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EVALUATING EFFICIENCY AND EQUITY CONCEPTS OF TRAFFIC CONTROL STRATEGIES: A NOVEL TRAFFIC CONTROL AND EQUITY MEASUREMENT APPROACH

Doctoral Dissertation
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July, 2013

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in loving memory of a passionate highway engineer,
Sabahattin Kesten (1952 – 2007)

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ABSTRACT

Traffic congestion has been a problem since 1950's in the developed countries and parallel to the economic development in the world it is becoming a serious trouble for the cities of developing countries as well. Intelligent transportation systems offer different traffic control measures such as traffic signal, ramp metering, speed control, route guidance, etc. and they are used to minimize the impacts of traffic jams such as additional emissions and fuel consumption and to optimize the performance of the traffic network. One of the common features of these strategies is that they are designed to be implemented in the developed world countries. However, the traffic control techniques designed for the developed countries requires high amount of investments on infrastructure besides extreme management and maintenance costs to keep the system operational. Generally, efficiency is regarded as the only objective in the design and the management of traffic control systems, which is another important subject to be discussed. There are some studies showing that the long delays could be observed on the metered ramps (Levinson and Zhang, 2002) and the control strategies can create equity concerns while achieving a gain in congestion alleviation (Kotsialos and Papageorgiou, 2004). To gain public acceptance and support for the employment of the traffic control strategies, equity begins to emerge as an important issue.

There are numerous theoretical algorithms available to control the traffic dynamically. However due to the complex nature, in practice only few of these sophisticated algorithms are in use. Therefore, simple, applicable and efficient control strategies should be sought for the traffic congestion relief. The number of studies in analysis of equity issues in traffic control is limited, and their findings are mostly qualitative and sometimes even conflicting. For instance, efficiency and equity are usually considered as two competing requirements: the more efficient a control system is, the less equitable it becomes (Kotsialos & Papageorgiou 2004) (Meng & Yang 2002). Apparently, there is a need to resolve these contradictory findings, and develop proper equity measures to be used in the evaluation and further design of control systems that consider the efficiency and equity.

The main objective of the present research consists; investigation of an optimal cycle time for fixed-time ramp metering and optimal speed limit for speed harmonization control strategies besides development of an alternative equity index for evaluation of equity properties of

traffic state. The proposed methodologies are examined within the traffic simulation environment. The traffic control strategies performances and equity index are compared with some common traffic control strategies and well-known indexes through simulations of real networks.

The research began with an extensive review of the fundamental relationships in traffic flow and traffic simulation. The models are classified according to level-of-detail and some key points are discussed. With respect to model applicability, microscopic simulation models are found more appropriate for this research that are more realistic and representative in terms of bottleneck formation. Hence, the traffic control and equity measurement analyses are performed with microscopic traffic simulation software.

For the traffic control theory, a fixed-time ramp metering control algorithm and harmonized speed control are proposed and the optimality of the cycle time and speed limits are analyzed through traffic simulation experiments. The fixed-time ramp metering considers bottleneck capacity, thus the underlying principle of control is the demand-capacity control. The offline demands are measured and the maximum admissible ramp volumes transformed into the green times for the control. The speed harmonization control applies the speed limits more particularly the speed ranges for the whole network. The optimum speed range is determined through a set of experiments in traffic micro-simulation program. The results implies that there is an optimal cycle length for fixed time ramp metering and optimum speed limit for harmonized speed controls that minimize the total travel time and maximize the average speed on the corridor which are found 15 seconds and 70km/h respectively.

In the equity measurement part of the study, the existing equity measurement methodologies developed for social sciences, economy and traffic researched are analyzed. It is found that the previous researches still did not have strong emphasis on the conceptualization of equity in traffic control where most of the indices are barrowed from econometric studies that are usually incorporating the differences of pairs among the mean values. There are also some other problems detected in terms of mathematical formulation of equity, for instance when the delay values are incorporated with Gini Index this would yield 0/0 indeterminate form if there is no delay in traffic. To overcome the drawbacks, a new equity index, namely K, is proposed to assess the equity of traffic considering the mathematical and theoretical subjects. The conceptualization of equity in K index is based on the notion that the ratio of the speed differences (vehicles or time frames) over a specific value which can be selected as a meaningful traffic state parameter such as free- flow speed. The equity index is structured on the notion that it should depend on the differences among the peers but not the mean value of the peers. The speed of a vehicle is found to be the most appropriate indicator for K index that gives the index flexibility to measure the equity of speed and delay distribution by taking the denominator as optimum speed or free-flow speed.

This research has presented the results of applying the fixed time ramp metering, speed harmonization and integrated control for the regulation of the inflow from on ramps to the mainstream of a highway corridor for the case study of Istanbul 0-1 corridor. Furthermore, extension of model predictive control strategies are derived and implemented as a series of freeway traffic control strategies as either a standalone strategy or the integrated ones, within the case study of Tokyo Route-3. The results also reveal that the speed increase (gain) in traffic comes up with an equity decrease (loss) for the ramp metering strategies. However, the trade-off is not the general pattern. The speed management control strategies can achieve increase in both efficiency and equity at the same time in expense of loss in efficiency compared to the ramp metering control. The integrated control strategies, which are the combination of ramp metering and speed management strategies, are more successful either in establishment of efficiency and equity at the same time.

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ABBREVIATIONS

FTC	:	Freeway Traffic Control
RM	:	Ramp Metering
SM	:	Speed Management
IC	:	Integrated Control
ITS	:	Intelligent Transportation Systems
RTMS	:	Remote Traffic Microwave Sensor
VR	:	Virtual reality
LOS	:	Level of service
VMS	:	Variable message signs
ALINEA	:	Asservissement Lineaire d'Entree Autoroutiere (French) stands for Linear Utilization for Highway Entrances
SWARM	:	System wide adaptive ramp metering
ARMS	:	Advanced Real-time Metering System
BEEX	:	Balanced Efficiency and Equity
MILOS	:	Multi-Objective, Integrated, Large-Scale, & Optimized System
SZM	:	Stratified Zone Metering
VSL	:	Variable speed limit
AWS	:	Active warning signs
PCMS	:	Portable changeable message signs
RGS	:	Route guidance systems
HSC	:	Harmonized speed control
MPC	:	Model predictive control
FTRM	:	Fixed-time ramp metering
MDG	:	Millennium Development Goal
SP	:	surplus procedure
TTS	:	Travel Time Savings
TTT	:	Total Travel Time
TWT	:	Total Waiting Time
MEX	:	Tokyo Metropolitan Expressway

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CHAPTER 1. INTRODUCTION

1.1 Background

Since 1950s, the rate of road transportation demand is increasing more than the road infrastructure supply, thus this unbalanced growth yields traffic congestion in most of the cities in the world. During peak hours every morning and every evening, the capacity of numerous motorways around the world is reached, resulting in traffic jams. These traffic jams induce costs to the society at several levels. As a response to the growing internationalization of the economy, the demand for mobility of people and goods as well as the motorization demand increases as it is shown in Figure 1.1 (Davis et al., 2011). Companies tend to locate their offices and production units near accesses to motorways in order to be quickly transport their goods and in order to be easily accessible for their employees and their clients. Delays caused by congestion result in a huge loss of productivity either in commercial or individual level. Most of the business sector requires a smooth flow of goods and the reliability of travel times. The delays caused by congestion force the companies to keep larger and thus more expensive stocks in order to guarantee the reliability of the production process.

While the more vehicles are queuing up in congestion, the emissions released from the vehicles have also some negative effects on environment. Especially, recent studies show that these emissions causes air pollution in most of the cities and are one of the major contributors to the global warming (Jacobson, 2009). Congested traffic operation results in an increased fuel consumption, which has both an economical and an ecological side (Smit, 2008).

Another important consequence of traffic congestion is the traffic accidents. Traffic congestion affects the magnitudes of other road-traffic externalities including accidents (Hensher, 2006 and Steimetz, 2008) and road damage (Hussain and Parker, 2006). However, there is a study suggests that traffic congestion has little or no impact on the frequency of road accidents (Wang et al, 2009). On the contrary, a more recent study explains the relationship between the levels of congestion and crashes, even the types of crashes, at the basic freeway segments by using regression trees (Hossain and Muromachi, 2013).

Different policies can be applied in order to tackle the congestion problem. The construction of new roads may help to eliminate some important bottlenecks or to fill 'missing links' in the existing motorway network. As an example to the construction of missing links, several stretches of Tokyo Metropolitan Expressway radial roads are under construction in order to build a more accessible and functional highway system (Omura, 2010). Bottlenecks can be reduced or eliminated by adding more lanes however the main disadvantages of this strategy are that the construction of new roads is very costly and adding more capacity at one location often merely shifts the congestion to another location (Fields et al, 2009).

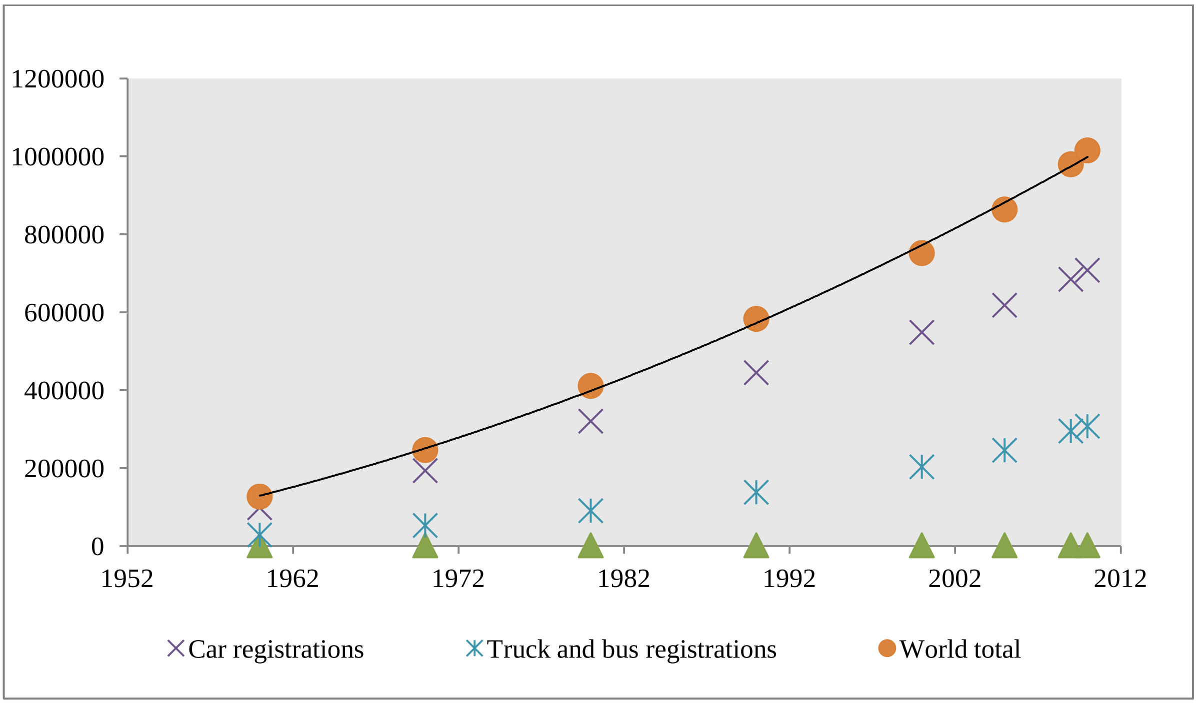

Figure 1.1: Historical trend of worldwide vehicle registrations (Davis et al., 2011).

Stimulate the use of other modes of transportation such as public transportation, car-pooling, transport of goods by train or by ship would be another solution. However, in order to realize a shift from road transportation to other modes of transportation, the other modes of transportation need to offer a good quality of service, which often requires large investments.

Demand control is another alternative in congestion alleviation. The demand for road transportation can be reduced by encouraging the public transportation use and inauguration of additional taxes or congestion charges. The former requires investments of the government while the latter is an unpopular and politically hard decision (Chen et al., 2001)

An improved use of the existing infrastructure, which can be implemented by control measures such as ramp metering, speed management and route guidance, should also be considered as viable alternatives to demand management policies. Implementation of Intelligent Transportation Systems (ITS), which apply a combination of control, telecommunication and computer technologies in order to assist the driver in reaching the desired destination while attempting to globally avoid congestion and improve safety are among the other alternatives (Weiland and Purser, 2000). Since ITS systems require on-board sensors, processing power, which is not widely available in developing countries. Furthermore, there are some concerns about the privacy and confidentiality of ITS data collection policies, therefore, in practice vehicle owners are not willing to accept some of these advanced technologies (Steinfeld, 2010).

Given the huge costs of congestion to the society and the urgent need for solutions, this research investigates a solution applicable in the short term. More in particular, fixed time ramp metering and low cost control strategies are considered as traffic control measures in an advanced traffic management system set-up that optimizes the use of the currently available traffic capacity (Cambridge Systematics, 2001 and Papageorgiou and Kotsialos, 2000). Ramp metering is merely an example of a traffic control measure that can be controlled using the model predictive control framework for advanced traffic management systems that is also present in this thesis. The fixed time ramp metering and speed harmonization control are selected as a traffic control measures in this thesis since its main advantages are that it can be realized in the short term and at a minimal cost. Fixed-time ramp metering use the existing infrastructure and only relatively limited investments would be necessary to run the system even without installation of loop detectors or any sort of sensors.

1.2 Problem Statement

Traffic congestion is a major problem in most cities around the world, especially in developing regions resulting in massive delays, increased fuel wastage and monetary losses. When unplanned development and rapid increase in population come across the limited resources such as freeways become more vulnerable.

The term sustainable transport came into use as a logical follow-on from sustainable development, and is used to describe modes of transport, and systems of transport planning, which are consistent with wider concerns of sustainability. There are many definitions of the sustainable transport, and of the related terms sustainable transportation and sustainable mobility (Green Paper, 1995). One such definition, from the European Union Council of Ministers of Transport, defines a sustainable transportation system as one that:

i. Allows the basic access and development needs of individuals, companies and society to be met safely and in a manner consistent with human and ecosystem health, and promotes equity within and between successive generations.

ii. Is Affordable, operates fairly and efficiently, offers a choice of transport mode, and supports a competitive economy, as well as balanced regional development.

Therefore, a sustainable control system for highway traffic should hold the total travel time spend in the network by all user groups as minimal by establishing the alleviation of traffic related emissions and other environmental problems and considering the differences between the groups (vehicles) and/or among the groups either at spatial or temporal level. However, establishment of the latter needs a consensus on what kind of measurement is required and which method can be applied to calculate the differences in between/among the groups.

Thus, definition of a sustainable control in traffic engineering can be restated. The control design considering the efficiency, equity and the environmental issues should aim to provide the most efficient, equitable and environmental friendly solution all together at the same time.

Benefits attributed to freeway traffic control in the literature include increased freeway speeds, decreased travel times, reduced delays, increased freeway capacity/throughput, reduction in accidents/improved safety, congestion reduction, cost-effectiveness, reduction in emissions/improved air quality, reduction in fuel consumption/improved fuel economy, and efficient use of capacity. The disadvantages caused by freeway traffic control in the literature include traffic diversion, increased ramp delays and spill back, equity issues, costs of installation, maintenance, enforcement, and public relations; on-ramp emissions and fuel consumption and mode choice issues.

In traffic control systems design, efficiency has been considered as the single most important measure for years. From the designer aspect, it is so natural to use measures like the total generalized cost, travel time, delay, utility of individuals or social welfare to compare and select the 'best' control strategy, that nearly all control systems since the classical work of Webster (1958) are designed with this philosophy. However, recent practices begin to question whether it is appropriate to design a control system relying solely on efficiency. In Table 1.1 the advantages and disadvantages of traffic control systems are tabulated (Arnold, 1998).

From the urban travelers point of view that are experiencing long delays on metered ramps efficiency concept started to become questionable. One of the most prominent examples of the controversial debate is Minnesota Twin Cities. With a legislature, the Minnesota DOT was mandated to conduct an eight-week ramp metering shut-off experiment to compare the system performances with and without metering (Levinson et al, 2002). This experiment confirmed that ramp metering would favor long-distance travelers at the expense of short-distance ones. Understandably, those controls are not considered "efficient" from the perspective of those short-distance travelers.

Table 1.1: Advantages and disadvantages of traffic control

	Measures	Trend
Advantages	Freeway mainline speed	Increases
	Accident rate & frequency	Decreases
	Freeway mainline occupancy	Decreases
	Overall travel time	Decreases
	Total delay time	Decreases
	Freeway mainline volume	Increases
	Benefit/cost (B/C) ratio	>1.0
Disadvantages	Ramp delays	Increase
	Induced overall travel demand	Increases
	Induced emissions	Increase
	Induced Fuel Consumption	Increase

Capital and maintenance costs for freeway traffic control can be substantial (Kang and Gillen, 1999). The amount depends on the type of control(s) being implemented. Enforcement of traffic control is also critical for a successful operation. Traditional enforcement methods are difficult to use in high-accident merge areas and can be quite expensive. Violation rates of 10% have been found to be acceptable, but higher numbers may significantly reduce the benefits of traffic control (Arnold, 1998).

The public support and acceptance are highly critical in a successful traffic control management. The benefits should be openly and clearly presented to the citizens since the driver behavior can be sensitive to the perceptions and beliefs that will directly affect the performance of the control (Levinson et al, 2002). Therefore, public relations campaign could be held before and during the implementation of freeway management system (Bertini et al, 2005).

In order to gain public acceptance and support, the control systems must not only consider the overall efficiency improvements, but also how the efficiency gains are distributed among the system's user groups who differ from one another in their departure time, origin-destination, trip purposes, and value of time. Some of those aspects are not easy to address however, this is a highly important issue that deserves to be considered in detail (Papageorgiou and Kotsialos, 2000).

Equity, or user fairness, begins to emerge as an important issue in traffic control design. Therefore, an ideal control strategy can be defined as; a system that is using advanced technologies and procedures that are more efficient and integrated to a context of freeway management strategies that seek to manage, operate, and maintain expressways in an efficient, cost-effective, and more equitable manner.

Compared with the overwhelming number of studies in addressing control efficiency, the analysis of equity issues in traffic control is far less, and their findings are mostly qualitative and sometimes even conflicting. For instance, efficiency and equity are usually considered as two competing requirements: the more efficient a control system is, the less equitable it becomes (Kotsialos & Papageorgiou 2004) (Meng & Yang 2002). However, some microscopic simulation studies (e.g. Yin et al, 2004) finds that the above claim may not be true because a control algorithm could be more equitable than another could, yet maintains a similar level of overall efficiency. Apparently, there is a need to resolve these contradictory findings, and develop proper equity measures to be used in the evaluation of control systems that measures the efficiency and the equity.

The entanglement around the two requirements originates from several threads. Firstly, several control strategies can be applied in developed countries. However, the sources are limited for most of the other countries in the world and there is a need to find a low-cost alternative to the sophisticated traffic control policies demanding high amount of investments. The control strategies should be inexpensive, easy to manage and durable.

Secondly, there is no consensus on the definition and measurement of equity in freeway traffic control. The equity measures applied in the literature are inherently deficient because they are borrowed mainly from social welfare studies in economics, where an appropriate measure is not generally agreed upon (Cowell, 2009). The equity of freeway traffic control should be defined and appropriate measures are also be considered for the evaluation.

1.3 Objectives

The necessity for development of a low-cost control strategies and new equity measurement has been briefly presented above and will be more fully elaborated in the remainder of this

dissertation. In response to the necessity discussed earlier, this research set four main objectives.
The first is to develop alternative control strategies to maximize the utilization of existing infrastructures (efficiency) which are also cost efficient.

The second is to define the equity in freeway traffic control context and develop an index for the evaluation of traffic equity.

Another objective is to evaluate the efficiency properties of the proposed control strategies and assessing the equity performances of them with the developed equity index within the traffic simulation environment.

Furthermore, developed traffic control strategies are compared with the existing traffic control strategies and the equity measurement method will be examined with all the strategies modeled in micro-simulation program.

1.4 Thesis Outline

This thesis is organized into seven chapters. The relation between the chapters is presented by the research flow illustrated in Figure 1.2.

Chapter 2 is a literature review of traffic flow modeling. In the beginning of the chapter, the fundamentals of traffic flow are briefly explained and traffic state measurements are defined. Later, the data collection methods in traffic engineering are presented and a literature review on traffic flow modeling is given emphasizing the model types and level of details of the models. Furthermore, traffic simulation programs are analyzed and briefly introduced. Finally, to give context to the proposed control strategies and new equity measurement approach, the conclusions are drawn and these discussions are referred to the next chapters.

Chapter 3 describes the freeway traffic control strategies namely ramp metering control, speed management and route information and guidance. A fixed-time ramp metering control algorithm and harmonized speed control are determined and the optimality of the cycle time and speed limits are analyzed through traffic simulation experiments. The fixed-time ramp metering considers bottleneck capacity, thus from this point the control system works like demand-capacity control. The offline demands are measured and the maximum admissible ramp volumes transformed into the green times for the control. The speed harmonization control applies the speed limits more particularly the speed ranges for the whole network. The optimum speed range is determined through a set of experiments in traffic micro-simulation program.

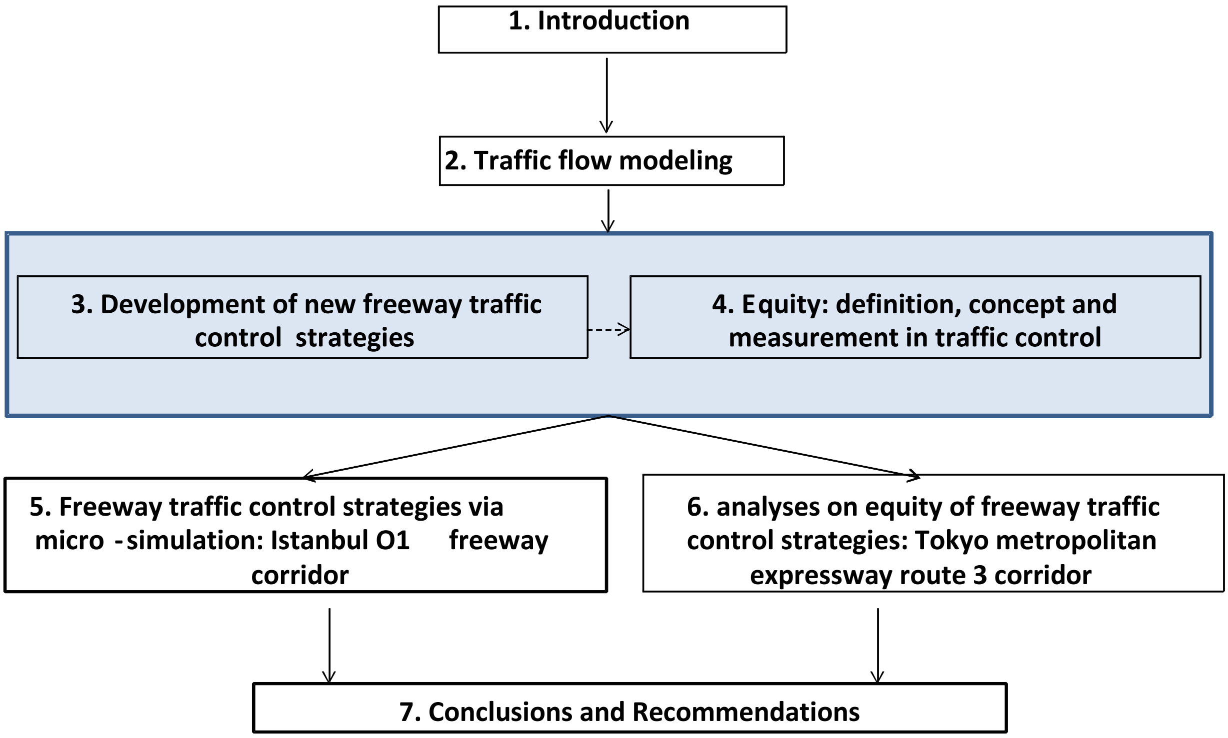

Figure 1.2: Organization of dissertation

Chapter 4 proceeds with the analysis of equity, equity in transportation studies and equity in traffic control. In the equity measurement part of the study, the existing equity measurement methodologies developed for social sciences, economy and traffic researched are analyzed. A new equity index, namely K, is proposed to assess the equity of traffic considering the mathematical and theoretical subjects. The most appropriate indicator for K index is also discussed in this chapter.

Chapter 5 describes the development of a traffic model for Istanbul O-1 corridor. The FTRM, HSC and IC are analyzed as traffic control strategies. The traffic simulation network is modeled and calibrated for the analyses. The optimality of fixed-time ramp metering and harmonized speed limit control is shown and the performance of equity and efficiency of traffic control simulation are discussed in this chapter.

In Chapter 6, two new extensions to the model predicted controls among the previously proposed control strategies are presented. The integration of standalone controls are examined though traffic simulation study. The traffic simulation model is constructed for Tokyo

Circular Expressway Route-3. The performance of equity and efficiency of traffic control strategies are computed and compared. The impacts of demand are also analyzed through the demand sensitivity analysis.

Finally, Chapter 7 presents overall conclusions, future direction and recommendations.

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CHAPTER 2. TRAFFIC FLOW MODELING

2.1 Introduction

This chapter provides an overview of traffic flow modeling that is of greater relevance to the propositions of this dissertation. Firstly in this chapter, the fundamentals of traffic flow are briefly explained and traffic state measurements are defined. In section 2.3 the data collection methods in traffic engineering are presented and in section 2.4 a literature review on traffic flow modeling is given emphasizing the model types and level of details of the models. In section 2.5 traffic simulation programs are analyzed and briefly introduced. Finally, to give context to the proposed control strategies and new equity measurement approach, in section 2.6 the conclusions are drawn and these discussions are referred to the next chapters of this dissertation.

2.2 Fundamentals of Traffic Flow

Tackling with the traffic congestion problem requires an extensive understanding of the properties of congestion phenomena as well as the traffic flow and road capacity. In recent decades with the help of technology advancement, there are varieties of sensing and communication technologies available to detect the traffic condition with a high precision (Klein et al, 2006). However, the analysis and the interpretation of the collected traffic data are not an easy job. Moreover, many traffic management and control strategies are based on traffic state estimations for the near or long-term future (Du et al, 2012). Thus, it is quite important to accurately estimate the traffic conditions in a road network during certain time periods and understand the evolution pattern of traffic conditions, i.e., the traffic dynamics. Therefore, there is a need for the development of traffic models.

A traffic model can be defined as a function that relates the movement of vehicles and/or drivers' behavior in general. There are several other parameters such as; vehicle type, vehicle compositions, network characteristics, link characteristics, weather conditions, traffic signals, guidance information, and interaction with other vehicles. The movement of a vehicle can be represented by its position at any time, i.e., its trajectory, from which its speed and acceleration rate can be obtained (Nagel and Schreckenberg, 1992). It would be possible to measure the performance of a link, corridor or a network e.g., travel time, level of service, congestion level, etc once the traveling vehicles trajectories are known. In figure 1.1, the relationship between the traffic system and traffic model is depicted (Jin, 2003).

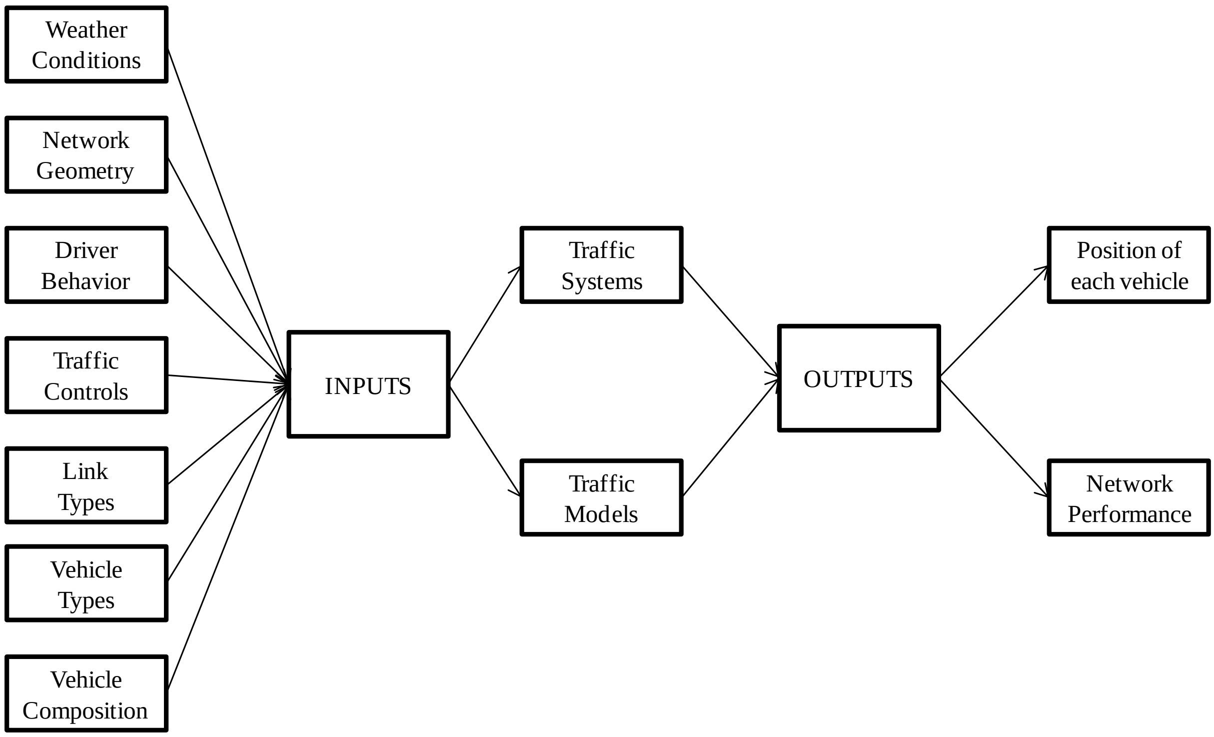

Figure 2.1 : Structural representation of traffic modeling

Traffic models are also subject to space and time limitations, i.e., initial traffic conditions and boundary conditions (Strub et al, 2006). Moreover, the calibration of the traffic model is critical in representation of the actual traffic system as realistically as possible. In validation, the output of a traffic model is compared with observation of real traffic (Toledo et al, 2003).

The role of traffic models can be used in two different ways in transportation engineering. Firstly, the models help researcher to observe the traffic dynamics, especially the occurrence of traffic congestion. Hence, the models can be used to identify the potential bottlenecks. Another way is that the models can be used as a simulation platform, where several strategies can be developed and evaluated. For example, an expansion of a road network plan can be evaluated through traffic simulation in a more cost-effective fashion. For another example, they are also helpful in determining the optimal ramp metering control strategy, which are designed to maximize the traffic efficiency and equity performance. Finally, traffic models can also be used to evaluate existing policies, networks and geometries.

2.3 Traffic state variables

For modeling the traffic, the parameters of the system, at least some of them, should be measurable. The state variables are essential to build a representative and a reliable model.

There are various variables that can be measured to describe the traffic state on the highway such as; traffic flow or number of vehicles, the speed of the vehicles, the traffic density, the occupancy level of the highway, the time headways and the distance headways.

One of the most common variables to measure is the traffic flow (veh/h). The traffic flow is measured as the number of vehicles that pass a fixed point on the motorway during a certain time period.

The mean speed (km/h) of the vehicles is another important parameter to assess the traffic situation on the motorway. In practice, a value for the mean speed is obtained by averaging the speeds of the vehicles passing the sensor over a certain time period. The mean speed can be measured for each lane separately or it can be measured for all the lanes together.

The traffic density (veh/km/lane) is the number of vehicles on the motorway per unit of length at a certain time instant. The traffic density can vary from lane to lane. The total traffic density on the motorway is the sum of the lane traffic densities. Note that if the traffic is homogeneously distributed over the lanes of the motorway the motorway traffic density is given by the lane traffic density times the number of lanes.

The occupancy of a lane or highway is measured as the fraction of the time that the vehicle occupies the detector. In some studies it is represented as a percentage of time.

The time headway (h) between two vehicles is the time that passes between the arrival of the front of a vehicle and the arrival of the front of the next vehicle at a reference point.

The distance headway (km) is the distance between the front of a first vehicle and the front of the second vehicle. The distance headway consists of the length of the first vehicle plus the length of the gap between both vehicles.

Depending on the setup and the features of the measurement system, several levels of detail can be distinguished. Average values or instantaneous values for each vehicle can be considered. The averages can be averages over time (the time mean speed is the average speed at a certain location averaged over a time period) or the averages can be averages over space (the space mean speed is the average speed in a motorway stretch at a certain time).

Depending on the application, the period over which the measurements are averaged can range from seconds to seconds, minutes, and hours and even to longer periods. The traffic parameters can be measured for every motorway lane separately or a value for the motorway as a whole can be obtained. Furthermore, sometimes vehicles are classified in several vehicle types or classes such as motorcycles, automobiles, medium goods vehicle, heavy goods vehicle, bus, truck.

For every vehicle class certain measurements can be collected to make a deeper analysis about the traffic flow. As a rule of thumb, the higher the level of detail required and the more parameters need to be measured, the higher the cost of the measurement system involved.

The most commonly measured traffic parameters are the traffic flow, the speed and the occupancy of the motorway traffic. The choice of these parameters is influenced by their importance in traffic theory as well as by the ease they can be measured with the most common traffic detector technologies.

In the next section, the most widely used traffic measurement systems or sensors are presented as well as the problem of collecting all the distributed measurements in a central database.

Many macroscopic traffic flow models use the traffic density on the motorway as a state variable. The traffic density is often not available as a traffic measurement since, by its definition, it needs to be measured along a distance and most popular sensors provide point measurements. However, there is a relation between the traffic density and the occupancy:

$$\rho = \frac{o}{l} = \frac{o}{\lambda + \lambda_0}. \tag{2.1}$$

Here o is the occupancy, l is the effective car length, λ is the weighted car length and λ_0 is the sensitivity range of the detectors.

2.4 Data collection methods

There exist a wide variety of technologies to measure the traffic parameters such as pneumatic detection systems, inductive loops, cameras, ultrasonic sensors, microwave sensors, active and passive infrared sensors, passive acoustic arrays, magnetometers.

Inductive loops were introduced as a traffic detection system around 1960 and have become the most widespread traffic detection system to date. An inductive loop consists of a conducting loop that is put in or on the surface of the motorway (see Figure 2.2). This loop is connected to an electronic circuit that produces an oscillating current through the loop. If a vehicle enters the loop, the metal parts of the vehicle cause the oscillation frequency of the electronic circuit to shift. This frequency shift is registered and the vehicle is detected.

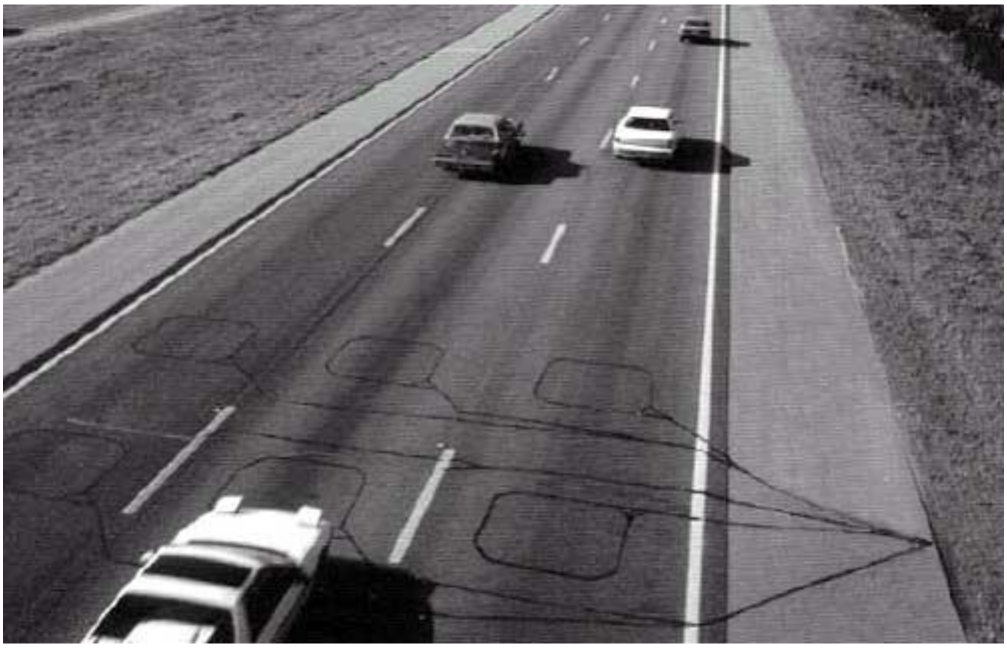

Figure 2.2: Double loop detector (Klein, 2003).

There are two main configurations of inductive loop systems on motorways: the single-loop and the double-loop configuration. In the single-loop configuration, the traffic flow and the occupancy level of the motorway can be measured.

Depending on the implementation, these measurements are available per lane or for all the lanes together. By defining an average length of a vehicle, an estimation of the speed can be computed based on the time a vehicle spends in the loop. However, using a single inductive loop setup and an average vehicle length, it is hard to distinguish a fast driving lorry from a slow driving car, since the time the vehicle is in sight of the detector (in the loop) can be the same. In order to solve this problem and in order to improve the accuracy of the measurements, inductive loops are also implemented in double-loop configurations. Once the speed and the occupancy time of a vehicle are known, the length of the vehicle can be computed allowing for a classification. The main advantages of the double inductive loop setup over the single-loop configuration are the improved accuracy and the additional traffic parameters that can be recorded. The main disadvantage is the higher cost with respect to the single-loop configuration. The main disadvantages of the inductive loop technology are the sensitivity of the loops during road maintenance works and the special installation and maintenance requirements resulting from the presence of the loops on the road surface.

The presence of the inductive loops in the road surface causes very specific problems regarding maintenance. Traffic detection using cameras emerged around 1980 and is a non-intrusive technology that is becoming more and more popular. A fixed camera is mounted above the motorway and its images are sent to a video processing unit that extracts the desired parameters using image recognition algorithms. A video snapshot image is given in Figure 2.3. If the video images are sent to the traffic center they can also serve as a visual inspection tool. The main advantages of the camera traffic detection technology are the higher average accuracy of the measurements and the visual inspection possibilities. However, the main disadvantages are the high cost of the installation and the reduced accuracy during weather conditions with low visibility.

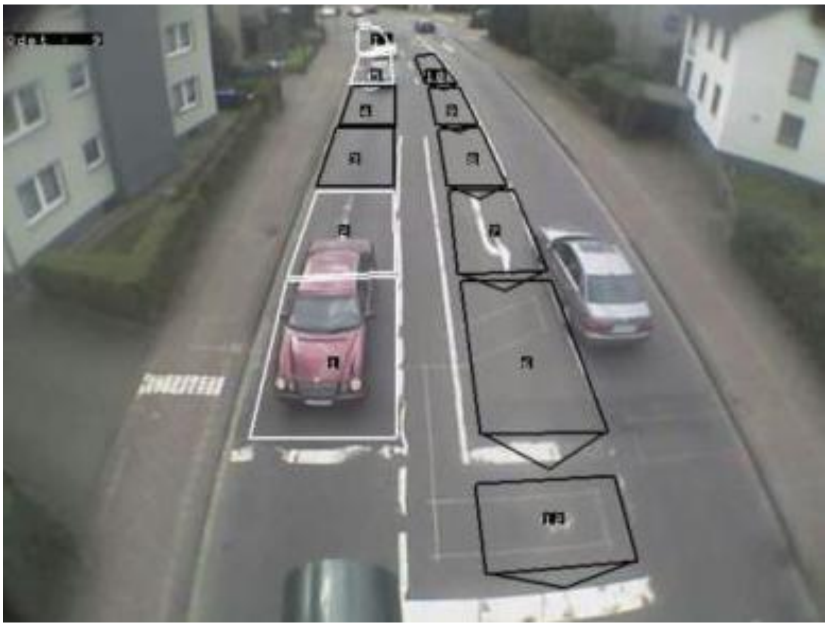

Figure 2.3: Video image processing (Aldridge traffic controllers, 2013)

Pneumatic traffic detection systems consist of a tube that is led over the road as shown in Figure 2.4. A vehicle driving over the tube produces pneumatic shock waves3 in the tube which are detected and processed.

