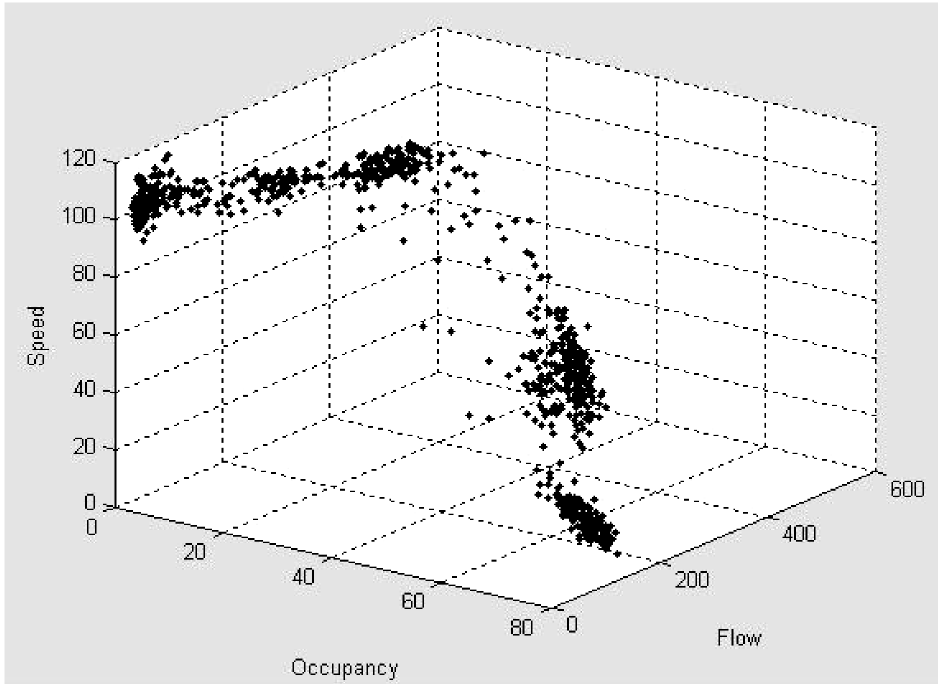

Figure 5.17. Three-dimensional plot for four day data of RTMS 12

Projections of this three dimensional plot, to each plane were given in Figure 5.18, 5.19 and 5.20. They represent the relationships between speed-volume, speed-occupancy and volume-occupancy, respectively.

