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Macroscopic Delay Models for the Analysis of Transportation System Management Options

By

**Khaled N. Helali
B.Sc. (Civil)**

A thesis submitted to
the Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of

MASTER OF ENGINEERING

**DEPARTMENT OF CIVIL ENGINEERING
CARLETON UNIVERSITY
OTTAWA, CANADA**

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The undersigned hereby recommend to the
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acceptance of the thesis entitled

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Transportation System Management Options”**

submitted by

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in partial fulfilment of the requirements for the degree of

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ABSTRACT

During the last two decades emphasis in transportation planning has shifted from long-term, capital-intensive construction projects to short and medium-term, relatively low-capital cost projects aimed at using existing transportation facilities more efficiently. This trend integrated Transportation System Management (TSM) activities into the overall transportation planning process as a prerequisite prior to initiating major capacity expansion types of capital works.

Although the TSM initiatives are undertaken in response to congestion (delay), energy, environment and safety concerns, reduction of delay is a key Measure Of Effectiveness (MOE). Despite, the importance of this MOE, there is a lack of efficient tools which are capable of estimating delay corresponding to TSM measures (i.e., low-cost planning process). Most if not all the existing delay models have been developed for detailed operational analysis. These models are data-intensive and require highly specialized human and machine types of resources.

In this thesis, multi-link streets and networks were synthesized and their performance in delay terms was estimate through micro-simulation. The output of such simulations, in conjunction with physical and traffic characteristics of these systems were used to develop multiple regression types of macro-simulation models of delay. The models developed are capable of estimating vehicle delay for both urban streets and networks. Field data were collected for model validation by using the videotape technique and maps. The models have proven to be valid and accurate. These macro-simulation models were successfully tested against the micro-simulation technique and the 1985 Highway Capacity Manual procedures. Finally a number of illustrative cases are included in this thesis which highlight the practical applications of the models.

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LIST OF SYMBOLS

AOV	Analysis of Variance
BSTOP	A Dummy Variable which Gives The Effect of The Location of Bus Stop on Total Delay
CBD	Central Business District
DEL	Total Vehicle Delay for The Entire Street (vehicle.minute)
FHWA	Federal Highway Administration
fps	Feet per Second
ft	Feet
HCM	Highway Capacity Manual
HCS	Highway Capacity Software
HV	Percent of Heavy Vehicles in Total Traffic (includes trucks and buses other than transit buses)
ITT	Ideal Travel Time (vehicle.minute)
K	Average Network Concentration (vehicle/lane.mile)
LT	Average Percent of Vehicles Turning Left for The Entire Street
man/hr	Maneuver per Hour
MOE	Measures of Effectiveness
NBUS	Number of Buses per Hour Running on That Particular Street
NLAN	Number of Moving Lanes for each Street
OCCUP	Link Average Occupancy in Terms of Vehicles or Link Mean Content in Terms of Vehicles
ped/hr	Pedestrian per Hour
Q	Average Network Flow (vehicle per hour)
RTAC	Roads and Transportation Association of Canada
sec	Second
sec/veh	Second per Vehicle
SIGPM	Average Number of Signals per Mile on That Particular Street
STOP	Percent of Vehicles Stopped at Least Once of Total Number of Vehicles and averaged over The Entire Street
TSM	Transportation System Management
V	Average Network Speed (mile per hour)
veh/hr	Vehicle per Hour
veh.min	Vehicle.Minute

CHAPTER 1

INTRODUCTION

1.1 Background

During the last two decades, events have focused attention to the need for providing safe, economic and efficient movement of people on existing road facilities [1]. The former practice of developing urban transportation infrastructures, without regard to cost-effectiveness, has changed to the efficient management of existing facilities and services. As a result, the concept of Transportation System Management (TSM) was developed.

In its original definition of the TSM, the Federal Highway Administration (FHWA) [40 Federal Register 42976-42984 (1975)], stated that the objective was to coordinate the individual elements through operating, regulatory and service policies in order to achieve maximum efficiency and productivity for the system [2].

Later, a modified TSM concept called the Urban Transportation System Management emerged. It is a process of coordinating the individual elements of a system through operating, regulatory, pricing, service and investment policies so as to achieve maximum efficiency and productivity for the whole system, with monitoring as an essential part of the process [2, 3].

While some researchers consider the TSM a short-range planning and implementation process, others, owing to infrastructure modification type of improvements, consider it as a middle-range planning process. Whether it is short-range or middle-range planning process, the main concern in this thesis is that TSM is a low-capital cost planning process and it is quite different from a long-range planning process. The main differences between TSM and long-range planning are given in Table 1.1.

Table 1.1 Key Differences Between TSM and Long-range Planning, [4]

	<u>TSM</u>	<u>Long-Range</u>
Problems	Clearly defined, observable	Dependent on growth scenarios and projected travel
Scale	Usually local, subarea or corridor	Usually corridor or regional
Objectives	Problem-related	Broad, policy-related
Options	Few specific actions	Several modal, network, and alignment alternatives
Analysis Procedures	Usually analogy or simple operational	Based on trip and network models
Response Time	Quick response is essential	Not critical
Product	Design for implementation	Preferred alternatives for further study or detailed design

TSM actions are initiated to accomplish several main goals which can be summarized as follows [5,6]:

- to maintain and/or improve the quality of the transportation services;
- to increase the efficiency of the existing system;
- to minimize the cost of maintenance and improvements;
- to minimize undesirable environmental impacts; and
- to promote desirable and reduce undesirable social and economic impacts.

These goals in turn lead to a large number of objectives such as:

- minimizing travel time,
- maximizing public transport (ridership, coverage, etc.),
- improving road safety,
- minimizing undesired fuel emissions and more.

The above objectives are assessed in terms of Measures Of Effectiveness (MOE) such as, vehicle delay, vehicle hours of travel, traffic volumes, number of accidents, and transit passenger-miles of travel.

One of the most important MOE is vehicle delay. Its importance stems from the following:

1. Delay is a very important measure of effectiveness in traffic, since it represents the direct cost of fuel consumption and the indirect costs of vehicle users [7, 8, 9].
2. Delay is a key MOE in the Highway Capacity Manual (HCM) for signalized intersections.
3. Delay was chosen as a proxy for congestion since it is widely accepted as a key MOE in major system improvement programs such as "The Future Program 1991 and beyond" sponsored by the FHWA [7].
4. Delay can be calculated from existing data and may be estimated from simulation studies.

A review of the literature on what delay models were used or suggested to be used by TSM users, resulted in the following summary.

Lockwood, Wagner, Voorhees and Associates gave a methodological framework for the TSM planning process suggesting the use of traffic simulation models in estimating delay [10].

The user's manual of Simplified Procedures For Evaluating Low-Cost TSM Projects (which is one of the most comprehensive reports on TSM) suggested the use of traffic simulation models, such as **TRANSYT**, **NETSIM**, **SOAP** and others for estimating delay [4].

Giuliano, in his analysis of the impacts of TSM program employed during the 1984 Los Angeles Summer Olympics, stated that traffic simulation models, such as **NETSIM** and **TRANSYT** were used [11].

Another approach followed for estimating delay is the use of the HCM 1985. For example, Levinson, Golenberg, and Zografos in their discussion of impact assessment techniques applied to TSM actions suggested the use HCM 1985 for estimating delay [12].

1.2 Review of Different Delay Models

As discussed above, both simulation models such as **SOAP**, **TRANSYT**, and **NETSIM**, and the **HCM** were used or suggested to be used by TSM users to estimate delay.

This section contains brief discussions of these delay models, including the well-known Webster's delay model. The Webster's delay model is the basic theoretic formulation among most of the simulation models in practice. It estimates delay for a single approach of an intersection controlled by a pretimed traffic signal. The model states that the average delay per vehicle for a single approach is a function of the degree of saturation (the ratio of the traffic flow to the maximum possible traffic flow that could be accommodated under the given settings of signal), the cycle length, the traffic flow (average number of vehicles passing a given point on the road in the same direction per unit time), and the effective green time (the sum of the green and the amber period minus

the lost time for a particular phase). It consists of three terms. The first term represents the average delay per vehicle assuming that vehicles arrive at uniform flow rate throughout the cycle. The second term captures the randomness of the vehicle arrivals during the cycle. The last term of the equation is an adjustment factor.