Based on the detected shock waves in the tube and the delays between consecutive shocks, an estimation of the traffic flow, the speed and even a classification of the vehicles can be made.

The main advantage of this technology is the mobility of the installation due to its ease of installation.

Figure 2.4: Pneumatic traffic detection systems (TCS for Surveys, 2013)

It allows for a quick set up of a short term measuring point along a motorway. Since the tubes are led on the road surface they are susceptible for wear and tear caused by the traffic, hence the pneumatic sensors are not suitable for long term operation. A disadvantage of pneumatic tube detectors is their limited accuracy.

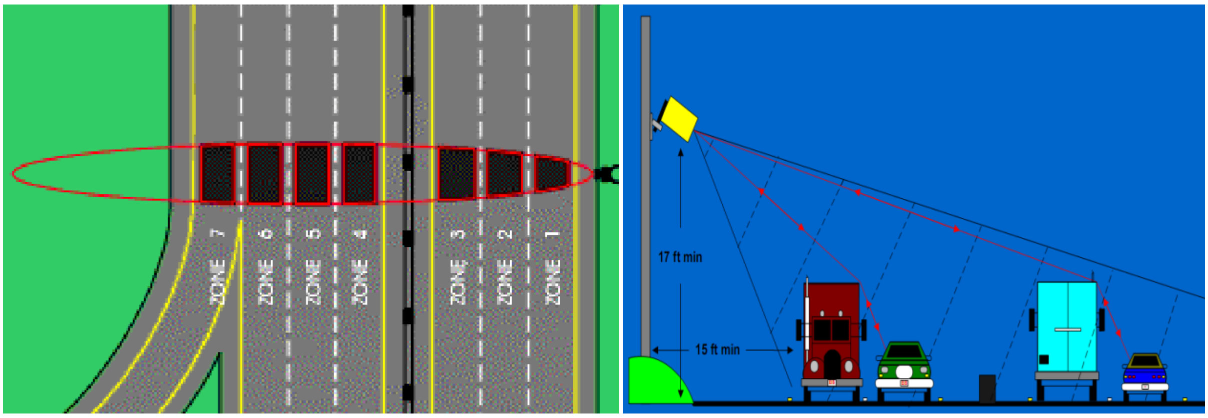

Figure 2.5: RTMS data collection (RTMS user manual, 2012)

RTMS (Remote Traffic Microwave Sensor) is a sensor used to collect the data from the section of the highway on which RTMS is set up. The RTMS is small roadside pole-mounted radar, operates in the microwave band. The sensor simultaneously provides volume, occupancy, speed and classification information from the detection zones to the central controller. Output information is provided to existing controllers via contact closure and to other computing systems by serial port or by other wireless options.

2.5 Traffic Flow Modeling

Due to the complexity of the traffic flow system, analytical approaches may not provide the desired results. Therefore, traffic flow simulation models designed to characterize the behavior of the complex traffic flow system have become an essential tool in traffic flow analysis and experimentation. To understand the relationship between the traffic parameters, a great deal of research has been done over the past several decades. The results of these researches yielded many mathematical models.

Papageorgiou (1998) convincingly argues that it is unlikely that traffic flow theory will reach the descriptive accuracy attained in other domains of science (e.g. Newtonian physics or thermodynamics). The only accurate physical law in traffic flow theory is the *conservation of vehicles* equation; all other model structures reflect either counter-intuitive idealizations or coarse approximations of empirical observations. Consequently, the challenge of traffic flow researchers is to look for useful theories of traffic flow that have sufficient descriptive power, where sufficiency depends on the application purpose of their theories.

2.5.1 Types of models

General mathematical models aimed at describing this behavior using mathematical equations include the following approaches (Papageorgiou (1998) as; purely deductive approaches whereby known accurate physical laws are applied, purely inductive approaches where available input/output data from real systems are used to fit generic mathematical structures (ARIMA models, polynomial approximations, neural networks) and intermediate approaches, whereby first basic mathematical model-structures are developed first, after which a specific structure is fitted using real data.

Traffic flow models may be categorized according to various criteria (level of detail, functionality, representation of the processes). We discuss several classes of traffic flow models, the order of which is based on the level-of-detail classification i.e. microscopic, mesoscopic, and macroscopic modeling approaches.

2.5.2 Level of detail

There are broad scope of models developed for different aspects of traffic flow operations, either by considering the time-space behavior of individual drivers under the influence of vehicles in their proximity (microscopic models), the behavior of drivers without explicitly distinguishing their time-space behavior (mesoscopic models), or from the viewpoint of the collective vehicular flow (macroscopic models). In addition to the controversy between these microscopic, mesoscopic, and macroscopic modeling streams, several researchers have joined the debate on the macroscopic modeling approach most suitable for a correct description of traffic flow.

2.5.2.1 Macroscopic traffic models

At the macroscopic level, the vehicles are not considered as separate entities. The traditional traffic flow model is highly relevant to the dynamic description of traffic. Initially, the macroscopic variables that translate the discrete nature of traffic into continuous variables are defined.

Macroscopic stream models represent how the behavior of one parameter of traffic flow changes with respect to another. Most important among them is the relation between speed and density. The first and most simple relation between them is proposed by Greenshield (1935). Greenshield assumed a linear speed-density relationship as illustrated in figure 2.6 to derive the model. The equation for this relationship is shown below.

$$v = v_f - \left[\frac{v_f}{k_j}\right].k \qquad (2.2)$$

where v is the mean speed at density k, v_f is the free flow speed and kj is the jam density. This equation 2.1 is often referred to as the Greenshields' model. It indicates that when density becomes zero, speed approaches free speed (ie. $v \rightarrow v_f$ when $k \rightarrow 0$).

Once the relation between speed and flow is established, the relation between flow and density can be derived. This relation between flow and density is parabolic in shape and is shown in figure 2.6 Also, we know that

$$q = k.v \qquad (2.3)$$

Now substituting equation 2.1 in equation 2.2, we get

$$q = v_f . k - \left[\frac{v_f}{k_j}\right] . k^2 \qquad (2.4)$$

Similarly we can find the relation between speed and flow. For this, put k = q/v in equation 2.1 and solving, we get

$$q = k_j . v - \left[\frac{k_j}{v_f}\right] . v^2 \qquad (2.5)$$

This relationship is again parabolic and is shown in figure 2.6.

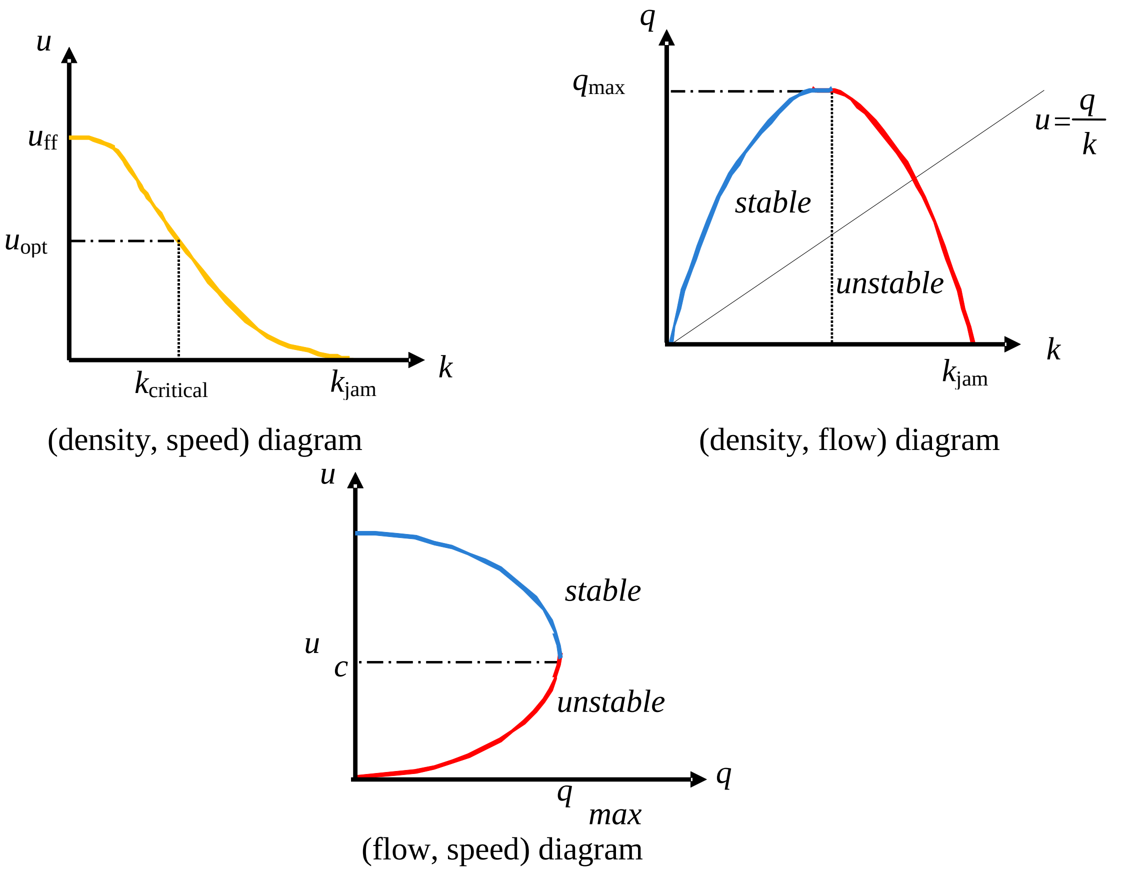

Figure 2.6: Fundamental relationships

Once the relationship between the fundamental variables of traffic flow is established, the boundary conditions can be derived. The boundary conditions that are of interest are jam density, free-flow speed, and maximum flow. To find density at maximum flow, differentiate equation 2.3 with respect to k and equate it to zero. ie.,

$$\frac{dq}{dk} = 0$$
$$v_f - \frac{v_f}{k_j}.2k = 0 \tag{2.6}$$
$$k = \frac{k_j}{2}$$

Denoting the density corresponding to maximum flow as k_0,

$$k_0 = \frac{k_j}{2} \tag{2.7}$$

Therefore, density corresponding to maximum flow is half the jam density. Once we get k_0, we can derive for maximum flow, q_{max}. Substituting equation 2.5 in equation 2.3,

$$\begin{aligned} q_{max} = & \ v_f.\frac{k_j}{2} - \frac{v_f}{k_j}.\left[\frac{k_j}{2}\right]^2 \\ = & \ v_f.\frac{k_j}{2} - v_f\frac{k_j}{4} \\ = & \ \frac{v_f.k_j}{4} \end{aligned} \tag{2.8}$$

Thus, the maximum flow is one-fourth the product of free flow and jam density. Finally, to get the speed at maximum flow, v_0, substitute equation 2.5 in equation 2.1 and solving we get,

$$\begin{aligned} v_o = & \ v_f - \frac{v_f}{k_j}\frac{k_j}{2} \\ v_o = & \ \frac{v_f}{2} \end{aligned} \tag{2.9}$$

Therefore, speed at maximum flow is half of the free speed.

When density decreases below critical density (ρ_c), the traffic flow maintains a free and stable status. As density exceeds the critical density, the total flow would decrease therefore traffic would enter the congestion zone or to an unstable state.

In order to use this model for any traffic stream, one should get the boundary values, especially free flow speed (v_f) and jam density (k_j). This has to be obtained by field survey and this is called calibration process. Although it is difficult to determine exact free flow speed and jam density directly from the field, approximate values can be obtained from a number of speed and density observations and then fitting a linear equation between them. Let the linear equation be{y = a+bx} such that y is density k and x denotes the speed v. Using linear regression method, coefficients a and b can be solved as,

$$b = \frac{n\sum_{i=1}^{n} xy - \sum_{i=1}^{n} x \,.\sum_{i=1}^{n} y}{n\sum_{i=1}^{n} x^2 - \sum_{i=1}^{n} y^2} \qquad (2.10)$$

$$v_o = \bar{y} - b\bar{x} \qquad (2.11)$$

Alternate method of solving for b is,

$$b = \frac{\sum_{i=1}^{n}(x_i - \bar{x})\,(y_i - \bar{y})}{\sum_{i=1}^{n}(x_i - \bar{x})^2} \qquad (2.12)$$

where x_i and y_i are the samples, n is the number of samples, and $\bar{x}$ and $\bar{y}$ are the mean of x_i and y_i respectively.

Table 2.1: Single-Regime Speed-Density Relationships (Wang, 2010)

Model	**Relationship**
Greenshields Model (1935)	$u = u_f\left(1 - \frac{\rho}{\rho_i}\right)$
Greenberg Model (1959)	$u = u_0 \ln\left(\frac{\rho_j}{\rho}\right)$
Underwood Model (1961)	$u = u_f \mathrm{e}^{\left(-\frac{\rho}{\rho_j}\right)}$
Newell Model (1965)	$u = u_f\left(1 - \mathrm{e}^{\left(-\frac{\lambda}{v_f}\left(\frac{1}{\rho} - \frac{1}{\rho_j}\right)\right)}\right)$
Northwestern Model (1967)	$u = u_f \mathrm{e}^{\left(-\frac{1}{2}\left(\frac{\rho}{\rho_j}\right)^2\right)}$
Pipes-Munjal Model (1967)	$u = u_f\left(1 - \left(\frac{\rho}{\rho_j}\right)^n\right)$
Modified Greenshields Model (1994)	$u = u_0 + \left(u_f - u_0\right)\left(1 - \left(\frac{\rho}{\rho_j}\right)^\alpha\right)$

Kerner-Kornhäuser Model (1994)	$u = u_f \left(\frac{1}{\left(\cdots \left(\frac{\rho}{\rho} - 0.25 \right) \right)} - 3.72x10^{-6} \right)$
Del Castillo Model (1995)	$u = u_f \left(1 - e^{\frac{\lvert C_j \rvert}{u_f}\left(1 - \frac{\rho_j}{\rho}\right)} \right)$
Van Ärde Model (1995)	$\rho = \frac{1}{c_1 + \frac{c_2}{u_f - u} + c_3 u}$
MacNicholas Model (2008)	$u = u_f \left(\frac{\rho_j^n - \rho^n}{\rho_j^n + m\rho^n} \right)$

The main critics of the existing speed-density models are their limited capability to satisfy both criteria at the same time: mathematical elegance and empirical accuracy. For example, the path-breaking Greenshields model presented above has an attractive mathematical simplicity but it is insufficient to match some empirical observations. In order to overcome this many researchers proposed their mathematical forms to represent the macroscopic relationships of traffic flow. A summary of single regime models are given in Table 2. 1.

2.5.2.2 Mesoscopic traffic models

A mesoscopic model does not distinguish nor trace individual vehicles, but specifies the behavior of individuals, for instance in probabilistic terms. To this end, traffic is represented by (small) groups of traffic entities, the activities and interactions of which are described at a low detail level. For instance, a lane-change maneuver might be represented for an individual vehicle as an instantaneous event, where the decision to perform a lane-change is based on e.g. relative lane densities, and speed differentials. Some mesoscopic models are derived in analogy to gas-kinetic theory. These so-called gas-kinetic models describe the dynamics of velocity distributions.

Mesoscopic flow models describe traffic flow at a medium detail level. Vehicles and driver behavior are not distinguished nor described individually, but rather in more aggregate-terms (e.g. using probability distribution functions). However, the behavior rules are described at an individual level. For instance, a gas-kinetic model describes velocity distributions at specific locations and time instants.

The dynamics of these distributions are generally governed by various processes (e.g. acceleration, interaction between vehicles, lane-changing), describing the individual driver's behavior. Three well known examples of mesoscopic flow models are the so-called headway distribution models, cluster models, and the gas-kinetic continuum models.

A time-headway is defined by the difference in passage times of two successive vehicles. In general, it is assumed that these time-headways are identically distributed independent random variables. Headway distribution models are mesoscopic in the sense that they describe the distribution of the headways of the individual vehicles, while neither explicitly considering nor tracing each vehicle separately. Examples of headway distribution models are Buckley's semi-Poisson model (Buckley, 1968), and the Generalized Queuing Model (Branston, 1976). Mixed headway distribution models distinguish between leading and following vehicles: time headways of leading drivers and following drivers are taken from different probability distributions.

Headway distribution models have been criticized for neglecting the role of traffic dynamics. More-over, these models assume that all vehicles are essentially the same. That is, the probability distribution functions are independent of traveler type, vehicle type, travel purpose, level of drivers' guidance, etc. To remedy this, Hoogendoorn and Bovy (1998) developed a headway distribution model for respectively multiclass traffic flow and multiclass multilane traffic flow.

Cluster models are characterized by the central role of clusters of vehicles. A cluster is a group of vehicles that share a specific property. Different aspects of clusters can be considered. Usually, the size of a cluster (the number of vehicles in a cluster) and the velocity of a cluster are of dominant importance. Generally, the size of a cluster is dynamic: clusters can grow and decay. The within cluster traffic conditions, e.g. the headways, velocity differences, etc., are usually not considered explicitly: clusters are homogeneous in this sense. Usually, clusters emerge because of restricted overtaking possibilities due to e.g. overtaking prohibitions, or interactions with other vehicles making overtaking impossible, or due to prevailing weather or ambient conditions (Botma, 1978).

Instead of describing the traffic dynamics of individual vehicles, gas-kinetic traffic flow models describe the dynamics of the velocity distribution functions of vehicles in the traffic flow. In this section, we first recall the seminal models of Prigogine and Herman (Prigogine, 1961), Prigogine and Herman, 1971), after which a few extensions to this model type are dealt with.

Hoogendoorn and Bovy (2000) also developed gas-kinetic multiclass traffic flow models that vehicles of relatively fast user-classes catch up with vehicles of slow user-classes more frequently than vice-versa.

2.5.2.3 Microscopic traffic models

A microscopic simulation model describes both the space-time behavior of the systems' entities i.e. vehicles and drivers, as well as their interactions at a high level of detail (individually). For instance, for each vehicle in the stream a lane-change is described as a detailed chain of drivers' decisions.

The development of microscopic traffic flow models started at1960s with the famous car-following models. These models are based on supposed mechanisms describing the process of one vehicle following another. There are three types of car-following models, namely safe-distance models, stimulus-response models, and psycho-spacing models.

Safe-distance car-following models describe the dynamics of a single vehicle in relation to its predecessor. In this respect, a very simple model is Pipes' rule Pipes, 1953; "A good rule for following another vehicle at a safe distance is to allow yourself at least the length of a car between you and the vehicle ahead for every ten miles an hour (16.1km/hr) of speed at which you are travelling".

A similar approach was proposed by Forbes et al. (1958). Both Pipes' theory and Forbes' theory were compared to field measurements. It was concluded that according to Pipes' theory, the minimum headways are slightly less at low and high velocities than observed in empirical data. Fellendorf, (1994) discusses a more refined model describing the spacing of constrained vehicles in the traffic flow. The braking distance is defined by the distance needed by a vehicle to come to a full stop, incorporating the reaction time of the driver, and the maximal deceleration.

In psycho-spacing models, the model assumes that the response equals zero as the velocity differences disappear, even if the distance between the consecutive vehicles is very small or large. The basic behavioral rules of such so-called psycho-spacing models can be described as; for large spacing, the following driver is not influenced by velocity differences and for small spacing, some combinations of relative velocities and distance headways do not yield a response of the following driver, because the relative motion is too small.

Wiedemann (1974) developed the first psycho-spacing model. He distinguished constrained and unconstrained driving by considering perception thresholds. Moreover, lane-changing and overtaking are incorporated in his modeling approach. Psycho-spacing models are the foundation of a number of contemporary microscopic simulation models.

Microscopic simulation models are better than the other level of models in the correct description of the capacity drop and thus of wide jams. Simple car following models do not have this feature, and are therefore not suited for correct description of congested traffic flow.

2.6 Traffic simulation programs

Simulation is defined by Shannon (1975) as the process of designing a model of a real system and conducting experiments with this model for the purpose either of understanding the behavior of the system or of evaluating various strategies (within the limits imposed by a criterion or set of criteria) for the operation of the system.

The Highway Capacity Manual (HCM, 2010) describes a simulation model as "A computer program that uses mathematical models to conduct experiments with traffic events on a transportation facility or system over extended periods of time."

In general, simulation is defined as dynamic representation of some part of the real world achieved by building a computer model and moving it through time (Drew 1968). The use of computer simulation started when D.L. Gerlough published his dissertation: "Simulation of freeway traffic on a general-purpose discrete variable computer" at the University of California, Los Angeles, in 1955 (Kallberg 1971). Since then, computer simulation has become a widely used tool in transportation engineering with a variety of applications from scientific research to planning, training and demonstration.

The five driving forces behind this development are the advances in traffic theory, in computer hardware technology and in programming tools, the development of the general information infrastructure, and the society's demand for more detailed analysis of the consequences of traffic measures and plans.

There are vast amount of parameters representing the traffic system those also make it difficult to analyze, control and optimize the characteristics of traffic. The systems often cover wide physical areas, the number of active participants is high, the goals and objectives of the participants are not necessarily parallel with each other or with those of the system operator (system optimum vs. user optimum), and there are many system inputs that are outside the control of the operator and the participants (the weather conditions, the number of users, etc.). In addition, traffic systems are implicitly having a dynamic nature that the system varies according to the time, and with a considerable amount of randomness. The great number of active participants at present at the same time in the system means a great number of simultaneous interactions whereas this yields a tremendous complexity.

Transportation systems are typical man-machine systems, that is, the activities in the system include both human interaction (interaction between driver-vehicle-elements) and man-machine-interactions (driver interaction with the vehicle, with the traffic information and control system and with the physical road and street environment). In addition, the laws of interaction are approximate in nature; the observations and reactions of drivers are governed by human perception and not by technology based sensor and monitoring systems.

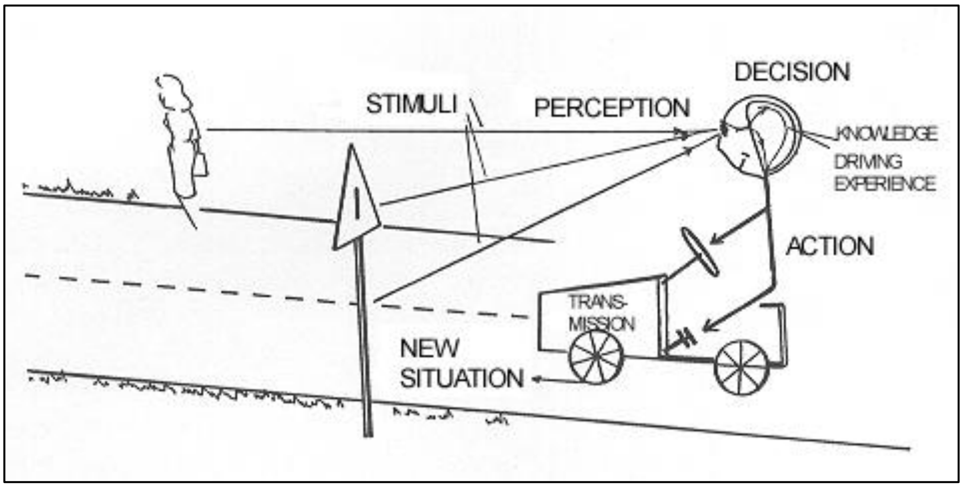

Figure 2.7: Basic Driver Perception-action Process (Häkkinen and Luoma 1991).

In all, traffic systems are an excellent application environment for simulation based research and planning techniques, an application area where the use of analytical tools, though very important, is limited to subsystem and sub problem level.

The traditional simulation problem with practical orientation in road and street traffic analysis is related to questions of traffic flow, that is, to capacity and operational characteristics of facilities. Delays and queue lengths at intersections are perpetual issues in analysis and simulation studies with a newly grown international interest in roundabouts.

In the area of traffic signal control, the classic Webster's formula (Webster and Cobbe 1966) is an example of early use of simulation with practical results. In this formula a simulation-based correction is added to an analytical delay formula derived by the use of queuing theory. Modern vehicle-actuated traffic signal controllers have added a new dimension to signal control simulation. In traditional fixed time signal control only the traffic was reacting to signals, now the signals are also reacting to traffic, and the analysis of controller reactions is as quite important as the analysis of traffic itself.

Most urban transportation problems are network related. In networks, one has to combine different kinds of intersections (signalized, unsignalized) and links (arterial roads, motorways, city streets). This makes the simulation quite complicated and the number of comprehensive simulation tools for network analysis is quite small in comparison to that of programs for isolated intersections and road sections. The most widely known package in this area is probably the American NETSIM from the 1970's (Byrne et al. 1982). Later examples of tools in this area are e.g. INTEGRATION and AIMSUN2 (Algers et al. 1997).

In link traffic flow analysis motorway simulation seems to be more common than simulation of ordinary two-lane two-way traffic roads. One of the reasons here is that in two-lane road environment the interactions between vehicles travelling in opposite directions have to be modeled. The platooning and overtaking are not only dependent on traffic situation but also on the road environment (sight distances, passing control). This way the problem is much more complicated than in the motorway environment. Probably the most well-known programs in this area are the Swedish VTI-model (Algers et al. 1996) and the Australian TRARR (Hoban et al. 1991), both basically developed in the 1970's.

Most traffic system simulation applications today are based on the simulation of vehicle-vehicle interactions and are microscopic in nature. Traffic flow analysis is one of the few areas, where macroscopic (or continuous flow) simulation has also been in use. Most of the well-known macroscopic applications in this area originate from the late 1960's or the early 1970's. The British TRANSYT-program (Byrne et al. 1982) is an example of macroscopic simulation of urban arterial signal control coordination and the American FREQ- and FREFLO-programs (Byrne et al. 1982; Payne 1971) plus the corresponding German analysis tool (Cremer 1979) are related to motorway applications. A mesoscopic approach with groups of vehicles is used in CONTRAM (Leonard et al. 1978), a tool for analysis of street networks with signalized and non-signalized intersections.

Traffic safety related questions have been quite a hard problem for simulation. In traditional simulation programs the drivers are programmed to avoid collisions. Thus, they do not exist. Some trials for analysis of conflict situations through simulation can be found (Karhu 1975;

Sayed 1997), but a general approach to the problem and widely used safety simulation tools are still missing. Traffic safety simulation belongs to the field of human centered simulation where the perception-reaction system of drivers with all its weak points has to be described. This kind of approach is sometimes called nanosimulation or submicroscopic simulation in order to separate it from the traditional microscopic simulation.

On the other hand, safety aspects and human reactions in different traffic situations have for long been analyzed using driving simulation systems, where the test subjects are exposed to artificial driving tasks in a simulated vehicle and traffic environment and the driver has to react to the given traffic (Moisio 1973). Here the developments in virtual reality technology will increase the possibilities for realistic simulations

A new application area is the simulation of the use and effects of telematic services in traffic. This is on the other hand related to the simulation of traffic flow, and on the other hand to the simulation of human behavior and decision-making (Algers et al. 1997). Even the effects of totally human-free driving are tested in this area.

In recent years another new area of traffic simulation has emerged, namely simulation of travel demand. This is an area, where the analytical tradition has gone from aggregate gravity modeling to individual based disaggregate choice models. In demand simulation, the question is to reproduce the trip pattern (the number, time of day, purpose, origin-destination pattern, modal split and use of routes) of the citizen population within an area by summing up the behavior of the individuals. Examples of this approach are the American SAMS and SMART, both still under development (Spear 1996). One of the most advanced modeling approaches, the American TRANSIMS, combines demand modeling and flow behavior on the streets and roads thus trying to describe the whole traffic system behavior in one simulation environment (Smith et al. 1995).

The development in traffic simulation from the early days in the 1950's and 1960's has been tremendous. This, of course, is partly related to the development of computer technology and programming tools. On the other hand, the research in traffic and transportation engineering has also advanced during this 40-year period. Simulation is now an everyday tool for practitioners and researchers in all fields of the profession.

There are some quite interesting new developments in the theoretical macroscopic models for fundamental traffic flow analysis, which give new insight to the fundamental speed-flow-density relationships (Helbing et al. 1997).

The simulated networks are enlarging in size and with respect to the increase in coverage of analyses different type of local and network wide systems are needed to be integrated into one system. This trend increases the need of computational power which allows more precise description of the physical road and street geometry, especially in local applications where it is critically important in simulation of intersections. In both these cases the use of graphic user interfaces and integration to GIS and CAD systems (Etches et al. 1998) are a feasible approach.

The American TRANSIMS development work is an example of a network approach. The simulation of the traffic system of a whole city is based on massive use of parallel computing (Nagel and Schleicher 1994), which again is a feature that is coming more common in modern applications (Argile et al. 1996). Parallel computing can be achieved for example through simultaneous use of several microcomputers communicating through a local network and the use of super computers are gaining popularity among academic institutes (Argile et al. 1996).

In addition to the parallel computing, the modern programming principles and methods have their effect on the simulation. Object-oriented programming has been found very suitable in the description of the great amount of practically parallel interactions in traffic. Objects, or agents, can be programmed to interact in a very natural way to produce accurate models of traffic flow behavior (Kosonen 1996).

TRANSIMS is an example of still another change in the approach. The traditional traffic flow descriptions are based on continuous speed and distance variables. TRANSIMS, in turn, uses a discrete approach where the road and street network is build from elements that can accommodate only one vehicle at a time unit.

In this cellular automata approach the vehicles move by "jumping" from the present element to a new one (Nagel 1996; Brilon and Wu 1998) according to rules that describe the driver behavior and maintain the basic laws of physics at present in vehicle movements (Figure 2.8).

Another way of looking at the need for system level simulations is to develop open environments where several analysis tools can be used interactively to solve the problems each one of them is most suitable. An example of this is the FHWA TRAF-program family and the FHWA Traffic Management Laboratory, whose primary goal is the development of a distributed, real-time testbed to simulate traffic conditions for Advanced Traffic Management Systems (FHWA 1994).

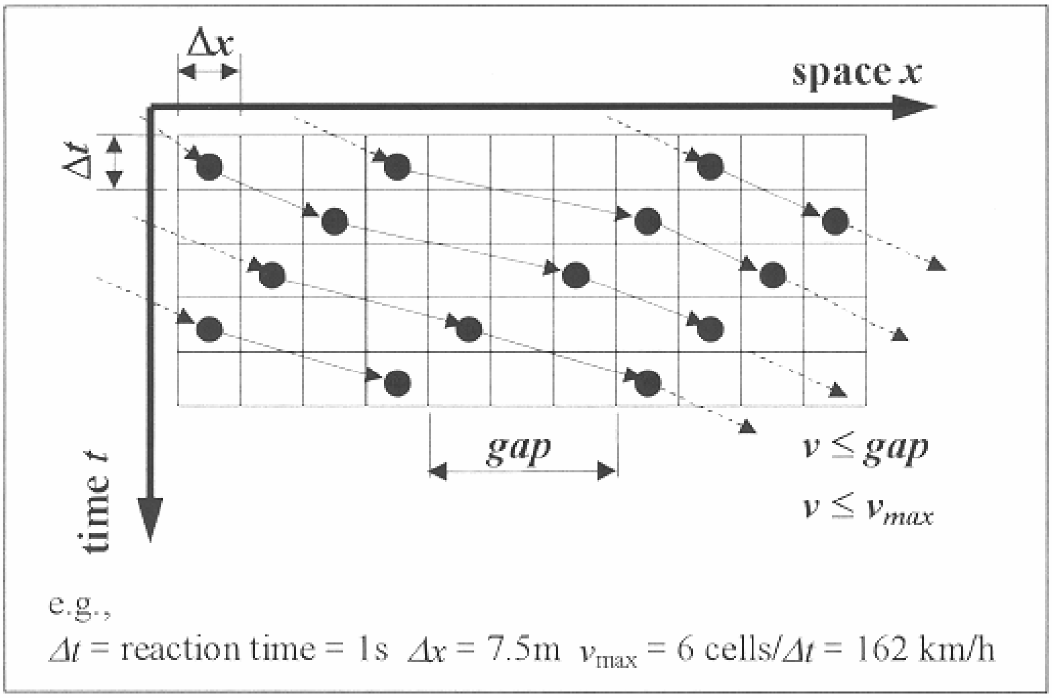

Figure 2.8: Principle of a Cellular Automaton (Brilon and Wu 1998).

In traffic flow simulation rule based approaches, like in HUTSIM and TRANSIMS, are coming more and more common. In this kind of framework the use of fuzzy logic to describe the human perception can easily be used, and there are several applications of fuzzy car-following models available (Kikuchi and Chakroborty 1992; Rekersbrink 1995; Wu et al.1998).

Simulation of control systems as a part of traffic operations is also coming more and more important with the wide ongoing research in transport telematics. The new control systems interact with traffic, and thus both the control system reactions and the driver reactions must be described in a true way. An especially important feature in driver reactions is the route choice decision that must be treated dynamically. In the future more and more simulation systems are embedded in control systems for the anticipation of the state of traffic flow and the effects of alternative control measures.

There are several traffic simulation programs based on various modeling premises available for different purposes and levels of analyses. Still it would be presumptuous to favor one simulation model over another one. A list of programs found in the literature is given in Table 2.2.

Table 2.2: Traffic simulation programs

PROGRAM	Developer	Country
AIMSUN 2	Universitat Politècnica de Catalunya	Spain
ANATOLL	ISIS and Centre d'Etudes Techniques de l'Equipement	France
ARTEMiS	University of New South Wales, School of Civil Engineering	Australia
ARTIST	Bosch	Germany
CASIMIR	Institut National de Recherche sur les Transports et la Sécurité	France
CORSIM	Federal Highway Administration	USA
DRACULA	Institute for Transport Studies, University of Leeds	UK
FLEXSYT II	Ministry of Transport	Netherlands
FREEVU	University of Waterloo, Department of Civil Engineering	Canada
FRESIM	Federal Highway Administration	USA
HUTSIM	Helsinki University of Technology	Finland
INTEGRATION	Queen's University, Transportation Research Group	Canada
MELROSE	Mitsubishi Electric Corporation	Japan
MICROSIM	Centre of parallel computing (ZPR), University of Cologne	Germany
MICSTRAN	National Research Institute of Police Science	Japan
MITSIMLab	Massachusetts Institute of Technology	USA
NEMIS	Mizar Automazione, Turin	Italy
PADSIM	Nottingham Trent University – NTU	UK
PARAMICS	The Edinburgh Parallel Computing Centre and Quadstone	UK
PHAROS	Institute for simulation and training	USA
PLANSIM-T	Centre of parallel computing (ZPR), University of Cologne	Germany
SIGSIM	University of Newcastle	UK
SIMDAC	ONERA – Centre d'Etudes et de Recherche de Toulouse	France
SIMNET	Technical University Berlin	Germany
SISTM	Transport Research Laboratory, Crowthorne	UK
SITRA-B+	ONERA – Centre d'Etudes et de Recherche de Toulouse	France
TRANSIMS	Los Alamos National Laboratory	USA
THOREAU	The MITRE Corporation	USA
VISSIM	PTV System Software and Consulting GMBH	Germany

Virtual reality systems and programming tools become in common use, especially in simulations where the driver reactions and behavior must be analyzed in great detail. Traffic safety related simulation would therefore probably be an area that greatly benefits from VR technology. There is, of course, no reason why VR tools could not be used in more traditional simulation tasks, as well. In planning applications VR gives new possibilities for the planning work, and for the demonstration of plans to decision-makers and public (Brummer et al. 1998).

The combination of traditional driving simulators and traditional traffic flow simulation systems becomes possible through virtual reality techniques. In traditional driving simulator the test driver has to react to the fixed traffic that he/she sees on the display. A more natural situation is achieved if the traffic also reacts to the test driver behavior, that is, the vehicle with the test driver comes an interactive part of the simulated traffic flow.

Finally, the simulation of travel demand is growing up rapidly. The basic research in time-use studies and trip chaining of individuals combined with disaggregate modeling form a theoretical basis for this new methodology. Demand simulation also uses GIS databases and tools for basic data input and demonstration of the results. The simulation approach is useful not only in the analysis of peak hour traffic in congested urban areas but also in the planning of special low demand transport services like demand responsive public transport.

2.7 Conclusions

This chapter provides an overview of the traffic flow modeling and presents some fundamental models. The models are classified according to level-of-detail and some key points are discussed. From the literature review, the microscopic models are perceived as better in off-line simulations such as testing the roadway geometry and assessment of traffic control strategies.

Macroscopic models are suited for large scale, network-wide applications, where macroscopic characteristics of the flow are of prime interest. Generally, calibration of macroscopic models can be considered as relatively straightforward but macroscopic models are generally too coarse to correctly describe microscopic details of bottleneck formation where the lane changing and weaving models are also have a great importance.

Mesoscopic flow models describe traffic flow at a medium detail level. Vehicles and driver behavior are not distinguished nor described individually, but rather in more aggregate-terms. However, the behavior rules are described at an individual level in mesoscopic models.

A microscopic simulation model describes both the space-time behavior of the systems' entities i.e. vehicles and drivers, as well as their interactions at a high level of detail. For instance, for each vehicle in the stream a lane-change is described as a detailed chain of drivers' decisions.

With respect to model applicability, microscopic simulation models are found more appropriate for this research that are more realistic and representative in terms of bottleneck formation. Hence, the traffic control and equity measurement analyses are performed with microscopic traffic simulation software.

Due to the availability of a microscopic level traffic simulation program, the traffic control strategies and the equity analyses will be performed through PTV-VISSIM software.

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CHAPTER 3. DEVELOPMENT OF NEW FREEWAY TRAFFIC CONTROL STRATEGIES

3.1 Motivation

The continuously increasing traffic congestion problem has led to application of various control strategies. Basically, these are formed by controlling the number of vehicles entering the freeway and/or by changing the speed limit of designated section along the freeway. Advanced urban traffic networks include both urban roads and freeways utilize control strategies like signal control, ramp metering, variable message signs and route guidance (Pesti et al, 2007). However, due to the costly nature of coordination and management in traffic control, some cost-efficient strategies should also be examined. In order to manage this low-cost management a fixed-time ramp metering strategy and harmonized speed control strategies are proposed. Fixed-time ramp metering strategy needs only simple adjustments on ramp layout and a signal device would be sufficient to manage the control. The drawback of this type of control is the unresponsiveness of the control system to the changes in the traffic demand. To overcome this problem there are two alternatives that can be considered, the first one is only the peak hour control can be applied and the second one is the cycle time can be optimized with respect to the peak hour traffic volumes. Thus, in this study the traffic control is mainly concentrated on peak-hours and the cycle time, or the released rates from the ramps in other words, are optimized.

The remainder of this section is organized as follows; the literature review on traffic control is presented, and later, the traffic control strategies namely; fixed-time ramp metering, harmonized speed limit control and integrated control strategies are demonstrated. Summary and the discussion of the chapter are presented lastly.

3.2 Freeway Traffic Control Strategies

Freeways had been delusively perceived as an infrastructure that can actually provide unlimited usability to road users, without the annoyance of flow interruptions by traffic lights (Papageorgiou et al, 2003).

Eventually, the rapid increase of traffic demand over supply, cause serious congestions, both at recurrent (occurring daily during rush hours) and non-recurrent (due to incidents) levels (Nissan, 2010). Increased travel times, delays, long queues, degraded infrastructure utilization, reduced safety and frustration of drivers which causes the decrease in quality of life are the direct effects of freeway traffic congestion (Sisiopiku et al 2004).