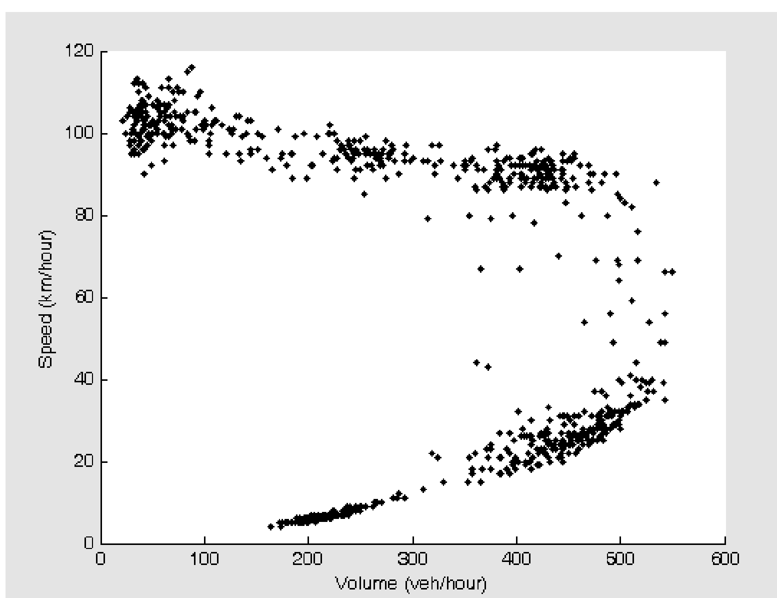

Figure 5.18. Relationship between speed-volume

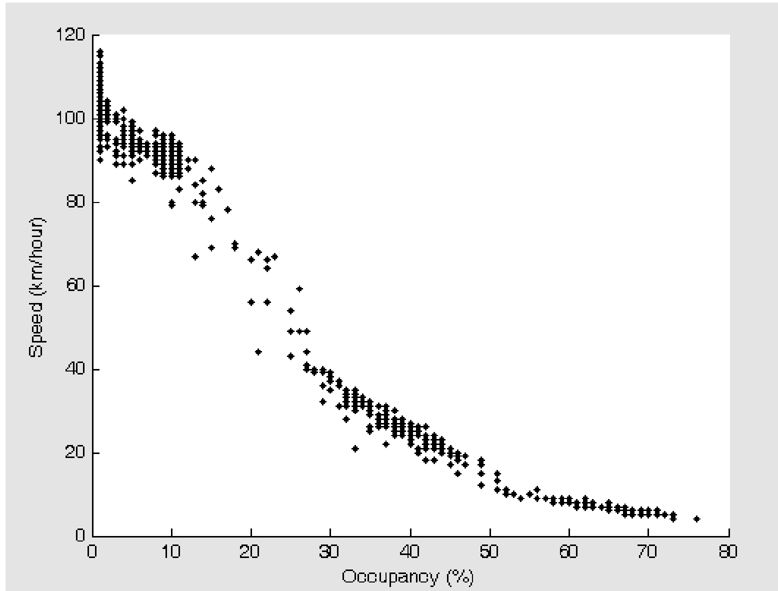

Figure 5.19. Relationship between speed-occupancy

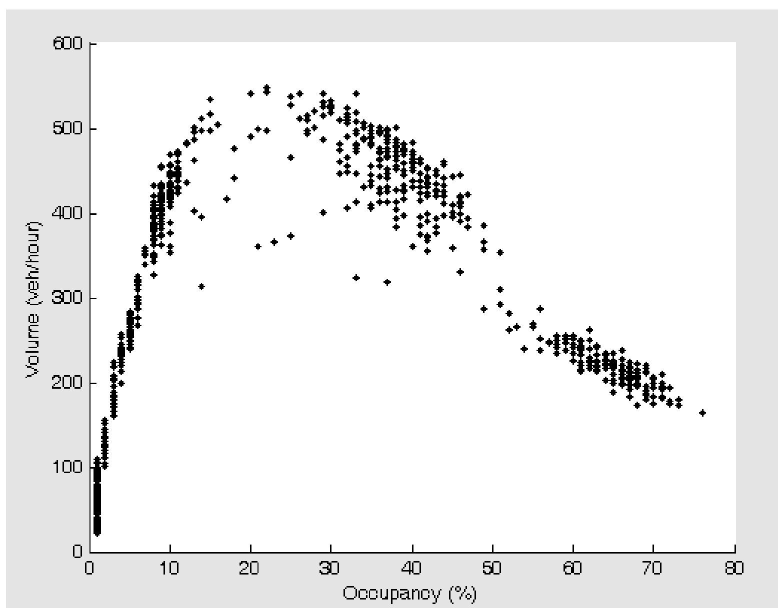

Figure 5.20. Relationship between volume-occupancy

5.6.1. Calibration of the Model

Out of these 809 data sets, a random chosen sample of 400 was used for the calibration process of the suggested model by Hall and the remaining 409 samples were used for the validation. In order to achieve a successful model the data obtained from the field had to be transformed, so the axis origins lie at the point where the fold begins to appear as shown in Figure 5.21. For the transformation, the maximum volume and speed and occupancy levels at that volume were to be determined. Then from each individual point these max volume, speed and occupancy values were subtracted (Ya-Ping and Yu-Long, 2003).