A wide variety of information of the prevailing road conditions is needed. This includes the amount and the distribution of the traffic movements, traffic composition, geometric characteristics, and the details of intersection signalization.

The Signal Operation Analysis Package (SOAP), is a computerized method for developing control plans and evaluating the operations of individual signalized intersections [13,14]. It is a deterministic, macroscopic model based on simple equations. The SOAP delay model is composed of Webster's first term for uniform arrivals and the random and saturation delay term for TRANSYT-6 (which is a function of the same variables as for Webster's delay model) with adjustments for actuated signal control and/or a combination of a high degree of saturation with low volumes [8].

Delay is defined in SOAP as the difference between the average travel time through the intersection and the travel time of a vehicle that is not stopped or slowed down by a signal. The data required by SOAP are almost the same as required by the Webster formulation, but more detailed information on the signalization characteristics is required.

The third delay model is a system optimization and performance estimation computer program called TRANSYT. It can be applied for both arterials and networks. TRANSYT is a macroscopic model that uses platoons in small time increments along with a platoon dispersion algorithm. The model is composed of three terms, uniform delay, random delay, and delay due to oversaturation. The first term of the model, which represents the uniform delay, is a function of the cycle length, queue length during one time step and the number of time steps in the cycle length. The second and the third terms of the model capture the effect of random arrival of the vehicles and oversaturation on delay. These two terms are functions of the degree of saturation and the volume on the link.

It should be noticed that it had been proven that the uniform delay term of

Webster's model is exactly the same as the uniform delay of TRANSYT [8,15]. Delay in TRANSYT is defined as the difference between total travel time and ideal travel time for each link, based on a designated target speed for a vehicle not stopped or slowed down by a signal.

Finally, TRANSYT is a data intensive program, which requires a considerable amount of data collection and reduction. It is very complex in comparison to the models mentioned earlier.

The last simulation model discussed in this section is NETSIM. It is stochastic, microscopic, digital simulation model that handles each vehicle separately. NETSIM is based on car-following and lane-changing rules. It considers different vehicle types and also recognizes conflicts between left turns and oncoming traffic as well as the impact of traffic that is backed up from the preceding intersection. NETSIM defines delay as the difference between the total travel time and idealized travel time for each link based on a designated speed [16,17]. More details on NETSIM are presented in the next chapter. Due to the microscopic nature of NETSIM, it is even more cumbersome and data-intensive than TRANSYT.

The HCM (1985) delay model presents another approach to estimate delay for TSM users. The model is based on a relationship that was derived from empirical basis and predicts the average stopping delay per vehicle for a given arrival pattern for approaching vehicles.

The HCM delay model also consists of two terms. The first term of the equation accounts for uniform delay (i.e., the delay that occurs if the arrival demand in the subject lane group is uniformly distributed over time). The second term of the equation accounts for incremental delay of random arrivals over uniform arrivals, and for the additional delay due to cycle failure. In order to estimate approach delay, stopped delay is multiplied by a factor 1.3 [8,18].

Similar to the above delay models, the HCM delay model needs detailed information on the roadway conditions, traffic conditions, and signalization conditions. Besides, all the signalization characteristics which are required by all the delay models in general, which include a full definition of the signal phasing, timing, and type of

control, a very important piece of information is needed for the HCM delay model which is the quality of progression on each approach of the intersection [8,18].

1.3 The Rationale for this Research

It is clear from section 1.2 that all the delay models described earlier have been developed mainly for operational analysis. In other words detailed information on all traffic, roadway, and signalization conditions for each intersection have to be available.

All of these models estimate delay within an intersection and few of them estimate delay for the entire street or network. Models that have the capability to estimate delay for central business district (CBD) streets or networks, such as NETSIM and TRANSYT, are more data intensive than those estimating delay for intersections. None of the these delay models mentioned above deal with aggregate level of input.

Given the existing models, a number of questions require answers. Do the staff of most public agencies have the time and skills to learn and understand complicated packages like NETSIM and TRANSYT?

Can the data requirements for these models be met with the existing resources of all users or do they need special data acquisition budgets?

Quite often, TSM planners need to estimate delay for the entire system whether it is a street or a network since the overall performance of any system is a very important factor in cost-effectiveness studies. There is a number of TSM actions, such as most demand-side TSM actions, which need to be assessed using delay. At the same time there is no need to go into signalization and traffic details required by the existing models.

From all that has been discussed so far, it is clear that there is a strong need to develop new delay models that are aggregate both in nature and in their input requirements. They should have macro level functions. Besides, they should be easy to use and consistent with TSM planning process which by definition is a low-capital planning process.

1.4 Objectives and Scope of this Research

The objectives of this research are:

1. to investigate the possibility of developing a delay model of macroscopic nature for multi-link streets;
2. to investigate the possibility of developing a delay model capable of estimating delay for the entire network based on a small number of network-level descriptors;
3. to study the effect of the different roadway, traffic and signalization characteristics on the total delay for CBD streets and networks; and
4. to investigate describing the road traffic characteristics in CBD networks by a small number of network-level descriptors and quantifying their interrelationship.

This research is mainly concentrated on Central Business District street networks. Roadway, traffic and signalization characteristics and their effect on urban street delay are studied.

The research carried out through this thesis is limited to peak hour conditions, since peak hour conditions are key factors in planning, design, and design of improvements to urban street networks.

1.5 Thesis Outline

The thesis is organized in eight chapters and two appendices. **Chapter 2** describes the research framework, and the simulation experiment design and setup. The methodology used to create the input data is discussed. A brief explanation of the nature of the simulation model NETSIM is given.

In **chapter 3**, the implementation of the simulation experiment is described. Traffic signal timing and optimization using **PASSER II-87** is noted. A description of the final run of NETSIM, resulting in the different MOE is reported.

Chapter 4 describes the preparation of the different MOE resulting from the simulation for the regression analysis. The development of the delay equations for urban

streets using multiple regression analysis is reported. The assessment of these equations are noted.

Chapter 5 discusses the fundamental traffic flow relationships for the network-level. A delay equation estimated for the entire network is presented.

In **Chapter 6**, the methodology of data collection and the model validation is given.

Chapter 7 describes the comparison of the developed delay model with the HCM delay procedure and NETSIM using field data. In addition, a number of illustrative examples for the model application are discussed.

Chapter 8 presents the conclusions and recommendations.

In **Appendix A**, the procedure of estimating the delay reduction factor for heavy vehicles is presented.

Appendix B, includes detailed calculations of the model validation.

CHAPTER 2

SIMULATION EXPERIMENT DESIGN AND SETUP

2.1 Research Needs

The discussion in Chapter One concluded that the existing delay models do not satisfy the requirements of TSM planners, since they are developed for operational analysis. None of them is suitable for planning purposes and, except for **NETSIM** and **TRANSYT**, they cannot be used to estimate delay for CBD streets or networks. **NETSIM** and **TRANSYT**, however, are complicated models which require special skills and experience to be used efficiently. They also require a large amount of information and special data acquisition budgets, which might not be available.

There is a strong need to develop new delay models with macroscopic functions which are capable of estimating delay for CBD streets and networks. These models should be easy to use and should not require special data acquisition.

2.2 The Use of Computer Simulation for this Research

In order to develop a delay model which is capable of estimating delays for both CBD streets and networks, field information of a large number of CBD networks for different cities is needed. Obtaining such data is not feasible for the following two reasons.

1. The resources required to collect such information, such as money, effort and time are enormous.
2. The nature of the data required is difficult to find in different transportation organizations. Since, in addition, to detailed information of traffic, geometric and signalization conditions, field vehicle delay estimates of CBD streets and networks

are required.

Due to these difficulties in obtaining real data, computer simulation was used.