The capacity of the infrastructure is highly important in determination of the utilization level. Therefore, as it was mention in the previous chapter, the capacity of highways has been researched since the study of Greenshileds, 1935. The authorities who are responsible for the management of highways are interested in the maximum possible service that they can provide and how efficient they can succeed in provision. Also the bottleneck sections are one of the special focuses, which can be simply defined as the weakest links of a chain.

Many studies showed that most of the highways are lacking of efficient and comprehensive management (Liu, et al, 1999). In other words, the expensive infrastructure is intended to deliver a nominal capacity that is not available (due to congestion), ironically, exactly at the time it is most urgently needed (during peak hours) (Papageorgiou et al, 2003).

The control of a traffic corridor, which consists of two major components—freeway system control and arterial street system control, is aimed at improving flows on both freeway and arterial streets, and has been demonstrated as an effective mean to increase the level of service of a corridor system during peak periods (Jin and Zhang, 2001).

The control measures that are typically employed in freeway networks are the ramp metering, activated via installation of traffic lights at on-ramps, link control that comprises a number of possibilities including lane control, variable speed limits, congestion warning, tidal (reversible) flow, keep-lane instructions, and driver information and guidance systems, either by use of roadside variable message signs or via two-way communication with equipped vehicles (Papageorgiou et al, 2003).

Traditionally, control strategies for each type of control measure are designed and implemented separately, which may result in antagonistic actions and lack of synergy among different control strategies and actions (Papageorgiou et al, 2003). However, modern traffic networks that include various infrastructure types are perceived by the users as an entity, and all included control measures, regardless of their type or location, ultimately serve the same

goal of higher network efficiency. Integrated control strategies should consider all control measures simultaneously toward a common control objective. Despite some preliminary works on this subject, the problem of control integration is quite difficult due to its high dimensions that reflect the geographical extension of the traffic network.
In conclusion, an ideal control methodology should possess the following properties:

i. A good system model describing freeway operations and control: The model should be able to describe both the operations and control in the freeway system accurately. It should capture major traffic flow phenomena that are critical to control design, such as criticality, shock waves, and drivers' response to controls.
ii. Sound theoretical foundation: The model should be based on reasonable assumptions and objectives, rigorous problem formulation, efficient and accurate solution methods.
iii. Proactive and balanced: The model should prevent congestion rather to react to congestion, and avoid happening of spillback of queues.
iv. Accuracy and robustness: The control actions should be effective to achieve the control objective, and degrades gracefully when some parts of the system, such as input links are down.
v. Computational efficiency and flexibility: Algorithms should be easy to program, implement and modify besides being computationally efficient.
vi. Cost-efficiency and simplicity: The model should use the simplest logic structure possible to reconcile the demands on realism and theoretical elegance and shouldn't yield an extreme investment for realization.

However establishment of an ideal control in traffic is not an easy task, more truly this is unrealistic. There is a contradiction that the more advanced a control system is, so the more crucial may be the contribution of the human operator (Bainbridge, 1983). Also, it is also reported that the global multiple optima caused by the each bottleneck section can contradict with the global optimum (Papageorgiou et al, 1990). ALINEA and METALINE were tested in simulation studies by Papageorgiou et al. (1990, 1991) for the Boulevard Peripherique in Paris, using METANET macroscopic traffic simulator. Both feedback control strategies were found to decrease the total travel time; METALINE resulted in a slightly better performance for non-recurrent congestion whereas ALINEA performed better in recurrent congestion. In conclusion, the development of a holistic traffic control solution is demanding an extreme amount of theoretical and experimental research.

3.2.1 Ramp Metering Control

A freeway corridor consists of mainline and entrance/exit ramps which connect the freeway to the arterial and local streets. It is designed to provide a high level of service (LOS) to the community (Fitzpatrick at al, 2003). However, many corridors in the world are congested, with the worst congestion problems usually arising during the two peak periods (morning and evening) (Schrank and Lomax, 2007). There are two types of traffic congestion observed: recurrent and non-recurrent. Recurrent congestion is due to excessive peak demands and non-recurrent congestion is due to capacity reduction caused by events such as accidents (Zhang et al, 2001).

Ramp metering can be defined as a basic traffic light or a two-section signal (red and green only) light together with a signal controller that regulates the flow of traffic entering freeways to maintain the capacity on the mainline or at the bottleneck section (Papageorgiou et al, 2000). Ramp metering, (or on-ramp/entrance ramp control), which is designed to determine a metering rate for each on-ramp by considering the traffic conditions as a part or whole of the corridor, has been considered a very important component of corridor traffic control (Ben-Akiva et al, 2003). Ramp metering systems are one of the major control techniques in traffic congestion alleviation and driving safety provision (Dingus and Hulse, 1993). It is also reported that ramp meters help the increase of speed and volume on freeways by reducing demand and by breaking up platoons of cars (Chaudhary and Messer, 2002).

The objectives of ramp metering control are (Zhang et al, 2001);

i. alleviation or elimination of congestion;
ii. improvement of freeway flow, traffic safety and air quality by the regulation of input flow to a freeway;
iii. reduction of total travel time and the number of peak-period accidents;
iv. regulation of the input demand of the freeway system so that a truly operationally balanced corridor system is achieved.

According to the control philosophy, there are two classes of control schemes; fixed-time/ pre-timed/ time-of-day control, in which metering rates are fixed according to clock time and feedback control/ traffic responsive control where the output of the system fed back through a sensor measurement to the reference value. For the recurrent congestion, fixed-time controls can provide good performance besides having the advantage of having a simple, cost-efficient environment. Nevertheless, most of the modern ramp metering strategies is based on traffic responsive philosophy yet this makes the systems more complex, expensive and difficult to maintain (Jin and Zhang, 2001).

Zhang et al (2001) classified ramp metering control systems into three major groups as; isolated or local system, in which control is applied to an on-ramp independently of any other on-ramps; coordinated system, in which control is applied to a group of on-ramps in a coordinated fashion, taking into consideration the traffic conditions in the whole system rather than the local conditions around independent on-ramps; integrated system, in which different types of control measures are used, such as ramp metering, signal timing, and route guidance via variable message signs (VMS).

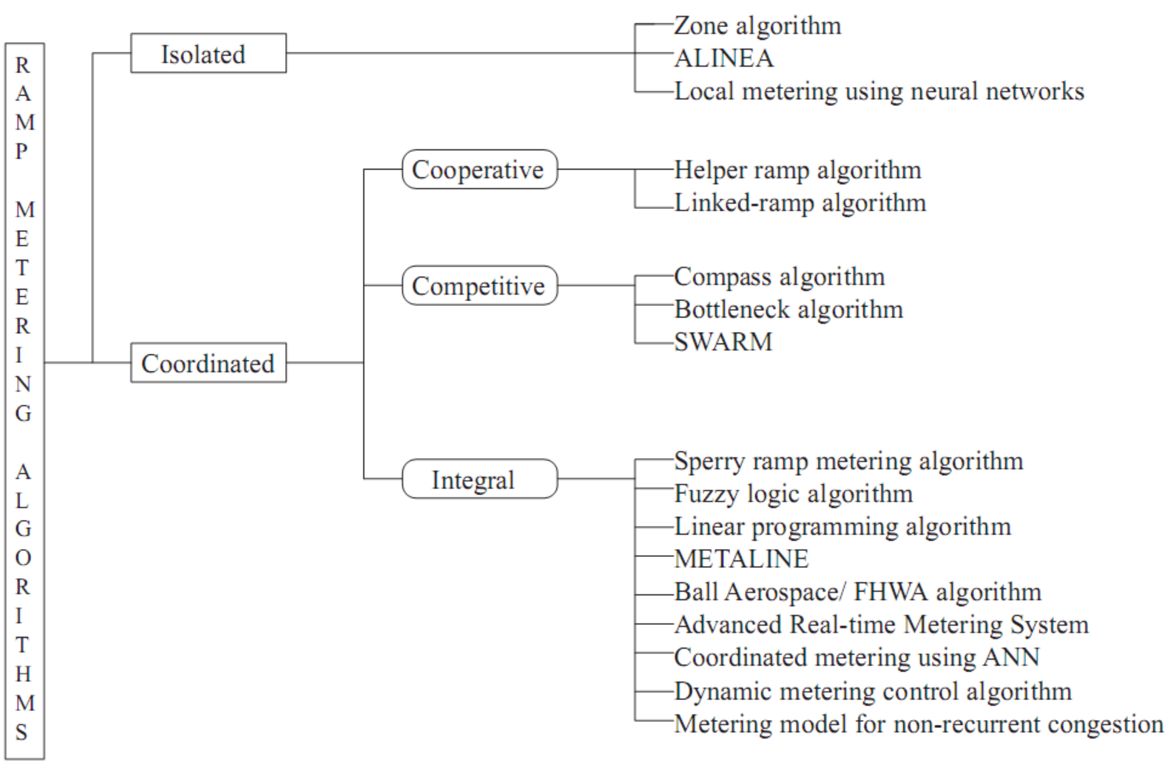

Figure 3.1: The categories of ramp metering algorithms (Zhang et al., 2001).

There are also some other classifications of ramp metering algorithms based on the coordination type, being open or closed loop control, geometry or operations level. Zhang et al, (2001) mainly classified the ramp metering algorithms into two categories; isolated and coordinated, which is presented in Figure 3.1. Classification of Lierkamp (2006) is also given in Figure 3.2.

Another classification of ramp metering algorithms is given by Tian (2002). Ramp metering algorithms are clustered into four initial categories such as; operation level, geometry, location and operations rule. The categorization is shown in Figure 3.3.

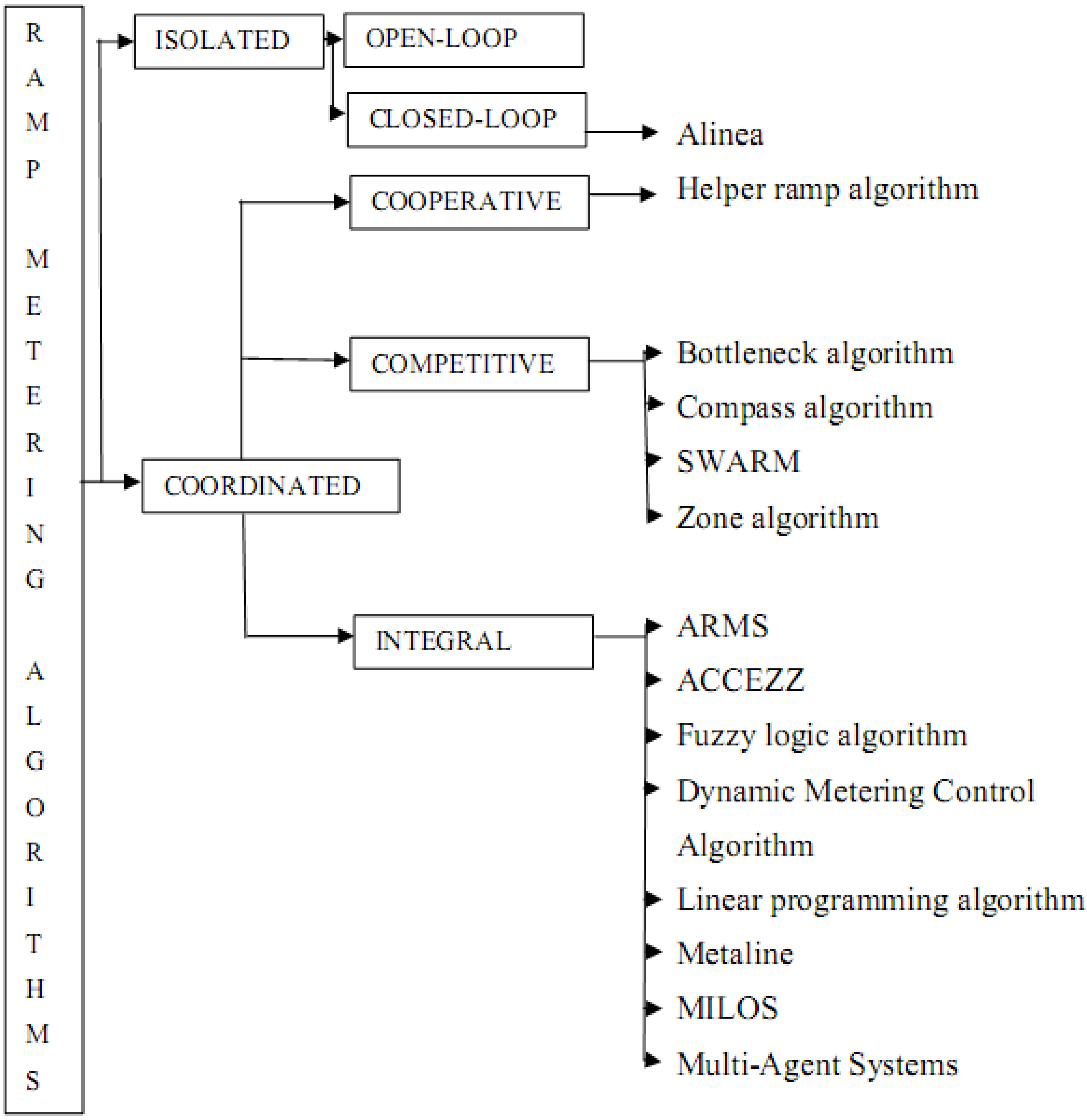

Figure 3.2: The categories of ramp meter algorithms (Lierkamp 2006).

3.2.1.1 Ramp Metering Control Algorithms

There exists a large number of ramp metering schemes in literature. While some of them were implemented in the real world, most of these algorithms are still awaiting further assessment. In this study the focus is given on the most common ramp metering algorithms observed in the literature and practice.

In isolated ramp-metering algorithms, a ramp metering rate for an on-ramp is determined based on its local traffic conditions, such as flow, occupancy, travel speed, and occasionally queue over- flow on the metered ramp. The reviewed algorithms include; ALINEA (Asservissement Lineaire d'Entree Autoroutiere-French for "Linear Utilization for Highway Entrances"), the demand-capacity, the percent-occupancy, the fixed-rate or time-of-day control, METALINE, the zone algorithm, Bottleneck algorithm, System wide adaptive ramp metering (SWARM), Helper ramp algorithm, Compass algorithm, Fuzzy Logic algorithm, linear programming, ARMS (Advanced Real-time Metering System) and, FLOW.

Most of the local control algorithms including ALINEA use feedback regulation to maintain a desired level of occupancy, or the target occupancy, which is usually chosen to be the critical occupancy. The traffic models are mostly macroscopic ones that utilize the kinematic wave theory with locally calibrated fundamental diagrams (Papageorgiou, 1998).

ALINEA is a widely examined ramp metering control algorithm either in theory or in the practice. The control input is based on the system output. The goal of ALINEA is to sustain near maximum flow downstream of the on-ramp by regulating the downstream occupancy to a target value, which is set a little below the critical occupancy at which congestion first appears. There are different versions of ALINEA such as, Q-ALINEA, FL-ALINEA, MALINEA, UF-ALINEA, UP-ALINEA and X-ALINEA/Q.

The metering rate r(k) is determined by the sum of the measured volume of vehicles the ramp meter is allowing onto the freeway mainline r at the interval k-Δt (computation iteration minus the ramp meter update cycle and the product of the coefficient with the difference between the desired downstream occupancy and the measured occupancy per computation iteration)

$$r(k) = r(k-1) + K_R[\hat{o} - o_{out}(k)] \tag{3.1}$$

Where; k is the computation iteration; r(k) is the metering rate per iteration; k-Δt is the measured metering rate at interval, where Δt is the update cycle of the metering rate; ô is a preset desired value of downstream occupancy and o_{out} is measured occupancy per iteration. K_R is a coefficient, normally set to 70 vph.

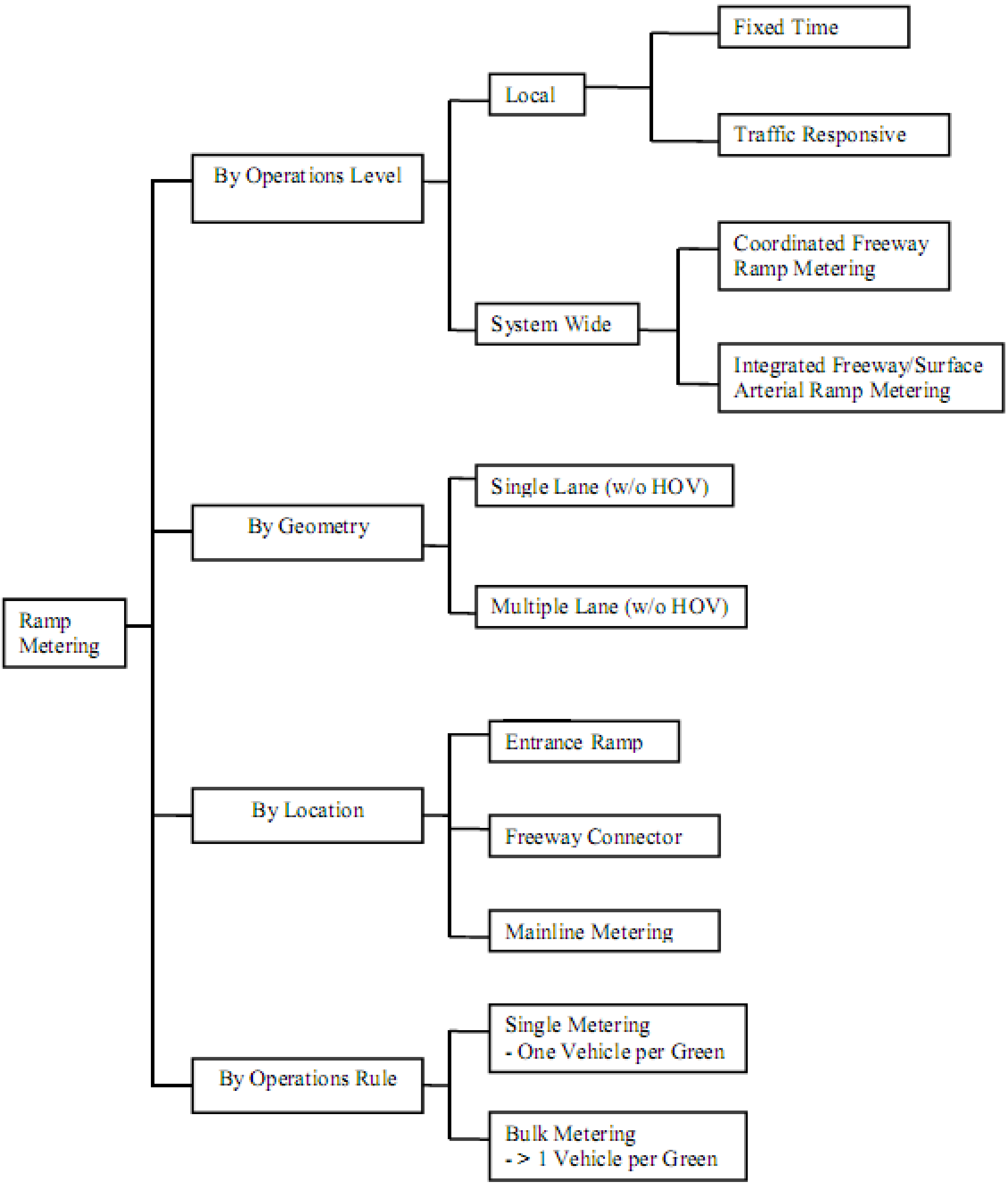

Figure 3.3: Ramp metering classifications (Tian, 2002).

The gain of the control loop is established by the coefficient K_R. As K_R is increases, the sensitivity and speed of response to changing inputs is increase. This tends to make the control more oscillatory and more sensitive to random variations and errors in the measured occupancy. Although an effective algorithm in reducing congestion, Chu and Yang (2003) advised that accurate implementation of ALINEA depends on correctly choosing three parameters k, $\hat{o}$ and the location of the downstream detector.

Kachroo and Krishen (2000) noted that as ALINEA was designed using linearization of the system dynamics it is valid only for the local region around the equilibrium. They warned that as ALINEA does not explicitly take ramp queues into consideration it needs a feedback control law that incorporates queue length. Hasan and Ben Akiva (2002) found that ALINEA performed worse at higher demand levels due to its inability to deal with downstream bottlenecks. They also found that ALINEA's performance deteriorated at demand levels below 80% and ramp queues over 75% the length of the ramp. Bogenberger and May (1999) found ALINEA to be inferior to coordinated ramp metering in the case of an incident. Papageorgiou and Smaragdis (2003) advised that modifications were needed so that ALINEA could use upstream instead of downstream measurements, flow-based rather than occupancy-based parameters, automatic real-time adaption of set values to optimize downstream flows and efficient ramp-queue control to avoid interference with ramp feeder roads.

Demand-Capacity is also another common ramp metering control algorithm. This traffic responsive algorithm controls the ramp flow with respect to the downstream occupancy. If it is above the critical occupancy, congestion is assumed to exist. The metering rate is then set to the min rate. Otherwise, the volume is measured upstream of the merge, and the metering rate is set to the difference between the downstream capacity and the upstream volume (Hadj-Salem et al, 1990).

Similar to Demand-Capacity, percent-occupancy strategy uses only upstream sensor occupancy measurements to identify and measure congestion. The critical occupancy is measured using historical data which may, under certain conditions, reduce the corresponding implementation cost (Pi et al, 2007).

In fixed-rate or time-of-day control, ramp meter timings are adjusted automatically by specified time-of-day parameters. As it is mentioned previously, this algorithm does not afford flexibility for changing traffic conditions (Rafferty and Treazise, 2007).

System-wide coordinated ramp metering schemes consider the metering rates based on conditions throughout the entire length of the metered corridor. This makes system-wide control more flexible in handling reductions in capacity that occur as a result of congestion or non-recurring incidents.

METALINE (Papageorgiou et al.) is an extension of the local control algorithm ALINEA. It was implemented on certain freeways in France, the United States and the Netherlands. The metering rate of each ramp is computed based on the change in measured occupancy of each freeway segment under METALINE control and the deviation of occupancy from critical occupancy for each segment that has a controlled on-ramp:

$$\vec{r}(k) = \vec{r}(k-1) - K_1\left(\vec{o}(k) - \vec{o}(k-1)\right) - K_2\left(\vec{O}(k) - \vec{O}^c\right) \tag{3.2}$$

where, $\vec{r}(k) \in$ Rm is the vector of metering rates for the m controlled ramps at time step k; $\vec{o}(k) \in$ Rn is the vector of n measured occupancies within the directional freeway segment at time step k; $\vec{O}$, $\vec{O}^c \in$ Rn are respectively the measured and desired occupancy downstream of m controlled ramps. K1,K2 are two gain matrices.

The zone algorithm has been employed by Minnesota DOT for many years and considerable experience has been gained with this particular algorithm, and is flexible due to possible adjustments for different situations. However, parameters for the algorithm have to be tuned carefully to suit local traffic and freeway characteristics, which may not be as easy as it appears because the relation between the control parameters and the control objective is not clear in the zone algorithm. Another significant drawback of the algorithm is that it does not consider the dynamic nature of traffic flow and for this reason may not perform well under conditions as happening of an incident when fast changes of traffic flow occur (Jin and Zhang, 2001).

The Seattle bottleneck algorithm (Jacobsen et al., 1989) was developed by the Washington Department of Transportation (WSDOT), and has been used to control a portion of I-5, north of the Seattle Central Business District. This algorithm also has a two-level structure. At the local level, a control strategy compares the upstream demand with the downstream supply (that is, the real-time capacity), then takes the difference of them as the locally determined metering rate. At the global level, a coordinate control strategy first identifies bottlenecks, decides the volume reduction for the bottleneck based on flow conservation, and then distributes the volume reduction to upstream ramps; the coordination determined metering rates, according to predetermined weights. The restrictive one of the locally and globally determined rates is selected to be realized. The Seattle bottleneck algorithm is conceptually one of the best heuristic ramp metering algorithms implemented in the field. It is real-time, coordinated, yet logically simple (based on supply-demand and flow conservation) and flexible (only a few adjustable parameters). Field operations with this control also show remarkable improvement in traffic conditions. Nevertheless, the Seattle algorithm can be improved by adopting a more robust local control strategy such as ALINEA, and real-time adjustment of volume reduction weights based on current O-D information. Further consideration of ramp queue spill over is also needed.

System wide adaptive ramp metering (SWARM) (Paesani et al, 1997) is developed by NET and is expected to be tested in Orange County, California. Like other heuristic coordinated control algorithms, SWARM also operates at two levels: the local control decides ramp metering rates based on local density; the global control decides the overall volume reduction from ramps upstream a critical bottleneck, and then distributes them to upstream ramps according to a set of predetermined fractions to obtain a new set of ramp metering rates; the most restrictive one of the two is selected for each ramp. SWARM has a built-in failure management module to clean faulty input data from detectors. It also allows further adjustment to accommodate queue spill-back handling. Both features enhance its robustness. Unlike previous two-level algorithms, SWARM identifies bottlenecks based on predicted traffic conditions rather than measured traffic conditions. Therefore it has the potential to nail congestion in the bud. On the other hand, it could also produce worse results than other non-anticipating algorithms (such as the Seattle Bottleneck Algorithm) if its predictions are poor. Good prediction models and accurate OD information are two key elements in the successful implementation of SWARM.

In Helper ramp algorithm (Lipp et al, 1991) a freeway corridor is divided into six groups consisting of one to seven ramps. In the local traffic responsive metering part of the strategy, each meter selects one of six available metering rates based on localized upstream mainline occupancy. In coordination part, if a ramp grows a long queue and is classified as critical, its metering burden will be sequentially distributed to its upstream ramps. The two-level structure of the helper algorithm makes it more capable and flexible when dealing with heavy congestion. Helper is a quite robust algorithm when accurate traffic flow models and origin-destination information are not available for the controller.

Compass algorithm was first implemented in the Toronto area, Canada in 1975. Locally, the compass algorithm determines the metering rates from an ad-hoc look-up table, which has seventeen levels for each ramp, determined by the local mainline occupancy, the downstream mainline occupancy, the upstream mainline volume as well as some pre-defined parameters that include thresholds for local and downstream occupancies, and upstream volume. Globally, coordinated control use off-line optimization to generate metering rates based on system-wide information. The more restrictive one of the two rates is selected. The compass algorithm addresses spillback through overriding restrictive rates: if the occupancy at a ramp queue detector exceeds its threshold value, the metering rate is increased by one rate level until the detected occupancy is back below the threshold level. The Compass algorithm is flexible, considers many types of constraints, and is straightforward to implement. However, it is not robust because of the use of look-up tables and predetermined metering rates.

Fuzzy Logic algorithm can also balance several performance objectives simultaneously, such as occupancy, flow rate, speed, and ramp queue. Fuzzy logic based ramp control (Taylor et al, 1998) has been implemented in Seattle and the Netherlands. Fuzzy logic algorithms convert empirical knowledge about traffic flow and ramp control into the so-called fuzzy rules. For example, a rule can be as the following; if the local occupancy is small, and ramp queue is small, then metering rate is high. The performance objectives are divided into finite categories and then rules are developed with different weighting factors to relate traffic conditions with metering levels. Fuzzy logic can anticipate a problem and take temperate, corrective action before congestion occurs. It is very powerful and robust if the right type of rules is used. With congestion indicators as inputs, the Fuzzy Logic can handle poor data, incidents, special events, and adverse weather without modifying the control parameters.

Linear programming based ramp control algorithms are developed by Yoshino et al. (1995) and are among the oldest algorithms in both research and practice. The linear algorithm maximizes the weighted sum of ramp flows where the weights are selected by the user to reflect his belief in the varying importance of the ramps. Also real-time capacities of each road segment are computed. This allows the algorithm to work under congested road conditions. The drawbacks of this algorithm are; its performance is heavily dependent on accurate origin-destination data, and it is static, i.e., it neglects the variation of travel time in its computation of ramp metering rates.

ARMS (Advanced Real-time Metering System) (Liu et al, 1993) developed by researchers from Texas Transportation Institute and works on two levels. In the first level, a system-wide control policy is to maintain free flow conditions. The novelty of this algorithm lies here that it incorporates a congestion risk factor into its formulation. It also projects traffic conditions to decide potential bottlenecks, which makes the algorithm proactive. In the second level, the algorithm works to resolve congestion once it develops. It does this by minimizing the congestion clearance time and queues on the controlled ramps.

The dynamic ramp metering model developed by Chen et al, (1997) has four elements: local control, area-wide control, state estimation and O-D prediction. Local control attempts to maintain traffic conditions close to the target traffic conditions that are provided by area-wide control. The area-wide control in the dynamic ramp metering model is a predictive (rolling horizon) optimal control algorithm. It obtains metering rates through minimizing the total system travel time that includes travel time on freeway and delay on ramps, subject to demand and queue capacity constraints. To know future travel demand and traffic conditions, a state estimation model and a O-D prediction model are also developed. Overall, this is perhaps the most complex and comprehensive ramp metering algorithm that we have reviewed in this report. It contains essentially all the elements that an ideal ramp metering algorithm has. It is system-wide, adaptive and predictive. Initial simulations indicate that the combined local/area-wide control model is more effective than each control model operating alone. It is yet to be seen, however, how smooth this control model will operate in the real world because its effectiveness depends heavily on the accuracy of the state estimation and O-D prediction models.

FLOW (Jacobson et al, 1989) is a competitive, bottleneck-based, area-wide ramp metering algorithm. For each ramp, FLOW calculates both a local metering rate and a bottleneck metering rate, and selects the more restrictive of the two rates. The local metering rate uses a percent occupancy algorithm. A lookup table is used to relate upstream occupancies with metering rates. The lookup table is determined using historical volume-occupancy relationships. The metering rate associated with each upstream occupancy is the difference between the capacity and volume associated with the occupancy on the fundamental diagram. For the bottleneck metering rate, bottleneck locations on the freeway must be determined.

Each bottleneck must have an influence zone with one or more on-ramps associated with it. Each metered on-ramp has a weighting factor associated with it, determined both by the distance of the ramp from the bottleneck, and the historical volume on the ramp. Loop detectors must be located both upstream and downstream of the influence zone, as well as on all on-ramps (both metered and un-metered) and off-ramps. In order for the bottleneck algorithm to be invoked, two conditions must be met. The first condition is that the downstream occupancy must be greater than threshold occupancy, indicating that the freeway section is operating above capacity. The second condition is that the freeway section must be storing vehicles, meaning that the sum of the vehicles entering the section and entering via on-ramps must be greater than the sum of the vehicles exiting the section and leaving via off-ramps. Because influence zones may overlap, each ramp may have more than one bottleneck metering rate associated with it. In that case, the most restrictive bottleneck metering rate is chosen. Finally, either the most restrictive bottleneck metering rate, or the local metering rate, whichever is more restrictive, is selected. FLOW uses a two step queue control process. The first part is queue adjustment. When the queue for a ramp reaches a certain length, the metering rate for that ramp is increased slightly. The second step is advance queue override. When the queue reaches its maximum permissible length, the ramp meter is shut off.

There are also some other control algorithms that is worth to mention such as; BEEX (Balanced Efficiency and Equity) seeks to minimize the total weighted travel time, which involves weighting both the freeway mainline travel time and the ramp delays, MILOS (Multi-Objective, Integrated, Large-Scale, & Optimized System) targets ramp metering rates to maximize freeway throughput, balance ramp queue growth rates, and minimize queue spill-back into the adjacent surface street interchanges and SZM (Stratified Zone Metering) is found effective in reducing ramp delays and queues, reducing freeway travel time and delay, increasing freeway speed, smoothing freeway flow, as well as reducing the number of stops.

Table 3.1. Characterization of local and coordinated ramp metering algorithms.

Ramp Metering	System characteristics	
Algorithms	Local	System-wide control
Fuzzy Logic	The rule base is rather simple and number of rules needed is limited.	The rule base can be complex. Developing a consistent set of rules is not always straightforward.
Bottleneck	Compares the upstream demand with the downstream supply, and then takes the difference of them as	First identifies bottlenecks, decides the volume reduction for the bottleneck based on flow conservation, and then distributes

	the locally determined metering rate.	the volume reduction to upstream ramps according to predetermined weights.
Compass	Determines the metering rates from an ad-hoc lookup table, which has multiple levels for each ramp, determined by the local mainline occupancy, the downstream mainline occupancy, and the upstream mainline volume.	Coordinated control use of off-line optimization to generate metering rates based on system-wide information. Compass addresses spillback through overriding restrictive rates. If the occupancy at a ramp queue detector exceeds its threshold value, the metering rate is increased by one rate level until the detected occupancy is back below the threshold level.
Dynamic metering control	Local control attempts to maintain traffic conditions close to the target traffic conditions that are provided by area-wide control.	Originally designed as a system-wide control where the constraints of objective of the control are system-wide parameters.
FLOW	Metering rate is the difference between the capacity and volume on the fundamental diagram.	The bottleneck metering rate for each ramp is calculated by subtracting the bottleneck metering rate reduction from the measured on-ramp flow during the previous interval.
Helper	Each meter selects one of six available metering rates based on localized upstream mainline occupancy.	If a ramp grows a long queue and is classified as critical, its metering burden will be sequentially distributed to its upstream ramps.
SWARM	Ramp metering rates determined based on local density.	When a bottleneck is detected, a new set of ramp metering rates are determined. Downstream ramp meters will be shut off and upstream ramp meters will have a more restrictive timing.
ZONE.	Utilizes the occupancy control logic.	Discritizes freeway into segments then calculates metering rates based on volume control in each zone.

3.2.2 Speed Management

Speed management is intended to improve safety and mobility of traffic through freeway bottlenecks (Lu et al, 2010). Speed harmonization techniques using VSL are common in many countries in the world. Such systems may be deployed to promote safer driving during recurring congested periods, incidents, or under adverse weather conditions. In the literature, two views on the use of speed limits can be found. The first emphasizes the homogenization effect whereas the second is more focused on the preventing traffic breakdown by reducing the flow by means of speed limits.

3.2.2.1 Speed Limit Control

The basic idea of homogenization is that speed limits can reduce the speed differences, by which a more stable (and safer) flow can be achieved (Hegyi et al, 2005). The homogenizing approach typically uses speed limits that are above the critical speed (i.e., the speed that corresponds to the maximal flow; see Figure 1 for an illustration). So, these speed limits do not limit the traffic flow, but only slightly reduce the average speed (and slightly increase the density). The homogenization approach can in theory increase the time to breakdown, but cannot suppress or resolve existing shock waves.

Figure 3.4: Variable Speed Control in Yokohama
Source: http://waytrip.web.fc2.com/hodo-nb.html

An extended overview of existing speed limit systems that aim at reducing speed differentials is given by Wilkie. The traffic breakdown prevention approach focuses more on preventing too high densities, and also allows speed limits that are lower than the critical speed in order to limit the inflow to the jammed area. By resolving these high density areas (bottlenecks) higher flow can be achieved in contrast to the homogenization approach (Hegyi et al, 2008).

3.2.2.2 Speed Harmonization Control

The ideal VSL system consists of sensors, variable speed limit signs, variable message signs, and a central processing unit to execute control actions. As shown in Figure 1, VMS are used to inform drivers of the traffic condition ahead and to display the enforced speed limit based on the VSL control strategies.

Depending on the approaching volume, driver compliance rate, and the resulting congestion, the central processing unit that integrates all system sensors and signs will compute the time-varying optimal speed limit for each VMS dynamically and display it in a timely fashion.

Figure 3.5: VSL Control in Istanbul

Source: http://www.isbak.com.tr/tr/icerik/degisken-mesaj-sistemi-vms

Common practice over the past several decades for work zone operations is to recommend or enforce a reduced speed limit via variable message signs (VMS), which may or may not respond to fluctuations in approaching traffic demand. To properly respond to traffic conditions and to increase the compliance rate of drivers, traffic professionals in recent years have experimented with variable speed limit (VSL) control in place of the conventional posted speed limit operations in highway work zones.

In brief, most existing VSL-related systems have been designed in response to traffic safety concerns but not for improving operational efficiency, such as to maximize the throughput from a work zone segment or to minimize the average delay for vehicles traveling through the entire highway segment plagued by the work zone-imposed traffic queue (Kang et al, 2004).

Hegyi et al. (2005) developed a method for optimal coordination of VSL and ramp metering. They demonstrated the potential benefits of variable speed limits in minimizing total travel time and suppressing shockwaves. Park and Yadlapati (2004) evaluated a number of variable speed limit control logics at work zones using microscopic simulation (VISSIM). They used a minimum safety distance equation as a surrogate safety measure. They found that VSL can improve both mobility and safety at work zones. Zhicai et al. (2004) tested various VSL strategies for various traffic demands and for different roadway geometry, lane closure, and incident scenarios. Another study by Abdel-Aty et al. (2006) investigated the safety benefit of VSL using traffic data from IH 4 in Central Florida. Various VSL strategies were tested at three individual locations. They found that VSL is most beneficial when speed limits upstream of the risk-prone location are greatly reduced, and the speed limits downstream of this location are increased.

3.2.2.3 Active speed warning signs

Active warning systems are traffic control devices consisting of sensors and variable message signs or flashing beacons with conventional warning signs. Warning signs may be divided into categories based upon the variability of the hazard identified and the uniformity of that hazard's relevancy among drivers (Pesti et al, 2008).

i. static hazards with uniform relevancy,
ii. variable hazards with uniform relevancy and
iii. static and variable hazards with non-uniform relevancy

Active warning signs (AWS) may be activated by certain predetermined threshold for speed or speed differentials. Then the drivers are informed through a Variable Message Signs (VMS) of either their speed or the hazard ahead.

The following section reviews some active warning systems that have been used for advance warning of slow or stopped traffic. McCoy and Pesti (2002) evaluated another portable, condition-responsive work zone traffic control system, the ADAPTIRTM. The system utilizes radar sensors mounted on three portable changeable message signs (PCMS) and an arrow panel at the merging taper to continuously measure speeds at four locations along the approach to a freeway work zone. Whenever the average speed at the next downstream radar sensor location was found to be more than 10 mph lower than the average speed at a PCMS, a speed advisory message was displayed indicating the downstream speed rounded down to the nearest 5 mph. The results of the analysis indicated that the messages were only slightly effective in reducing speeds. It was concluded that their effectiveness could have been improved if the distances between the PCMSs had been shorter.

A Work Zone Safety ITS system capable for adaptive queue warning was recently developed and evaluated by the University of Michigan. The system was a distributed, queue-warning system that automatically adapted to the current traffic-flow situation within and upstream of the work zone. The study focus is on finding two critical elements of the system: an inexpensive but sufficiently accurate speed sensor and a simple but effective signaling system. Three prototype speed sensors are developed and evaluated in a limited field study. The researchers are also evaluated active infrared, passive infrared and magnetic sensor technologies, respectively. The active infrared system is found to be the most accurate but consumed the most power.

Driving simulator results suggested that drivers find the adaptive systems more helpful than static road signs. Systematic positive change in their driving performance, that is indicative of enhanced safety, was also observed. The technology shows promise in addressing problems of work zone rear-end collisions (Vahidi, and Eskandarian, 2003).

3.2.3 Route information and guidance

Freeway, urban, or mixed traffic networks include a large number of origins and destinations with multiple paths connecting each origin-destination pair. During rush hours, the travel time on many routes changes considerably due to the congestion and less congested routes may become plausible alternative. Regular travelers in a network can easily optimize their individual routes based on their past experience, which yields the well known user-equilibrium conditions first observed by Wardrop (1952). But demands in the network vary greatly with respect to the weather conditions, events and most commonly the incidents. Thus, this causes the underutilization of the overall network's capacity, where some links are heavily congested while others still remains in flux. Route guidance systems (RGS) may be employed to improve the network efficiency through direct or indirect recommendation of alternative routes via VMS, traffic radio or some other communication tools (Bluetooth, infrared...etc.) (Kotsialos and Papageorgiou, 2004).