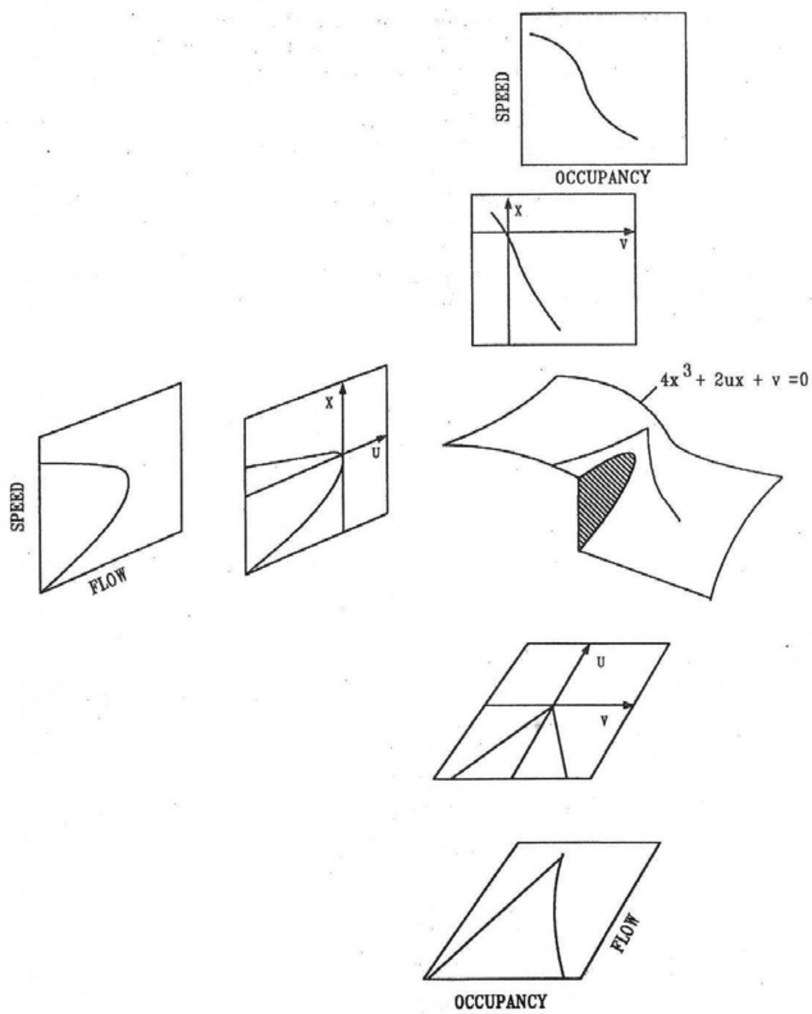

Figure 5.21. Catastrophe surface and the possible path to traffic operations with transformation (Forbes and Hall, 1989)

With this new set of transformed data, the calibration of the Hall's model was done with the use of a statistical program.

The suggested critical surface of Hall

$$4aX^3 + 2bXY + cZ \,, \tag{5.10}$$

where,

X: Speed (km/hour),
Y: Volume (veh/hour),
Z: Occupancy (%),

can be rewritten in an explicit form,

$$Z = \frac{-4aX^3 - 2bXY}{c} \,. \tag{5.11}$$

For this model, the suggested parameter values after iterations were given in Table 5.10.

Table 5.10. Estimated parameters after calibration

Parameter	Estimate	Std. Error	95% Confidence Interval	
			Lower Bound	Upper Bound
a	,114	84449	-166022	166022
b	-1,504	1117765	-2197481	2197478
c	3466,8	3E+009	-5064348912	5064355846

The critical surface becomes,

$$Z = \frac{-0,456X^3 + 3,008XY}{3466,8} \,, \tag{5.12}$$

while the model is,

$$0,114X^4 - 1,504X^2Y + 3466,8XZ\ . \tag{5.13}$$

The ANOVA table for this calibration (See Table 5.11) shows good results. The model had a 96 per cent explanation power for this calibration data. After this, the validation process was done.

Table 5.11. ANOVA Table for the calibration analysis

Source	Sum of Squares	df	Mean Squares	R2
Regression	257611	3	85870	0,959
Residual	10356	397	26,085	
Uncorrected Total	267966	400		
Corrected Total	251489	399		

5.6.2. Validation of the Model

For the validation the remaining 409 data points were used. This data had to be also transformed for consistency. The related analysis of variance table is shown in Table 5.12 The high value of R^2 is a good indication for the fit. It indicates that the model explains 94,8 per cent of this validation data.

Table 5.12. ANOVA Table for the validation analysis

Source	Sum of Squares	df	Mean Squares	R2
Regression	229109	3	76370	0,948
Residual	12172	406	29,981	
Uncorrected Total	241281	409		
Corrected Total	232977	408		

6. CONCLUSIONS AND RECOMMENDATIONS

The main conclusions of this study were summarized below.