The main reasons for choosing computer simulation to investigate and develop the proposed model are that simulation models can yield potentially useful insights into a wide variety of situations and conditions that would otherwise require enormous resources. Computer simulation programs provide the capability of looking at urban networks under controlled conditions at relatively low costs. Simulation experiments can guide further theoretical development as well as assist in the judicious selection and design of observational studies to validate key assumptions and results [19].

TRAF-NETSIM is the computer simulation package used. More details on its nature and how it works are discussed later in section 2.4.

2.3 Research Framework

In order to investigate and develop the proposed model, three computer packages were used, **NETSIM**, **PASSER II-87**, and **STATGRAFICS**. The overall framework of the research is outlined in the flowchart shown in Figure 2.1.

First, field data for Ottawa's CBD was collected. Simultaneously the creation of a data base for the simulation experiment was performed.

Secondly, a preliminary run of the computer simulation program **NETSIM** was undertaken. It should be noted that the signal timing plan for the first run included one cycle length for the whole network. Besides, the green time was split equally between the intersecting streets for all the intersections in the network.

NETSIM's first run was performed to obtain traffic volume on each link on the network which is an essential input to **PASSER II-87**, the traffic signal optimization program used.

The optimum traffic signal plan estimated by **PASSER II-87** was input into **NETSIM** to perform its second run.

Total vehicle delay for the entire network was compared between **NETSIM**'s first run and second run. If the total delay for the second run was less than that of the first

* Total network delay.

Figure 2.1 Research Framework Flowchart

run, the next step was to be executed. Otherwise, another traffic signal plan had to be obtained from **PASSER II-87**.

The delay model was developed using multiple regression analysis. The statistical computer program, **STATGRAFICS**, was used to perform the regression analysis. The developed model was validated using real data collected using the videotape technique. Finally, the developed model was compared with **NETSIM**, and the **HCM** using field data.

2.4 The TRAF-NETSIM Model

In order to have a better understanding of the nature of the procedure, it is appropriate to give a brief introduction to the **TRAF-NETSIM** computer program before describing the setup and design of the simulation experiment.

NETSIM is a microscopic, stochastic simulation model of traffic operation on urban street networks. This program was formerly called **UTCS-1** and was widely used by both practical planners and researchers.

NETSIM is a fixed-step, microscopic network simulation model, in which each vehicle in the system is treated separately during the simulation. Vehicle behaviour is governed by a set of microscopic car-following, queue-discharge and lane switching rules.

Vehicles are initially emitted onto the network via "entry" links surrounding the area to be studied. After passing through the network they are discharged via a set of "exit" links around the periphery. The model does not include any detailed assignment algorithm for allocating a particular travel path to vehicles as they traverse the network. Vehicles are simply emitted onto the network at pre-specified "input-flow-rates" or "entry volumes" along a set of input links and move through the street system according to turning movements assigned stochastically at the head of each link. This assignment is based upon average, percentage turning movements at each downstream node. The model has the capability of generating as an output, a final resultant origin-destination matrix which is based on these input-flow-rates and assignments of turning movements [16].

As each vehicle enters the network, a set of performance characteristics is assigned to it stochastically. These characteristics include classification of vehicle type (i.e., automobile, truck, bus), and specific characteristics such as its average discharge headway and average acceptable gap. A detailed "vehicle array" is maintained for each vehicle, containing a record of its statistical history as it moves through the network. This array is updated once every time step. This record includes specification of the type of vehicle, its cumulative trip-time and travel distance, its cumulative delay and number of stops incurred to date, its present position within the network, its position in "queue" and its projected turning movement at the next intersection.

At the same time a "link-array" is maintained containing detailed information on the traffic conditions on each link within the network. This array includes information on the present occupancy of the link, a cumulative count of turning movements, the number of vehicles presently queued in each lane and an indication of the status of the signal at the downstream intersection. The data in these arrays are also updated every time step. All vehicles are processed once every second and their time space trajectories are recorded to a resolution of 0.1 sec.

However, the microscopic nature of **NETSIM** introduces a significant difference, in that each vehicle is assigned an individual target speed, ranging from 75-127% of the link travel speed [8,14,16,17]. The travel time of a vehicle is thus influenced not only by the traffic signal, but also by friction among individual vehicles within the traffic stream.

NETSIM works with a "node-link" configuration. Nodes are typically intersections and links represent urban streets.

In the early eighties, **NETSIM** was integrated within **TRAF** simulation system and renamed **TRAF-NETSIM**. The creation of **TRAF** did not involve new model development work, but the enhancement of what was regarded as the best available logic of traffic simulation. This logic is in the form of modularized subroutines which are stored in a master file. A program tailored to a particular application can then be generated by an operating system which selects the needed subroutines, adjusts their dimensions, and integrates them. This flexibility minimizes the waste of the computer resources because the programs contain only the user's selected features and dimensions required by the

desired applications [20].

It should be noted that since **NETSIM** integration into the **TRAF** system, it was continuously updated under the FHWA-sponsored **TRAF** software maintenance and support activity. Several enhancements have been incorporated into **TRAF-NETSIM**. The most important enhancement from this research point of view is the identical traffic streams. The importance of this enhancement is that users can perform a series of runs with traffic streams that exhibit identical routing patterns and driver-vehicle characteristics. This is necessary so that the **TRAF-NETSIM** user can test different control strategies with the assurance that each such strategy is applied to the same traffic stream and that any change in performance is due solely to a change in control variable, rather than random variation [21].

The reasons for choosing **NETSIM** in this research are noted here. First, it is a network simulation model. Secondly, it has many features that are not available in the other software programs including **TRANSYT-7F** which was a candidate option. The most important feature that is not available in **TRANSYT-7F** is the difficulty in modelling transit operations accurately in the networks. **TRANSYT-7F** is limited in its ability to account for the delay caused by passengers loading, skip-load operation and other elements of transit [22]. Table 2.1 shows the main different features for software packages.

2.5 NETSIM Validation and Its Wide Application

NETSIM is an accurate computer simulation program that was proved to be valid fifteen years ago. Since then users and researchers have been using it as a powerful tool on a very wide basis.

The validation of **NETSIM** (**UTCS-1** formerly) was proved in Washington D.C and the results are shown in Table 2.2 [24,25]. **NETSIM** has always been used as a trustful tool to evaluate different operational, planning and design options.

Hagerty and Maleck used it as an evaluation tool in Michigan Department of Transportation (MDOT). According to them, " The time for model simulation as a serious

**Table 2.1 Comparison of TRAF-NETSIM with Other
Traffic Engineering Programs, [22]**

Program Features	TRAF-NETSIM	TRANSYT-7F	PASSER II	SOAP
Simulates isolated intersections	Yes	Yes	Yes	Yes
Simulates fixed-time signals	Yes	Yes	Yes	Yes
Simulates actuated signals	Yes	No ^a	No	No
Simulates STOP and YIELD signs	Yes	Yes	No	No
Simulates network:	Yes	Yes	No	No
Simulates signalized intersections with different cycle lengths	Yes	No	No	No
Simulates saturated conditions	Yes	No	No	No
Simulates pedestrians	Yes	No	No	No
Simulates buses	Yes	Yes	No	No
Simulates lane closures	Yes	No	No	No
Simulates parking	Yes	No	No	No
Simulates any combination of the above features	Yes	No	No	No
Optimizes timing plans	No	Yes	Yes	Yes
^a Approximation only				