A route guidance system may be viewed as a traffic control system. Based on real-time measurements, sufficiently interpreted and extended within the surveillance block, a control strategy decides about the routes to be recommended to the road users. Short computation times are highly important due to the real-time nature of the operation.

Route guidance strategies may be classified according to various aspects. Reactive strategies are based only on and react to current measurements without the real-time use of mathematical models or other predictive tools; predictive strategies attempt to predict traffic conditions sufficiently far in the future. Iterative strategies run several model simulations in real time, each time with suitably modified route guidance, to ensure that the control goal will be achieved as accurately as possible. Single step strategies may either be reactive, in which case they typically perform simple calculations based on real-time data, or they may be predictive, where they run one single time a simulation model to increase the relevance of their recommendations (Papageorgiou et al,2003).

Route guidance strategies may aim at either system optimal or user optimal traffic conditions. In the first case, the control goal is the minimization of a global, system optimal, objective function. In the second case, user-optimal conditions imply equal cost on all utilized alternative routes connecting any two nodes in the network.

3.3 Development of alternative freeway traffic control strategies

Historically, the solution of the conflicts between the multidirectional flows of traffic is sought by considering the allocation of saturation, time or delay among all movements. The use of traffic signal establishes an orderly movement of traffic and increases the capacity and safety of intersections thoroughly. The design process of timing plans for signalized intersections in Highway Capacity Manual (2010) treats the traffic merely as static volumes of conflicting movements that require right-of-way alternatively. With a given phase sequence and phase groups, the method can determine how much green time within a cycle will be allocated to each phase, or the green splits. One fundamental difference of these methods is the design logic to allocate green splits; and these logic will affect how efficient and equitable a timing plan can be. Three major logics have been developed are; equal-saturation strategy where the green time is determined in such a way that the phase duration will be proportional to its critical Volume/Capacity (V/C) ratio, delay minimization strategy which is a policy that minimizes the total intersection delay and the capacity maximization policy maximizing the intersection capacity through balancing the traffic pressures of conflicting approaches where the pressure is defined as the product of the approach capacity and its average delay for each approach link (Webster, 1958, Allsop,1971, Papageorgiou and Papamichail, 2007)

3.3.1 Optimal fixed-time ramp metering control

Likewise traffic lights, ramp metering control utilize traffic signals at freeway on ramps or freeway interchanges to manage the rate of vehicles entering the freeway.

Ramp metering algorithms typically deliver the admissible ramp flow value r(t) (in veh/h) to be applied during the next period t. The next step of a ramp metering installation is the actual implementation of this value, i.e., the operation of the traffic lights so as to allow $T \cdot r(t)/3{,}600$ vehicles to pass the ramp exit during the next T seconds, T denoting the ramp metering update period. In practice, there are several alternatives are available for translating the r(t) into the cycle plans. Each of these policies has advantages and disadvantages as in terms of traffic safety and performance.

In one car per green metering policy, the green phase T_G is fixed, e.g. to a value of 2 seconds to allow exactly one car to pass on each lane at each cycle. Typically, the metering policy is displayed to the drivers through a traffic sign at the upstream section of the traffic lights. Under this metering policy, only the cycle needs to be calculated, while the red phase T_R can be found by considering the excess demand. The main advantage of the one-car-per-green metering policy is that it releases one car at a time, rather than platoons of vehicles that may perturb the mainstream traffic while merging therein. The main disadvantage of the one car per green metering policy is that it certainly sets the admissible ramp flow more specifically; the maximum implementable ramp flow under this policy would be limited to 720 veh/h. The limitation of the one car per green metering policy could be partially solved by incorporating a pre-specified number n (e.g. n = 2) of cars to exit per green phase. Thus, this policy releases n cars at a time, which is an accordingly small platoon. Whereas, the admissible ramp flow can get higher values (Papageorgiou and Papamichail, 2007).

Under full traffic cycle metering policy, the traffic cycle c is always equal to the metering period T, hence only the green phase T_G needs to be calculated. The admissible ramp flow usually much higher than the one / n cars per green metering policy and this represents the major advantage of the full-cycle policy. On the other hand, this metering policy releases all vehicles during one single green phase which may perturb the mainstream traffic flow. This disadvantage may be partially relaxed if the ramp metering cycle is chosen relatively short, e.g. c = 20 seconds, in which case the green phases are accordingly short and the platoons of released vehicles are limited in size.

However, there is no established method for the specification of optimum cycle (length) time for fixed time ramp metering. Therefore, the traditional analytical models developed for fixed time intersection control are examined and capacity maximization approach is modified for fixed time ramp metering simulation experiment.

Ramp metering algorithms aim to set the allowable ramp flow value r (in veh/h) which can be basically defined as (cr/3,600) vehicles where c denotes cycle time in seconds (Papageorgiou and Papamichail, 2007). Traffic lights are operated on the basis of a traffic cycle consisting of a green phase T_G, an amber phase T_A, a red phase T_R, and a red-amber phase T_{AR} which are adjusted in seconds such that:

$$c = T_G + T_A + T_R + T_{AR} \tag{3.3}$$

The number of vehicles that the signals allow off the ramp is calculated as the difference between the actual demand at the bottleneck (D_a), more specifically the sum of the mainline demand (D_m) and ramp demand (D_r) and the pre-specified capacity of the road (C).

$$D_a = D_m + D_r \tag{3.4}$$

The most critical point is the specification of the bottleneck capacity since it varies over time. Nevertheless, in most cases bottlenecks are also considered as having the same capacity as basic freeway segments which takes values between 1800 and 2200 veh./h/lane. The excess demand (D_e) would be determined from:

$$D_e = D_a - C \qquad (3.5)$$

where D_a is the actual demand (in veh./h) including ramp and mainline flows and C (in veh./h) is the capacity of the downstream section of the bottleneck. Resulting from (3.5), the admissible ramp flow value (r) would be:

$$r = D_r - D_e \qquad (3.6)$$

The translation of the ramp flow value r into a corresponding green phase under a full traffic cycle plan, where the traffic cycle c is always equals to the metering period, would be based on the green ratio (f):

$$f = r/D_r \qquad (3.7)$$

Therefore, the green time leads to:

$$T_G = cf \qquad (3.8)$$

In order to determine the optimal cycle time and green time for fixed time ramp metering control and examine the cycle time duration effects on network performance, a set of simulation experiments is designed. At each ramp, the green times are calculated for the average flows of entire simulation period by varying the signal cycle time from 5 to 20 seconds.

3.3.2 Harmonized speed control

The definition of harmonized speed control given by Fuhs, 2010 that the control alters the speed limits dynamically and automatically by utilizing the regularly spaced, over lane speed and lane control signs in the areas of congestion, construction work zones, accidents, or special events to maintain traffic flow and reduce the risk of collisions due to speed differentials at the end of the queue and throughout the congested area. Most of the focus is given in the literature is on the harmonized speed control impacts on traffic safety.

In general, VSL control is implemented to homogenize traffic flow, improve safety, and reduce driver stress. Many VSL control strategies have been put into action in USA, UK, the Netherlands, Germany, Australia, Austria, Japan and Turkey (Lee et al, 2006). There are several recent studies investigating the impact of VSL on safety and traffic flow [Lee and Abdel-Aty, 2008, Abdel-Aty et al, 2008, Allaby et al, 2007). Much of the focus of VSL system evaluation studies has been on safety. German motorway data is utilized by Papageorgiou et al, 2008 to investigate the impact on aggregate traffic flow behavior considering the impact on the shape of the flow–occupancy diagram and efficiency increase. It is concluded that employment of speed limits below the critical occupancies decreases the slope of the flow–occupancy diagram and shifts the critical occupancy to higher values. However, with respect to the potential increase in efficiency, the results are indeterminate, as at some sections an increase have attained, while at other sections no increase are noted. In conclusion, there appears to be limited empirical evidence and affirmative studies on the effects of VSL on traffic flow efficiency.

It is reported that the capital costs for these systems can vary substantially depending on the desired system. The infrastructure costs would be composed by signage requirements; gantry/sign bridge spacing, upgrades to ancillary equipment, and associated emergency refuge areas if desired. Therefore, it is critical to develop the overall system concept first and then determine the requirements and operational approach of the system to fully understand and estimate system costs (Fuhs, 2010).

Our harmonized speed control strategy intends to address this critical issue of efficiency of freeway traffic management. The proposed HSC system adjusts the set of displayed the speed limits for the congested peak periods where the speed limits are pre-determined that aims to minimize the selected measures of effectiveness for the examined corridor. The main difference of this control is the intervention to the speed limit boundaries. The conventional speed limit controls only deals with the upper speed limit. However the lower speeds are also considered as a critical part of an effective control and these limits are also needed to be examined. The research question focus is the investigation of the bounded speed limits effects on prevention of congestion and on congestion alleviation.

In order to test the research question, similar to the ramp metering control experiment a bunch of rationale speed limits from 60km/h to 100km/h are employed with 10km/h increments in traffic micro-simulation program. In this study it is assumed that all the drivers are driving with in the speed limits posted within a 5% upper and lower margin. In order to achieve this in simulation environment the speed profiles are adjusted linearly for every speed limit examined.

Speed restriction areas are defined along the corridor in order to test the speed limit control in simulation environment and a perfect compliance rate of drivers to the displayed speed limits is assumed.

3.4 Other freeway control strategies

In addition to the proposed control methods given above, ALINEA is examined as a ramp metering control and a variable speed limit control strategy which takes the occupancies of bottleneck sections is implemented for comparison purposes.

3.4.1 Model predictive ALINEA ramp metering control

MPC models predict the change in the dependent variables of the modeled system that will be caused by changes in the independent variables. MPC is an optimal control method applied in a rolling horizon framework. Optimal control has been success-fully applied by several researchers in traffic control over the years.

Both optimal control and MPC have the advantage that the controller generates control decisions that are optimal according to a controller-supplied objective function. However, MPC offers some important advantages over conventional optimal control. First, optimal control has an open-loop structure, which means that the disturbances (in our case: the traffic demands) have to be completely and exactly known before the simulation, and that the traffic model has to be very accurate to ensure sufficient precision for the whole simulation. MPC operates in closed-loop, which means that the traffic state and the current demands are regularly fed back to the controller, and the controller can take disturbances into account and correct for prediction errors resulting from model mismatch.

Second, adaptivity is easily implemented in MPC, because the prediction model can be changed or replaced during operation. This may be necessary when traffic behavior significantly changes (e.g., in case of incidents, changing weather conditions, lane closures for maintenance).

Third, for MPC a shorter prediction horizon is usually sufficient, which reduces complexity, and makes the real-time application of MPC feasible (Hegyi et al, 2005).

In our control algorithm a time series model is used to predict the occupancies of the next time frame. The time series model can be expressed as;

$$O_t = \beta_1 O_{t-1} + \beta_2 O_{t-2} + \beta_3 O_{t-3} \tag{3.9}$$

where, O_t is the dependent variable and O_{t-1}, O_{t-2} and O_{t-3} are the independent variables to predict the occupancies of the further time frames. β_1, β_2, and β_3 are the coefficients determined with the ordinary least square regression analysis.

For this approach, ALINEA ramp metering strategy aims to keep the occupancy rate at a level of desired occupancy rate which is generally taken 0.30 and the regulation parameter K_r is taken as 140 as it proposed by the developers of the algorithm. The predicted occupancies are incorporated to the control strategy as;

$$r(t) = r(t-1) + K_R[\hat{o} - O_t] \tag{3.10}$$

where, o_{pre} is the predicted occupancy of bottleneck downstream. A fixed cycle time is preferred to be able to compare the dynamic nature of algorithm with the proposed one.

3.4.2 Model predictive variable speed limit control

The model predictive control explained above is adopted for variable speed limit control. To perform an optimal dynamic VSL control, a set of traffic models must capture the complex interactions between traffic-state evolution and all control parameters. In particular, those traffic-state evolution equations should be mathematically formulated to represent the actual operational constraints.

As recognized in many studies such as Sisiopiku, (2001) and Smulders, (1990) traffic density and speed have been taken as state variables, of which the former is a key factor affecting drivers' choice of speed and the VSL system's selection of appropriate speed limits.

However, instead of taking speed or volume as a control variable occupancy measure is proposed as an optimal control variable of VSL algorithm for congestion alleviation. The algorithm includes the time dependent occupancies and variable speed limit intervals as control variables and parameters. Table 3.2 shows the VSL strategies employed in this study.

The strategy takes the occupancy measurements at the downstream section of ramp bottleneck and translates them into the control decision.

Table 3.2 Implemented variable speed control strategies

Strategy	Occupancy Rate (%)	Posted Speed Limits(km/h)
MP-VSL	0.1	120
	0.15	110
	0.2	100
	0.25	90
	0.3	80
	0.35	70

3.5 Integration of control strategies

A comprehensive traffic control philosophy should consider the full range of available operational strategies; including the various ways these strategies can be integrated together and among existing infrastructure, to actively manage the transportation system to achieve system performance goals.

Besides the stand alone strategies which are the proposed FTRM, SHC, MP_ALINEA and MP_VSL controls, the combination of the stand alone strategies are also modeled and implemented in microscopic traffic simulation program. For the integrated strategies there is no coordination is employed between the stand alone strategies. There are four stand alone and four integrated control strategies are compared with no control case. The control strategies examined are presented in Table 3.3.

Table 3.3 Implemented integrated control strategies

Number	Strategy	Explanation	Type
1	No Control	Base Case	-
2	FTRM	Fixed Time Ramp Metering	Stand alone
3	HSC	Harmonized Speed Control	
4	MP-ALINEA	Model Predictive ALINEA	
5	MP-VSL	Model Predictive Variable Speed Limit Control	
6	FTRM + HSC	Fixed time ramp metering with harmonized speed limit control	Combined
7	FTRM + MP-VSL	Fixed time ramp metering with Model Predictive Variable Speed Limit Control	
8	MP-ALINEA + HSC	Model Predictive ALINEA with harmonized speed limit control	
9	MP-ALINEA + MP-VSL	Model Predictive ALINEA with Model Predictive Variable Speed Limit Control	

3.6 Conclusions

This chapter mainly deals with the analysis of traffic control strategies considering the effects of traffic control in terms of efficiency and equity. FTRM and HSC strategies are developed as cost efficient alternatives to the existing control methods. Fixed-time ramp metering strategy needs only simple adjustments on ramp layout and a signal device would be sufficient to manage the control.

The admissible ramp flow under full traffic cycle metering policy usually much higher than the one / n cars per green metering policy and this represents the major advantage of the full-cycle policy. However, there is no established method for the specification of optimum cycle (length) time for fixed time ramp metering. Therefore, the traditional analytical models developed for fixed time intersection control are examined and capacity maximization approach is modified for fixed time ramp metering simulation experiment.

The proposed HSC system adjusts the set of displayed the speed limits for the congested peak periods where the speed limits are pre-determined that aims to minimize the selected measures of effectiveness for the examined corridor. The main difference of this control is the intervention to the speed limit boundaries. The conventional speed limit controls only deals with the upper speed limit. However the lower speeds are also considered as a critical part of an effective control and these limits are also needed to be examined. The research question focus is the investigation of the bounded speed limits effects on prevention of congestion and on congestion alleviation.

The optimality of the FTRM and HSC strategies are searched with respect to the minimization of the total travel times. A set of traffic simulation experiments is designed and applied in Chapters 5 and 6. The fixed-time ramp metering considers bottleneck capacity, thus from this point the control system works like demand-capacity control. The offline demands are measured and the maximum admissible ramp volumes transformed into the green times for the control. The speed harmonization control applies the speed limits more particularly the speed ranges for the whole network.

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CHAPTER 4. EQUITY: DEFINITION, CONCEPT AND MEASUREMENT IN TRAFFIC CONTROL

4.1 Introduction

In traffic control, in some cases it is observed that when the efficiency is targeted as the single objective, the consequences of traffic control may not be consistent with the socially sustainable transportation principles. Therefore, some efficient traffic control strategies would not be acceptable due to its negative impacts on equity. Thus, equity issues should be taken into account in traffic management and control. Before integrating equity issues to traffic control the broader definition of equity and specific meaning in traffic control should be put forward. Nevertheless, the definition and measurement of equity is one of the most controversial debates of humankind where a consensus is far over the horizon. There are some recent trials on integration of equity objectives in transportation planning and traffic engineering. The first studies of equity integration in transportation covered the fairness of transportation policies and later, road network design evaluated the equity issues. There are also some studies covered the equity issues in traffic control. The most common drawback of these researches is the single measure dependent nature, where most of them employed only GINI index. Hence, in this chapter the equity concept is extensively reviewed either in social sciences or in economics, later on the reflections of equity concept in transportation research is also reexamined. Following that, the motivation of development of an alternative equity concept is given by emphasizing the importance of indicator selection. Finally, the developed methodology is implemented on a generic corridor through traffic micro-simulation software and conclusions are drawn.

4.2 Equity and Equity in Transportation Studies

Equity has been the everlasting debate since the beginning of humankind and even it is the one of the biggest sources of each conflict on Earth. Not only humans but also the animals have a concept of equity (BRAUER et al, 2009). There are some other researchers also addressed the same issue by testing capuchin monkeys who exchanged tokens for food (Brosnan and de Waal, 2003). In their study the subject always received a cucumber for the token, however a competitor either received the same type of food (equity) or instead a more favored food for a token (inequity). It was found that subjects rejected potential exchanges for the less-preferred food when their competitors had received better food for the same token. Hence it can be concluded that equity is as a natural instinct rather than an engineered social concept.

Equity is a complex idea that resists simple formulations. It is strongly shaped by cultural values, by precedent, and by the specific types of goods and burdens being distributed. To understand what equity means in a given situation we must therefore look at the contextual details. It is a common way that a research begins with an explicit definition of the terms and objectives. However, before the definition of the 'equity' issue, the existence of the abstract subject should be examined first. According to Young (1995), the arguments against existence of equity take three different forms.

- *The first is that equity is merely a word that hypocritical people use to cloak self-interest. It has no intrinsic meaning and therefore fails to exist.*
- *The second argument is that, even if equity does exist in some notional sense, it is so hopelessly subjective that it cannot be analyzed scientifically. Thus it fails to exist in an objective sense.*
- *The third argument is that, even granting that equity might not be entirely subjective, there is no sensible theory about it, and certainly none that is compatible with modern welfare economics. In short, it fails to exist in an academic sense.*

Therefore according to the arguments above, it can be resolved that setting a quantitative and mathematical ground to that tricky problem is almost unlikely. However equity is, after all, an everyday concern and often used in everyday conversations such as equities of the taxation, the health system, pricing of services, transportation fares so on and so forth.

Equity in the sense of social order is concerned with the proper distribution of resources, rights, duties, opportunities, and obligations in society at large. This large topic has visited by many political philosophers since antiquity, from Plato's and Aristotle's conceptions of the ideal state, to the social contract theories of Hobbes, Locke, and Rousseau, to the more modern theories of Rawls, Nozick, and Walzer.

The equity theories can be considered as it is applicable only to social justice. However, there are ways to quantify and measure the equity, at least some portions of it in some other fields. In other words, it is possible to analyze the meaning of equity in the small without resolving what social justice means in the large. I would like to give the conceptual methodology on how the transportation supply distribution problem can be analyzed by giving a special emphasis on freeway traffic control strategies, in the subsequent parts of this chapter.

There are several definitions on equity but the most common ones are selected. According to the web dictionary (http://www.thefreedictionary.com/equity) the etymology of the word is given as early 14c., from O.Fr. equite (13c.), from L. aequitatem (nom. aequitas). Aequitas (genitive aequitatis) is the Latin concept of justice, equality, conformity, symmetry, or fairness. It is the origin of the English word equity. In ancient Rome, it could refer to either the legal concept of equity or fairness between individuals and in modern times the equity is mostly defined as the 'ideal of being just, impartial, and fair.'

4.2.1 Equity in Social Sciences and Economics

Equity is the concept or idea of fairness in economics, particularly as to taxation or welfare economics. More specifically it may refer to equal life chances regardless of identity, to provide all citizens with a basic minimum of income/goods/services or to increase funds and commitment for redistribution (Bird, 2009).

Inequality and inequities have significantly increased in recent decades, possibly driven by the worldwide economic processes of globalization, economic liberalization and integration (Jones, 2010). This has led to states 'lagging behind' on headline goals such as the Millennium Development Goals (MDGs) and different levels of inequity between states have been argued to have played a role in the impact of the global economic crisis of 2008-2009 (Jones, 2010).

Equity is based on the idea of moral equality that looks at the distribution of capital, goods and access to services throughout an economy and is often measured using tools such as the Gini index. Equity may be distinguished from economic efficiency in overall evaluation of social welfare. Equity is considered as a welfare distribution problem thus, it is studied as inequity aversion in economics.

Low levels of equity effects the quality of life of the society and may have direct impacts to the social exclusion and the resulting poor access to basic services and intergenerational poverty resulting in a negative effect on growth, financial instability, and crime and increasing political instability (Jones, 2010).

The state often plays a key role in the essential redistribution of the common wealth between all peers, stakeholders and citizens, but applying this in practice is highly complex and involves litigious choices. However, a great deal of consensus can be reached on three main issues; equal life chances, equal concern for people's needs and meritocracy (positions in society and rewards should reflect differences in effort and ability, based on fair competition).

Economists distinguish between three types of equities; universal equity measured through an inequity index, compensatory equity applies where universal equity cannot be achieved, and, status equity illustrated by the special care delivered to disabled (Deutsch,1975).

In public finance horizontal equity is the idea that people with a similar ability to pay taxes should pay the same or similar amounts. It is related to the concept of tax neutrality or the idea that the tax system should not discriminate between similar things or people, or unduly distort behavior (Musgrave, 1990).

Vertical equity usually refers to the idea that people with a greater ability to pay taxes should pay more. If the rich pay more in proportion to their income, this is known as a proportional tax; if they pay an increasing proportion, this is termed a progressive tax, sometimes associated with redistribution of wealth (Musgrave, 1990).

Horizontal equity means providing equal healthcare to those who are the same in a relevant respect (such as having the same 'need'). Vertical equity means treating differently those who are different in relevant respects (such as having different 'need'), (Culyer, 1995).

Equitability in fair division means every person's subjective valuation of their own share of some goods is the same. The surplus procedure (SP) achieves a more complex variant called proportional equitability. For more than 2 people a division cannot always both be equitable and envy-free (Brams et al, 2006).

An issue of social justice as the division of common property is highly important topic. Every society has rules for sharing goods and difficulties in distributing the goods among its members. Moreover, as economies become more complex, notions of common property become incomprehensible.

There are three general theories of justice that figure prominently in discussions about equity. The oldest and most prominent is Aristotle's equity principle, which states that goods should be divided in proportion to each claimant's contribution. A second theory of justice is classical utilitarianism, which asserts that goods should be distributed so as to maximize the total welfare of the claimants (the greatest good for the greatest number). A third approach to social justice that meets these objections to some extent is due to John Rawls (1971). It is impossible to do justice in this short space to Rawls's theory, which is intricately constructed and contains many subtle and important qualifications. However, the central distributive principle may be simply stated: the least well-off group in society should be made as well off as possible. This is known as the maximin or difference principle.

The conceptual difficulties posed by the utilitarian and Rawlsian principles have led some economists to adopt an entirely different approach to distributive justice. A distribution is said to be envy-free if no one prefers another's portion to his own. This concept does not require interpersonal comparisons of utility, because each person evaluates every other person's share in terms of his own utility function.

4.2.2 Equity measurement

The concept of inequality is distinct from that of poverty and fairness. Income inequality metrics or income distribution metrics are used by social scientists to measure the distribution of income, and economic inequality among the participants in a particular economy, such as that of a specific country or of the world in general.

Income distribution has always been a central concern of economic theory and economic policy. Classical economists such as Adam Smith, Thomas Malthus and David Ricardo were mainly concerned with factor income distribution, that is, the distribution of income between the main factors of production, land, labor and capital.

Modern economists have also addressed this issue, but have been more concerned with the distribution of income across individuals and households. Important theoretical and policy concerns include the relationship between income inequality and economic growth. In the economic literature on inequality four properties are generally postulated that any measure of inequality should satisfy:

i. Anonymity is an inequality metric that doesn't change with the individual's identity.
ii. Scale independence states that richer economies should not be automatically considered more unequal by construction.
iii. Transfer principle expresses that if some income is transferred from a rich person to a poor person then the measured inequality should not increase. Rather the measured level of inequality should decrease.

4.2.3 Equity Indexes

There have been many indexes proposed to calculate the equity or inequality in distribution. Perhaps the simplest measure of dispersion would be the range between the highest and lowest observations of a particular variable of interest. Given the fact that it is mathematically simple and easy to understand, it is a very limited measure due to the explanation of whole by using only two observations.

Range ratio can be calculated for a certain variable by dividing the value at a certain percentile (usually above the median) by the value at a lower percentile (usually below the median). One of the popular range ratios is the inter-quartile range ratio. Subtracting the observation at the 25th percentile by the observation at the 75th percentile results in a quantity known as the inter-quartile range, and dividing the observation at the 75th percentile by the 25th percentile calculates the inter-quartile range ratio. Range ratios can measure all sorts of inequalities and the percentiles can be constructed in any manner. A range ratio can take on any value between one and infinity, and smaller values reflect lower inequality. Range ratios are easy to understand and simple to compute however, like the range, range ratios only look at two distinct data points, throwing away the great majority of the data.

Another common index is the coefficient of variation which is simply the standard deviation of a variable divided by the mean. A distribution that is accumulated around the mean shows a high peak, and the coefficient of variation will be small. Therefore, it can be concluded that when the coefficient of variation gets smaller values this shows a more equitable distribution.

Among the most common metrics given in the Table 4.1 to measure inequality; the Gini index (also known as Gini coefficient), the Theil index, and the Hoover index will be briefly introduced.

The Gini coefficient is a measure of the inequality of a distribution, a value of 0 expressing total equality and a value of 1 maximal inequality. The Gini coefficient is usually defined mathematically based on the Lorenz curve, which plots the proportion of the total income of the population (y axis) that is cumulatively earned by the bottom x% of the population. The line at 45 degrees thus represents perfect equality of incomes. The Gini coefficient can then be thought as the ratio of the area that lies between the line of equality and the Lorenz curve (marked 'A' in the diagram) over the total area under the line of equality (marked 'A' and 'B' in the diagram); G=A/(A+B).

Table 4.1 : Common Inequality Metrics and Indexes

Measure	**Definition**	**Properties**[1] T	SI	TI
Range	$R = Y_{max} - Y_{min}$			
Variance	$V = \frac{1}{n}\sum_{i=1}^{n}\left(Y_i - \bar{Y}\right)^2$	+	−	−
Covariance	$c = \frac{\sqrt{V}}{\bar{Y}}$	±	+	−
Relative Mean Deviation	$M = \frac{1}{n}\sum_{i=1}^{n}\left\lvert\frac{Y_i}{\bar{Y}} - 1\right\rvert$	+	+	−
Logarithmic Variance	$\upsilon = \frac{1}{n}\sum_{i=1}^{n}\left(\log\left(\frac{Y_i}{\bar{Y}}\right)\right)^2$	−	+	−
Variance of Logarithms	$\upsilon_\iota = \frac{1}{n}\sum_{i=1}^{n}\left(\log\left(\frac{Y_i}{\bar{Y}}\right)\right)^2$	−	+	−
Gini	$G = \frac{1}{2n^2 \cdot \bar{Y}}\sum_{i=1}^{n}\sum_{j=1}^{n}\left\lvert Y_i - Y_j\right\rvert$	±	+	−
Theil	$T = \frac{1}{N}\sum_{i=1}^{N}\frac{Y_i}{\bar{Y}}\,log\left(\frac{Y_i}{\bar{Y}}\right)$	+	+	−
Atkinson	$A_\varepsilon = 1 - \left\lvert\frac{1}{n}\sum_{i=1}^{n}\left[\frac{Y_i}{\bar{Y}}\right]^{1-\varepsilon}\right\rvert^{\frac{1}{1-\varepsilon}}$	+	+	−
Kolm	$K_\alpha = \frac{1}{\alpha}\,log\left(\frac{1}{N}\sum_{i=1}^{n}exp\left(\alpha\left(\bar{Y} - Y_i\right)\right)\right)$	+	+	+

[1]T: Transfer, SI: Scale Invariance, TI: Translational Invariance.

The Gini coefficient can range from 0 to 1; it is sometimes expressed as a percentage ranging between 0 and 100. More specifically, the upper bound of the Gini coefficient equals 1 only in populations of infinite size. In a population of size N, the upper bound is equal to $1 - 2 / (N + 1)$.

The Gini coefficient's main advantage is that it is a measure of inequality by means of a ratio analysis. This makes it easily interpretable, and avoids references to a statistical average or position unrepresentative of most of the population, such as per capita income or gross domestic product. As a disadvantage, the Gini index only maps a number to the properties of a diagram, but the diagram itself is not based on any model of a distribution process. The "meaning" of the Gini index only can be understood empirically. Additionally the Gini does not capture where in the distribution the inequality occurs. As a result two very different distributions of income can have the same Gini index.

Figure 4.1: Lorenz Curve

A Theil index of 0 indicates perfect equality. A Theil index of 1 indicates that the distributional entropy of the system under investigation is almost similar to a system with an 82:18 distribution. This is slightly more inequal than the inequality in a system to which the "80:20 Pareto principle" applies. The Theil index can be transformed into an Atkinson index, which has a range between 0 and 1 (0% and 100%), where 0 indicates perfect equality and 1 (100%) indicates maximum inequality.

The Theil index is an entropy measure. As for any resource distribution and with reference to information theory, "maximum entropy" occurs once income earners cannot be distinguished by their resources, i.e. when there is perfect equality. In real societies people can be distinguished by their different resources, with the resources being incomes. The more "distinguishable" they are, the lower is the "actual entropy" of a system consisting of income and income earners. Also based on information theory, the gap between these two entropies can be called "redundancy". It behaves like negative entropy.

The Hoover index is the simplest of all inequality measures to calculate: It is the proportion of all income which would have to be redistributed to achieve a state of perfect equality. In a perfectly equal world, no resources would need to be redistributed to achieve equal distribution: a Hoover index of 0. In a world in which all income was received by just one family, almost 100% of that income would need to be redistributed (i.e., taken and given to other families) in order to achieve equality. The Hoover index then ranges between 0 and 1 (0% and 100%), where 0 indicates perfect equality and 1 (100%) indicates maximum inequality.

4.2.4 Equity in Transportation studies

Equity in transportation is mostly researched from the fairness point of view, which indicated the vertical equity. Transportation equity is neither a new concept nor a new goal. The history dates back to the beginning of urbanization. In recent years, many studies introduced the concept of equity mostly on social and economic justice contexts. For example, many low income group citizens who are seriously suffering the congestion either on freeways and public transport are demanding better and cheaper transportation alternatives. However, the claims on public transportation, freeways or local roads are not clear and the distribution of rights or priorities is of a big problem for authorities.

Transportation equity concerns extend to disparate outcomes in planning, operation and maintenance, and infrastructure development. Transportation is a key component in addressing poverty, unemployment, equal opportunity goals, and ensuring equal access to education, employment, and other public services. In the real world, all transportation decisions do not have the same impact on all groups. Costs and benefits associated with transportation developments are not randomly distributed. Transportation equity is concerned with factors that may create and or exacerbate inequities. Environmental justice focuses on measures to prevent or correct disparities in benefits and costs. Disparate transportation outcomes can be subsumed under three broad categories of inequity: procedural, geographic, and social.

Procedural inequity deals with the question that do the rules apply equally to everyone? In this respect, freeway traffic control treats some group of vehicles depending on their location and time differently. Therefore procedural inequity is one of the concerns in freeway traffic control. Geographic Inequity considers the distributive impacts of policies (positive and negative) that are geographic and spatial, such as rural vs. urban vs. central city. Some communities are physically located on the “wrong side of the tracks” and often receive substandard services. There are some group of people, especially living close to the controlled ramps and city centers, or some specific spatial areas are affected more in comparison to some others.

Social inequity questions the distribution of transportation benefits across the population. Generally, transportation benefits fall to the wealthier and more educated segment of society, while transportation burdens diminish disproportionately to the poor and less educated part of the society. For traffic control, within the context of geographic equity some social groups with different social structures can be affected more than others depending on the impacted region.

These factors significantly affect equity evaluation. Analysis conclusions may change depending on how people are categorized, which impacts are considered, and how they are measured. There is no single correct way to evaluate transportation equity. It is generally best to consider various perspectives, impacts and analysis methods.

Evaluation of transportation equity is not unidirectional and it depends on the type of equity, how people are categorized, which impacts are considered and how they are measured, as summarized in Table 4.2.

Some examples of transportation equity topics, types, categories, impacts and measurement units are given in Table 4.3. Freeway traffic control can be evaluated under almost in every type, category, and impact with several different measurement units.

Table 4.2: Equity Evaluation Variables

Types of Equity	Categories	Impacts	Measurement Units
• Horizontal • Vertical wrt income • Vertical wrt disability	• Demographics • Income • Geographic location. • Disablity • Mode • Vehicle type • Trip characteristics	• Price or fare structure. • Tax • Service quality • External costs • Economic	• Per capita. • Per veh-km • Per pass-km • Per trip. • Per peak-period trip. • Per subsidy

There is also various numbers of studies on equity in transportation, some of them listed at Table 4.4. According to Litman, (2009) horizontal equity concerns the distribution of impacts between individuals and groups considered equal in ability and need. According to this definition, equal individuals and groups should receive equal shares of resources, bear equal costs, and in other ways be treated the same. It means that public policies should avoid favoring one individual or group over others.

Table 4.3: Equity Evaluation Cases

Case	Type of Equity	Category	Impact	Measurement Unit
Highway cost allocation	Horizontal equity	Vehicle type	Financial costs	Per veh-km
Environmental justice	Vertical wrt income	Income	Market and non-market costs	Per capita
Freeway traffic control	Horizontal Vertical wrt income Vertical wrt disability	Demographics Income Geographic location. Disability Mode Vehicle type Trip characteristics	Tax Service quality External costs Economic	Per capita. Per veh-km Per pass-km Per trip. Per peak-period trip. Per subsidy

Vertical equity concerned with the distribution of impacts between individuals and groups that differ in abilities and needs, in this case, by income or social class. Therefore, transport policies should favor economically and socially disadvantaged groups to be considered as equitable (Litman, 2009).

Horizontal equity simply requires that the consumers (users) pay the costs imposed by their activities. Higher fuel tax, road and parking pricing, and distance based fees, can increase equity by making prices more accurately reflect costs, taking into account factors such as vehicle type, time and location (Litman, 2005).

Transportation price increases are often criticized as being regressive, since a particular fee represents a greater portion of income for lower-income people than for higher income people. Overall equity impacts depend on how prices are structured, the quality of transport alternatives available (Golub 2010), how revenues are used, and whether driving is considered a necessity or a luxury (Litman 1996; Rajé 2003). If there are good alternatives, revenues are used to benefit the poor, and disadvantaged people are given discounts, price increases can be progressive overall.

There is a long history of incorporating vertical equity objectives into transport pricing with targeted discounts that benefit lower-income people. Adam Smith (1976), the founder of modern economics, wrote that, "*When the toll upon carriages of luxury coaches, post chaises, etc. is made somewhat higher in proportion to their weight than upon carriages of necessary use, such as carts, wagons, and the indolence and vanity of the rich is made to contribute in a very easy manner to the relief of the poor, by rendering cheaper the transportation of heavy goods to all the different parts of the country*."

There is often debate over the equity of road and parking pricing, particularly when fees are introduced on previously underpriced, mostly free, facilities. The drivers are asked to pay the external cost they contributed but there are difficulties in determination of the actual external or marginal cost and the distribution of the cost among the users.

Critics argue that road pricing represents "double taxation" since they already pay fuel taxes that fund roads. However, road and parking pricing is usually applied in areas where the costs of providing facilities is particularly high, such as in city centers and new highways. Such fees can be considered a surcharge for these higher-than-average costs.

Table 4.4: Equity studies in transportation literature

Equity Studies	**Authors**	**Year**
Mobility for		
Disadvantaged Groups	Stanley, *et al*	2011
Women's Employment Access	Dobbs	2005
Public Funding Allocation	Chen	1996
Non-Drivers Accessibility	Case	2011
Inclusive Planning Analysis	Mann	2011
Transportation Improvement		
Benefit Distribution	Fruin and Sriraj	2005
Parking Requirement	Litman	2005
Transportation Cost Analysis	FHWA	1997
Transportation Cost Burdens	BLS	2000
Traffic Impacts	VTPI	2005
Planning Biases	Beimborn and Puentes	2003
Economic Opportunity	Gao and Johnston	2009
Transportation Pricing	TRB	2011
Spatial Analysis	Rodier, *et al*	2010
Climate Change		
Emission Reduction	Lin	2008
VMT Reduction Strategies	Carlson and Howard	2010
Road Funding	Schweitzer and Taylor	2008
Fairness in a Car		
Dependent Society	SDC	2011
Right To Basic Transport	KOTI	2011

Pricing proponents emphasize that motorists receive benefits, such as reduced traffic congestion, and that pricing is optional. For example, motorists may have a choice between free but congested highway lanes, and uncongested but priced lanes. Similarly, they may be able to choose between convenient but priced parking, and less convenient but free parking. This is called value pricing. Whether motorists have adequate alternatives is often an important issue in pricing equity analysis.

Pricing reforms can benefit disadvantaged people if they reduce negative impacts on disadvantaged neighborhoods or improve travel options for non-drivers. For example, Kain

(1994) predicts that congestion pricing can benefit lower income commuters and on-drivers overall by improving transit and rideshare services. Cameron (1994) concludes that a 5¢ per mile road user fee in Southern California is not regressive because all residents benefit from reduced congestion and pollution.

The report, Equity Analysis of Land Use and Transport Plans Using an Integrated Spatial Model (Rodier, et al. 2010), used the Activity Allocation Module of the PECAS (Production, Exchange, and Consumption Allocation) Model to evaluate the equity effects of land use and transport policies intended to reduce greenhouse gas emissions.

4.3 Equity in freeway traffic control

In fall 2000, the ramp meters in the Twin Cities were turned off for 8 weeks so that an assessment of their effectiveness could be made. Although this assessment focused on the efficiency of the system, considering mobility and safety particularly, a transportation equity analysis of the delay distribution across space could also be made. The relationship is estimated between mobility and equity for O-D pairs on Route 169, a suburb-to-suburb limited access highway connecting the north and south legs of the region's beltway, with and without ramp meters.

In this discussion the equity and mobility for specific trips are considered. The mobility measures include total travel time, speed, and delay. All the measures are computed for each 5-min interval for each OD pair. Since the number of trips on each OD pair is not available, all the averaging is done without weighting by number of trips between origin and destination. The analysis is performed for 105 OD pairs. Figure 4.2 shows the equity and speed profiles in peak hours with and without metering.

What ramp meters bring in terms of mobility and equity can be shown by the comparison of the two cases. Previous research indicates that ramp meters can increase the mobility of freeway networks, which is confirmed by the findings here. With ramp metering, the average travel speed (taking ramp delay into account) of the highway increases from 60 mph to 100 mph (37 km/h to 62 km/h); travel delay per mile decreases from 136 s to 112.5 s and the average travel time for one trip decreases from 610 s to 330 s. Another interesting result of this study is the Gini coefficient decrease without ramp metering. This result suggests that the system becomes fairer when meters are removed.