1. The speed vs. flow rate graphs of the calibrated functions were found to be quite similar to those in the Highway Capacity Manual (Transportation Research Board, 2000). The main difference between those two is that the graphs in the manual are more insensitive to flow changes, the speeds do not change up to a flow rate level of 1300 pc/hour/lane, while in this study they started to change between 500-1000 pc/hour/lane. The explanation for this can be that the graphs of Highway Capacity Manual do not represent any theoretical equation, but instead represent a generalization of empirical results, drawn by hand.
2. The final functions were not designed for different road types, like TEM and E-5, instead according to the free flow speed levels at that link, namely 80 and 100 km/hour.
3. Link-capacity functions differ from region to region, for that reason every country should derive the appropriate functions for its own roads and use them. With this study link-capacity functions for Istanbul freeways were calibrated.
4. The final graphs of link-capacity functions (See Figure 5.11) were drawn to each group’s capacity level, since after exceeding capacity, travel times may increase exponentially, but for the results of this study they still remain realistic.
5. The application of the catastrophe theory gave an insight for further applications. According to this theory it is quite obvious that the critical variable is speed.
6. The occupancy data obtained from the field was not converted to density, since the relationship between them is quite complex and somewhat arbitrary if the detector on the field is not a magnetic loop detector. Therefore the speed and volume data were used for the analysis.
7. It was seen that there can be a variation in free flow speed within a section, because of the geometric properties of the road, weather conditions, speed variation during

day and night, and the profiles of drivers. In order to find the most appropriate value, speeds achieved during a flow rate level less than or equal to 1000 pc/hour/lane were averaged.

8. The established models were as follows:

- Group A: Free Flow Speed ≈ 80 km/hour and capacity =2011 pc/hour/lane

$$S = \frac{3600}{45,90\left[1+0,158(V/C)^{3,843}\right]} \tag{5.8}$$

- Group B: Free Flow Speed ≈ 100 km/hour and capacity =2174 pc/hour/lane

$$S = \frac{3600}{36,32\left[1+0,397(V/C)^{3,333}\right]} \tag{5.9}$$

9. The catastrophe theory was applied to a freeway section and the results looked quite promising. The model for location of RTMS 12 was as follows:

The critical surface becomes,

$$Z = \frac{-0,456X^3 + 3,008XY}{3466,8} \tag{5.12}$$

While the model is,

$$0,114X^4 - 1,504X^2Y + 3466,8XZ \tag{5.13}$$

where,

X: Speed (km/hour),
Y: Volume (veh/hour),
Z: Occupancy (%),

Some recommendations for further research could be:

- Most of the RTMS devices were positioned without understanding the nature of the device and because of that, devices were used inefficiently, which means waste of resource and money. It would be much better to decide where to locate these radars in such a way that the data will be readily usable.
- The further research should be done in order to determine the general mathematical expression for the cusp generated on the model. The projection of this cusp on the speed-flow plane will be the actual two dimensional shape of this relation. The projection to the other planes will provide the relationship between speed vs. occupancy and flow rate vs. occupancy.

APPENDIX A: SPEED VS. FLOW CURVES FOR THE SELECTED SECTIONS

This section includes the Speed vs. Flow Rate plots of individual data and the corresponding fitted curves.