**Table 2.2 Model Validation : Network-Wide
Comparisons, [24]**

Characteristics	Run 1				Run 2				Run 3			
	Field	Amount	Differ- ence	percent	Amount	Differ- ence	percent	Amount	Differ- ence	percent	Amount	Differ- percent
Total link output.												
vph												
Peak	36.108	36.345	237	0.7	36.399	291	0.8	36.368	260	0.7		
Off peak	27.433	27.731	298	1.1	27.930	497	1.8	28.007	574	2.1		
Vehicles in network.												
Peak	328.4	327.9	-0.5	-0.2	332.0	3.6	1.1	329.5	1.1	0.3		
Off peak	276.4	246.3	-30.1	-10.9	243.3	-33.1	-12.0	238.7	-37.7	-13.6		
Vehicle minutes.												
Peak	10.506	10.502	-4	0.0	10.616	110	1.0	10.539	33	0.3		
Off peak	8.841	7.891	-950	-10.7	7.781	-1.060	-12.0	7.634	-1.207	-13.7		
Vehicle miles.												
Peak	1.701	1.705	4	0.2	1.710	9	0.5	1.710	9	0.5		
Off peak	1.285	1.293	8	0.6	1.302	17	1.3	1.306	21	1.6		
Total delay.												
vehicle min												
Peak	6.475	6.455	-20	-0.3	6.561	86	1.3	6.483	8	0.1		
Off peak	5.806	4.831	-975	-16.8	4.700	-1,1106	-19.0	4,541	-1.265	-21.8		
Average travel time.												
min vehicle												
Peak	5.4	5.5	0.1	1.9	5.4	0.0	0.0	5.4	0.0	0.0		
Off peak	6.4	6.0	-0.4	-6.3	5.2	-1.2	-18.8	5.1	-1.3	-20.3		
Average speed.												
mph												
Peak	16.43	16.08	-0.35	-2.1	15.99	-0.44	-2.7	16.03	-0.40	-2.4		
Off peak	15.35	16.48	1.13	7.4	16.88	1.53	10.0	16.71	1.36	8.9		

Note: 1 mile = 1.6 km

analytic tool has arrived. The growth and acceptance of **NETSIM** have exceeded all expectations. **NETSIM** was initially used as a supplement to reinforce traditional analytical methods. Today, the results from **NETSIM** analyses stand on their own merit" [26].

Labrum used the **NETSIM** model extensively to evaluate traffic control strategies for single intersections, arterials and grid networks in Utah. It was mentioned that **NETSIM** was a very useful tool and it was recommended for solving a wide variety of control problems [27].

Dudeck, Goode and Poole compared **TRANSYT-7F** with **NETSIM**, and they concluded that **NETSIM** represents the real world more accurately [28].

Mahmassan, Williams and Herman used **NETSIM** in their investigation of network-level traffic flow relationships, and they mentioned that **NETSIM** was used because it was accepted as a validated model and it has been used extensively by traffic researchers and engineers [19].

Chris Claterbos [8,29] conducted studies in three cities using **NETSIM** to determine the effectiveness of the computer programs, **MAXIBAND**, **SIGOP-3** and **TRANSYT-7F** (release 2) which are designed to optimize traffic signal timing.

Chang and Messer used **NETSIM** to evaluate the advantages and the disadvantages of combining bandwidth maximization procedure and the delay minimization technique. The program was selected for its ability and complexity in changing traffic signal timing to achieve specific experimental conditions for field validation [30].

In their comparison of arterial and network software programs, Sadeh, Radwan and Matthias used **NETSIM** for the comparative assessment because of its reliability [31].

Edmond C.-P. Chang, Stephen L. Cohen, Charles Liu, Nadeem A. Chanhary, and Carrol Messer used **NETSIM** to simulate the network performance under both sets of timing plans of **MAXIBAND** and **TRANSYT-7F** [32].

Yauch, Gray and Lewis used **NETSIM** to evaluate the effects of drawbridge openings on adjacent signalized intersections [33].

Sadeh and Radwan in their comparative assessment of 1985 HCM delay model, **SOAP** and **TRANSYT-7F**, used **NETSIM** as the evaluation tool. They stated that

"researchers felt that **NETSIM** calculation of average delay is reliable and trustworthy, compared to estimates by the other macroscopic models" [8].

Panos G. Michalopoulos used **NETSIM** to judge the realism and estimate the expected accuracy of his proposed dynamic macroscopic methodology of analyzing traffic flow at congested intersections and arterial streets [34].

Huel-Sheng Tsay used **NETSIM** to evaluate Traffic Responsive and Uniform Surveillance Timing System (TRVSTS), which is a new urban, on-line traffic control system that has been successfully performed in Taiwan [35].

Machane and Roess suggested using **NETSIM** in estimating arterial running time and intersection approach delay, because they considered it a well established and calibrated model [36].

2.6 Summary of Literature on Different Simulation Experiments

It is always desirable to use real data in studying different traffic problems. On the other hand, it may not be easy to collect enough data for studying a certain traffic problem which requires large amounts of money and effort.

Therefore researchers had to rely on their own experimental design for studies that needed an enormous amount of data. Some researchers have chosen real arterials or networks with their existing configuration to perform their experiments. They changed different characteristics other than the geometric and topological characteristics. Some of these researchers did not need to change these characteristics, and others did not change them for the sake of simplification.

Ramey Q. Rogness [37] used an existing arterial geometry and changed the spacings between intersections, used different cycle lengths, and used different phasing patterns in his evaluation of **PASSER II**.

In their comparison between **TRANSYT-7F** and **NETSIM**, Dudeck, Goode and Poole [28] used twenty-five signalized intersections in the CBD of Salisbury, North Carolina, and one hundred combinations of traffic conditions, to perform the simulation experiment.

Chang and Messer [30], in their investigation of the effectiveness of the enhancements in **PASSER II** used an existing arterial named Skillman. A large number of combinations were created by changing the spacing between intersections and the signal cycle length.

Other researchers had to develop their own networks or arterials to perform their traffic simulation work. Mao, Messer and Rogness [38], in their evaluation of signal timing variables by using a signal timing optimization program, had to assume a uniform arterial grid spacing with traditional two phase signal operation, three twelve-foot traffic lanes for each approach consisting of a separate left-turn lane, a through lane and a combined through plus right-turn lane. Saturation flow rates per lane of 1750 passenger car per hour of effective green (pcphg) for through traffic plus right-turns and 1200 pcphg for left-turns. Turning movements used were 10 percent for left-turn and right-turn and 80 percent for through movements. Different combinations of signal spacing and cycle lengths were chosen. A total of one hundred and forty runs were executed to analyze this study.

Mahmassani, Williams and Herman [19] in their investigation of network-level traffic flow relationships developed their own network which was a grid network with 400-ft equal spacings between intersections. Two scenarios were considered. The first assumed that one-third of vehicle traffic turned left, one-third turned right and one-third continued through. In the second scenario, 10 percent turned left, 15 percent turned right and the remaining 75 percent continued through. A 50 second (sec) cycle length with 50/50 split was considered in both scenarios (see Figure 2.2).

Wellander, Strombom, Fromm, Deardrof, Bonner, and Berg, in their development of a traffic modelling system for detour planning on a downtown Seattle project, created their own network data and simulated detailed reflections of the existing conditions. They collected real data for model calibration and validation [22].

Figure 2.2 Network Configuration, [19]

2.7 Experiment Setup Procedure

Resources, including money, effort and time required to obtain different traffic, roadway and signalized conditions data for different urban grid networks may be enormous. Therefore, researchers tried to develop their own data sets and simulation experiments to avoid such costs. Since the number of variables in urban grid networks is large, the cost of covering all possible combinations through simulation will be as high as collecting real data. As a result, researchers chose only a limited number of combinations which were compatible with their resources. Consequently, the possibility of losing some interaction between these different variables is large. Furthermore, the randomness and the variability which exist in real world may disappear.

Due to such limitations, a different approach to develop these data for the simulation experiment was used. Most of the different network characteristics (variables) were chosen on a random basis. The methodology of implementing the above approach is given in the following points.

1. Random numbers were generated for each variable (e.g., block length) separately in the range of 0-99. The generation of the random numbers was done manually using random number tables [39]. Except for two variables, (percent of turning movements and intra-link volumes), it was done by STATGRAFICS (which is statistical computer package) [40], since 21,000 random numbers were required.
2. The range of values from which each variable was to be chosen, was specified based on logic, experience, collected data of Ottawa CBD and the computer simulation model requirements and limitations.
3. The generated random numbers for each variable were divided into a number of intervals. A single value (e.g., 400 ft) was specified for each interval of the random numbers (e.g., 15-25).
4. Allocating generated random numbers to their corresponding variables was done manually, except for the two variables mentioned above. The reason for doing this manually, although it is time consuming, is because of the complexity of CBD network characteristics and their interaction. For the two variables mentioned above

a FORTRAN program was written to allocate the generated random numbers to their corresponding characteristics.