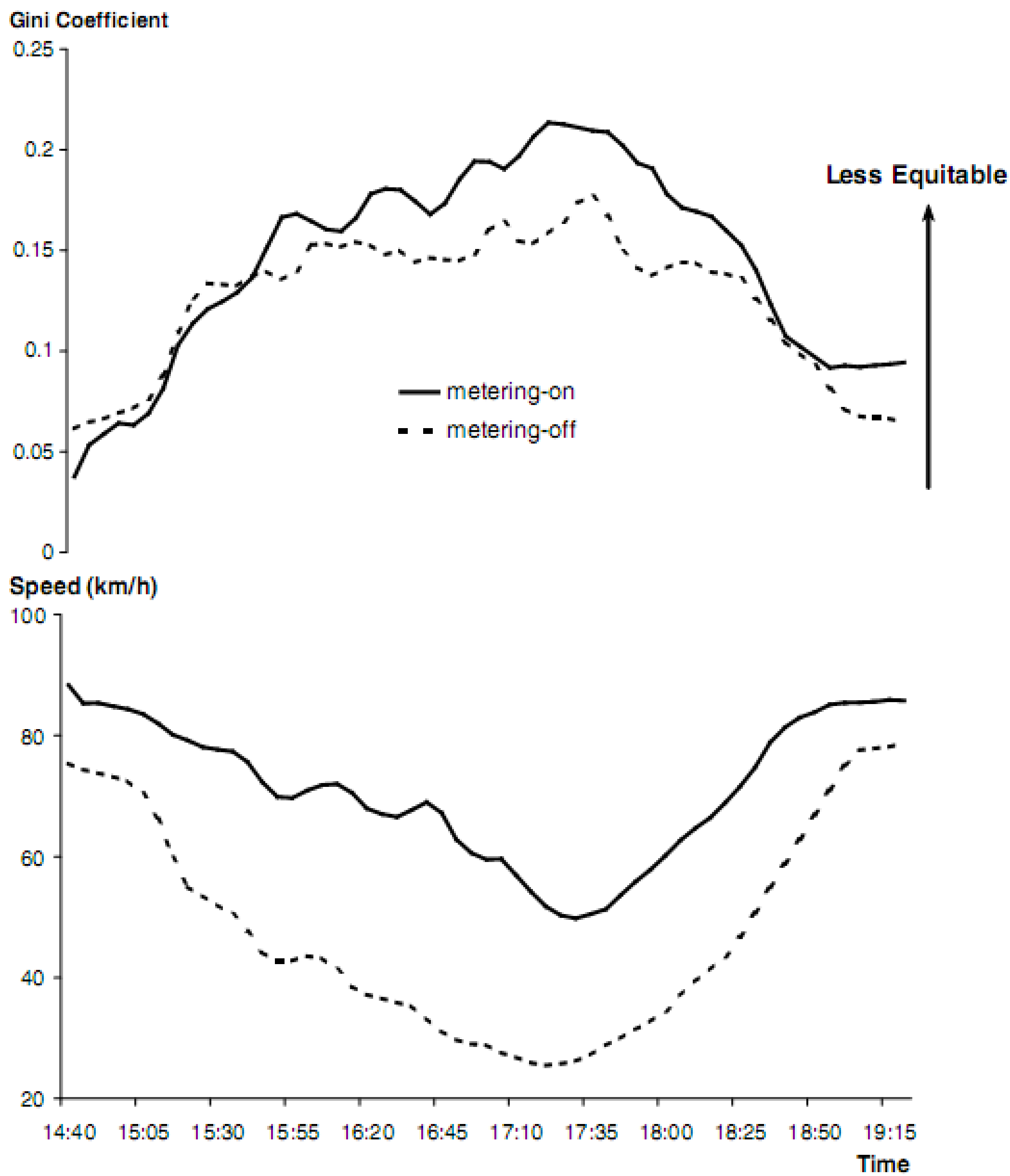


Figure 4.2: Relationship between temporal Gini coefficient and speed (Levinson and Zhang, 2004)

Figure 4.3 illustrates the trends in the change of mobility and equity with and without metering for trips. It should be noted that in Figure 6, the shortest trips (those on the right side of the graph) actually are hurt in mobility terms by ramp metering, whereas the longest trips (those on the left side) benefit the most.

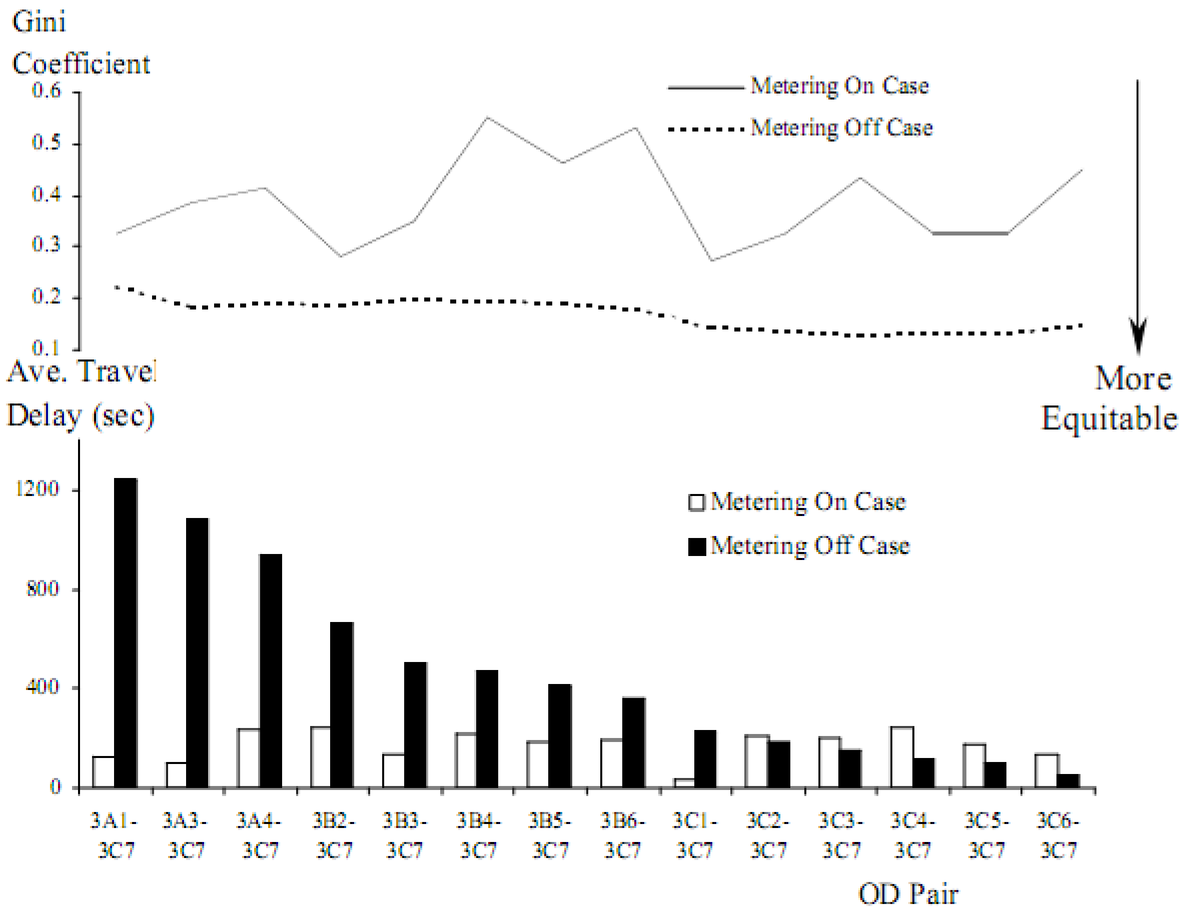


Figure 4.3: Relationship between temporal equity and mobility (travel delay), Route 169 (Levinson and Zhang, 2004).

Papageorgiou, 2004 implemented the Advanced Motorway Optimal Control (AMOC) strategy for optimal freeway network-wide ramp metering applied to the ring-road of Amsterdam, The Netherlands, investigated the interesting problems arising in ubiquitous ramp metering systems. A number of suitably chosen scenarios along with a thorough analysis, interpretation, and suitable visualization of the obtained results provide a basis for the better understanding of some complex interrelationships of competing performance criteria. The strategy's efficiency and equity properties as well as their trade-off are studied and their partially competitive behavior is discussed.

Scenario 0 is the base case and corresponds to the no-control case. In scenario 1; the maximum permissible storage is set to 40 vehicles for the urban on-ramps and 100 vehicles for the freeway-to-freeway ramps; in scenario 2; these storage capacities are equal to 80 and 200 vehicles, respectively, while in scenario 3 no maximum queue constraints are considered. Finally, considered in scenario 4; the maximum queue length allowed on-ramps is 40 vehicles which is applied at freeway to freeway ramps.

This trade-off is implicitly addressed by then AMOC strategy through consideration of the available ramp storage space and may be used as a tool to establish a desired policy of the systems' efficiency versus equity. Table 4.5 shows the Travel Time Savings (TTS), Total Travel Time (TTT) and Total Waiting Time (TWT) achieved by each control strategy.

Table 4.5: TTS, TTT and TWT for each scenario (Kotsialos and Papageorgiou, 2004)

Scenario #	TTS veh.h	TTT veh.h	TWT veh.h
0	13226	10832	2394
1	9032	7712	1320
2	7930	6619	1311
3	7466	6247	1219
4	10383	8682	1701

In this study the equity is defined as the variance of average travel times which is set as the travel time starting at each on-ramp needed for queuing and traveling a 6.5km mainstream distance.

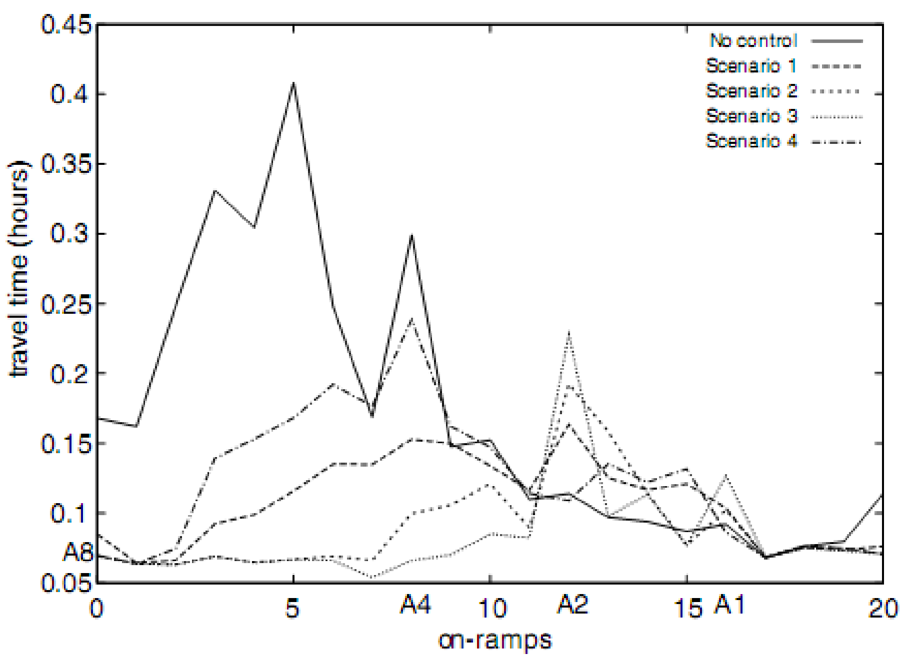


Figure 4.4: Average travel times at each on-ramp (Kotsialos and Papageorgiou, 2004)

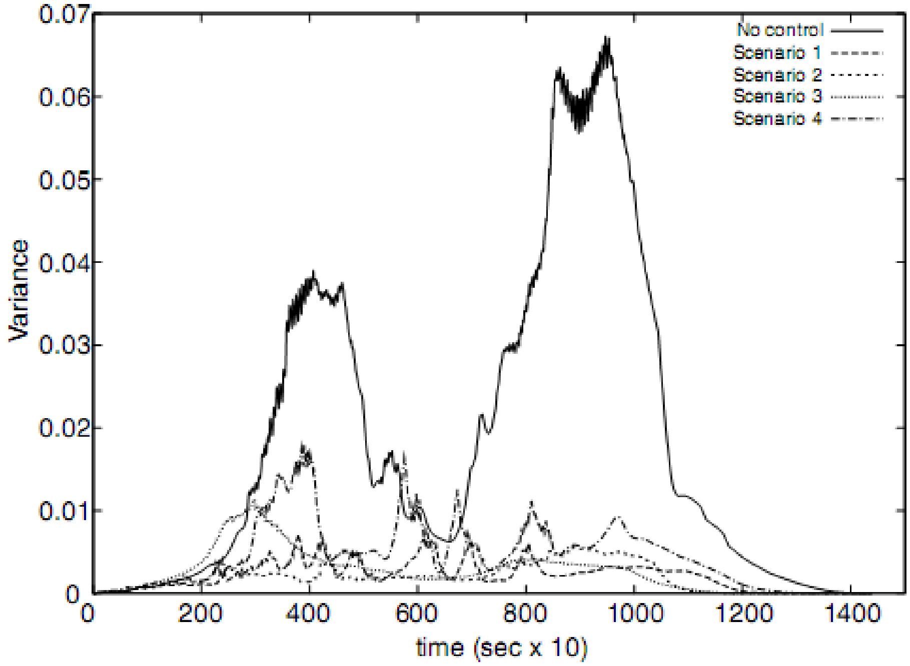


Figure 4.5: Variance of travel times (Kotsialos and Papageorgiou, 2004)

Under these terms, Figures 4.5 illustrate also the competitive nature of equity versus efficiency. Scenario 3 is the most efficient achieving a 43.5% improvement on the TTS, but the most unfair one (excluding scenario 4); scenario 1 is the most fair of the considered scenarios but at the cost of achieving 31.7% reduction of the TTS; finally, scenario 2 is in the middle of scenarios 1 and 3 concerning efficiency and equity. Scenario 4 is the most unfair one from all the considered scenarios due to the uncontrolled on-ramps.

4.4 Development of a new equity index for traffic control evaluation

The transportation research and the practice usually concern the efficiency of transportation policies; however the equity issue is also an integral part of the research and practice. On the other hand, as it is mentioned above there is neither a straightforward definition of the concept of nor there is a consensus on the measurement. Therefore, the main drawbacks from the equity of FTC can be summarized as;

i. Conceptualization of equity and the values (min, mean, max and thresholds) differs among researchers. This also yields transferability and reliability concerns in a broader view.

ii. Especially on last two example studies the researchers focused on either only network or only ramp zones which cannot satisfies the fully comprehension of the equity distribution among different user groups (mainline and ramp) and also different control strategies (ex ante).

iii. Widely used Gini coefficient doesn't represent the complete equality in distribution. Therefore it can be said that it is biased. For example, the proportion of the average ramp speeds (delay, travel time, waiting time, number of stop, etc) over average mainline speeds cannot be captured by Gini Index.

iv. Ramp metering greatly contributes the smoothing of traffic flow which also minimizes the speed variance. Therefore, utilizing Gini coefficient would be biased since the smoothing effect of RM suppress the speed variance even though the average speed differences can vary at each segments of the network.

v. The design speed of highway stretches varies with respect to the geometric standards and functions such as mainline, on-ramps and off-ramps. The equity of different stretches of a highway should be based on link type properties for an unbiased comparison.

vi. The indicators are also affecting the results i.e., the equities calculated through the use of delay, travel time and speed would be different.

vii. The delay which is commonly used in equity measurement cannot be used because of a mathematical fallacy. The reason validity fails with a division by zero is hidden in the denominator; when the average delay is equal to zero and any vehicles are not experiencing delay the value yields to the 0/0 indeterminate form. Therefore, the delay values of each vehicle cannot be taken for the Gini type index calculations as it is performed by Levinson, Meng and other several others.

In conclusion, it is noticed that the previous researches still did not have strong emphasis on the conceptualization of equity in traffic control where most of the indices are barrowed from econometric studies that are usually incorporating the differences of pairs among the mean values. There are also some other problems detected in terms of mathematical formulation of equity, for instance when the delay values are incorporated with Gini Index this would yield 0/0 indeterminate form if there is no delay in traffic.

Consequently, the reasons above indicate the motive of a novel equity concept and an appropriate metric establishment especially for evaluating the equity of traffic network control. The definition and properties of the new index is given in the following section.

4.4.1 Definition and properties of K index

To overcome the drawbacks explained in previous section, a new equity index, namely K, is proposed to assess the equity of traffic considering the mathematical and theoretical subjects. The conceptualization of equity in K index is based on the notion that the ratio of the speed differences (vehicles or time frames) over a specific value which reflects the optimal flow. The equity index is structured on the notion that it should depend on the differences among the peers but not the mean value of the peers.

The K index uses the free flow speed or the optimum speed values instead of the mean speed that is used in Gini index. From the assumptions of Gini; the mean must be greater than zero which may not be valid in traffic flow when the delay measurements are incorporated. It is possible to have two different traffic situations may have the same Gini values with different average speeds. On the other hand, the K equity index will be always the same when the sum of the differences and free-flow or optimal speed is the same.

When Gini index used as a measure of traffic equity measure, the most unequal traffic state will be one in which a single driver receives 100% of the delay, which is practically meaningless, and the remaining drivers experience no delay (G=1); and the most equal traffic state will be one in which every driver receives the same delay, regardless of the magnitude of delay (G=0).

In order to overcome the shortcomings of Gini in measuring the equity of traffic state some modifications are proposed in terms of the value of mean value, the normalization of the measure and the indicator selection. Therefore, the mathematical form for a mainline section of a highway is given in 4.1 as;

$$\mathrm{K_m} = \frac{1}{2nu_{ff_m}} \sum_{\mathrm{i}=1}^{\mathrm{n}} \sum_{j=1}^{n} \left(\left| \mathrm{u_{i_m}} - \mathrm{u_{j_m}} \right| \right) \tag{4.1}$$

where; $\boldsymbol{u_{ff_m}}$ is the free-flow speed of the mainline section and $\mathbf{u}_{\mathbf{i}_\mathbf{m}} - \mathbf{u}_{\mathbf{j}_\mathbf{m}}$ are the measured speeds of two vehicles on the mainline of a highway.

The distinctiveness of the K index compared to other indices is therefore twofold. Firstly, the index measures the ratio of the sum of the speed differences of each vehicle and the free-flow speed, which indicates the equity of the delay in the network. Secondly, when the denominator part is altered with the optimum speed $\boldsymbol{u_{op}}$ where the capacity is reached, then the index shows the equity of speed in the network.

As it is the case for the Gini index, the delay values cannot be directly used in K index due to the possible 0/0 indeterminate form. However, given the definition of delay, the equity of delay can be determined through the free-flow speed and actual speeds.

Since the K index have similarities in its form, the smaller values indicates better equity (more equal traffic state) and whereas the higher value points indicate more inequality in traffic state. Similar to Gini, the K index also doesn’t deal with the magnitude of indicator which is related with the form and definition of equity. When there is no speed difference between each vehicle in the traffic this indicates the perfect equity with the value K=0 and the upper limit of the index determined by the free-flow or optimum speeds. As a small example for the upper boundary of K index, let’s consider two vehicles having a speed of 110 km/h and 10 km/h where the free-flow speed is 100km/h. Then the K index yields a value of K=0.25 which can be considered as the highest inequality limit. The main difference of Gini and K is; with K index it is possible to compare the equity of different stretches of a highway without a bias. Since the design, free-flow and the optimum speeds are different among different stretches, the equity index should have taken these differences into account. For example the free-flow speeds could be around 120 km/h on an alignment section of a basic freeway segment whereas the free-flow speed an on-ramp would only reach 60 km/h. While calculating the equity of these two sections of a highway by using Gini the results would be in favor of mainline section since the average speed has a higher value. On the other hand, the K index distinguishes the link properties of the highway and this provides a more realistic comparison bases.

The transfer principle is the assumption states that if some gain is transferred from one peer to another, while still preserving the order of the ranks, then the measured inequality should not increase. In its strong form of this principle the measured level of inequality should decrease. The

K index can be considered as it satisfies the transfer principle. However, K index doesn't satisfy the scale invariance and translational invariance principles.

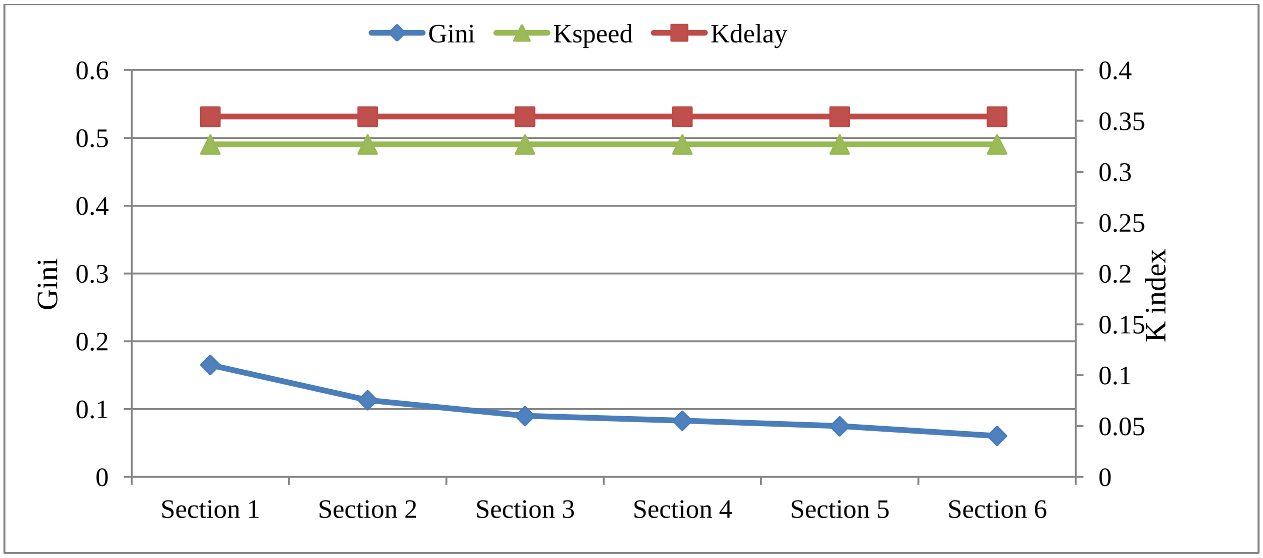
In table 4.6 hypothetical values are shown where those represent the 15 minute average speeds of the morning peak hours from 08.00 to 10.00 am for six different sections along a highway. The sums of the differences of the average speeds are the same for all the sections but the average speed values varies. The free-flow speed is taken as 90 km/h and the optimum speed is selected as 65 km/h for all the sections.

Table 4.6: Comparison of Gini and K index with hypothetical speed values

Time	**Section 1**	**Section 2**	**Section 3**	**Section 4**	**Section 5**	**Section 6**
08.00 -08.15	26.0	34.0	45	45	45	55
08.15 - 08.30	27.0	34.0	40	45	45	60
08.30 - 08.45	36.0	43.8	55	60	55	65
08.45 - 09.00	29.0	35.0	55	60	45	60
09.00 - 09.15	18.0	27.0	45	50	62	73
09.15 - 09.30	14.0	30.0	35	40	55	65
09.30 - 09.45	28.0	50.0	45	45	55	77
09.45 - 10.00	15.0	28.0	35	40	65	72
Average (km/h)	24.1	35.2	44.4	48.1	53.4	65.9
Sum of differences	510.0	510.0	510.0	510.0	510.0	510.0
Gini	0.165	0.113	0.089	0.082	0.074	0.060
K_{delay}	0.354	0.354	0.354	0.354	0.354	0.354
K_{speed}	0.490	0.490	0.490	0.490	0.490	0.490

In Figure 4.6 it can be seen that, the Gini index decreases with respect to the increase in average speed values even though the sum of the differences among the peers remains constant for all the sections. The differences among the peers should be the determinant part for the equity analyses however when the Gini is used to evaluate the equity as in this case, it wouldn't be possible to comprehend the values without knowing the average values. On the other hand, the K_{delay} and K_{speed} values are not changed with the change of the average since this metric takes the free-flow and optimum section speeds into account. Therefore, K index is more meaningful than other indices since it regards the physical characteristics of a highway section which enables to make a more meaningful comparison.

Figure 4.6: The changes in equity with respect to average values in Gini and K



4.4.2 Traffic micro-simulation of a generic corridor as an explanatory example

A simple generic network consists of two onramps and an off ramp located in between is created for the analyses can be seen in Figure 4.7. The mainline has 3 lanes with the width of 3.5m and ramps are single lane having the same width. The links are selected as urban (motorized) type in Vissim.

Table 4.7: Vehicle Inputs and Time Dependent O-D Matrix

Decision No.	Route No.	Destination Link	Length [m]	Hourly Volumes (sec)		
				0 - 3600	3600 - 7200	7200 - 10800
1	1	3	270.71	800	900	850
1	1	5	2885.12	3200	3300	3250
2	2	3	274.216	400	700	450
2	2	5	2884.28	800	900	950
3	4	5	2884.65	1200	1600	1300

The data is generated by considering the morning peak hours. (V/C varies between 0.9 and 1.05). The traffic composition is set as 80% of vehicles are automobiles, 10% HGV and 10%Bus with the desired speeds of 80km/h (75.0,110.0), 50km/h(48.0,58.0) and 60km/h(58.0,68.0) respectively. Volumes are selected as to have the same downstream traffic at the upstream of bottleneck locations. The volumes inserted are given at Table 4.7 and Figure 4.8.

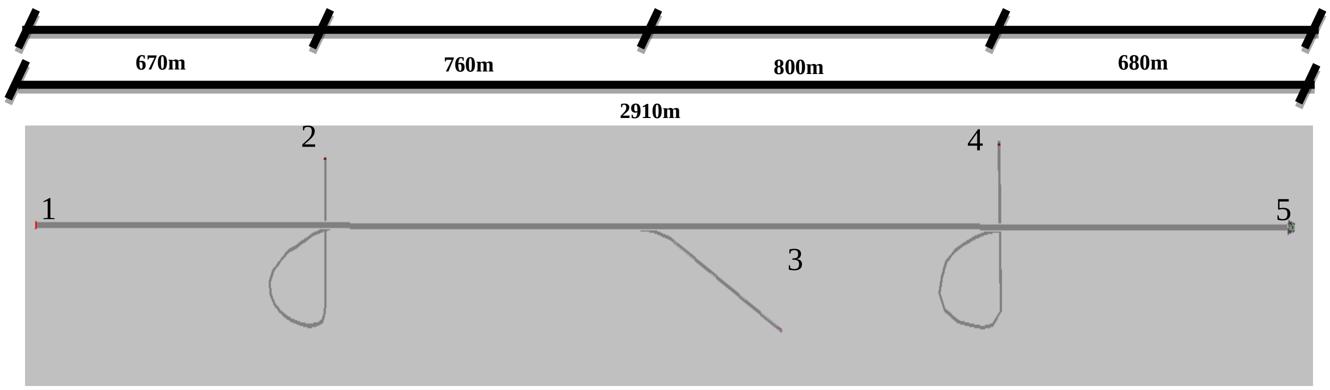


Figure 4.7: Generic Network

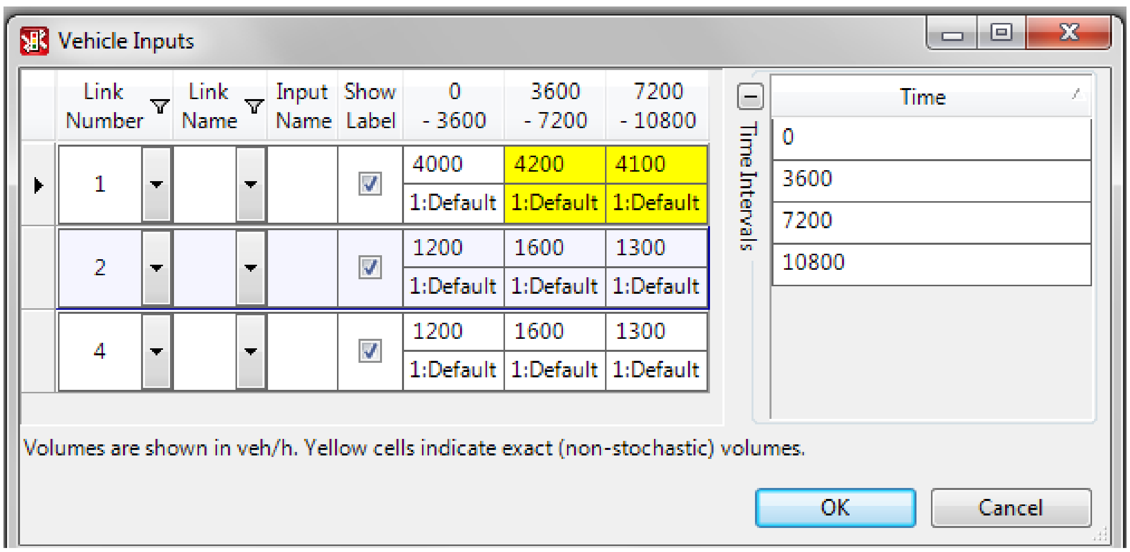


Figure 4.8: Hourly Exact Volumes

The data collection points are placed to collect data for evaluations of number of vehicles, occupancy rate, and maximum and mean queue delay time, and minimum and mean speed for all vehicle types (Figure 4.9). The evaluation files are selected with data collection points, network performance analyses and travel time measurement. Travel time data is collected with 15min intervals for the entire simulation period.

The measures of effectiveness are selected as; average delay time per vehicle in seconds, average number of stops per vehicles average speed in km/h, average stopped delay per vehicle in seconds, total delay time in hours, total distance traveled in kilometers, total stopped delay in hours, total travel time in hours, number of stops, number of vehicles in the network and number of vehicles that have left the network. The screenshot views are given in Figure 4.9.

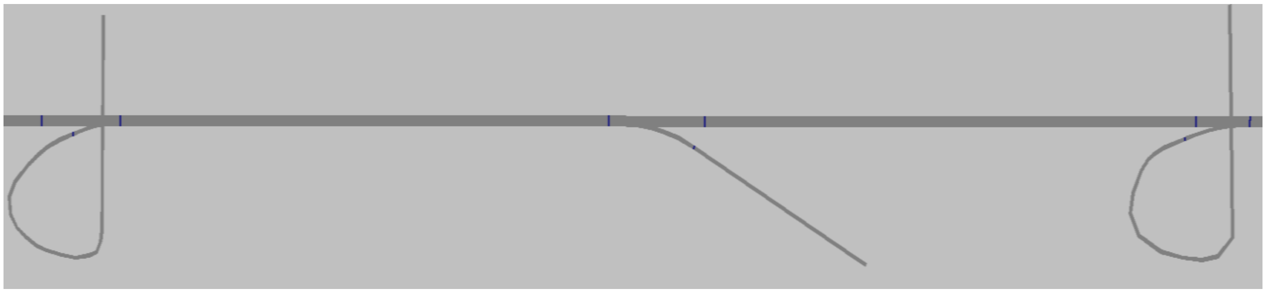


Figure 4.9: Data Collection Points

The freeway traffic controls proposed in Chapter 3 are applied for the analyses. Firstly FTRM control is applied for every isolated entrance. At each ramp, the signal cycle time is set 10 sec with the 5sec green and 5sec red time. Later, SHC is employed for mainline speed management. For the speed management the speed limit of 70 km/h is employed on the network. Speed restriction areas are defined along the corridor in order to implement speed control. The speed distribution is set as default for the simplicity and comparability. Finally, proposed control strategies; FTRM and SHC are combined for the integrated control scenario. In integrated control strategy in this study the fixed time ramp metering control and speed management are implemented independently.

Two levels of analyses are presented; first one is network level performance measures' analyses and the second one is on equity performance of the control strategies. The efficiency performance results are shown in Table 4.8. and Figure 4.11. It can be clearly seen that with all control strategies the average speeds increased, the total travel time decreased and the average delay time per vehicle decreased exclusively.

However, with FTRM the average number of stops per vehicles increased due to the stop and go mechanism of control. Despite using the same cycle time and green split in integrated control the use of HSC improved the number of stops even better than the base case. The total delay time is substantially decreased with all the control strategies implemented. The largest decrease is achieved with the integrated control where the total delay time is decreased almost 50%.

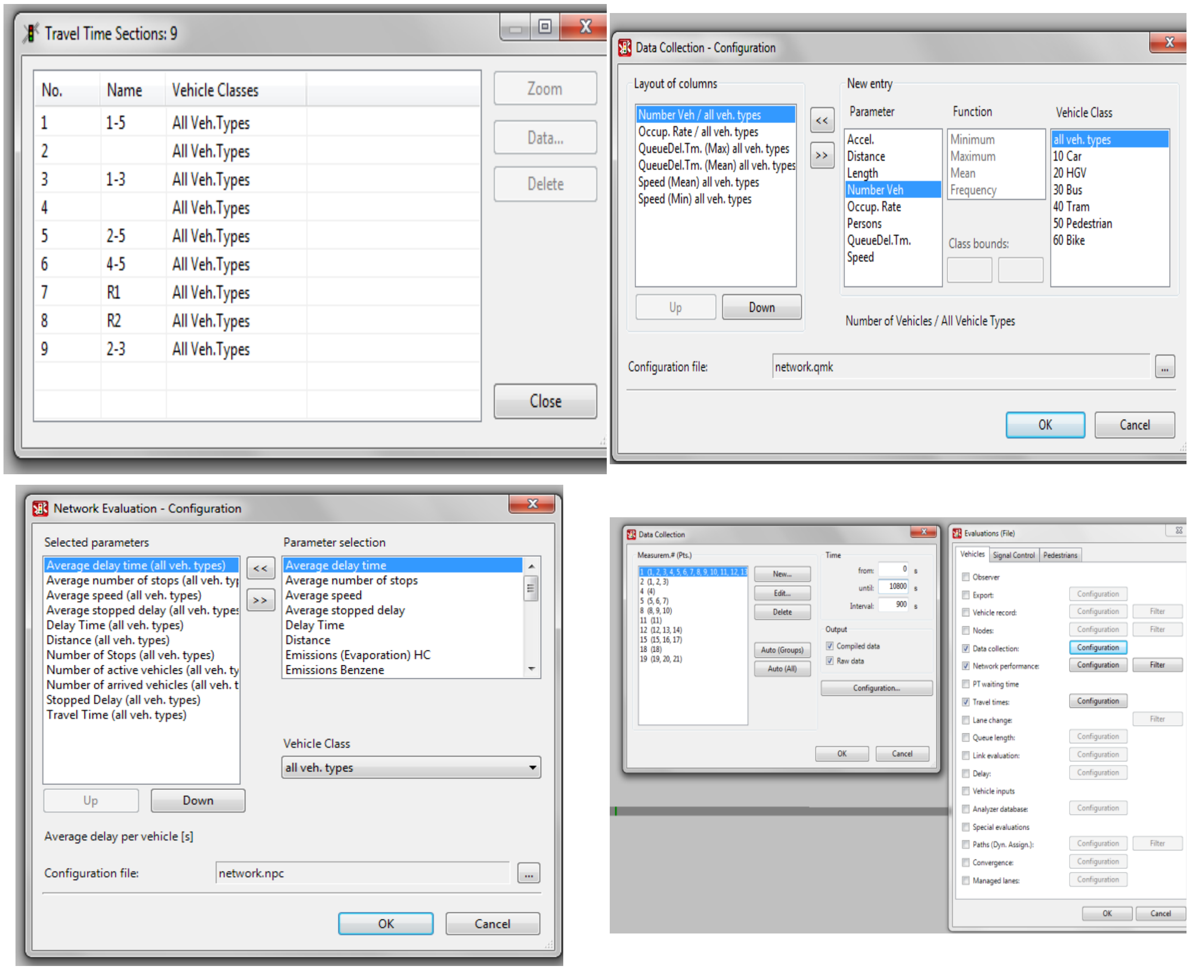


Figure 4.10: Settings for Evaluation of Travel Time and Network Performance

Table 4.8: Network Performance with Control Strategies

Measures of Efficiency	No control	Ramp Metering	Speed Management	Integrated Control
Average delay time per vehicle [s]	91.335	74.637	57.9	47.672
Average number of stops per vehicles	0.139	0.142	0.119	0.124
Average speed [km/h]	42.113	46.783	46.471	49.95
Average stopped delay per vehicle [s]	0.173	0.186	0.143	0.164
Total delay time [h]	453.375	356.31	293.004	228.415
Total Distance Traveled [km]	41106.165	41147.006	41691.198	41279.29
Number of Stops	2489	2443	2166	2144
Number of vehicles in the network	333	282	320	279
Number of vehicles that have left the network	17537	16904	17898	16970
Total stopped delay [h]	0.859	0.889	0.724	0.786
Total travel time [h]	976.101	879.52	897.139	826.414

The total distances traveled are slightly increased with the implementation of traffic control strategies. However the changes compared to the other measures of efficiency are miniscule. The biggest achievement attained with the integrated control where the gain is only about 0.5%.

The total stopped delays are increased with the ramp metering control strategy due to the control nature and the best performance observed for the speed limit control when the total stopped delays are taken into consideration. Total travel times are fairly decreased with the traffic control strategies and the largest decreased is detected with the integrated control as 9%.

The indexes are calculated according to the speeds collected from the detectors defined on the corridor. The free-flow speed is taken as 110 km/h and the optimum speed is taken as 70 km/h. The equity performances of the FTCS are given in Tables 4.9 – 4.12 and a comparison of equity performances with K index is presented in Figure 4.12.