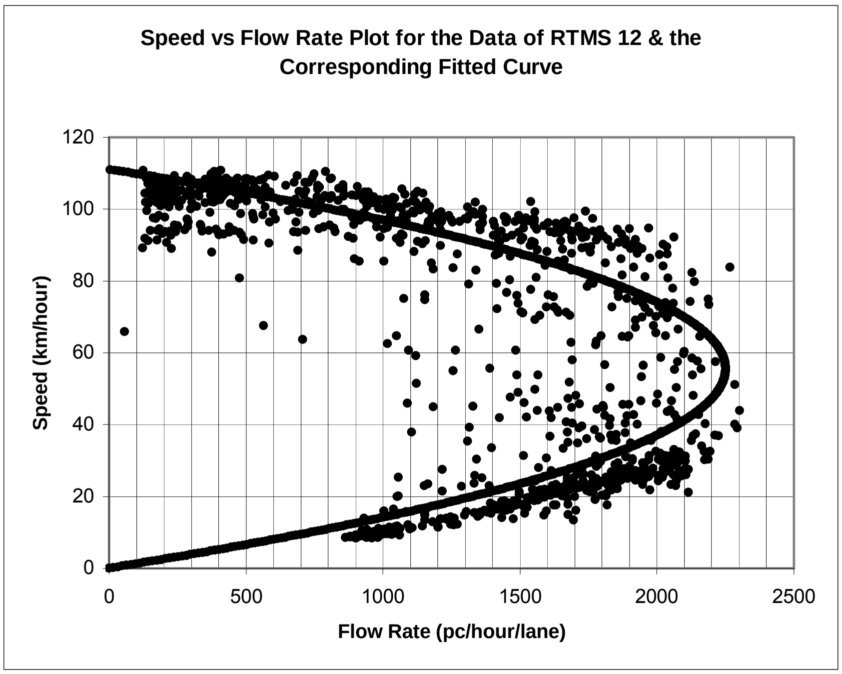

Figure A.1. Speed vs. Flow Rate Data plot for RTMS 12 and fitted curve

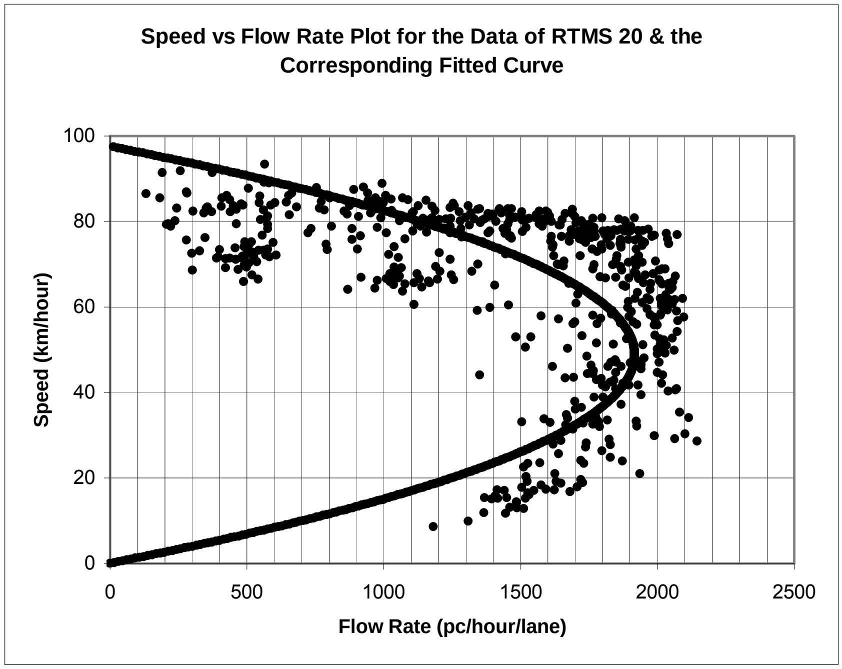

Figure A.2. Speed vs. Flow Rate Data plot for RTMS 20 and fitted curve

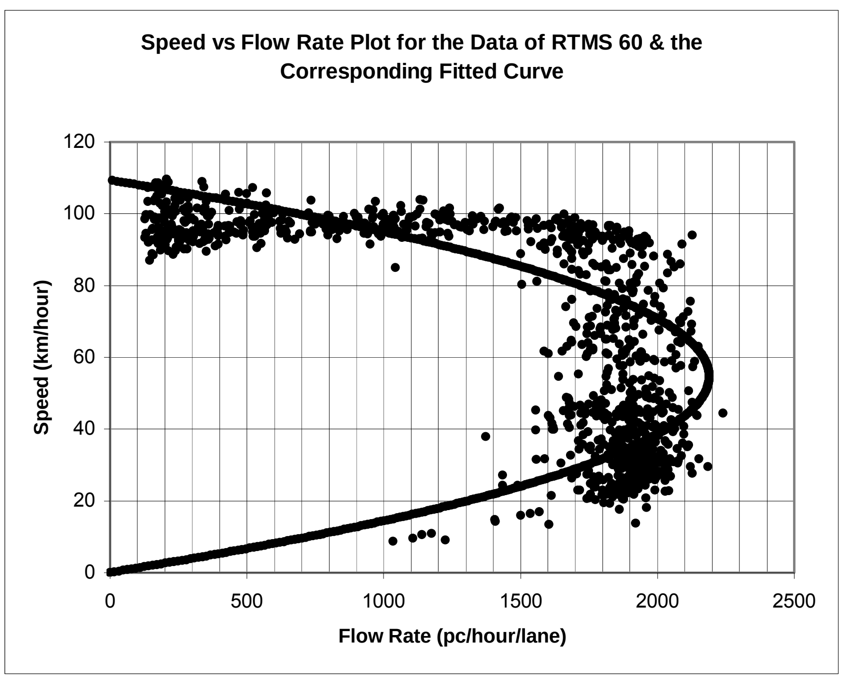

Figure A.3. Speed vs. Flow Rate Data plot for RTMS 60 and fitted curve