5. Extensive checks were performed to ensure the reasonableness of the interaction between the variables (characteristics). For example the location of the bus-stop was checked versus curb parking and turning movements were checked versus network configuration (one-way streets or two-way streets).

On the other hand, there were several variables which could not be chosen randomly. For example, bus routes were allocated based on guidelines [41,42] and engineering judgement, and traffic signals were designed using PASSER II-87. This will be discussed later in chapter 3.

The following sections discuss in more detail, the different variables forming the input data of NETSIM. These input data include the following features [17].

- Topology of the roadway,
- Geometrics of each roadway component,
- Channelization of traffic on each roadway component,
- Circulation pattern of traffic on the roadway system,
- Specification of the traffic devices and their operational characteristics,
- Traffic volumes entering the roadway system,
- Traffic composition,
- Specification of bus transit system including routes, stations and frequency of service.

Before discussing the generation of the input data, the following assumptions are stated.

- The different characteristics of the urban network are discrete, and independent variables.
- All links have no grade (g) i.e., $g=0$.
- All links are straight i.e., no curved links.
- No change in the number of lanes along the streets.
- No links have any right-turn or left-turn bays.

2.7.1 Network Configuration

Twenty-four networks, divided into three scenarios, were developed. Each scenario represents a network configuration. Each network consisted of 25 nodes (intersection) arranged in a 5 node by 5 node square, connected with two-way or one way streets depending upon the scenario. Due to the nature of NETSIM which uses uni-directional links in representing the network, a two-way street is represented by two uni-directional links.

The main directions were assumed such that the top of the paper represents the north direction. Streets were given names to make it easier when dealing with them. North-south streets were given names first to fifth avenue respectively. Streets from east to west were named first to fifth street respectively.

The three different scenarios or network configuration were as follows:

- the first scenario consisted of a two-way street network (i.e., all the streets and avenues in the network were two-way streets);
- the second scenario consisted of two-way avenues and one-way streets;
- the third scenario consisted of one-way streets network. That is, all streets and avenues in the network were one-way streets. The middle streets and the middle avenues were two-way streets.

The basis of scenario 3 is that in CBD networks, which consist mainly of one-way streets, it is more likely to find at least one two-way street in each direction. For simplicity, the two-way street was chosen in the middle of each direction in each of the three scenarios. Figures 2.3, 2.4 & 2.5 give the different network configurations. In these three figures, links connected to nodes numbered 8000 and over are either entry or exit links. These links were mentioned previously in NETSIM's description and no MOE are estimated for them. The twenty five nodes that represent the networks are numbered between 1 and 750 (NETSIM's notation) [17].

Figure 2.3 Network Configuration 1 (Scenario 1)

Figure 2.4 Network Configuration 2 (Scenario 2)

Figure 2.5 Network Configuration 3 (Scenario 3)

2.7.2 Distance between intersections or block lengths

Block lengths were chosen in the range of 200-1000 ft with 100 ft intervals, each length with an equal probability to occur. Figure 2.6 shows the exact definition of the block length.

This range of block lengths or distances between intersections were selected for the following reasons:

1. Field measurements for Ottawa CBD streets resulted in a range of 200-800 ft.
2. Measurements to scale from maps of Los Angeles, Hartford, and Charlotte of U.S.A cities were in the range of 300-1000 ft [28]. See figures 2.7, 2.8 & 2.9.
3. Williams, Mahmasanni and Herman, in their analysis of urban networks used 400 ft and 1000 ft for simulation experiments [44].
4. Moa, Messer and Rogness, in their analysis of possible enhancements for **PASSER II**, used spacings of 330 ft and 990 ft for urban arterials [38].

2.7.3 Number of Moving lanes

Moving lanes are those lanes used for vehicle movement only, i.e., parking lanes are not included. The number of moving lanes for each street including both directions (in case of two-way streets) were selected randomly, all with equal probability to occur. In case of two-way streets, there were only two options, either 2-lanes or 4-lanes. The reasons for skipping the 3-lane option were simplicity and that two-way streets with 3-lanes are not common in the CBD networks. In the case of one-way streets, the number of moving lanes were two lanes, three lanes, or four lanes.

2.7.4 Traffic Volumes (Entry volumes)

In TRAF-NETSIM model, vehicles are emitted onto the network via links called entry links located on the periphery of the studied area. Traffic volumes are discharged from these links but not modelled on them (i.e., no MOE are estimated for them). Then these traffic volumes are distributed and discharged through the network, depending on the different link characteristics.

Figure 2.6 Measurement of Length for Urban Links, [16]

Figure 2.7 Parking Garages - Downtown Los Angeles, [43]

**Figure 2.8 Off-Street Parking Facilities for CBD
Hartford, Connecticut, [43]**

**Figure 2.9 Off-Street Parking Facilities for
CBD Charlotte, N.C., [43]**

The entry volume was between 100 and 500 veh/hr/lane with 50 veh/hr/lane intervals. The lower limit of 100 veh/hr/lane, was chosen as a reasonable minimum number of vehicles for peak hour conditions, based on traffic volume data for the CBD of Ottawa. On the other hand, the higher limit of 500 veh/hr/lane was selected as a reasonable number to represent the street capacity, using the HCM computer software (HCS) [45], for signalized intersections and using the operation analysis with different roadway, traffic characteristics. Furthermore, the maximum volume of Ottawa's CBD peak traffic volume data that has been reached is 420 veh/hr/lane.

2.7.5 Heavy Vehicles

Heavy vehicles were expressed as percentages of total vehicles at entry links. These percentages of heavy vehicles did not include any transit buses, which were treated separately in section 2.7.9. The percentage of heavy vehicles was between 0% and 10% of total traffic with 2% intervals. The selection of this range was based on data for Ottawa's CBD network.

2.7.6 Percentages of Turning Movements at Intersections

Percentages of turning movement were chosen randomly from the following range of values:

- Percentage left turn (LT) (0 - 35%) with 5% intervals;
- Percentage right turn (RT) (5 - 35%) with 5% intervals.

Through traffic percentage was calculated by subtracting the total percentages of turning movements from one hundred percent, satisfying the limits of the through volume which are:

- Percentage of through traffic $\geq 40\%$
 $\leq 95\%$.

It is clear that the minimum percent of LT is different from the minimum percent of RT (i.e., minimum % LT = 0% and minimum % RT = 5%). The reason for this is that it is more likely to have LT prohibited in CBD streets than to have RT prohibited. The limits chosen were based on the following:

1. Using RTAC'S classification of roads [46], it could be concluded that most streets in CBD networks could be considered urban collectors. RTAC defines urban collector functions as serving mobility and accessibility for the same degree. In another location [41,42], it was stated that urban collectors provide primarily land access service and minor amount of through traffic (mobility). Considering both definitions and interpreting then into percentages of turning movements and through traffic the minimum 40% through traffic was estimated;
2. The highest turning movement observed in Ottawa's CBD traffic volume data was 37% (with the other turning movement equal to zero);
3. Williams, Mahnassani, and Herman in their studies of urban network characteristics used 33% LT and 33% RT in one scenario [19].

2.7.7 Pedestrian Volumes

Pedestrian volumes for each intersection were chosen randomly from one of the following three options with equal probabilities:

- Light pedestrian flow rate (100 - 200 ped/hr);
- Moderate pedestrian flow rate (250 - 500 ped/hr);
- Heavy pedestrian flow rate (above 500 ped/hr).