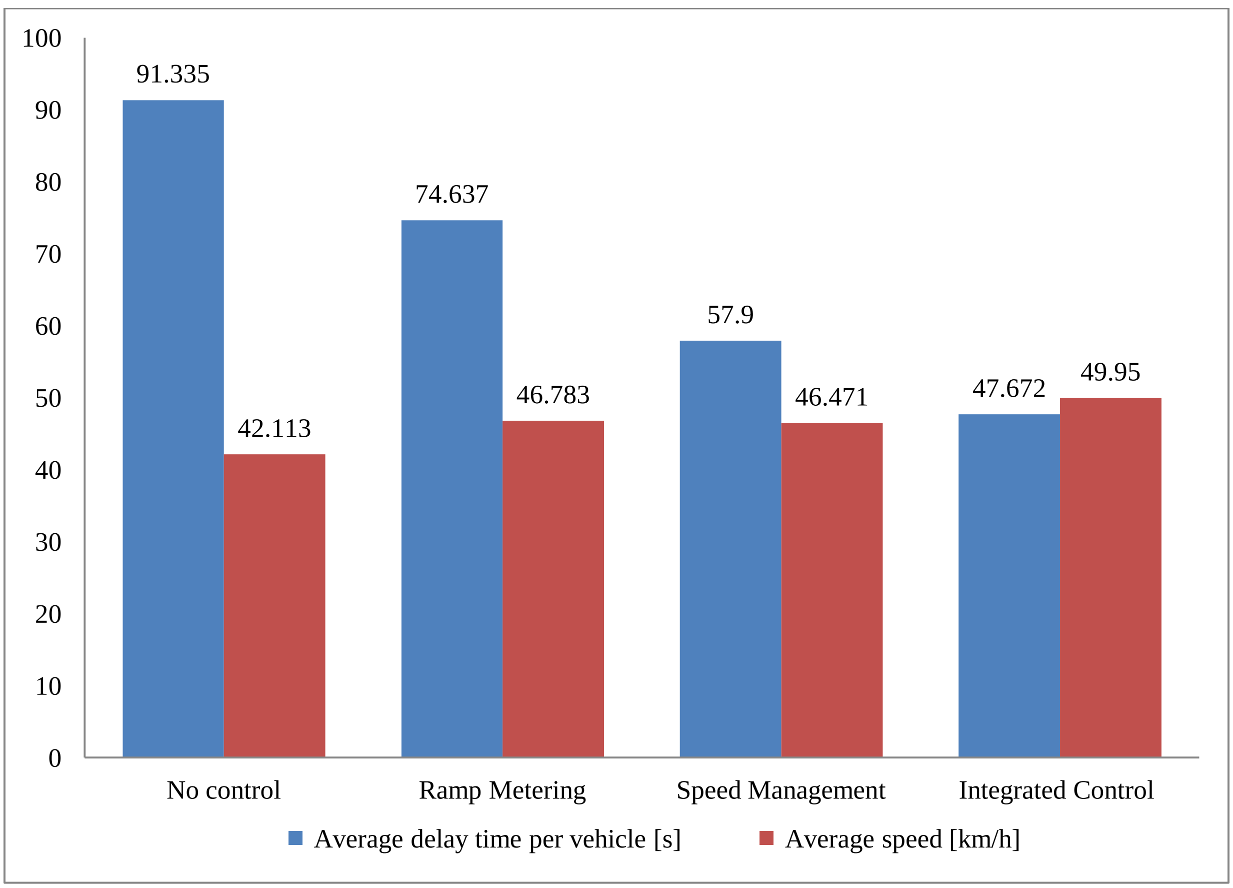


Figure 4.11: Network Performance with Control Strategies

Table 4.9: Equity Indexes of Base Case (No Control)

Indexes	**1-> 5**	**2**	**1 -> 3**	**4**	**2-> 5**	**4-> 5**	**R1**	**R2**	**2->3**
Range	14.940	28.558	18.277	17.406	18.765	22.457	9.741	26.971	13.278
Variance	27.598	83.715	30.703	36.543	25.017	42.293	6.823	60.109	12.597
Coefficient of Variance	0.100	0.297	0.176	0.120	0.168	0.157	0.255	0.244	0.168
Relative Mean Deviation	0.091	0.412	0.399	0.114	0.433	0.214	0.805	0.394	0.596
Logarithmic Variance	0.002	0.074	0.056	0.003	0.067	0.016	0.524	0.064	0.162
Variance of Logarithms	2.197	1.543	1.585	2.148	1.523	1.900	0.592	1.582	1.181
K index	0.054	0.163	0.149	0.064	0.176	0.125	0.173	0.126	0.115
Theil's Entropy	0.002	0.018	0.006	0.003	0.005	0.005	0.012	0.012676	0.005334
Atkinson	0.022	0.125	0.051	0.030	0.034	0.059	0.060	0.125557	0.033895
Kolm	0.498	1.210	0.493	0.636	0.336	0.815	0.112	1.064934	0.19111

Table 4.10: Equity Indexes of FTRM

Indexes	**1-> 5**	**2**	**1 -> 3**	**4**	**2-> 5**	**4-> 5**	**R1**	**R2**	**2->3**
Range	4.497	11.498	10.050	4.279	8.732	9.061	4.366	6.420	7.040
Variance	2.409	13.516	10.036	2.505	5.182	5.845	1.309	2.929	3.509
Coefficient of Variance	0.025	0.061	0.060	0.025	0.072	0.118	0.101	0.139	0.082
Relative Mean Deviation	0.021	0.062	0.153	0.020	0.495	0.673	0.820	0.804	0.633
Logarithmic Variance	0.000	0.001	0.006	0.000	0.089	0.240	0.559	0.507	0.192
Variance of Logarithms	2.379	2.317	2.160	2.387	1.550	1.114	0.635	0.695	1.223
K index	0.013	0.033	0.083	0.013	0.129	0.157	0.195	0.172	0.163
Theil's Entropy	0.000	0.001	0.001	0.000	0.001	0.003	0.002	0.003764	0.001361
Atkinson	0.001	0.009	0.009	0.002	0.009	0.021	0.017	0.026812	0.011917
Kolm	0.040	0.227	0.175	0.042	0.078	0.086	0.021	0.04493	0.054119

Table 4.11: Equity Indexes of HSC

Indexes	**1-> 5**	**2**	**1 -> 3**	**4**	**2-> 5**	**4-> 5**	**R1**	**R2**	**2->3**
Range	16.706	30.341	20.978	19.355	21.249	8.781	11.545	14.179	15.085
Variance	38.086	99.837	38.339	51.035	32.812	6.518	9.713	18.044	16.306
Coefficient of Variance	0.111	0.316	0.189	0.134	0.184	0.038	0.293	0.064	0.182
Relative Mean Deviation	0.100	0.433	0.412	0.130	0.442	0.199	0.809	0.189	0.602
Logarithmic Variance	0.002	0.086	0.062	0.004	0.071	0.006	0.544	0.006	0.169
Variance of Logarithms	2.257	1.549	1.610	2.195	1.554	2.505	0.605	2.488	1.211
K index	0.058	0.080	0.159	0.073	0.084	0.073	0.180	0.192	0.131
Theil's Entropy	0.003	0.020	0.007	0.004	0.006	0.000	0.015	0.000935	0.006191
Atkinson	0.026	0.139	0.057	0.037	0.038	0.004	0.070	0.013258	0.037213
Kolm	0.633	1.359	0.570	0.819	0.398	0.146	0.145	0.437646	0.225749

Table 4.12: Equity Indexes of Integrated Control

Indexes	1-> 5	2	1 -> 3	4	2-> 5	4-> 5	R1	R2	2->3
Range	3.200	5.000	16.800	3.400	72.100	70.500	63.500	67.200	66.400
Variance	1.124	2.939	29.401	1.164	309.361	310.757	257.346	290.549	279.970
Coefficient of Variance	0.007	0.043	0.053	0.009	0.058	0.081	0.086	0.093	0.069
Relative Mean Deviation	0.006	0.739	0.331	0.215	0.979	0.424	0.236	0.233	0.588
Logarithmic Variance	0.000	0.341	0.031	0.011	0.088	0.025	0.009	0.009	0.041
Variance of Logarithms	3.398	1.587	2.784	3.021	4.576	3.982	3.722	3.701	4.175
K index	0.004	0.186	0.098	0.081	0.129	0.125	0.106	0.106	0.127
Theil's Entropy	0.000	0.000	0.001	0.000	0.001	0.002	0.002	0.002045	0.001091
Atkinson	0.000	0.004	0.007	0.000	0.011	0.027	0.027	0.038192	0.016088
Kolm	0.008	0.021	0.204	0.008	3.175	3.602	2.470	3.332109	2.779573

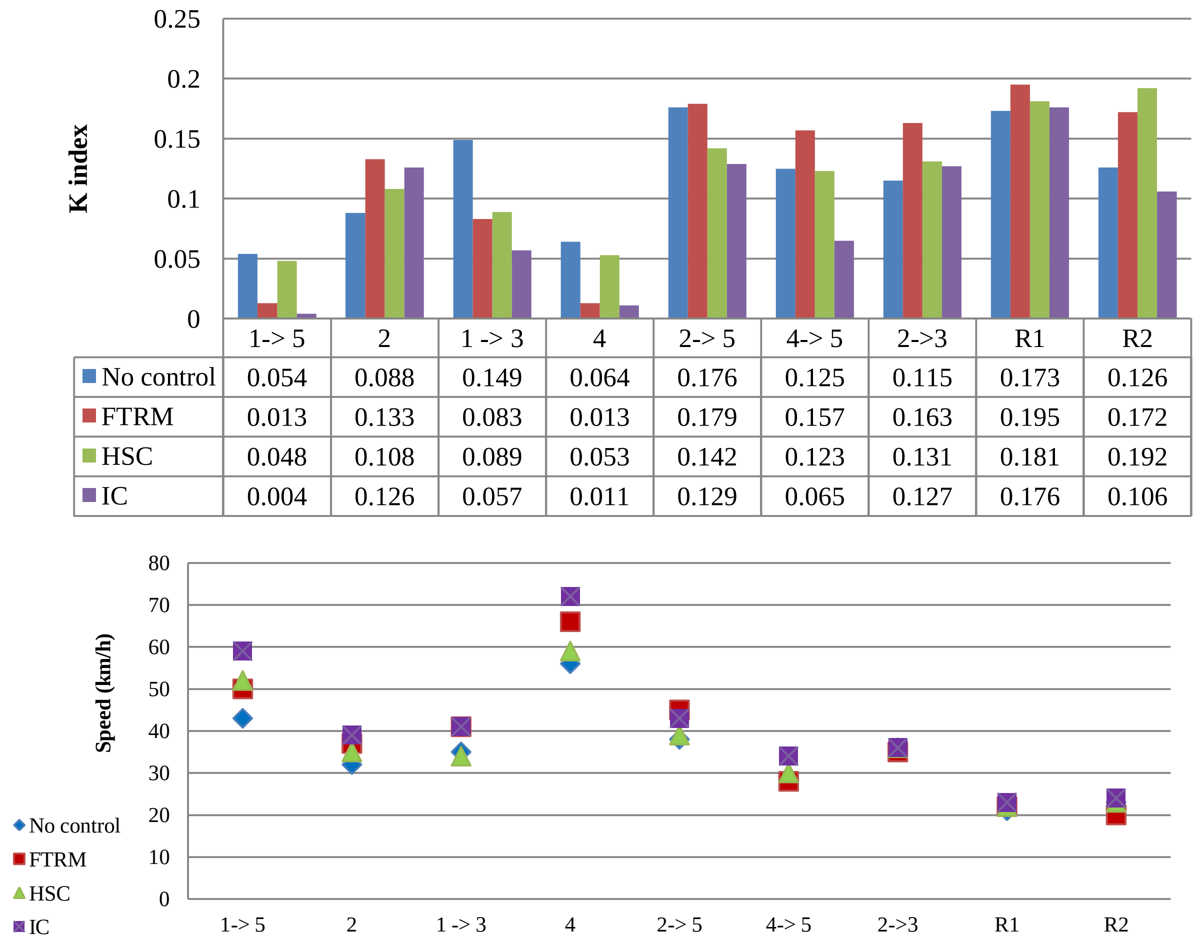


Figure 4.12: Comparative of FTCS in terms of efficiency and equity

It can be seen that in some cases the trade-off between equity and efficiency is observed for the FTCS analyzed. The speed increase (gain) comes up with an equity decrease (loss) at the first bottleneck downstream (numbered as 2). However, the trade-off is not the general pattern. There are some situations where both the speed and the equity increased as it is observed for the O-D pair 1-5 and also there is a case showing that the speed and equity can remain almost unchanged.

4.4.3 Spatio-temporal dimension of K index

The K Index can be calculated for each O-D pair and in between O-D pairs. When the speed of each vehicle is compared in time direction the index gives the equity in temporal dimension. Whereas, incorporation of the speeds measured from different locations will yield the spatial equity. As for an example, the diagonal matrices of K index are calculated for no control scenario, given in Tables 4.13 and 14.

Table 4.13: K Spatial Matrix

O-D Pairs	1-> 5	2	1 -> 3	4	2-> 5	4-> 5	R1	R2	2->3
1-> 5	0.055	0.366							
2	0.366	0.128							
1 -> 3	0.079	0.384	0.081						
4	0.106	0.279	0.135	0.065					
2-> 5	0.271	0.579	0.254	0.357	0.038				
4-> 5	0.472	0.168	0.488	0.393	0.658	0.018			
R1	0.096	0.418	0.080	0.180	0.220	0.518	0.052		
R2	0.690	0.434	0.701	0.636	0.809	0.324	0.721	0.032	
2->3	0.187	0.511	0.171	0.272	0.117	0.600	0.137	0.773	0.041

The spatial and temporal equity can be useful in assessment of the FTCS equity performance evaluation. The equity of departure time or equity of a specific location in comparison to the other locations can be determined through the K index.

4.5. Summary and Discussions

In this chapter, the equity concept is discussed in social, economical and transportation related sciences. The emphasis is given on the equity of FTCS and the motivation of developing a new index is explained. From the traffic control literature on equity issues, there is conceptualization problem of equity in traffic control where most of the indices are barrowed from econometric studies that are usually incorporating the differences of pairs among the mean values.

Table 4.14: K Temporal Matrix

O-D Pairs	900	1800	2700	3600	4500	5400	6300	7200	8100	9000	9900	10800
900	0.251											
1800	0.340	0.344										
2700	0.338	0.339	0.330									
3600	0.347	0.342	0.333	0.332								
4500	0.340	0.332	0.323	0.322	0.307							
5400	0.352	0.331	0.320	0.319	0.304	0.293						
6300	0.347	0.326	0.316	0.315	0.299	0.289	0.284					
7200	0.353	0.333	0.322	0.321	0.306	0.296	0.292	0.298				
8100	0.354	0.332	0.321	0.319	0.305	0.294	0.290	0.297	0.294			
9000	0.349	0.330	0.319	0.318	0.303	0.294	0.290	0.296	0.295	0.293		
9900	0.348	0.329	0.319	0.318	0.304	0.295	0.290	0.297	0.295	0.294	0.294	
10800	0.346	0.327	0.316	0.316	0.300	0.291	0.286	0.294	0.292	0.291	0.291	0.286

There are also some other problems detected in terms of mathematical formulation of equity, for instance when the delay values are incorporated with Gini Index this would yield 0/0 indeterminate form if there is no delay in traffic.

In order to solve the problems encountered, a new equity index, namely K, is proposed to assess the equity of traffic considering the mathematical and theoretical subjects. The conceptualization of equity in K index is based on the notion that the ratio of the speed differences (vehicles or time frames) over a specific value which reflects the optimal flow. It is assumed that perfectly equitable state must be attained at the optimal flow not at the completely stopped traffic.

The distinctiveness of the K index compared to other indices is therefore twofold. Firstly, the index measures the ratio of the sum of the speed differences of each vehicle and the free-flow speed, which indicates the equity of the delay in the network. Secondly, when the denominator part is altered with the optimum speed $\boldsymbol{u_{ff}}$ where the capacity is reached, then the index shows the equity of delay in the network.

Consequently, the reasons above indicate the motive of a novel equity concept and an appropriate metric establishment especially for evaluating the equity of traffic network control. It is considered that the developed K index is a useful, meaningful and mathematically relevant index to use in traffic control studies. The proposed index is regarded as a strong alternative to the common methodologies, especially to the most common one, the Gini index.

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CHAPTER 5. MODELING FREEWAY TRAFFIC CONTROL STRATEGIES VIA MICRO-SIMULATION: ISTANBUL O1 FREEWAY CORRIDOR

5.1 Introduction

There are two ways to evaluate the performance of ramp metering systems: field operational tests and computer simulations. Although field tests provide more realistic results, due to the high costs and time consuming nature, traffic simulation studies are becoming more popular. In this study, widely accepted, discrete, stochastic, time step based microscopic traffic flow simulation software, VISSIM, is employed to test the performance of control strategies and compare their performances.

The remainder of this section is organized as follows; the traffic model of Istanbul O-1 corridor is presented, and later, the selection of model parameters and traffic assignment process is explained. The optimality of FTRM, HSC and IC strategies are investigated and the simulation results are given. Finally the conclusions and the discussion of the chapter are presented.

5.2 Traffic model of Istanbul O-1 Bosporus bridge crossing

The corridor investigated has approximately 7 km of length, where there are 6 entrance ramps and 2 exit ramps up to the Bosporus Bridge. There are 4 main junctions entering / exiting to/from O1 Route and the bottlenecks are mostly occurring at around the downstream sections of the junctions (Uzuncayir, Acibadem, Altunizade and Beylerbeyi) due to the merging flow. In morning hours, especially weekdays the queue length may reach kilometers long and the average speed on the corridor can decrease down to 5km/h which indicates a complete hyper-congestion.

5.2.1 Study area and data

The data is collected by the students and staff of Istanbul Technical University on 14th of March, 2011. The traffic flow is observed from 6:00 a.m. to 11:30 a.m. through video recordings. Later, manual counts aggregated to 15 min. and inserted to commercial spreadsheet program (Microsoft Excel). The volumes at ramps are high between 6:00 a.m. and 7:00 a.m. especially most of the vehicles are medium type vehicles (minibus or midibus) which are used as service vehicles. Service vehicles could be classified as a special type of car sharing model which is mostly provided by companies free of charge to their employees. The hourly volumes of some ramps, Acibadem which provides freeway to freeway connection and the ramps at Altunizade, even have higher volumes per lane than the mainline for a short time period.

The on-ramps have two different geometries as; 5m width single lane and dual lanes with 3.5m width. How-ever, a virtual lane occurs at every single ramp during congested hours. The congestion starts at 6:45 .m. and the flow decreases down to 400 veh./h/lane at the bottle-neck downstream sections. It is observed that, once the breakdown occurs along the O1 Route, the congested flow remains invariant regardless of the time of the day which is verified by Sahin et al. (2005).

The speeds are also calculated through the recorded videos despite the measurements are not precise. The average speed is represented by randomly taken cars (medium, heavy) for a time period. The speed profiles are only used in visual conformity check and not considered for calibration purposes due to possible measurement errors.

However, the results indicate that the speed profiles at ramps are relatively lower than the mainline speed. The average mainline speed decreases down to 20 km/h after 6:45 a.m. and oscillates between 30 to 40 km/h for automobiles afterwards. The average ramp speed is around 20 km/h for automobiles and after 9:00 a.m. the speed increases to 30km/h.

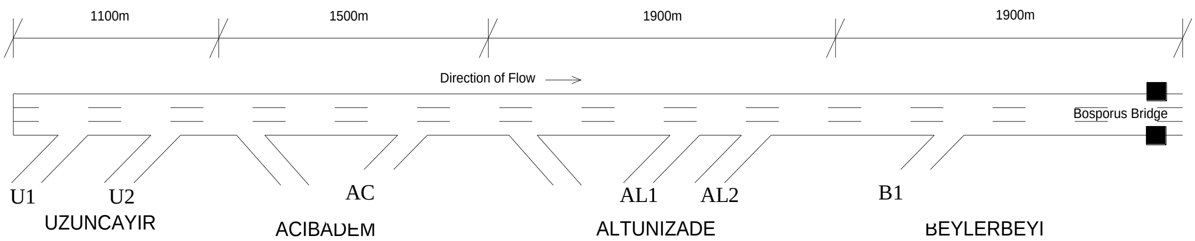

Figure 5.1: Study Area – O1 Route Westbound Direction

5.2.2 Calibration

The study corridor is simulated for the morning peak hours which start from 6:30 a.m. to 9:30 a.m. and performance measurement interval is selected as 15 minutes.

In the model calibration process, model parameters are altered until a qualitative and a quantitative balance between the simulation and the observation is reached. Traditionally, calibration requires several runs based on engineering judgment and experience. A three step calibration procedure is applied in this study, which are; calibration of driving behavior models, OD estimation and model fine-tuning.

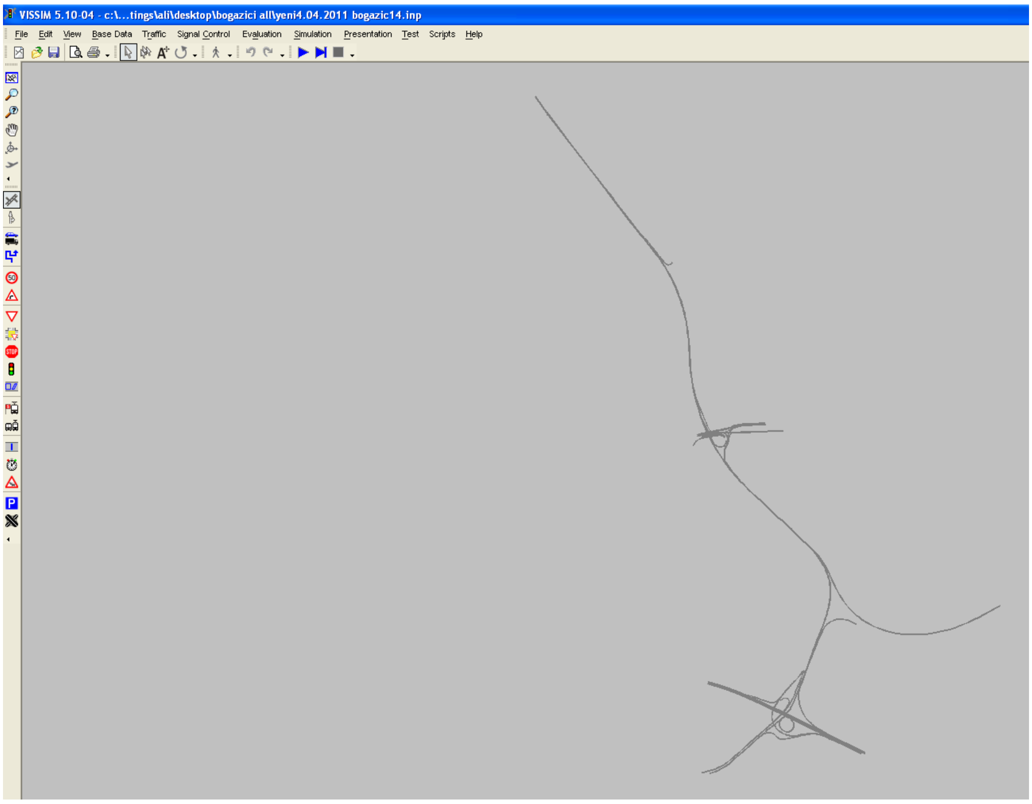

Figure 5.2: Simulated corridor

In calibration process GEH index is often used to test the relative difference between observed (Qo) and simulated (Qs) link volumes. GEH formula can be calculated with equation (2),

$$\mathrm{GEH} = \sqrt{\left(\frac{(2(Q_o - Q_s)^2)}{(Q_o + Q_s)}\right)} \tag{5.1}$$

The simulation model is acceptable if the GEH scores are smaller than 5 in 85% of the links and smaller than 4 for the sum of all link counts. The GEH scores are below 5 for all the links.

5.3 Selection of model parameters and traffic assignment process

VISSIM utilizes psychophysical car following models which combines a perceptual driver behavior model with a vehicle dynamic model (1974, 1999). The traffic composition and the priorities at the ramp weaving areas are set through the analyses of video recordings. It is observed that the vehicles entering to the mainline are more aggressive than the vehicles cruising on the right most lanes. Therefore the priority is given to the ramp flows over mainline flows in simulation. It is also watched that, if there is an enough gap the drivers tend to change the left most lane within the minimum possible distance. Typically, the drivers are highly aggressive and breaking and acceleration values are taken higher than the default values.

Lane changing is also highly strong in Istanbul traffic and drivers are frequently cutting in and overtaking. The car following model is selected as Wiedmann 1999, which has ten driver behavior parameters labeled CC0 – CC9. Several driver behavior parameters are reported to have significant impacts on roadway capacity and speed profiles thus, the parameters need to be optimized to attain the visual conformity and numerical correlation between the observation and simulation (Lownes and Machemehl, 2006).

The mean target headway and driver reaction time, which are the key user specified parameters in the car-following and lane changing models, can drastically influence overall driver behaviors of the simulation (Lownes and Machemehl, 2006). The calibrated values of the two parameters are 0.6 sec and 1.5 sec in this study. The calibration of ten parameters in car following model could be performed through some optimization techniques in order to achieve the most representative model. However, the calibration needs more precise data, especially the speed measurements should be based on a more valid method. Likewise, the local arterial roads are not included in the studied network hence, route choice is not considered in this calibration process.

The observations of Chu et al, (2004) confirmed that the precise geometry of merging angle and connector link length have an impact of simulation accuracy. Proportion of each vehicle type, vehicle characteristics and performance, such as the acceleration and deceleration rate, driving restrictions, the speed limits and the driving lane restriction, the look-back distance at merging and bifurcation weaving area, the priorities and traffic flow bases on conflicting areas also effects the simulation results.

The required number of runs can be calculated according to the mean and standard deviation of a performance measure of these runs, which is estimated from;

$$N = \left(t_{\frac{\alpha}{2}} \frac{\delta}{\mu\varepsilon}\right)^2 \tag{5.2}$$

where μ and δ are the mean and standard deviation of the performance measure based on the already conducted simulation runs; ε is the allowable error specified as a fraction of the mean μ; tα/2 is the critical value of the t-distribution at the confidence interval of 1-α. It is found that 10 different simulation runs are required. Therefore, the random seeds are chosen by creating 10 random numbers between 0 and 100. Therefore, the random seeds are chosen by creating 10 random numbers between 0 and 100 are listed in Table 1.

Table 5.1: Random seeds used in simulation

Number	1	2	3	4	5	6	7	8	9	10
Random Seeds	17	27	34	40	48	58	62	73	88	93

For the OD estimation and fine tuning the methodology given by Chu and Yang (2003) is adapted. The major difference from the Chu's study is that the reference OD matrix is also created with the help of traffic counts in this study. The volumes are inputted as 15min exact volumes in order to represent the exact static routing decisions. The priority settings are decided according to the video analyses and adjusted for each merge, Figures 5.2. The detectors are placed to collect the data for the analyses.

Table 5.2: O-D matrix input

	Static O-D Matrix													
Time a.m.	1-5	1-8	1-	2-5	2-8	2-	3-5	3-8	3-13	6-8	6-13	9-13	10-13	12-
6:30 - 6:44	505	197	756	339	238	637	524	195	840	334	1078	767	937	1526
6:45 - 6:59	449	145	713	390	240	618	605	195	628	404	1147	1004	820	1402
7:00 - 7:14	389	80	638	489	243	770	425	182	999	332	1503	745	800	1844
7:15 - 7:29	388	85	684	526	272	831	317	163	1135	427	1610	977	860	1952
7:30 - 7:44	455	68	665	645	266	880	270	124	915	422	1592	1301	876	1938
7:45 - 7:59	508	78	666	681	276	859	248	115	809	389	1348	1214	924	1579
8:00 - 8:14	512	72	514	678	264	790	290	121	867	425	1326	1324	916	1565
8:15 - 8:29	678	119	806	689	302	788	302	118	852	521	1502	1042	904	1849
8:30 - 8:44	605	290	720	672	392	682	272	205	774	554	1243	1074	1016	1464
8:45 - 8:59	890	278	814	742	302	589	461	208	945	712	1201	601	1004	1803
9:00 - 9:14	603	211	941	647	323	831	341	157	1117	815	1437	525	900	1945
9:15 - 9:29	708	329	778	787	466	954	390	181	872	453	1205	448	984	1616

5.4 Optimality search of cycle times for fixed-time ramp metering control strategy

For example, at Beylerbeyi junction (S12), the hourly volumes of 3 lane mainline are 5020, 5076 and 4980, veh./h and the single lane ramp volumes are 953, 926 and 986 veh./h respectively. In order to maintain the capacity flow which is observed around 1800 veh./h/lane at the downstream of the section, the ramp volumes should be limited as the amount of the exceeding volumes. Hence, the excess volumes for each hour are calculated as 573, 602 and 566 veh./h respectively and the average excessive volume is determined as 580 veh./h consequently. Since the signal is placed to control only the ramp, the green time is calculated to restrict the flow only at ramp.

Figure 5.3: Priority setting at Beylerbeyi

The green ratio which is the ratio of effective green time to cycle length would be 0.67 and the corresponding green time for 10 second cycle time plan, which should be rounded-off to the closest integer value, would be 7 seconds. The signal timing plan for each section is shown in Table 5.3.

In order to determine the optimal cycle time and green time for fixed time ramp metering control and examine the cycle time duration effects on network performance, a set of simulation experiments is designed. At each ramp, the green times are calculated for the average flows of entire simulation period by varying the signal cycle time from 5 to 20 seconds.

Figure 5.4: Detector locations

Table 5.3: Green times with varying cycle times

		Bottleneck Sections					
		U1	**U2**	**AC**	**AL1**	**AL2**	**B1**
Bottleneck Downstream Capacity (veh./h/lane)		1800	1800	1800	1800	1800	1800
Average Excess Demand (veh./h/lane)		182	234	527	680	368	580
Green Ratio		0.90	0.87	0.71	0.62	0.80	0.68
Cycle Time Variations	Green Times (5sec Cycle Time)	5	4	4	4	4	4
	Green Times (10sec Cycle Time)	9	8	7	7	8	7
	Green Times (15sec Cycle Time)	14	11	12	10	12	11
	Green Times (20sec Cycle Time)	18	15	14	13	16	14

5.5 Optimality search of speed harmonization and integrated control strategy

With respect to the speed management, similar to the ramp metering control experiment a bunch of rationale speed limits from 60km/h to 100km/h are employed with 10km/h increments. Speed restriction areas are defined along the corridor in order to test the speed limit control in simulation environment. One of the important issues is compliance of drivers to the displayed speed limits. The speeds of the vehicles are set within a 5% upper and lower margin of the speed limits. In order to achieve this in simulation environment the speed profiles are adjusted linearly for every speed limit examined.

In integrated control strategy in this study, the fixed time ramp metering control and speed management are implemented together without any coordination. As it is mentioned previously, the sophisticated control techniques are not considered as feasible in short range therefore, the focus is given to the applicable control strategies. The results of ramp metering and speed management control are used to design the experimental setup for the integrated control. Ramp metering control is found more effective than the speed limit control. The 15sec cycle time has the best performance among the other cycle times thus; the speed limits are tested from 50km/h to 80km/h with 15sec ramp metering cycle time.

5.6 Simulation results

The objective of the freeway traffic control process is to optimize a performance index that mostly consists of efficiency measures. Performance index can be selected that minimize the travel times, delays, number of stops, or some other parameters such as fuel consumption and environmental pollution. In a more social context the optimization of temporal and spatial equity along the network could also be considered as an objective. However, in this study the efficiency and the equity properties are analyzed for each control strategies in this chapter.

5.6.1 Efficiency performance

The first performance measure is selected as total travel time. The total travel time is calculated in hours for all active and arrived vehicles. In addition to the total travel time, the total delay in hours, the total number of stops and the average speed in km/h are evaluated by averaging values of 15min. intervals for each simulation run.

Table 5.4 compares the performance measure of control strategies investigated. It shows that all the traffic control strategies significantly increase the network performance. According to the results obtained, only 50km/h speed limit control (SL_50kph) performs worse than no control case in total travel time, number of stops and average speeds considering all the control scenarios. Nevertheless, even for the 50km/h speed limit control, the total delay gets smaller values than no control case. The best network performance achieved for ramp metering control at 15sec cycle time (RM_15sec). When the 15sec cycle time control is compared with no control case, it can be seen that the total travel time, the total delay and the number of stops decrease by 32%, 60% and 80% respectively and the actual average speed increase from 29.2km/h to 44.7km/h.

In case with the speed limit control, the best performance is attained at 70km/h limit control where average speed increase from 29.2km/h to 35.4km/h. and 19.8% decrease in total travel time. With speed limit control, the total travel time, the total delay and the number of stops are reduced by 17%, 31% and 26% respectively a dramatic increase in all measures for integrated control. The average speed of the network increased to 58.7km/h and 91.1%.

Table 5.4: Performance Measures of Control Strategies

	Performance Measures			
Control Strategy	Total travel Time [h]	Total Delay [h]	Number of Stops	Average speed [km/h]
No Control	4942.8	2910.1	411772	29.2
RM_5sec	4875.0	2784.4	385791	29.6
RM_10sec	3598.1	1436.7	139553	41.5
RM_15sec	3368.5	1190.3	81634	44.7
RM_20sec	3595.2	1440.5	153010	41.4
SL_100kph	5158.1	2732.0	445168	27.7
SL_90kph	5077.0	2679.2	438311	28.2
SL_80kph	4718.6	2656.6	381011	30.6
SL_70kph	4125.7	2020.3	306180	35.4
SL_60kph	4437.1	2115.1	336023	32.7
SL_50kph	5505.1	2644.4	499931	25.7
Int_50kph_15sec	4046.3	1026.1	152968	36.9
Int_60kph_15sec	2957.9	406.5	51882	51.1
Int_70kph_15sec	2586.6	386.6	45709	58.7
Int_80kph_15sec	3331.5	1123.6	69012	45.3

The results indicated that the total travel time can be significantly diminished by the examined control methods. Figure 5.5 shows the resulting total travel times for each control strategy and the comparison of best scenarios with no control case. From the results, for any scenario of ramp metering control the overall performances are increased. Nevertheless, for speed limits of 100km/h, 90km/h and 50km/h the performance of total travel times, the number of stops and the average speed measures are decreased. Interestingly, the total delays of these speed limits still have smaller values when it is compared with no control. This is possibly explained by the fact that speed harmonization may affect the total delays on the traffic network.

When the best scenarios of all types of control are compared, it can be concluded that the integrated control shows better performance than the ramp metering and speed management in all measures analyzed. With the best scenario of the integrated control, 70km/h speed limit combined 15sec cycle time ramp metering control (Int_70kph_15sec), the travel time reduces 48%, the total delay and the number of stops decreases 87% and 89% and the average speed increases from 29.2km/h to 58.7km/h when the integrated control is compared with no control case.

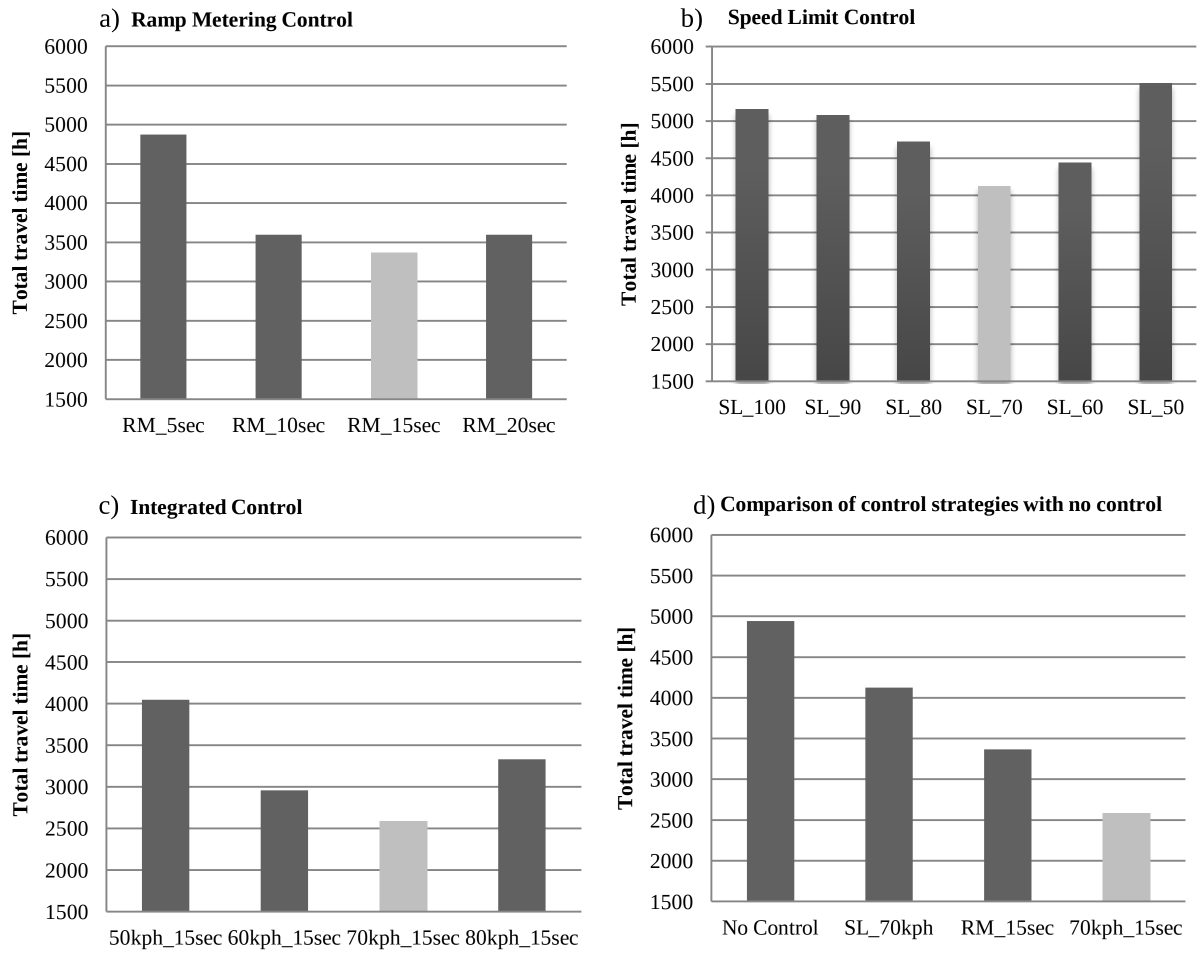

Figure 5.5: – Simulation results for the total travel times: a) Ramp Metering Control b) Speed Limit Control c) Integrated Control d) Comparison of Strategies with no control

The traffic control strategies evaluated in this chapter show that the freeway delays can be reduced through the increased capacity at bottlenecks which is remarked by previous studies (Cassidy and Rudjananakanoknad, 2002, Zhang and Levinson, 2010).

It is clear that the integrated control plays a positive role in increasing the average speeds. Under the best scenario of the integrated control, the average speed remains stable around 60km/h for the entire simulation period. In addition, the capacity breakdown at bottleneck locations of Uzuncayir and Acibadem (section 3 and 6) are prevented with the integrated control.

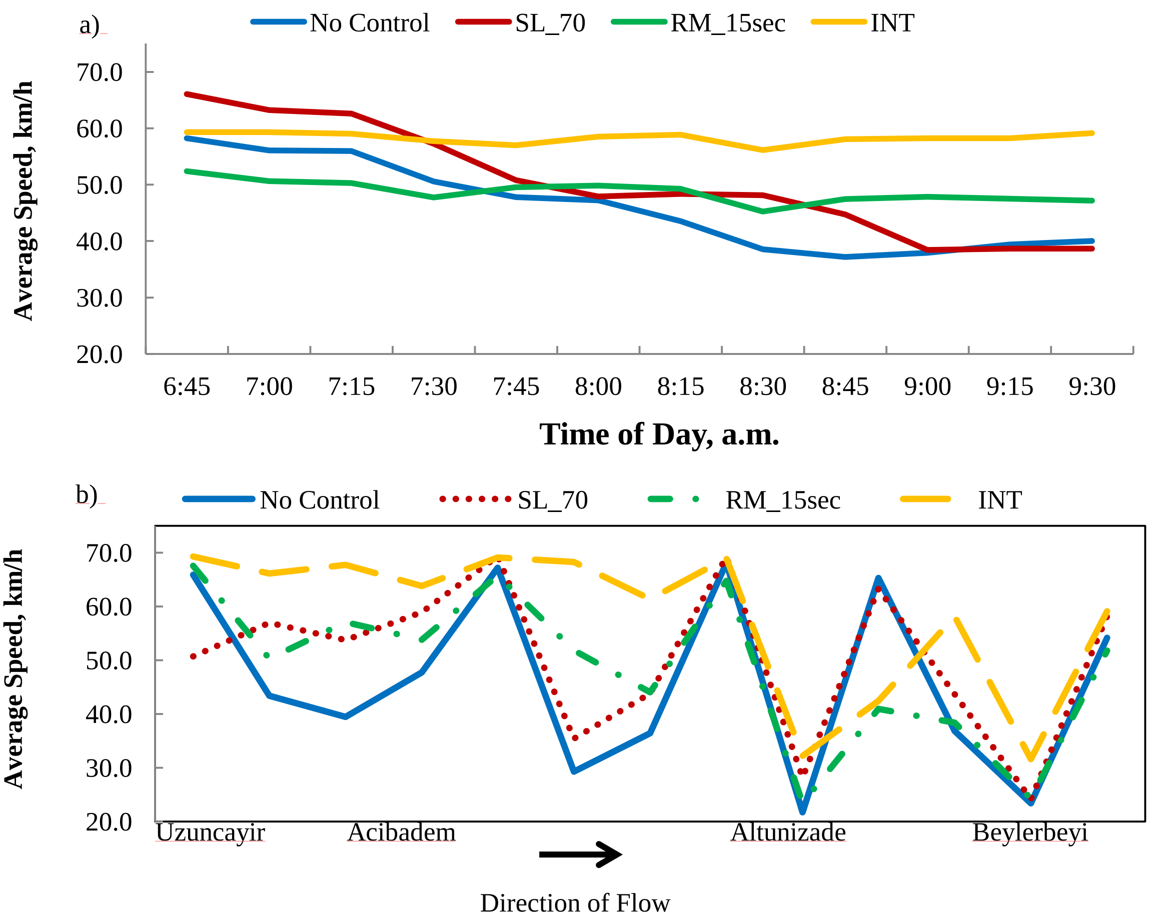

Figure 5.6: Average speed profiles of control strategies: a) Time of day b) Along the sections

The integrated control increases the capacity of active freeway bottlenecks by either postponing or sometimes eliminating the bottleneck activation. This result is complied with findings of the previous studies on ramp metering control (Zhang and Levinson, 2010). However due to the excessive demand, the congestion cannot be fully avoided at Altunizade and Beylerbeyi (section 9-10 and 12) on ramps even with the presence of the integrated control but it is delayed compared to previous scenarios. Figure 5.6 plots the relationship between speed profiles of control strategies and the time of day (a) and sections (b).