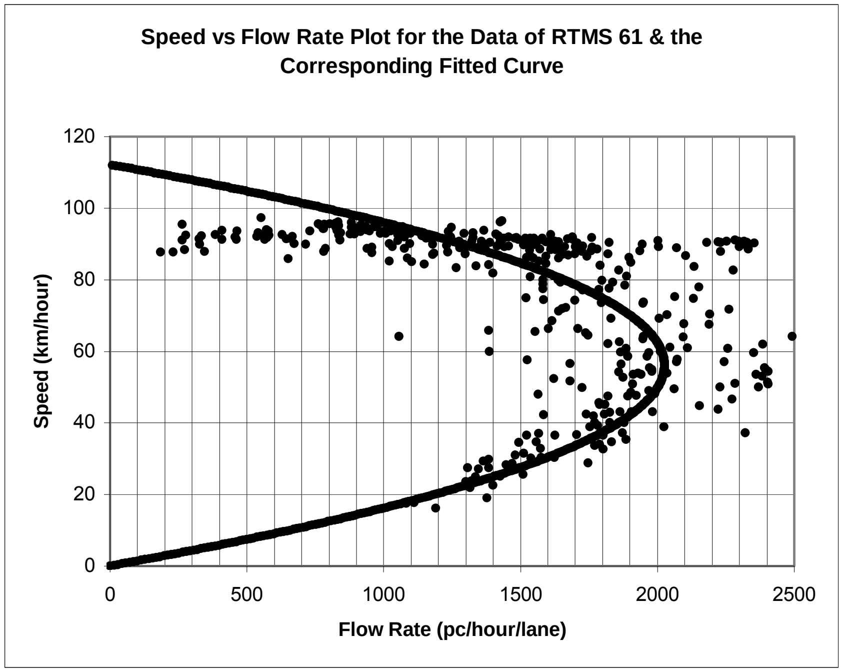

Figure A.4. Speed vs. Flow Rate Data plot for RTMS 61 and fitted curve

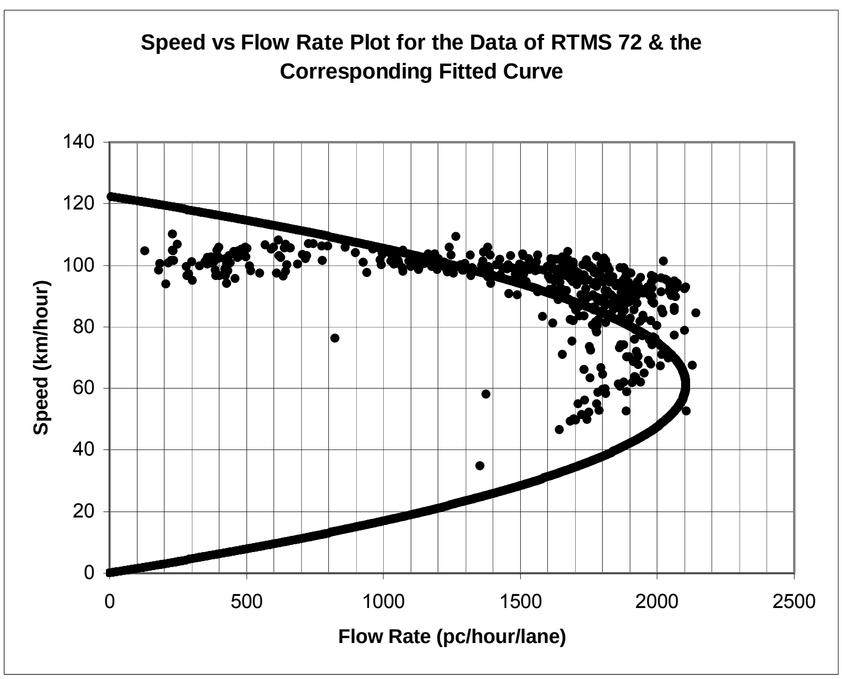

Figure A.5. Speed vs. Flow Rate Data plot for RTMS 72 and fitted curve

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