2.7.8 Parking Maneuvers

Parking maneuvers cause friction as well as blocking the flow in the adjacent lane when vehicles move into or out of parking spaces. Curb parking in CBD areas constitute about 40% from the total lineal feet available [41,42]. Based on this statement, it could be reasonably stated that the probability of having no parking on a certain link would be 60%. The range of parking maneuvers/hr was chosen 0 -40 as in the HCM, chapter 9 [18] with intervals 10 man/hr. To make this clearer, the following illustration is given :

Random number	parking man/hr
0 - 60	0
61 - 73	20 man/hr
74 - 86	30 man/hr
87 - 99	40 man/hr

The reason for skipping the option of 10 man/hr is that TRAF-NETSIM, does not consider the effect of parking maneuver for less than 14 maneuvers/hr. As mentioned above, the procedure for developing the input data was not only based on random selection, but also on engineering judgement and rules. Therefore the following restrictions and guidelines on parking were followed :

- no parking on two-way two-lane streets;
- parking on one side only of one-way streets [26,27,47,48];
- parking on links with local bus routes will be prohibited in most cases, especially on two-way streets;
- parking will never extend further than 50 ft from the stop line of the stop line of the downstream or the upstream approach [49].

The last point indicates the high degree of detailing required by NETSIM.

Finally, curb parking was prohibited for the entire network for 10% of the twenty four created networks, as it is possible to find CBD networks where parking is prohibited totally during peak hours.

2.7.9 Local Transit Buses

To account for the effect of local transit buses blocking lanes at bus stops, a number of transit buses were assumed to run on some links of the networks. The number of buses were chosen randomly between 10 and 40 bus/hr, each with equal probability to be chosen. This range is similar to that in HCM [18] used to calculate the effect of bus blockage on the capacity of signalized roads. In case of exclusive bus lanes, the choice was limited only to either 30 or 40 bus/hour [18].

The location of the bus stops for each network were the same type and were chosen from the three traditional types:

- Near side bus-stop (just prior to intersection);
- Far side bus-stop (just beyond the intersection);
- Mid block bus-stop.

These locations for each network were systematically chosen in the order given above. About 50% of the bus stops were protected (with a bay) while the other 50% were unprotected. The selection was based on a random basis with one restriction for two-way two-lane streets where the bus stops had to be protected (built in logic of NETSIM).

The allocation of bus routes to the links were based on engineering judgement, logic and the following guidelines:

- in case of near-side bus stop, the distance between that bus stop and the downstream stop line is 20 ft [17,50];
- in case of far-side bus stop, there must be at least a distance of 55 - 60 ft from the approach, as shown in Figure 2.10;
- in case of networks where exclusive bus lanes are placed, there should be at least two lanes available for other traffic in same direction [18].

A number of assumptions were made concerning the different aspects of the local transit bus. These are:

1. each uni-directional link will have only one bus route;
2. only one bus-stop is to be located within each block, since spacing of blocks are in the range of 200 - 1000 ft with an average of 600 ft, and spacing of bus stops in the CBD is recommended to be between 500 and 1000 ft [41,42];
3. dwell time for each bus will be assumed 30 sec [18];
4. each bus will stop at all bus stops on its route, which means that it will stop at each block;
5. in each direction (e.g east bound) there will be at least two bus routes.

2.5.10 Intralink Volumes

Since NETSIM processes each vehicle on the network, it is possible to replicate (or to represent) in detail, events that occur between intersections. In CBD networks, it is most likely to have parking lots, garages, small side streets (which are not presented

in the simulation) and alleys on each link between signalized intersections. These facilities are represented in **NETSIM** as sinks that absorb traffic from the links or as sources which emit traffic flow to links. Figure 2.11 shows how these facilities are presented in **NETSIM** .

Using this capability, networks were made to look more like real world networks. Studying Ottawa's CBD and the available parking lots and garages and looking at maps of parking facilities for different cities such as, Los Angeles, Charlotte and Hartford [43] (see Figures 2.6, 2.7 & 2.8), it was estimated that 66% of the CBD network links include sinks or sources and the remaining 33% do not include any sinks or sources. Therefore, each link had three options with equal probabilities of occurrence. These are -50, 0 or 50 veh/hr. The negative sign indicates sink volume and the positive sign indicates source volume.

The 50 veh/hr was chosen because:

1. if a source emits 50 veh/hr onto a link carrying the upper limit of traffic volumes which is 500 veh/hr, then that extra volume can be accommodated by the link;
2. if this source or sink represent a small driveway or a small street, its logical that it emits half the minimum traffic assumed for normal streets.

Figure 2.10 Far-Side Bus Stop Location, [51]

The equivalent representation is shown below:

Figure 2.11 NETSIM Representation of Intralink Gains/Losses in Traffic Volumes, [16]

CHAPTER 3

EXPERIMENT IMPLEMENTATION

3.1 General

In the previous chapter the input data and the approach used to develop these were discussed in detail. In this chapter the computer experiment is illustrated. Both the computer packages **NETSIM** and **PASSER II** are used in the experiment. Additionally, details on **PASSER II** are given. The results of the final run of **NETSIM** are the MOE used in developing the proposed delay model.

3.2 NETSIM First Run

Due to the nature of **NETSIM** in accepting traffic volumes as entry volumes from entry links, a preliminary run had to be performed to estimate traffic volumes on each uni-directional link.

3.2.1 Cycle Length

A cycle length between 50 - 90 seconds were used in running the computer experiments. In other words this was the range of cycle lengths from which the optimum cycle was to be estimated using **PASSER II**.

The higher limit of this range was selected based on the following considerations. In CBD networks, cycle lengths are chosen short so as to minimize pedestrian waiting times [52]. In general, it never reaches this 90 sec, and the collected data of Ottawa CBD showed a maximum cycle length of 80 sec. On the other hand, the lower limit was

selected because it is not feasible to operate a traffic signal with cycle length of 40 sec [36] and the collected data for Ottawa indicated that the minimum cycle length was 50 sec.

3.2.2 Signal Type and Green Split

All traffic signals used in these simulation experiments were two-phase pre-timed traffic signals. It is more likely to have two-phase pre-timed traffic signals in CBD areas [53]. The green time was equally divided between the two intersecting streets (i.e., the green split was 50/50). This equal green split was assumed for simplicity.

3.2.3 Executing NETSIM First Run

As mentioned in the previous chapter, there were three scenarios which represented three different network configurations. Each scenario included eight different cases with different roadway, traffic, and signalization characteristics. In other words twenty four cases or practically speaking twenty four networks were created and prepared for simulation. Each network was given a notation which indicates the corresponding scenario and its order in that particular scenario.

The data entry procedure for the first run of **NETSIM** was a very time consuming procedure. It took about 6-8 hours to complete entering the input data for each network. This can be explained due to the nature of **NETSIM**, where the input stream consists of a sequence of "card types". Each card type contains a specific set of data items. These card types are assembled into groups, wherein each group of card types defines a classification of input information. Figure 3.1 illustrates the layout of **NETSIM** input stream.

A cycle length of 50 sec was used for the preliminary run of **NETSIM**. The reason being, that by choosing this cycle length, it can be assured that if the second run of **NETSIM**, which is coded with the new cycle length estimated by **PASSER II**, gives less overall delay for the whole network, it should be the best cycle length available.

Finally, the phasing sequence was to be decided before executing the first run. It was decided to choose the directions with higher entry volumes as the main direction (i.e.,

Figure 3.1 Schematic Layout of TRAF-NETSIM Input Stream, [16]

the direction to be given green phase first).

A 15 minute simulation time was executed. The actual time of running was 2 to 2.5 hours on an IBM compatible computer with 640 K RAM and 20 Mbyte hard disk. The output of NETSIM includes MOE for each link as well as for the entire network. A detailed discussion on the nature of the output will be given in a later section. However, at this step the most important output of the first run was the average traffic volume for each link which is an essential input in **PASSER II**.

3.3 PASSER II-87

Before continuing the next step of the experiment implementation, a brief explanation about the nature of **PASSER II** is given.

PASSER II is an acronym for the Progression Analysis and Signal System Evaluation Routine. **PASSER II** is a computer program for the optimization of arterial signal timing. It can be classified as a macroscopic deterministic model. It uses a platoon level representation for fixed (uniform) traffic volumes and speeds [37]. The optimization procedure is an implicit enumeration of minimum interference values. This procedure uses a variant of half-integer synchronization approach for relative offsets [37].