5.6.2 Equity performance

The results of the equity performances of on-ramps, mainline and the corridor are given for the indicators of speed and delay in Table 5.5 and 5.6. The free-flow speed is taken as 110 km/h and it is assumed that the optimal flow can be reached at the speed of 65 km/h. These values are incorporated in K index calculations. From the evaluation results of on-ramps, mainline and corridor wide performances, it is obvious that the FTRM increased the inequality on ramps when the spot speeds are taken into account. However, when the equity of the on-ramps is compared through speed and delay as indicators, the results are dramatically favoring the equity of no-control policy. The possible explanation for this controversy is the measurement methodology of the indicators. IC provides more equity than the FTRM and HSC manages to establish the most equitable traffic state out of the FTCS evaluated.

The equity measures employed from the existing literature are basically calculating the distribution of differences from the mean value. The speeds are relatively stable in no control case, which actually show the heavy congestion on ramps. FTRM causes the fluctuation at on-ramps depending on the arrival rates which causes a higher inequality. Therefore, the selection of spot speed detection location gains an extreme importance due to this fluctuating nature of FTRM control. On the other hand, delay indicators create even more equal tableau in general since the indicator calculations are based on a lower denominator.

5.7 Conclusions

This study demonstrates the potential of implementing traffic control strategies in alleviating the traffic congestion on an urban freeway. The FTRM, HSC and IC are analyzed as traffic control strategies. The traffic simulation network is modeled and calibrated for the analyses. The FTRM, HSC and IC are successfully increased the effectiveness of the traffic flow referring the total travel time, the total delay, the number of stop and average speed.

Table 5.5: Equity Performance of FTCS

	FTCS											
	NO CONTROL			HSC			FTRM			IC		
Measures	**ON-RAMPS**	**MAIN LINE**	**CORRIDOR**	**ON-RAMPS**	**MAIN LINE**	**CORRIDOR**	**ON-RAMPS**	**MAIN LINE**	**CORRIDOR**	**ON-RAMPS**	**MAIN LINE**	**CORRIDOR**
Range	27.550	14.250	16.860	41.650	22.780	27.630	10.463	8.725	8.938	2.945	6.080	3.031
Variance	102.100	24.200	37.300	191.600	66.400	89.300	5.250	4.625	4.250	0.665	3.515	0.855
Coefficient of Variance	0.290	0.110	0.140	0.320	0.160	0.190	0.063	0.050	0.050	0.019	0.029	0.019
Relative Mean Deviation	0.260	0.090	0.120	0.260	0.140	0.160	0.050	0.038	0.038	0.010	0.029	0.010
Variance of Logarithms	1.805	2.028	2.051	1.985	2.166	2.149	2.475	2.721	2.666	2.074	2.254	2.191
K delay	0.036	0.015	0.017	0.047	0.023	0.028	0.006	0.005	0.005	0.002	0.004	0.002
K speed	0.054	0.022	0.022	0.078	0.034	0.045	0.008	0.008	0.007	0.004	0.006	0.003

Table 5.6: Equity states of bottlenecks, ramps, mainline and corridor

SECTION	TCS	U1	U2	AC	AL1	AL2	B1	ON-RAMPS	MAIN LINE	OFF-RAMPS	ALL-RAMPS	CORRIDOR
Range	NO Control	41.5	54.2	48.0	38.3	11.1	4.1	27.6	14.3	7.1	20.6	16.9
Variance		231.6	586.0	399.7	190.0	13.9	1.1	102.1	24.2	4.6	49.7	37.3
K delay		0.184	0.325	0.342	0.309	0.031	0.008	0.036	0.015	0.019	0.080	0.017
K speed		0.289	0.523	0.506	0.464	0.052	0.014	0.054	0.022	0.028	0.145	0.022
Range	HSC	8.6	18.2	27.1	20.4	6.0	6.7	8.4	7.0	7.7	7.3	7.2
Variance		7.0	23.3	78.1	38.1	4.0	3.0	4.2	3.7	5.5	3.8	3.4
K delay		0.026	0.051	0.089	0.150	0.026	0.016	0.047	0.023	0.020	0.022	0.028
K speed		0.038	0.080	0.144	0.260	0.039	0.023	0.078	0.034	0.029	0.039	0.045
Range	FTRM	45.1	56.5	55.2	56.9	11.2	2.8	41.7	22.8	0.6	31.2	27.6
Variance		290.6	560.2	610.7	533.3	14.0	0.5	191.6	66.4	0.0	108.0	89.3
K delay		0.154	0.235	0.359	0.397	0.031	0.005	0.006	0.005	0.001	0.123	0.005
K speed		0.233	0.372	0.556	0.641	0.049	0.008	0.008	0.008	0.001	0.219	0.007
Range	IC	4.5	9.1	1.7	16.6	2.0	3.9	3.1	6.4	1.3	2.3	3.2
Variance		1.7	5.9	0.2	29.4	0.5	1.5	0.7	3.7	0.1	0.4	0.9
K delay		0.010	0.015	0.004	0.093	0.010	0.011	0.002	0.004	0.003	0.006	0.002
K speed		0.015	0.024	0.006	0.145	0.016	0.019	0.004	0.006	0.004	0.011	0.003

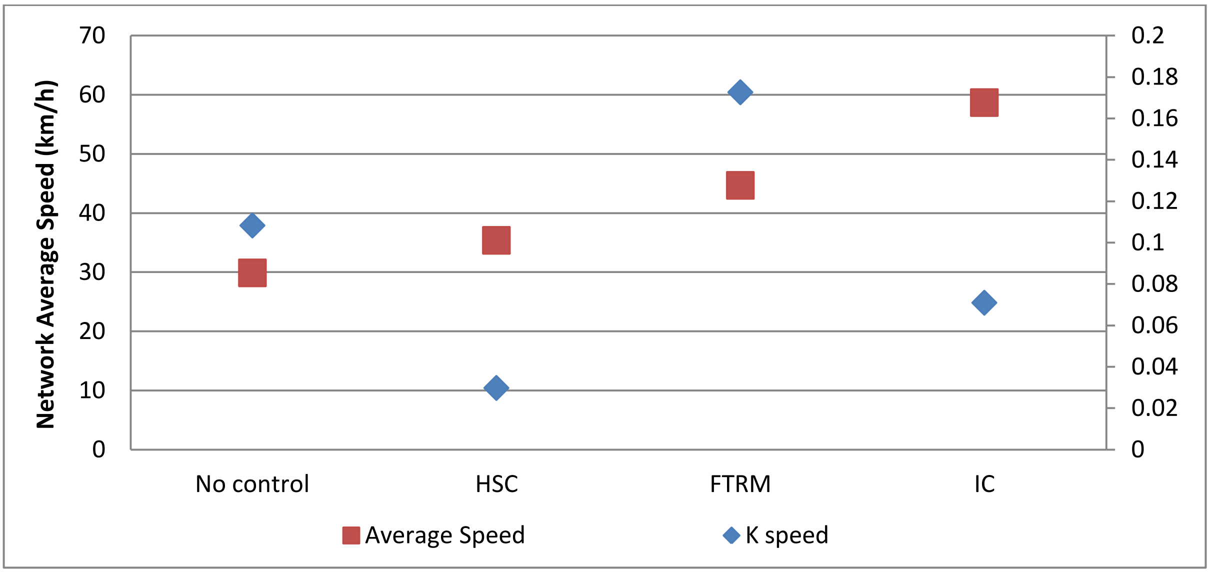

Figure 5.7: Average speed and K_{speed} profiles of FTCS

The results show that; there is an optimum cycle time that can be determined for each on ramp considering the bottleneck downstream capacity and flows and there is an optimal speed limit for HSC. This study also shows that the superposition of the optimal traffic control strategies also yields to an optimal integrated control.

From the Figure 5.7, it can be seen that in some cases the trade-off between equity and efficiency is observed for the FTCS analyzed. The speed increase (gain) comes up with an equity decrease (loss) for the FTRM.

However, the trade-off is not the general pattern. The HSC and IC either manage to increase the speed or maintain the speed besides increasing the equity state at the same time. The results are justifying the results of previous chapter. The prevailed results from the simulation lead to the following conclusions.

i. There is an optimum cycle time that can be determined for each on ramp considering the bottleneck downstream capacity and flows. The short cycle lengths have a tendency to increase the start-up lost times and limits the merging rate, therefore the delay increases rapidly. Long cycle lengths allows platoon of vehicles entering the mainline which also contributes an increase in delay. The result of the study is highly congruent with previous findings on the sensitivity of delay to cycle length for intersections exhibited on 16-16 at Highway Capacity Manual (TRB, HCM, 2000). Figure 5.8shows the delay sensitivity to cycle time for signalized intersections and for FTRM.

ii. Speed limit control strategy has also an optimal value which optimizes the performance measures. One of the possible reasons behind this is, at low speed limits the average speed drops under the effective level and at higher speeds drivers are applying more aggressive breaking which causes sudden shock waves.

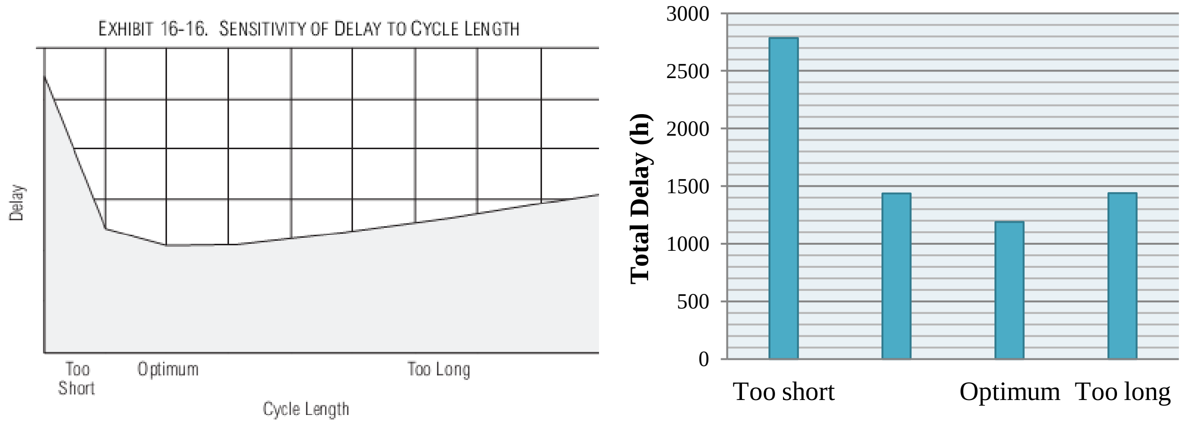

Figure 5.8: Delay sensitivity to cycle time for signalized intersections and for FTRM

iii. Ramp metering and speed limit control integration significantly increase the performance even without any coordination and ramp metering depicts a relatively better performance increase in total travel time savings than speed limit control.

iv. Probably the most practical output of this analysis is that all the control strategies analyzed increase the effectiveness of the traffic flow referring the total travel time, the total delay, the number of stop and average speed, in a satisfied manner.

v. A possible reason for this high amount of increase in efficiency might be the exclusion of queue spillover and the delays at the arterial and local roads. The ramp queues would be extended due to the control implementation in reality which yields a decrease in on ramp flow performance. Besides this, strict metering (high rate of ramp metering) brings some other concerns for the ramp users and may yield in modal shift or trip cancellation

vi. Another important output of this study is the results accord with the existing literature suggesting that the ramp control brings equity concerns for ramp users when both delays and speeds are taken into account. From the results, it is also observed that the equity of ramps is worsened with the FTRM control strategy when it is compared to the equity of mainline and overall corridor.

vii. The change in magnitude of equity with respect to the measure and indicator taken into account could be vast. Thus, first the equity indicators and measures should be evaluated carefully and then the performance should be interpreted accordingly to make a conclusive judgment.

viii. Finally the integrated control both achieved the increase in efficiency and equity at the same time that shows that the contradicting nature reported by Zhang and Levinson is not valid for all type of traffic controls especially if the implemented control strategies alleviate the congestion at specific sections.

As evaluation results indicated the controversial nature of selection and evaluation of efficiency and equity, there is still a need to a careful examination of other traffic control strategies such as ALINEA, VSL and other integrated control strategies.

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CHAPTER 6. ANALYSES ON EQUITY OF FREEWAY TRAFFIC CONTROL STRATEGIES: TOKYO METROPOLITAN EXPRESSWAY ROUTE 3 CORRIDOR

6.1 Introduction

In this chapter, the traffic from Shibuya to Yoga direction on Route 3, which is one of the radial routes of the Shuto Expressway system in the Tokyo area in Japan, is analyzed with respect to different demand scenarios. In previous chapter, Istanbul O-1 case, due to the unavailability of the occupancy data the model predicted control strategies couldn't be examined. The purpose of this chapter is to include the dynamic and model predictive control strategies to the analysis and evaluate the performances of efficiency and equity of each strategy. Moreover, the geometry of roads, demand patterns, and changes in the driving behavior are also completely different in Istanbul and Tokyo thus in this chapter, the proposed control strategies and equity index are examined for the Route 3 Tokyo.

The remainder of this section is organized as follows; the Tokyo metropolitan expressway route-3 is introduced, the data and the calibration of simulation model is demonstrated. The model predictive control (MPC) and integration of MPC to ALINEA and VSL is presented. The integrated control strategies are described and the simulation results are given. Furthermore, the performances of FTCS with respect to the demand levels are analyzed. In conclusion part, the simulation results of efficiency and equity are discussed with an emphasis on the demand sensitivity analyses.

6.2 Tokyo metropolitan expressway route-3

Traffic congestion is a common problem; particularly in metropolitan areas. It is especially true for the Tokyo Metropolitan Freeway (MEX) which handles about 1.2 million vehicles per day. Thirty percent of these vehicles use the MEX central circular and radial routes causing chronic traffic congestion because MEX operate with peak traffic demands that exceed capacity. This results in congestion and delays. There are two options to improve freeway operations. The first option involves an increase in the supply and the second option involves the control of demand. Increasing the supply (constructing and/or widening roads) is often an undesirable alternative due to physical constraints, cost considerations and environmental impacts.

6.2.1 Study area and data

Route 3 (also known as the Shibuya Route) is one of the radial routes of the Shuto Expressway system in the Tokyo area. Route 3 runs southwest from Tanimachi Junction (with the Inner Circular Route) in Minato-ku and runs for almost 12km through Shibuya-ku, Meguro-ku, and Setagaya-ku. The Route 3 designation ends at the Yoga Rampway (Tokyo Interchange) and the expressway continue as the intercity Tomei Expressway to Nagoya.

Figure 6.1: Shuto Expressway system and Route3

The demand scenarios are created as the low, medium and high levels for afternoon peak hours. The medium demand traffic flow data is created by averaging the flows of afternoon peak hours from Shibuya loop detector data. Data is gathered from actual loop detectors of Shibuya outbound direction from 18:30 to 20:30 collected in 2006 gathered from www.trafficdata.info.

Table 6.1: Demand Scenario (volumes in veh/hr)

O	D	Simulation Time (seconds)							
		900	1800	2700	3600	4500	5400	6300	7200
1	2	483	217	546	567	609	693	525	504
1	3	483	217	546	567	609	693	525	504
1	6	242	109	273	284	305	347	263	252
1	7	483	504	546	567	609	693	525	504
1	8	725	756	819	851	914	1040	788	756

The corridor investigated is starting from Shibuya and the length of the analyzed section is around 7 km. There are 2 entrance ramps and 4 exit ramps up to Yoga. The hypothetical flows are inserted to microscopic traffic simulation program as 15 min. exact volumes and the vehicle composition are selected in order to represent the real situation given in Table 6.1

6.2.2 Calibration of simulation model

The car following model is selected as Wiedemann 1999, which has ten driver behavior parameters labeled CC0 – CC9. Several driver behavior parameters are reported to have significant impacts on roadway capacity and speed profiles thus, the parameters need to be optimized to attain the visual conformity and numerical correlation between the observation and simulation [13].

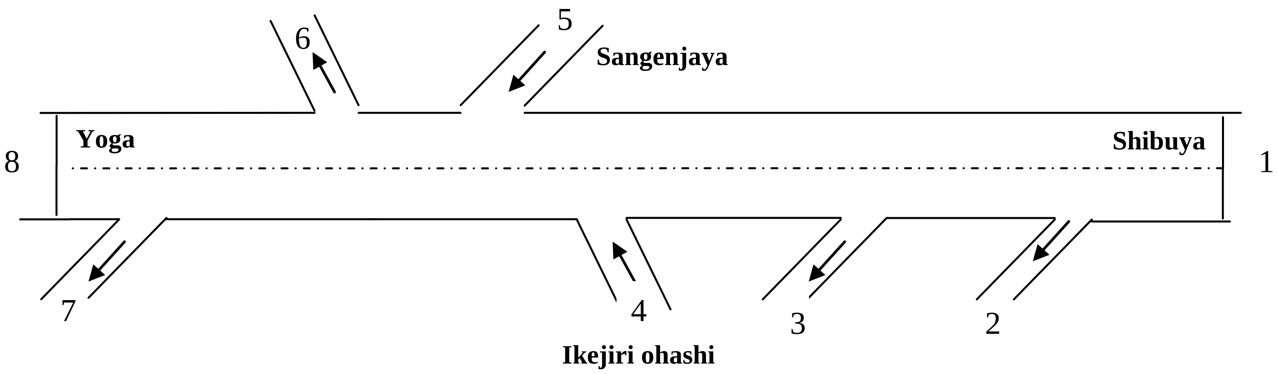

Figure 6.2: Schematic representation of the Route3

The traffic comprises of 85% is automobile and mini truck (weight dist. 800kg to 4000t) 5% Heavy Goods Vehicle and 10% Bus. The speed distributions are selected as 80kph (75 – 110 km/h) for automobile and mini trucks, 60kph (58 – 68 km/h) for HGV and 70kph (68 – 78 km/h) for buses.

In the model calibration process, model parameters are altered until a qualitative and a quantitative balance between the simulation and the observation is reached. Traditionally, calibration requires several runs based on engineering judgment and experience. The required number of runs can be calculated according to the mean and standard deviation of a performance measure of these runs, which is estimated from;

$$N = \left(t_{\alpha/2}\frac{\delta}{\mu\varepsilon}\right)^2 \tag{6.1}$$

where μ and δ are the mean and standard deviation of the performance measure based on the already conducted simulation runs; ε is the allowable error specified as a fraction of the mean μ; $t_{\alpha/2}$ is the critical value of the t-distribution at the confidence interval of 1-a. It is found that 10 different simulation runs are required.

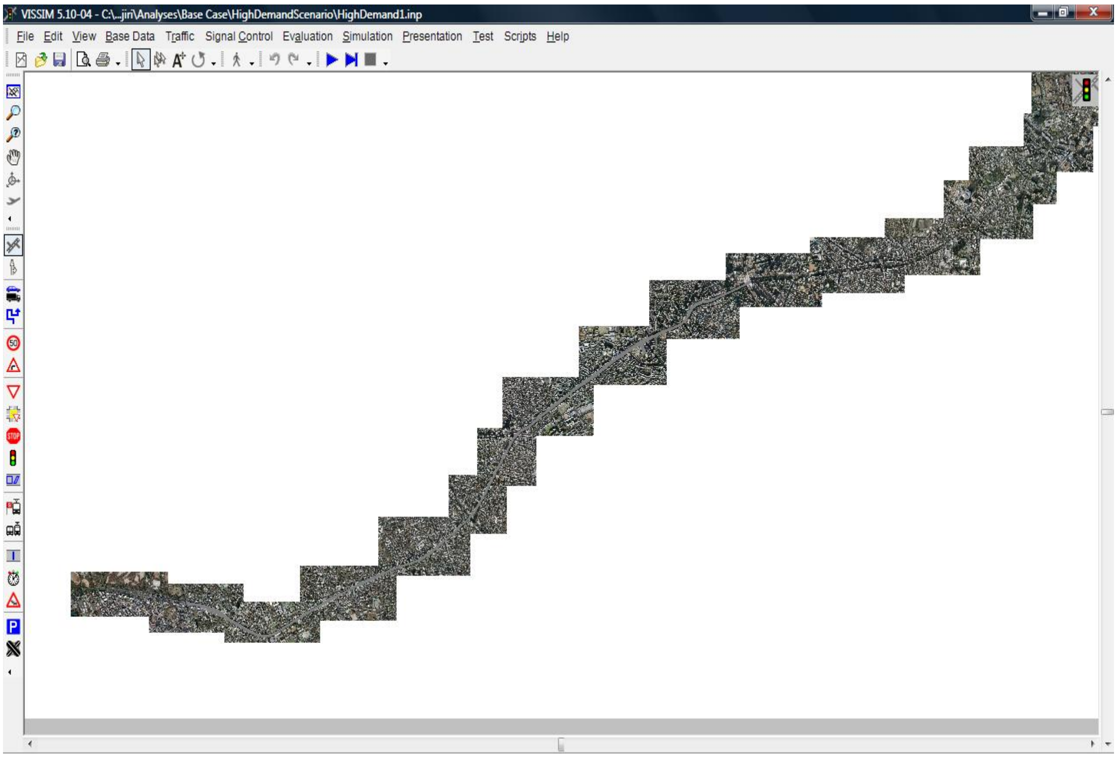

Figure 6.3: Simulated Network

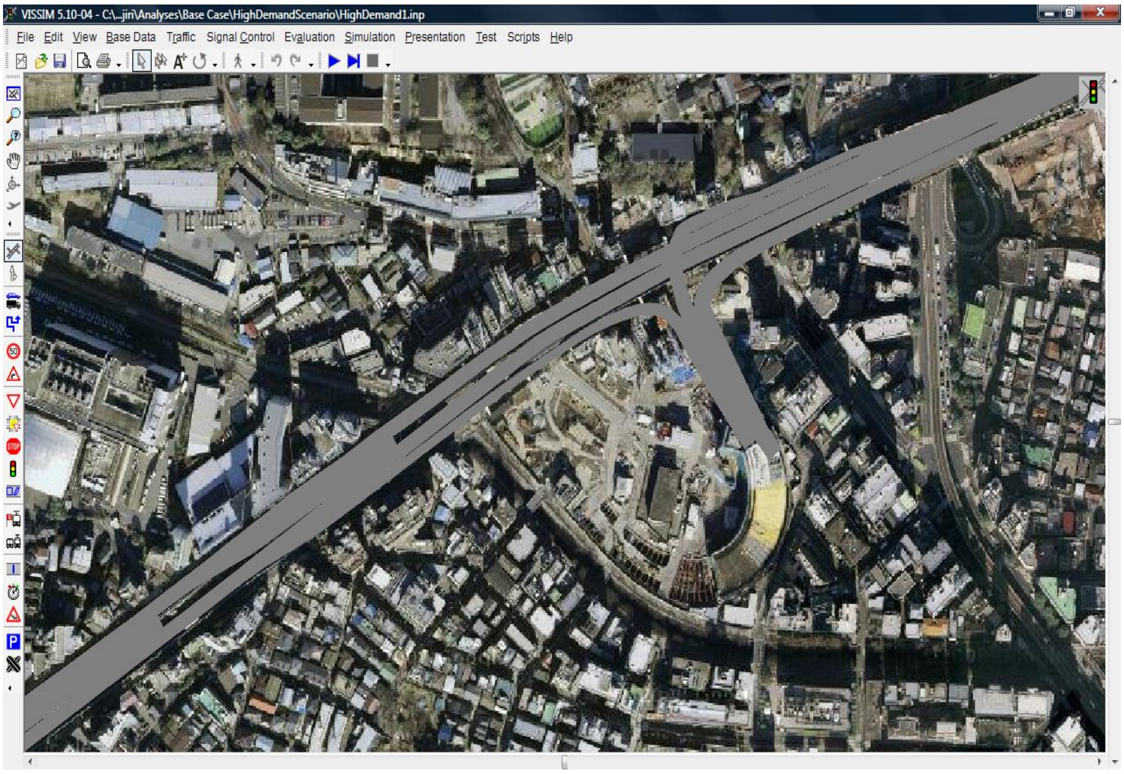

Figure 6.4: Ikejiri-ohashi junction

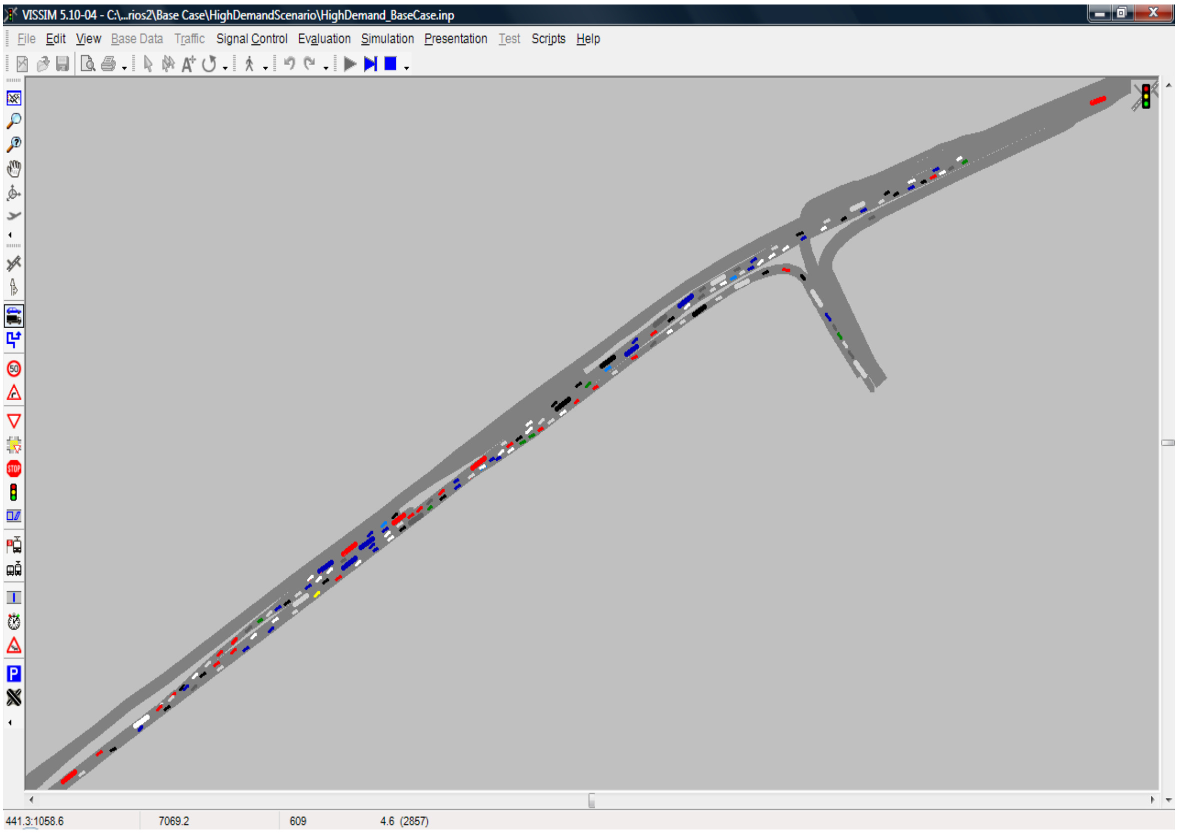

Figure 6.5: Formation of congestion at Sangenjaya and Ikejiri-ohashi entrances

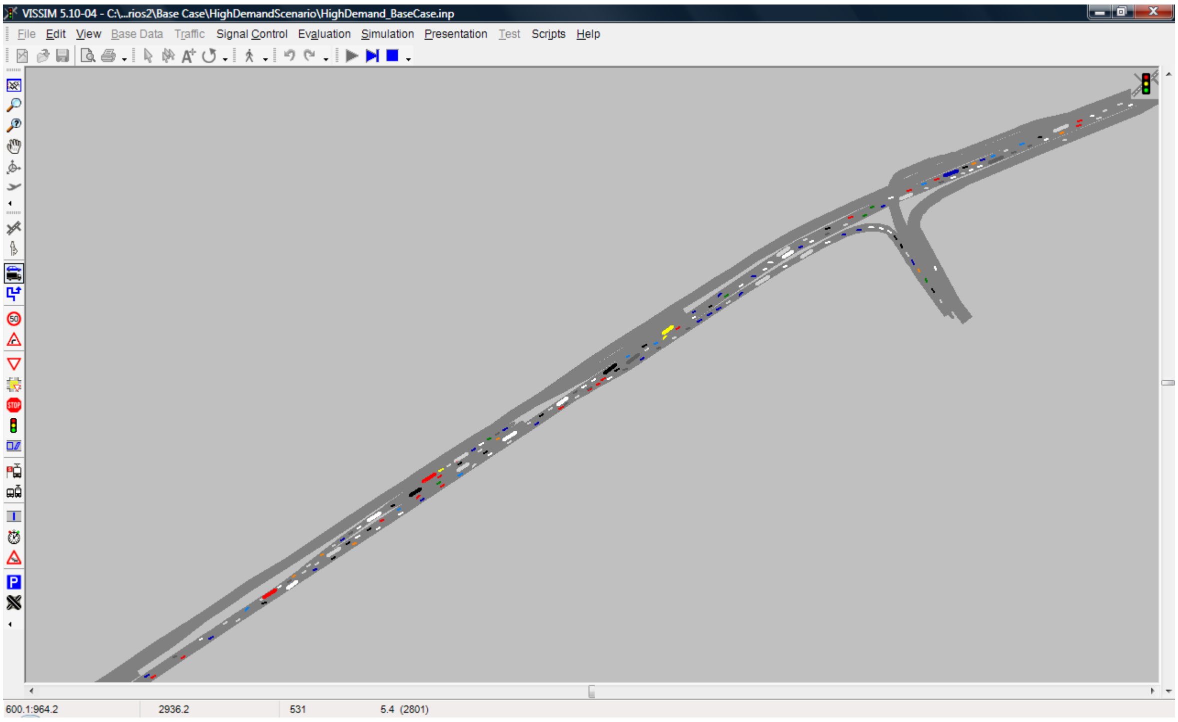

Figure 6.6: Congestion propagation to Shibuya direction

Therefore, the random seeds are chosen by creating 10 random numbers between 0 and 100 are listed in Table 6.2.

Table 6.2: Random seeds used in simulation

Number	1	2	3	4	5	6	7	8	9	10
Random Seeds	5	15	25	35	45	55	65	75	85	95

In calibration process GEH index [14] is often used to test the relative difference between observed (Q_o) and simulated (Q_s) link volumes. GEH formula can be calculated with equation (3) and the GEH values are tabulated in Table 6.3.

$$GEH = \sqrt{(2(Q_o - Q_s)^2)/(Q_o + Q_s)} \tag{6.2}$$

The simulation model is acceptable if the GEH scores are smaller than 5 in 85% of the links and smaller than 4 for the sum of all link counts. The GEH scores are below 5 for all the links.

Table 6.3: GEH Values

	Sections		
Hours	**Ramp**	**Mainline**	**Corridor**
17.30 - 18.29	2.8	4.2	3.8
18.30 - 19.29	4.2	4.8	4.7
19.30 - 20.29	4.6	4.9	4.6

6.3 Selection of model parameters and traffic assignment process

The volumes are inputted as 15 min exact volumes in order to represent the exact static routing decisions. Desired speed decisions are calibrated along the corridor in order to test the speed limit control in simulation environment. In this study the speeds are bounded within a 5% upper and lower margin. In order to achieve this in simulation environment the speed profiles are adjusted linearly for every speed limit examined.

Unfortunately, many of the simulation parameters used in VISSIM are difficult to measure in real traffic, which substantially affects the model's performance. For instance; it is really difficult to capture the start-up lost time, the queue discharge rate, the car-following sensitivity factors, the time to complete a lane change or acceptable gaps. The car following model is selected as Wiedmann 1999 for urban motorized way and the simulation parameters are taken from a study conducted in Japan (Terada, 2008) given in Table 6.4.

Table 6.4: Simulation parameters

Driving Behavior Parameter	**Value**
CCO Standstill Distance	1.70 m
CC1 Headway Time	1.5 s
CC2 Following Variation	4.00 m
CC3 Threshold for Entering	– 8.00
CC4 Negative Following Threshold	-0.18
CC5 Positive Following Threshold	0.18
CC6 Speed Dependency Oscillation	1.14
CC7 Oscillation Acceleration	0.06 m/s2
CC8 Standstill Acceleration	3.60 m/s2
CC9 Acceleration at 80km/h	1.50 m/s2

6.4 Incorporation state estimation into model predictive control

Model predictive control (MPC) is an increasingly significant and popular control approach because of its use of a possibly nonlinear multivariable process model and its ability to handle constraints on inputs, states and outputs. It uses open-loop constrained optimization of finite-horizon control criteria in a receding horizon approach. A model is used to predict the future behavior of the system up to the horizon, starting from its current state, and a constrained optimization based on the prediction yields an optimal open-loop control sequence over the complete horizon. Only the first element in this sequence is applied to the plant. New measurements available at the next sample time permit the calculation of an updated initial state value, and the optimization is then resolved. The introduction of each output measurement, via the mechanism of state update, results in the overall method yielding a closed-loop control.

The receding horizon approach behind MPC relies on state estimation even though most analyses of the stability, feasibility and performance of these schemes treat the controller as a full-state-feedback strategy (Mayne et al, 2000). From MPC's early linear unconstrained variants such as generalized predictive control and its connection to LQG (Clarke et al, 1987; Bitmead et al, 1990), the formulation of MPC has included state estimation either inherently or explicitly in the construction of the predictor using observer polynomials or via the Kalman filter. However, to our knowledge, there has been no satisfactory treatment of the inclusion of state estimation into the formulation of the full constrained problem other than our own (Yan & Bitmead, 2002) and some very interesting recent work of van Hessen and Bosgra (2002), both of which we build on here. This is in spite of the industrial impetus for MPC being its application in process control, where the rejection of stochastic disturbances is the central control objective. The formulations cited above are certainty-equivalence controllers, in which the state estimate is used as a replacement for the exact state in the control law. The natural sequencing of the receding horizon control calculation would see the one-step-ahead state prediction used as the initial state in the computation of the MPC control sequence.

6.4.1 Construction of model predictive control

In our control algorithm a time series model is used to predict the occupancies of the next time frame. The time series model can be expressed as;

$$O_t = \beta_1 O_{t-1} + \beta_2 O_{t-2} + \beta_3 O_{t-3} \tag{6.1}$$

where, O_t is the dependent variable and O_{t-1}, O_{t-2} and O_{t-3} are the independent variables to predict the occupancies of the further time frames. β_1, β_2, and β_3 are the coefficients determined with the OLS regression. The result of the regression analyses yield that without an intercept since the occupancies would be zero initially, the coefficients are found as 0.66, 0.15 and 0.19 respectively for β_1, β_2, and β_3 and the R^2 is found as 0.97 thus, this model can be considered as highly representative. The summary of the regression analyses is given below in Tables 6.5 and 6 and the comparison between the observed occupancies and predicted occupancies are given in Figure 6.7.

Table 6.5: Regression statistics

Multiple R	0.98
R Square	0.97
Adjusted R Square	0.97
Standard Error	2.39
Observations	570
Durbin Watson Number	2.04

In order to detect the serial correlation the Durbin Watson test is run. The Durbin Watson test is simply the ratio of the sum of squared differences in successive residuals to the residual sum of squares. According to the results the existence of positive autocorrelation cannot be rejected.

Table 6.6: Coefficient statistics

	Coefficients	Standard Error	t Stat	P-value
O_{t-1}	0.66	0.041	16.00	7.1E-48
O_{t-2}	0.15	0.049	2.96	0.0031
O_{t-3}	0.19	0.041	4.47	9.38E-06

6.4.2 Model predictive ALINEA control

The ramp metering strategy proposed by Papageorgiou, M., Hadj- Salem, H. and Blosseville, J. M. in 1991 is modified by incorporating the model predictive control method. The ALINEA strategy is a robust and efficient control model however; the model can be improved if the occupancies in the next time frames could be predicted.

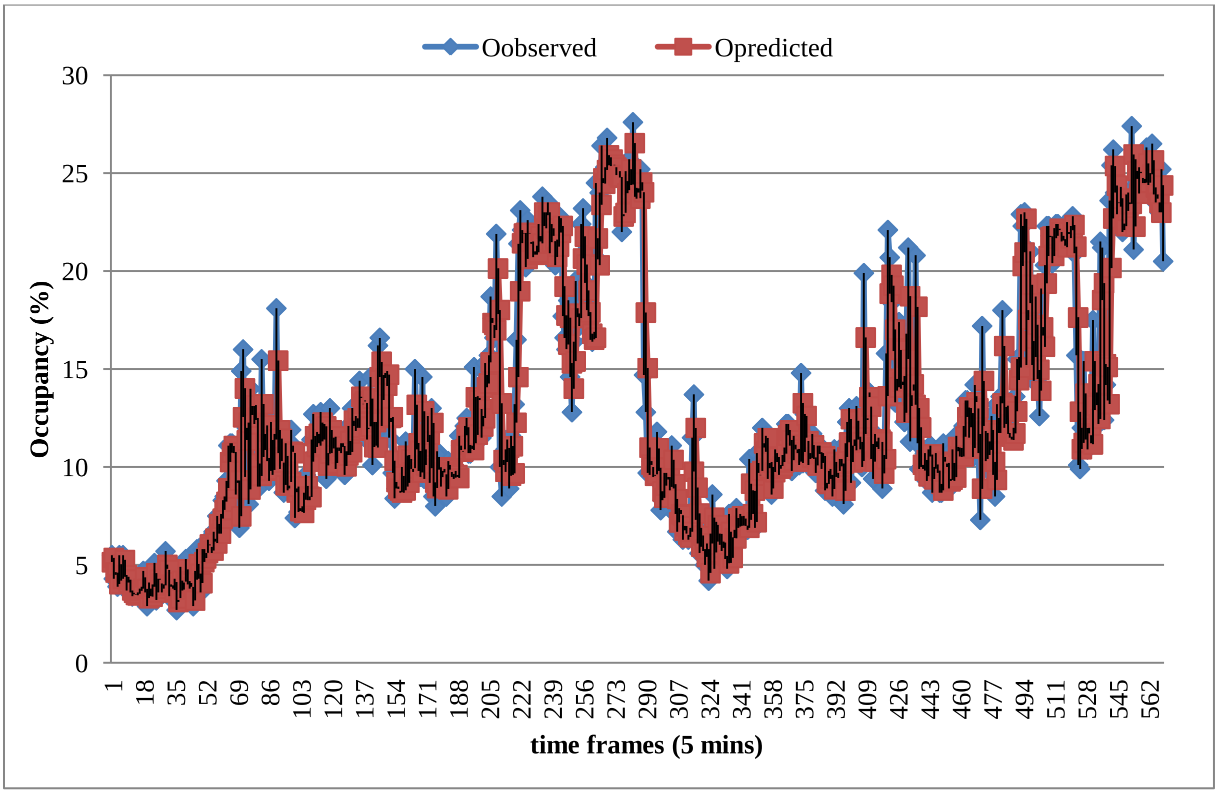

Figure 6.7: Comparison of observed and predicted occupancies

The ALINEA strategy aims to keep the occupancy rate at a level of desired occupancy rate generally set as 0.30 in many other studies. From the data analyzed, the congestion occupancy is also observed at the 30% level therefore, the desired occupancy is set to 0.30. The feedback occupancy is predicted through a linear regression model given in 6.1 where the independent variables are also the measured occupancies of previous time frames. The algorithm of model predictive MP-ALINEA can be expressed as;

$$r(t + \Delta t) = r(t) + K_r(\hat{o} - o(t + \Delta t)) \tag{6.2}$$

where $r(t + \Delta t)$ is called the metering rate to release to the downstream at the next cycle, $r(t)$ is the metering rate which is released at the current cycle, K_r is a regulator parameter which is taken generally 70 by the authors (Papageorgiou, 1990) and the applicators of this algorithm, ô is the desired occupancy rate and $o(t + \Delta t)$ is the predicted occupancy for the next cycle.

In order to apply this to the VAP module for PTV VISSIM, in simulation detectors having 5 meter lengths are placed at the bottleneck downstream sections where the data collection loggers are also placed.

6.4.3 Model predictive rule-based variable speed limit control

To perform an optimal dynamic VSL control, a set of traffic models must capture the complex interactions between traffic-state evolution and all control parameters. In particular, those traffic-state evolution equations should be mathematically formulated to represent the actual operational constraints. As recognized in many studies traffic density and speed have been taken as state variables, of which the former is a key factor affecting drivers' choice of speed and the VSL system's selection of appropriate speed limits.

However, instead of taking speed or volume as a control variable occupancy measure is proposed as an optimal control variable of VSL algorithm for congestion alleviation.

Table 6.7: VSL strategies

Strategy	Occupancy Rate (%)	Posted Speed Limits(km/h)
MP-VSL	0.1	120
	0.15	110
	0.2	100
	0.25	90
	0.3	80
	0.35	70

The algorithm includes the time dependent occupancies and variable speed limit intervals as control variables and parameters. Table 6.7 shows the VSL strategies employed in this section. The rules are changes with respect to the occupancy levels.

6.4.4 Definition of FTCS and integrated controls

The FTCS are examined as a single strategy and as integrated strategies of ramp metering and speed management. The strategies employed are given in the Table 6.8. The FTCS should be appropriately developed either as a stand-alone control measure or in combination with available or new ramp metering and speed management policies. The developed integrated control strategy should be able to combine FTCS with any other available control measures for best synergy in terms of achievable efficiency and equity.

Table 6.8: FTCS for simulation

Number	Strategy
1	No Control
2	FTRM
3	HSC
4	MP-ALINEA
5	MP-VSL
6	FTRM + HSC
7	FTRM + MP-VSL
8	MP-ALINEA + HSC
9	MP-ALINEA + MP-VSL

Firstly the no control case is simulated as a base case. The other strategies efficiency and equity performances are compared with respect to the no control case. Each of FTRM, HSC, MP-ALINEA and MP-VSL control strategies are examined as single strategy and later, the single ramp metering control strategies are combined with the speed control strategies. Other than those tested here, there are also some integration combinations possible such as implementation of various ramp metering strategies at different ramp locations and employment of speed management strategies at different stretches of the corridor. However, these possible combinations are further researched.

6.4.5 Determination of optimal cycle length for FTRM and optimal speed limit for HSC

The flows on O-Hashi junction and Ikejiri entrance are metered to maintain the capacity. The green ratios and the corresponding green times are calculated with the method given in chapter 3. The results show that the optimal cycle time for Route-3 is around 10 seconds. Hence, the cycle time is set as 10sec and green time is determined for each on ramp.

The HSC is also examined by employing speed limits varying from 50 km/h to 90 km/h. It is found that the optimal speed limit is around 70 km/h/. The integrated control, takes the superposition of the FTRM and HSC strategies where the 10 sec. cycle time for FTRM and 70 km/h speed limit for HSC is applied together.

6.5 Simulation results of efficiency performance

The performance measures of efficiency are selected as follows; average delay time per vehicle [s], average number of stops per vehicles, average speed [km/h], average stopped delay per vehicle [s], total delay time [h], total number of stops, total stopped delay [h] and total travel time [h] which are evaluated by averaging values of 15min. intervals for each simulation run.

Table 6.9-10 and Figure 6.8 presents the performance measure of control strategies investigated. It shows that all the traffic control strategies significantly increase the network performance. According to the results obtained, the maximum performance is obtained with the Strategy 9. The total travel time is reduced by 31.5 percent and the average speed in the corridor increased around 50 percent. All the strategies evaluated are positively affected the efficiency however there are some points that the average stopped delay per vehicle and the total stopped delay can increase for FTRM and ALINEA stand alone controls. These controls are ramp metering strategies and given the fact that the stop and go mechanism is the control base, increase in these measures are concurrent with the literature.

When the FTCS compared, it can be concluded that integrated controls shows better performances than the stand alone controls. Furthermore, the model predictive control provides the better results than the other controls. It can be concluded that the model prediction enhances the traffic control strength by the future traffic state estimation. It is also noticed that the control strategies analyzed increases the capacity of active freeway bottlenecks however none of them managed to completely eliminate the congestion activation. In addition, the model predictive integrated controls are the most responsive strategies towards the congestion formation. However, at some locations like Sangenjaya where the on-ramp merges to the rightmost lane (fast lane) the sudden speed decrease in traffic is almost inevitable most of the times.

6.6 Simulation results of equity performance

The results of the equity performances of on-ramps, mainline and the corridor is given for the indicators of variance, K_{speed} and K_{delay} in Table 6.11. From the evaluation results of on-ramps, mainline and corridor wide performances, the ramp metering control strategies causes more inequity at corridor wise.

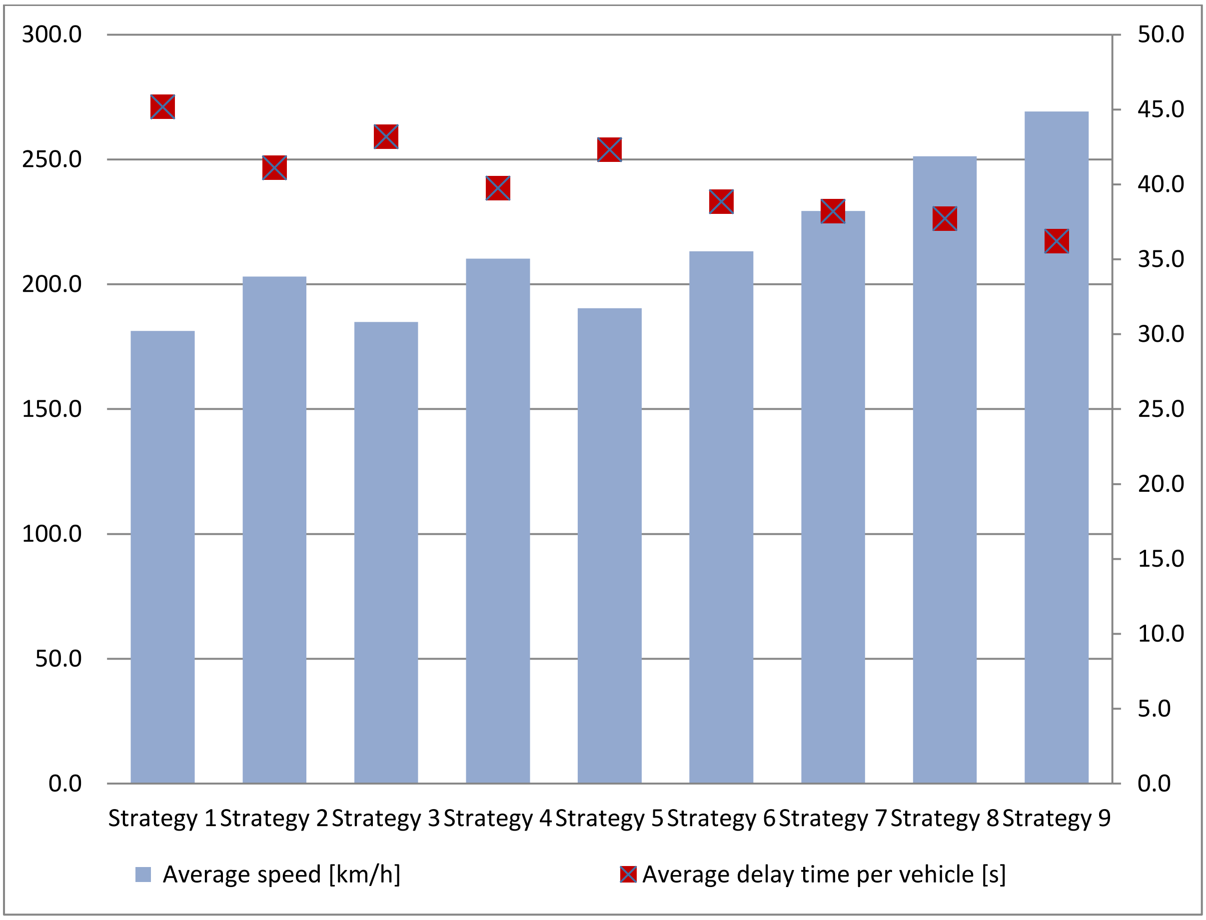

Figure 6.8: The average speed and average speed times of FTCS evaluated

However due to the reduced speed at ramps, the distribution of speed are becoming more even therefore this is considered as more equal when the variance is taken into account. When the K indexes are taken into account the free-flow speed and the optimal speeds are also considered which yields different conclusions than variance.

Table 6.9: Efficiency of FTCS

Measures of efficiency	**S1**	**S2**	**S3**	**S4**	**S5**	**S6**	**S7**	**S8**	**S9**
Average delay time per vehicle [s]	271.0	246.6	259.1	238.5	253.9	233.1	229.2	226.3	217.3
Average number of stops per vehicles	13.2	12.7	12.3	13.6	12.1	13.1	13.0	12.2	11.6
Average speed [km/h]	30.2	33.8	30.8	35.1	31.7	35.5	38.2	41.9	44.9
Average stopped delay per vehicle [s]	38.8	40.7	33.4	39.2	32.7	35.7	35.5	33.6	31.7
Total delay time [h]	785.3	722.5	738.2	675.4	723.4	702.1	640.2	569.3	552.2
Number of Stops	137974	128316	124177	126936	121693.1	123625	118133	111152	105992
Total stopped delay [h]	112.4	119.2	106.8	114.1	104.7	111.9	108.3	102.5	100.4
Total travel time [h]	1236.2	1137.3	1174.3	1100.2	1150.9	1044.8	934.9	885.6	846.8

Table 6.10: % Change in efficiency of FTCS

Measures of efficiency	% Change in comparison to no control							
	S2	**S3**	**S4**	**S5**	**S6**	**S7**	**S8**	**S9**
Average delay time per vehicle [s]	-9.0	-4.4	-12.0	-6.3	-14.0	-15.4	-16.5	-19.8
Average number of stops per vehicles	-4.0	-7.0	3.0	-8.9	-1.0	-1.5	-7.6	-12.0
Average speed [km/h]	12.0	2.0	16.0	5.1	17.6	26.5	38.6	48.5
Average stopped delay per vehicle [s]	5.0	-14.0	1.0	-15.7	-8.0	-8.4	-13.5	-18.3
Total delay time [h]	-8.0	-6.0	-14.0	-7.9	-10.6	-18.5	-27.5	-29.7
Number of Stops	-7.0	-10.0	-8.0	-11.8	-10.4	-14.4	-19.4	-23.2
Total stopped delay [h]	6.0	-5.0	1.5	-6.9	-0.5	-3.7	-8.8	-10.7
Total travel time [h]	-8.0	-5.0	-11.0	-6.9	-15.5	-24.4	-28.4	-31.5

The free-flow speed is taken as 90 km/h and it is assumed that the optimal flow can be reached at the speed of 55 km/h. These values are incorporated in K index calculations.

When equity of the on-ramps is compared through speed and delay as indicators, the results are dramatically favoring the equity of speed management policies. The possible explanation for this controversy is the harmonization effect of the control. The speeds of the vehicles in the traffic are getting closer to each other which cause either slight efficiency or considerable equity increase.

With respect to the integrated controls, it is observed that the equity of the traffic either remains the same as no control condition or there is a slight gain in equity. The most equitable FTCS among the integrated controls is the S9 which consists of MP-ALINEA and MP-VSL.

Table 6.11: Equity properties of FTCS

	On-ramp			**Mainline**			**Corridor**		
	Variance	$\mathbf{K_{delay}}$	$\mathbf{K_{speed}}$	**Variance**	$\mathbf{K_{delay}}$	$\mathbf{K_{speed}}$	**Variance**	$\mathbf{K_{delay}}$	$\mathbf{K_{speed}}$
S1	62.404	0.082	0.125	17.584	0.046	0.070	54.600	0.023	0.036
S2	48.686	0.084	0.154	17.714	0.042	0.068	32.131	0.027	0.047
S3	24.265	0.015	0.023	8.249	0.014	0.022	14.265	0.010	0.017
S4	46.643	0.081	0.127	12.253	0.039	0.061	28.596	0.024	0.037
S5	28.246	0.016	0.023	7.268	0.015	0.024	21.136	0.016	0.025
S6	38.424	0.076	0.116	9.264	0.035	0.054	20.262	0.019	0.030
S7	37.247	0.072	0.110	8.286	0.035	0.055	23.063	0.018	0.028
S8	40.255	0.065	0.099	7.251	0.029	0.046	28.243	0.016	0.025
S9	38.326	0.059	0.090	6.289	0.028	0.044	21.156	0.015	0.023

6.7 Demand sensitivity analyses

One of the most important characteristics for measuring the performance of FTCS is to evaluating the demand responsiveness of it. The demand levels would be changing the traffic flow patterns, congestion formation and duration in different ways.

In order to elucidate the effects of demand level on efficiency and equity properties of FTCS, the input demand data are varied to lower and higher levels given in Tables 6.12 and 13. The change in the traffic flows are based on the traffic flows given in Table 6.1.

Table 6.12: Low Demand Scenario (volumes in veh/hr)

O	D	Simulation Time (seconds)							
		900	1800	2700	3600	4500	5400	6300	7200
1	2	460	207	520	540	580	660	500	480
1	3	460	207	520	540	580	660	500	480
1	6	230	104	260	270	290	330	250	240
1	7	460	480	520	540	580	660	500	480
1	8	690	720	780	810	870	990	750	720

Table 6.13: High Demand scenario (volumes in veh/hr)

O	D	Simulation Time (seconds)							
		900	1800	2700	3600	4500	5400	6300	7200
1	2	507	228	573	595	639	728	551	529
1	3	507	228	573	595	639	728	551	529
1	6	254	114	287	298	320	364	276	265
1	7	507	529	573	595	639	728	551	529
1	8	761	794	860	894	960	1092	827	794

Table 6.14-18 shows the performance measure of control strategies evaluated with different demand levels. The results show that there is a significant relationship with the efficiency of FTCS and the demand levels. It is detected that the highest efficiency is achieved at the medium demand level.

For each demand level, the maximum performance is obtained with the Strategy 9. The total travel time is reduced by 27.4 and 21 percent and the average speed in the corridor increased 46.5 and 41.4 percent in low demand and high demand scenarios respectively. Although these results are lower than the medium demand level, the overall efficiency performance can be considered as more than sufficient. All the strategies evaluated are positively affected the efficiency however, as it is the case in medium demand level, there are some points that the average stopped delay per vehicle and the total stopped delay increases with FTRM and ALINEA stand alone controls.

The results of the equity performances of the studied corridor are given for the indicators of variance, K_{speed} and K_{delay} in Table 6.14. From the evaluation results of corridor wide performances, the general pattern in equity with variance measure is 'the higher the demand level, the higher the equity'. The congestion causes the vehicles to travel with lower and closer speeds which yield an increase in equity state. On the other hand, when the K indexes are considered, the equity state is more stable despite a slight increase is observed with the increase in demand levels.

Table 6.14: Equity properties of the corridor with different demand levels

	Low Demand			**Medium Demand**			**High Demand**		
	Variance	K_{delay}	K_{speed}	**Variance**	K_{delay}	K_{speed}	K_{delay}	K_{speed}	K_{delay}
S1	79.076	0.025	0.039	74.600	0.023	0.036	71.616	0.018	0.028
S2	97.659	0.032	0.050	92.131	0.027	0.047	88.446	0.017	0.027
S3	57.521	0.022	0.034	54.265	0.010	0.017	52.094	0.019	0.030
S4	93.912	0.038	0.060	88.596	0.024	0.037	85.052	0.016	0.025
S5	64.804	0.014	0.022	61.136	0.016	0.025	56.267	0.015	0.023
S6	85.078	0.026	0.041	80.262	0.019	0.030	77.052	0.016	0.025
S7	88.047	0.025	0.039	83.063	0.018	0.028	79.740	0.015	0.024
S8	82.938	0.024	0.038	78.243	0.016	0.025	75.113	0.014	0.022
S9	86.025	0.023	0.036	81.156	0.015	0.023	77.910	0.013	0.020

6.8 Conclusions

This chapter presented two new extensions to the model predicted controls among the previously proposed control strategies. The integration of standalone controls are examined though traffic simulation study. The equity and efficiency performances of each strategy is computed and compared.

The results reveal that the speed increase (gain) comes up with an equity decrease (loss) for the ramp metering strategies. However, the trade-off is not the general pattern. The speed management control achieves the increase in both efficiency and equity at the same time. The results are concurrent with the results of previous chapters.

The overall efficiency gain with FTCS in all demand levels is substantial. All the strategies evaluated are positively affected the efficiency however, as it is the case in medium demand level, there are some points that the average stopped delay per vehicle and the total stopped delay increases with FTRM and ALINEA stand alone controls.

Table 6.15: Efficiency of FTCS with low demand

Measures of efficiency	**S1**	**S2**	**S3**	**S4**	**S5**	**S6**	**S7**	**S8**	**S9**
Average delay time per vehicle [s]	233.6	220.0	231.2	212.8	223.5	208.0	204.5	201.9	193.8
Average number of stops per vehicles	11.5	11.4	11.0	12.2	10.7	11.7	11.7	11.0	10.4
Average speed [km/h]	38.2	42.2	38.4	43.7	41.1	44.3	47.7	52.2	56.0
Average stopped delay per vehicle [s]	26.0	27.5	22.5	26.4	21.8	24.1	24.0	22.6	21.4
Total delay time [h]	738.2	684.2	699.1	639.6	676.0	664.8	606.2	539.1	523.0
Number of Stops	130468.2	120486	114251	116790	110480.7	113743	108691	102267	97520
Total stopped delay [h]	104.5	108.8	97.5	104.2	94.3	102.2	98.9	93.6	91.7
Total travel time [h]	1178.1	1153.0	1143.0	1111.4	1105.3	1055.4	944.4	894.6	855.4

Table 6.16: % Change in efficiency of FTCS with low demand

Measures of efficiency	% Change in comparison to no control							
	S2	**S3**	**S4**	**S5**	**S6**	**S7**	**S8**	**S9**
Average delay time per vehicle [s]	-5.8	-1.1	-8.9	-4.3	-11.0	-12.5	-13.6	-17.0
Average number of stops per vehicles	-1.4	-4.5	5.8	-7.6	1.7	1.2	-5.1	-9.6
Average speed [km/h]	10.5	0.6	14.4	7.4	16.0	24.8	36.7	46.5
Average stopped delay per vehicle [s]	5.7	-13.4	1.6	-16.3	-7.4	-7.8	-12.9	-17.8
Total delay time [h]	-7.3	-5.3	-13.4	-8.4	-9.9	-17.9	-27.0	-29.2
Number of Stops	-7.7	-12.4	-10.5	-15.3	-12.8	-16.7	-21.6	-25.3
Total stopped delay [h]	4.1	-6.7	-0.3	-9.8	-2.3	-5.4	-10.4	-12.3
Total travel time [h]	-2.1	-3.0	-5.7	-6.2	-10.4	-19.8	-24.1	-27.4

Table 6.17: Efficiency of FTCS with high demand

Measures of efficiency	S1	S2	S3	S4	S5	S6	S7	S8	S9
Average delay time per vehicle [s]	320.7	308.8	324.4	298.6	318.3	291.8	287.0	283.4	272.0
Average number of stops per vehicles	15.5	15.6	15.1	16.7	15.0	16.1	16.0	15.0	14.3
Average speed [km/h]	24.9	26.6	24.2	27.5	24.9	27.9	30.0	32.9	35.3
Average stopped delay per vehicle [s]	57.8	59.8	52.3	56.2	53.4	53.1	51.8	52.2	50.6
Total delay time [h]	857.2	808.7	814.5	803.2	812.4	796.5	784.5	776.6	751.4
Number of Stops	149764.6	151343	143515	150038	143498.0	142993	141636	133469	132295
Total stopped delay [h]	123.9	129.4	112.9	120.1	110.0	116.2	114.6	113.1	109.4
Total travel time [h]	1332.1	1231.5	1298.3	1200.2	1283.4	1189.2	1159.2	1106.5	1052.4

Table 6.18: % Change in efficiency of FTCS with high demand

Measures of efficiency	% Change in comparison to no control							
	S2	S3	S4	S5	S6	S7	S8	S9
Average delay time per vehicle [s]	-3.7	1.2	-6.9	-0.7	-9.0	-10.5	-11.6	-15.2
Average number of stops per vehicles	0.9	-2.2	8.3	-3.0	4.1	3.6	-2.9	-7.5
Average speed [km/h]	6.7	-2.9	10.5	-0.2	12.0	20.5	32.0	41.4
Average stopped delay per vehicle [s]	3.4	-9.6	-2.8	-7.7	-8.2	-10.5	-9.8	-12.5
Total delay time [h]	-5.7	-5.0	-6.3	-5.2	-7.1	-8.5	-9.4	-12.3
Number of Stops	1.1	-4.2	0.2	-4.2	-4.5	-5.4	-10.9	-11.7
Total stopped delay [h]	4.4	-8.9	-3.1	-11.2	-6.2	-7.5	-8.7	-11.7
Total travel time [h]	-7.5	-2.5	-9.9	-3.7	-10.7	-13.0	-16.9	-21.0

The maximum efficiency can be reached around the demand levels close to the capacity. The equity of the traffic state around capacity flows shows more equitable conditions than the lower demand situations nevertheless the equity state is not as good as the high demand levels.

Finally, the change in magnitude of equity with respect to the demand level is varies. The highest efficiency is achieved at medium demand level however, the traffic states turn to more equal when the demand levels becomes higher. The possible reason behind these results is the congestion formation at the corridor. When the demand is manageable at the corridor, the efficiency of control strategies remains limited and the speeds differences can be high as well. If the demand exceeds the capacity, the FTCS are not useful in alleviating the congestion anymore thus the speed differences are getting smaller with a lower average speed meaning a more equitable traffic state.

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CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

In this section the objectives are restated and the findings are summarized.

Provided that the congestion is one of the biggest problems, alleviation of it is highly important for almost every city in the world. Engineers are mostly keen on solving the problems of a system by understanding the components and the relationships between the components of the system. Usually the solutions to a problem lie under the control of the system. However, the control systems should be viable. Even though some traffic control systems are feasible in the long run, due to the excessive costs of congestion, the initial costs are still unbearable for most of the countries in the world. Therefore the low-cost traffic control strategies should be developed.

In some other disciplines of engineering, the control mechanisms apply to the objects, materials or data. However, the biggest concern in traffic control is when the vehicles in the traffic are controlled; this implicitly means that the people inside these vehicles are also controlled. Therefore, an extreme attention should be given to the assessment of the consequences of traffic control before implementation. The first step in this direction would be determination of the consequences in terms of travel time savings, reduction in delay and emissions and the equity of the traffic. Most of these measures can be easily measured but the measurement of equity is probably the one of the oldest debates in the history of humankind and yet there is not a widely accepted straightforward approach established.

In this dissertation, alternative traffic control strategies and a novel equity measurement approach are been addressed in chapters 3 and 4. Subsequently the proposed approaches are tested with two real-world examples in traffic micro-simulation environment as case studies from Istanbul, Turkey and Tokyo, Japan.

An important advantage of the application of FTRM control in traffic management is the potential to improve the traffic operation on the motorway at minimal costs. The acceptance of ramp metering traffic control by the public is not hindered by the need for the drivers to invest in in-vehicle technology. From the control point of view it is important to note that ramp metering can easily be enforced, e.g., by a red-light running camera. Since the control can be enforced, we can assume that a control signal that is applied to the system is also followed up exactly. This simplifies the development of the controller.

From the traffic control literature on equity issues, there is conceptualization problem of equity in traffic control where most of the indices are barrowed from econometric studies that are usually incorporating the differences of pairs among the mean values. There are also some other problems detected in terms of mathematical formulation of equity, for instance when the delay values are incorporated with Gini Index this would yield 0/0 indeterminate form if there is no delay in traffic.

In order to solve the problems encountered, a new equity index, namely K, is proposed to assess the equity of traffic considering the mathematical and theoretical subjects. The conceptualization of equity in K index is based on the notion that the ratio of the speed differences (vehicles or time frames) over a specific value which reflects the optimal flow. The equity index should be based on the concept that the equity state should be depending on the differences among the peers but not the mean value of the peers.

The distinctiveness of the K index compared to other indices is therefore twofold. Firstly, the index measures the ratio of the sum of the speed differences of each vehicle and the free-flow speed, which indicates the equity of the delay in the network. Secondly, when the denominator part is altered with the optimum speed where the capacity is reached, then the index shows the equity of speed in the network.

FTRM and HSC strategies are developed as cost efficient alternatives to the existing control methods. Fixed-time ramp metering strategy needs only simple adjustments on ramp layout and a signal device would be sufficient to manage the control.

The admissible ramp flow under full traffic cycle metering policy usually much higher than the one / n cars per green metering policy and this represents the major advantage of the full-cycle policy. However, there is no established method for the specification of optimum cycle (length) time for fixed time ramp metering. Therefore, the traditional analytical models developed for fixed time intersection control are examined and capacity maximization approach is modified for fixed time ramp metering simulation experiment.

The proposed HSC system adjusts the set of displayed the speed limits for the congested peak periods where the speed limits are pre-determined that aims to minimize the selected measures of effectiveness for the examined corridor. The main difference of this control is the intervention to the speed limit boundaries. The conventional speed limit controls only deals with the upper speed limit. However the lower speeds are also considered as a critical part of an effective control and these limits are also needed to be examined. The research question focus is the investigation of the bounded speed limits effects on prevention of congestion and on congestion alleviation.

The optimality of the FTRM and HSC strategies are searched with respect to the minimization of the total travel times. A set of traffic simulation experiments is designed and applied in Chapters 5 and 6. The fixed-time ramp metering considers bottleneck capacity, thus from this point the control system works like demand-capacity control. The offline demands are measured and the maximum admissible ramp volumes transformed into the green times for the control. The speed harmonization control applies the speed limits more particularly the speed ranges for the whole network.

This research has presented the results of applying the fixed time ramp metering, speed harmonization and integrated control for the regulation of the inflow from on ramps to the mainstream of a highway corridor.

The summary of results is given below;

i. There is an optimum cycle time that can be determined for each on ramp considering the bottleneck downstream capacity and flows. The short cycle lengths have a tendency to increase the start-up lost times and limits the merging rate, therefore the delay increases rapidly. Long cycle lengths allows platoon of vehicles entering the mainline which also contributes an increase in delay. The result of the study is highly congruent with previous findings on the sensitivity of delay to cycle length for intersections exhibited on 16-16 at Highway Capacity Manual.

ii. Speed limit control strategy has also an optimal value which optimizes the performance measures. One of the possible reasons behind this is, at low speed limits the average speed drops under the effective level and at higher speeds drivers are applying more aggressive breaking which causes sudden shock waves.

iii. Ramp metering and speed limit control integration significantly increase the performance even without any coordination and ramp metering depicts a relatively better performance increase in total travel time savings than speed limit control.

iv. Another important output of this analysis is that all the control strategies analyzed increase the effectiveness of the traffic flow referring the total travel time, the total delay, the number of stop and average speed, in a satisfied manner. One of the reasons behind this extreme performance increase is not regarding the queue spillover effect. The ramp queues would be extended due to the control implementation in reality which yields a decrease in on ramp flow performance. Besides this, strict metering (high rate of ramp metering) brings equity concerns for ramp users and makes ramp metering a difficult policy to be accepted by society.

v. The change in magnitude of equity with respect to the measure and indicator taken into account could be vast. Thus, first the equity indicators and measures should be evaluated carefully and then the performance should be interpreted accordingly to make a conclusive judgment.

vi. The traffic control strategies evaluated in this thesis shows that the total delays along the stretch can be reduced through the increased capacity at bottleneck location. According to the results, the conventional speed limit control strategy performs better than the no control case in every performance measures taken account in this paper. It is considered that the speed harmonization effect can help to alleviate the congestion even without a dynamic control approach.

vii. Another interesting result of this analysis is that the (MP-VSL) control strategy increases the effectiveness of the traffic flow referring the total travel time, the total delay, the number of stop and average speed, in a satisfied manner and even performs better than the conventional speed limit control. Especially the remarkable decrease in total delay and average stop delay per vehicle measures shows that the proposed algorithm establishes a smoother traffic flow.

viii. The change in magnitude of equity with respect to the measure and indicator taken into account could be vast. Thus, first the equity indicators and measures should be evaluated carefully and then the performance should be interpreted accordingly to make a conclusive judgment.

ix. The results also reveal that the speed increase (gain) comes up with an equity decrease (loss) for the ramp metering strategies. However, the trade-off is not the general pattern. The speed management control achieves the increase in both efficiency and equity at the same time. The results are concurrent with the results of previous chapters.

x. The overall efficiency gain with FTCS in all demand levels is substantial. All the strategies evaluated are positively affected the efficiency however, as it is the case in medium demand level, there are some points that the average stopped delay per vehicle and the total stopped delay increases with FTRM and ALINEA stand alone controls.

xi. The maximum efficiency can be reached around the demand levels close to the capacity. The equity of the traffic state around capacity flows shows more equitable conditions than the lower demand situations nevertheless the equity state is not as good as the high demand levels.

xii. Finally, the change in magnitude of equity with respect to the demand level varies. The highest efficiency is achieved at medium demand level however, the traffic states turn to more equal when the demand levels becomes higher. The possible reason behind these results is the congestion formation at the corridor. When the demand is manageable at the corridor, the efficiency of control strategies remains limited and the speeds differences can be high as well. If the demand exceeds the capacity, the FTCS are not useful in alleviating the congestion anymore thus the speed differences are getting smaller with a lower average speed meaning a more equitable traffic state.

Through the proposed control strategies and equity index, this thesis is considered to bring a substantial contribution to the enrichment of the theoretical and practical area of the traffic control design, as well as to the ways of improving the assessment of traffic control implementation in practice.

7.2 Future Directions

There are some topics could be further researched utilizing the input of this study. First of all, the effects of other traffic control strategies such as route guidance, vehicle-vehicle, vehicle-infrastructure communication systems and other ramp metering control algorithms on efficiency and equity could be further analyzed. Especially route guidance and the dynamic traffic assignment models are may give some more insights about the efficiency and equity properties of traffic control. Re-routing and incorporation of the induced demand in traffic flow model or transport model by considering the other modes would be more comprehensive and realistic.

The effects of different level traffic flow models such as mesoscopic and macroscopic models can be examined with the same methodologies and the results can be compared. Also investigation of the effects of different freeway merging and lane changing models on the efficiency and equity would be an interesting research topic. There are also some simulation parameters considered as critically important as the compliance rate of drivers and dangerous driving.

The environmental considerations besides efficiency and equity are also crucial for sustainable freeway traffic. Thus, the changes in environmental aspects with traffic control strategies in terms of emissions and traffic related noised could be further analyzed.

Finally, in order to establish a balanced traffic management weighting of the efficiency and equity measures are highly decisive. Drivers evaluate the traffic and the congestion subjectively and make decisions based on those subjective experiences; future research should be aimed to understand the underlying reasons of the differences in evaluation and decision making. The valuation of equity and the valuation of ramp or mainline waiting among the efficiency are also regarded as critical in traffic management.

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***

APPENDICES

APPENDIX A

```
PROGRAM MP_ALINEA;

VAP_FREQUENCY 1;
ARRAY regConst[4]=[0, 0.66, 0.15, 0.19],
    OccupancyRate[5]=[0,0,0,0,0];

IF FirstRun = 0 THEN
        Trace(variable(FirstRun));
        FirstCycle := 0;
        minGreenTime:=5;
        rSaturated:=2000;
        desiredOccupancyRate:=0.30;
        measuredOccupancyRate:=0;
        KR:=70;
        cycleTime:=15;
        greenTime:=15;
        cTstart:=0;
        cycleSecond:=cTstart+2;
        cTend:=cTstart+cycleTime-1;
        gTstart:=cTend-greenTime+1;
        gTend:=cTend;
        FirstRun:=1;
END;
IF cycleSecond < cTend THEN
        IF (gTstart<=cycleSecond) AND (cycleSecond<gTend) THEN
                Sg_green(1);
                Sg_green(2);
        ELSE
                Sg_red(1);
                Sg_red(2);
        END;
```

```
        oRate:=(Occup_rate(1)+Occup_rate(2)+Occup_rate(3)+Occup_rate(4)+Occup_rate(5)
+Occup_rate(6))/6;
        measuredOccupancyRate:=measuredOccupancyRate+oRate;

        Trace(variable(oRate));
        Trace(variable(cycleSecond));
        set_cycle_second(cycleSecond);
        cycleSecond:=cycleSecond+1;
ELSE
        IF FirstCycle = 0 THEN
                r:=rSaturated;
                Trace(variable(FirstCycle));
                FirstCycle:=1;
        ELSE
                OccupancyRate[1]:=OccupancyRate[2];
                OccupancyRate[2]:=OccupancyRate[3];
                OccupancyRate[3]:=OccupancyRate[4];
                OccupancyRate[4]:=measuredOccupancyRate/cycleTime;

OccupancyRate[5]:=OccupancyRate[4]*regConst[4]+OccupancyRate[3]*regConst[3]+Occup
ancyRate[2]*regConst[2]+OccupancyRate[1]*regConst[1];
                r:=r+KR*(desiredOccupancyRate-OccupancyRate[5]);
                IF r>rSaturated THEN
                        r:=rSaturated;
                ELSE
                        IF r<rSaturated*(minGreenTime/cycleTime) THEN
                                r:=rSaturated*(minGreenTime/cycleTime);
                        END;
                END;
        END;
        Trace(variable(r));
        Trace(variable(cycleSecond));
        set_cycle_second(cycleSecond);
        cycleSecond:=cTstart;
        greenTime:=(r/rSaturated)*cycleTime;
        IF greenTime<minGreenTime THEN
                greenTime:=minGreenTime;
```

```
        ELSE
                IF greenTime>cycleTime THEN
                        greenTime:=cycleTime;
                ELSE
                        greenTime:=greenTime;
                END;
        END;
        gTstart:=cTend-greenTime+1;
        measuredOccupancyRate:=0;
END;
Trace(variable(greenTime));
Trace(variable(gTstart)).
```

***

APPENDIX B

```
PROGRAM MP_VSL;

VAP_FREQUENCY 1;
ARRAY regConst[4]=[0, 0.66, 0.15, 0.19],
    OccupancyRate[5]=[0,0,0,0,0];
IF FirstRun = 0 THEN
        measuredOccupancyRate:=0;
        cycleTime:=20;
        cTstart:=0;
        cycleSecond:=cTstart+2;
        cTend:=cTstart+cycleTime-1;
        FirstRun:=1;
ELSE
        IF cycleSecond < cTend THEN
                /* */

                /* */

        occup:=(Occup_rate(1)+Occup_rate(2)+Occup_rate(3)+Occup_rate(4)+Occup_rate(5)
+Occup_rate(6))/6;
                measuredOccupancyRate:=measuredOccupancyRate+occup;
                cycleSecond:=cycleSecond+1;
        ELSE
                OccupancyRate[1]:=OccupancyRate[2];
                OccupancyRate[2]:=OccupancyRate[3];
                OccupancyRate[3]:=OccupancyRate[4];
                OccupancyRate[4]:=measuredOccupancyRate/cycleTime;

        OccupancyRate[5]:=OccupancyRate[4]*regConst[4]+OccupancyRate[3]*regConst[3]
+OccupancyRate[2]*regConst[2]+OccupancyRate[1]*regConst[1];
                /* */
```

```
IF OccupancyRate[5]<0.1 THEN
        desiredSpeedForCarsMainStream:=1;
        desiredSpeedForHGVMainStream:=1;
        desiredSpeedForCarsRamp:=1;
        desiredSpeedForHGVRamp:=1;
ELSE
        IF OccupancyRate[5]<0.15 THEN
                desiredSpeedForCarsMainStream:=2;
                desiredSpeedForHGVMainStream:=2;
                desiredSpeedForCarsRamp:=2;
                desiredSpeedForHGVRamp:=2;
ELSE
        IF OccupancyRate[5]<0.20 THEN
                desiredSpeedForCarsMainStream:=3;
                desiredSpeedForHGVMainStream:=3;
                desiredSpeedForCarsRamp:=3;
                desiredSpeedForHGVRamp:=3;
ELSE
        IF OccupancyRate[5]<0.25 THEN
                desiredSpeedForCarsMainStream:=4;
                desiredSpeedForHGVMainStream:=4;
                desiredSpeedForCarsRamp:=4;
                desiredSpeedForHGVRamp:=4;
ELSE
        IF OccupancyRate[5]<0.30 THEN
                desiredSpeedForCarsMainStream:=5;
                desiredSpeedForHGVMainStream:=5;
                desiredSpeedForCarsRamp:=5;
                desiredSpeedForHGVRamp:=5;
ELSE
        IF OccupancyRate[5]<0.35 THEN
                desiredSpeedForCarsMainStream:=6;
                desiredSpeedForHGVMainStream:=6;
                desiredSpeedForCarsRamp:=6;
                desiredSpeedForHGVRamp:=6;

                        END;
```

```
                END;
            END;
        END;
        Set_des_speed(1,10,desiredSpeedForCarsMainStream);
        Set_des_speed(1,20,desiredSpeedForHGVMainStream);
        Set_des_speed(2,10,desiredSpeedForCarsMainStream);
        Set_des_speed(2,20,desiredSpeedForHGVMainStream);
        Set_des_speed(3,10,desiredSpeedForCarsMainStream);
        Set_des_speed(3,20,desiredSpeedForHGVMainStream);
        Set_des_speed(4,10,desiredSpeedForCarsMainStream);
        Set_des_speed(4,20,desiredSpeedForHGVMainStream);
        Set_des_speed(5,10,desiredSpeedForCarsRamp);
        Set_des_speed(5,20,desiredSpeedForHGVRamp);
        Set_des_speed(6,10,desiredSpeedForCarsRamp);
        Set_des_speed(6,20,desiredSpeedForHGVRamp);

        Trace(Variable(OccupancyRate[4]));
        Trace(Variable(desiredSpeedForCarsMainStream));
        Trace(Variable(desiredSpeedForHGVMainStream));
        Trace(Variable(desiredSpeedForCarsRamp));
        Trace(Variable(desiredSpeedForHGVRamp));
        /* */
        cycleSecond:=cTstart;
        measuredOccupancyRate:=0;
    END;
END;
set_cycle_second(cycleSecond).
```

***