A unique advantage in **PASSER II** program over other optimization programs is that it can be used to consider and select multiple phase sequence. **PASSER II-87** is the latest version of **PASSER II**. It seeks to maximize arterial two-way progression and minimize delay by pursuing a series of arterial signal timing optimization processes. A range of cycle lengths can be examined for the optimal progression operation in each run. It will select the cycle length that can provide the maximum progression efficiency. Delay-minimization guidelines are provided as program output for selecting a narrow range of cycle lengths which will yield delay efficient solutions. Finally, the program automatically fine-tunes the coordinated progression offsets to further minimize delay to the arrival patterns of the traffic flow. This fine-tuning results in a reduction of 5-15% in delay to arterial's progressive movements [54].

Most of the data requirements for **PASSER II-87** are nearly the same as for HCM

operation analysis for signalized intersections. Volumes, saturation flow, percentages of turning movements, percentages of heavy vehicles, number of parking maneuvers per hour, number of transit buses per hour and the geometric characteristics are needed.

In addition to these input data mentioned, other signalization and phasing data are required. A range of cycle lengths from which **PASSER II** estimates the optimum cycle length should be given. Besides, the National Electrical Manufacturers Association (NEMA) phase movement designation, the selection of signalization designation, and the phase sequence input are important input requirements as well.

PASSER II-87 is programmed to help the user in selecting the appropriate signal phasing patterns in an intelligent manner according to the user input data.

3.4 Running PASSER II-87

In order to use **PASSER II** which is an arterial signal optimization program, each network had to be divided into five arterials. The streets carrying the highest traffic volumes were considered the main directions (or the arterials) where the progression was done.

The different traffic and geometric characteristics for each arterial, including the traffic volumes estimated by **NETSIM**, were input into **PASSER II**. The time needed to complete recording all the input data requirements for one arterial was 20 to 30 minutes.

One of the important input data items in **PASSER II** is the saturation flow. This latest version of **PASSER II** includes the HCM procedure for calculating the saturation flow. It was decided to use this method, as no saturation flow was to be assumed. As a result of using this procedure, the ideal saturation flow was assumed to be 1800 vehicle per hour green per lane as in the HCM.

In order to estimate the best signal timing plan for each arterial, **PASSER II** had to be executed several times. The reason for this is the nature of **PASSER II-87** which searches for the best cycle length within a 10 sec range of cycle length in each run and the total range from which the cycle length is to be chosen is 50 to 90 sec.

For each 10 sec range of cycle lengths, **PASSER II** gives the cycle length which

gives the best timing plan for the arterial. The efficiency¹ of the progression and the total delay in veh/sec for the arterial are given for each run as an output. See Figures 3.2 , 3.3 and 3.4 which are examples of the program output. Among the four runs executed to cover the total range of cycle lengths (50-90 sec), the cycle length that gave the best efficiency and the minimum delay for the arterial was chosen. In cases where the cycle length that gave the highest efficiency was not the one for minimum delay for the arterial, the course of action described below was followed.

If the cycle length that gave the highest efficiency was not the one that gave the minimum delay for the arterial but, its associated delay was not that far from the minimum delay, it was chosen. But, if the delay associated with cycle length of maximum efficiency was too far from the minimum delay, the cycle length that gave the minimum delay was to be chosen if its efficiency was not too bad. If both steps were not satisfied, a third cycle length which gave the highest efficiency and almost the minimum delay was chosen.

3.5 Reasons for Choosing PASSER II-87

This program is widely used for research and practical applications. Example applications are noted below.

1. **PASSER II-87** is used in timing and optimizing traffic signals in Ottawa-Carleton Region.
2. **PASSER II-87** is used for timing and optimizing traffic signals Metropolitan Toronto.
3. **PASSER II-87** has been used in preparing traffic signal timing plans for **TRANSYT** and **NETSIM**, in a similar way its use in this research [8].

¹ The progression efficiency is defined as the percentage of the bandwidth to the cycle length

(ART.SUMY) TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION
 PASSER II-87 MULTIPHASE ARTERIAL PROGRESSION - 145101 VER 1.0 JUL 88

**** PASSER-87 BEST PROGRESSION SOLUTION SUMMARY ****
 Dallas Skillman Avenue DISTRICT 18 06/22/87 RUN NO. 1

CYCLE LENGTH = 95 SECS (MAXIMIN CYCLE = 99 SECS)
 EFFICIENCY = .38 (GREAT PROGRESSION)
 ATTAINABILITY = 1.00 (INCREASE MIN. THROUGH PHASE)

BAND A = 33 SECS AVERAGE SPEED = 32 MPH
 BAND B = 38 SECS AVERAGE SPEED = 36 MPH

NOTE: ARTERIAL PROGRESSION EVALUATION CRITERIA

 EFFICIENCY 0.00 - 0.12 - "POOR PROGRESSION"
 0.13 - 0.24 - "FAIR PROGRESSION"
 0.25 - 0.36 - "GOOD PROGRESSION"
 0.37 - 1.00 - "GREAT PROGRESSION"

ATTAINABILITY 1.00 - 0.99 - "INCREASE MIN THRU PHASE"
 0.99 - 0.70 - "FINE-TUNING NEEDED"
 0.69 - 0.00 - "MAJOR CHANGES NEEDED"

(INT.SUMY) TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION
 PASSER II-87 MULTIPHASE ARTERIAL PROGRESSION - 145101 VER 1.0 JUL 88

**** INTERSECTION PERFORMANCE SUMMARY ****

CYCLE LENGTH = 95 SECS SYSTEM MAXIMIN CYCLE = 99 SECS

INT NO	CROSS STREET INTERSECTION	PHASE ART CRS	MIN. DELAY CYCLE (SECS)	INTERSECTION V/C RATIO	AVERAGE DELAY (SECS/VEH)	INT NO
1	Mockingbird	3 3	97	.92	34.6	1
2	University	1 2	69	.76	14.3	2
3	Lovers Lane	3 1	87	.88	27.0	3
4	Southwest	4 1	99	.93	32.7	4

NOTE: PHASE SEQUENCE CODE FOR ARTERIAL (ART) CROSS STREET (CRS)

 1 - LEFT TURN FIRST OR DUAL LEFTS LEADING OR DUAL LEFTS (1+5)
 2 - THROUGH FIRST OR DUAL THRU LEADING OR DUAL THRU (2+6)
 3 - LEADING GREEN OR NO. 5 LEADING OR LT 5 LEADS (2+5)
 4 - LAGGING GREEN OR NO. 1 LEADING OR LT 1 LEADS (1+6)

Figure 3.2 Sample of PASSER II-87 Output
 (Arterial Summary), [54]

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 PASSER II-87 MULTIPHASE ARTERIAL PROGRESSION - 145101 VER 1.0 JUL 88

*** TOTAL ARTERIAL SYSTEM PERFORMANCE ***

Dallas Skillman Avenue DISTRICT 18 06/22/87 RUN NO. :

CYCLE LENGTH = 95 SECS BAND A = 33 SECS BAND B = 38 SECS
 AVERAGE PROGRESSION SPEED - BAND A = 32 MPH BAND B = 36 MPH

.38 EFFICIENCY 1.00 ATTAINABILITY

AVERAGE INTERSECTION DELAY TOTAL SYSTEM DELAY TOTAL NUMBER VEHICLES
 28.2 SECS/VEH 99.5 VEH-HR/HR 12717.

TOTAL SYSTEM FUEL CONSUMPTION TOTAL SYSTEM STOPS MAXIMUM CYCLE
 347.42 GAL/HR 9973. STOPS 99 SECS

(ART.MOE) TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION
 PASSER II-87 MULTIPHASE ARTERIAL PROGRESSION - 145101 VER 1.0 JUL 88

EFFICIENCY VERSUS CYCLE LENGTH

	CYCLE LENGTH	CUMULATIVE EFFICIENCY
	85	.33
	90	.38
	95	.38
BEST SOLUTION	95	.38

Figure 3.3 Sample of PASSER II-87 Output
 (Arterial MOE), [54]

(BEST.SOLN)

TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION
 PASSER II-87 MULTIPHASE ARTERIAL PROGRESSION - 145101 VER 1.0 JUL 88

**** BEST SOLUTION.... NEMA PHASE DESIGNATION ****

*** INT. 1 .0 SEC OFFSET ART ST PHASE SEQ IS LT 5 LEADS (2+5)
 Mockingbird .0 % OFFSET CROSS ST PHASE SEQ IS LT 3 LEADS (3+8)

	ARTERIAL STREET				CROSS STREET			
	2+5	2+6	1+6	TOTAL	3+8	4+8	4+7	TOTAL
CONCURRENT PHASES								
PHASE TIME (SECS)	10.1	23.3	15.0	48.4	25.7	10.9	10.0	46.6
PHASE TIME (%)	10.6	24.5	15.8	50.9	27.1	11.5	10.5	49.1
----- MEASURES OF EFFECTIVENESS -----								
PHASE (NEMA)	5[5]	6	1[5]	2	3[5]	4	7[5]	8
PHASE DIRECTION	EBLTPR	WBTHRU	WBLTPR	EBTHRU	NBLTPR	SBTHRU	SBLTPR	NBTHRU
PHASE TIME (SEC)	10.1	38.3	15.0	33.4	25.7	20.9	10.0	36.6
V/C-RATIO	.81	.88	.26	.27	.62	.61	.40	.87
LEVEL OF SERVICE	D	E	A	A	B	B	A	E
DELAY (SECS/VEH)	73.4	33.4	38.4	24.7	35.6	37.1	44.5	33.6
LEVEL OF SERVICE	E	C	C	B	C	C	D	C
QUEUE (VEH/LANE)	1.8	10.3	.5	2.0	2.4	5.8	.5	14.5
STOPS (STOPS/HR)	109.	1029.	43.	195.	203.	481.	40.	1387.
TOTAL INTERSECTION DELAY					MINIMUM DELAY CYCLE			
34.6 SECS/VEH					97 SECS			

(BEST.SOLN)

TEXAS DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION
 PASSER II-87 MULTIPHASE ARTERIAL PROGRESSION - 145101 VER 1.0 JUL 88

**** BEST SOLUTION CONTINUED.... NEMA PHASE DESIGNATION ****

*** INT. 2 25.7 SEC OFFSET ART ST PHASE SEQ IS DUAL LEFTS (1+5)
 University 27.1 % OFFSET CROSS ST PHASE SEQ IS DUAL THRU (4+8)

	ARTERIAL STREET				CROSS STREET			
	1+5	1+6	2+6	TOTAL	4+8	3+8	3+7	TOTAL
CONCURRENT PHASES								
PHASE TIME (SECS)	10.1	.0	63.9	74.0	21.0	.0	.0	21.0
PHASE TIME (%)	10.6	.0	67.3	77.9	22.1	.0	.0	22.1
----- MEASURES OF EFFECTIVENESS -----								
PHASE (NEMA)	5[5]	6	1[5]	2	3[1]	4	7[1]	8
PHASE DIRECTION	EBLTPR	WBTHRU	WBLTPR	EBTHRU	NBLTPR	SBTHRU	SBLTPR	NBTHRU
PHASE TIME (SEC)	10.1	63.9	10.1	63.9	.0	21.0	.0	21.0
V/C-RATIO	.94	.67	.10	.17	.00	.24	.00	.71
LEVEL OF SERVICE	A	B	A	A		A		C
DELAY (SECS/VEH)	48.5	7.4	38.3	6.0	.0	33.4	.0	41.0
LEVEL OF SERVICE	D	A	C	A		C		D
QUEUE (VEH/LANE)	.8	3.0	.1	.6	.0	1.0	.0	3.8
STOPS (STOPS/HR)	57.	685.	10.	113.	0.	88.	0.	300.
TOTAL INTERSECTION DELAY					MINIMUM DELAY CYCLE			
14.3 SECS/VEH					69 SECS			

Figure 3.4 Sample of PASSER II-87 Output
 (Best Solution), [54]

4. **PASSER II** is used as a comparison base for other signal optimization models for its efficiency. For example, it was compared recently with a new developed model **TRUSTS** in Taiwan [35].

3.6 NETSIM Second Run

The new timing plan obtained from **PASSER II-87** included the optimum cycle length, the green splits of the different phases, and the offsets. Entering all the new signal timing plans for the five arterials for each network, the input data were ready for **NETSIM**'s second run. Again **NETSIM** was run for 15 minutes of simulation, which was the same period of first run. The actual time of running the second run was in the range of 2.5 to 3.0 hours which is slightly more than the first run. It should be noted that sometimes a third run for **NETSIM** had to be executed as the timing plan estimated by **PASSER II** did not result in less delay for the entire network. The outputs of the final run of **NETSIM** included the MOE used in developing the delay models using multiple regression analysis.

CHAPTER 4

DEVELOPING THE DELAY MODEL FOR THE STREET LEVEL USING REGRESSION ANALYSIS

4.1 Introduction

In the previous chapter, the implementation of the simulation experiment was described. The output of the second run of **NETSIM** consisted of a large number of MOE. In this chapter these different MOE are used to estimate the delay equations using multiple regression analysis.

4.2 Procedure for Preparing Data for Regression Analysis

The **NETSIM** program provides a wide range of MOE on a link-specific basis and aggregated over the entire network. In addition, separate statistics are provided for bus transit operations. Figure 4.1 is an example of output generated by **NETSIM**. It includes link-specific MOE as well as MOE for the entire network. Figure 4.2 shows the MOE of bus transit operations which is another example of **NETSIM**'s output.

The first step in the data preparation was the aggregation of the data. In order to get the MOE of the entire street, the MOE of every four links were added together. It should be clear that the number of links for an entire street in this research is four, as the networks consist of five squared nodes. This first step was applicable to the following

CUMULATIVE NETBIM STATISTICS AT TIME 7:45.0																	
ELAPSED TIME 18 0:15.0 (900 SECONDS),				TIME PERIOD 1 ELAPSED TIME 18 900 SECONDS				AVERAGE VALUES									
VEHICLE		VEHICLE MINUTES		RATIO		MINUTES/MILE		SECONDS/VEHICLE		STOP		STOPS VPH		SPEED OCCUP		CONGESTION	
LINK	MILES TRIP	MOVE TIME	DELAY TOTAL TIME	MOVE/ TOTAL	TOTAL TIME	DELAY TIME	TOTAL TIME	DELAY TIME	QUEUE TIME	STOP TIME	STOP	(PCT)	VPH	MPH	VEN.	(PCT)	FAIL
(44, 45)	23.17	136	47.4	212.2	259.6	.18	11.21	9.14	114.6	93.6	71.2	60.6	96	544	5.4	17.4	23.1 14
(45, 46)	34.45	213	75.0	181.6	256.6	.29	7.00	4.94	71.6	50.7	32.1	27.7	81	840	8.4	14.9	21.8 7
(51, 52)	2.75	73	5.9	18.6	24.5	.24	8.92	6.77	20.2	15.3	9.9	8.1	41	242	6.7	2.0	22.8 0
(52, 51)	3.75	99	8.0	14.8	24.8	.32	6.63	4.48	15.1	10.2	8.7	4.9	39	396	9.1	2.2	23.8 0
(52, 53)	24.74	131	53.1	121.7	174.8	.30	7.07	4.92	80.1	55.0	37.8	31.9	84	524	8.5	11.9	26.2 6
(53, 52)	21.72	115	46.4	43.1	89.7	.52	4.13	1.98	44.8	22.5	12.7	10.9	47	440	14.5	4.4	14.1 0
(53, 54)	3.91	104	8.4	21.3	29.7	.28	7.58	5.44	17.1	12.3	6.7	4.2	37	416	7.9	2.3	28.8 0
(54, 53)	3.24	83	6.9	18.8	25.7	.27	7.93	5.80	18.2	13.2	8.6	7.8	40	340	7.4	2.1	23.2 0
(54, 55)	19.59	115	42.0	27.2	67.3	.61	3.54	1.39	34.1	14.2	5.8	4.6	35	440	17.0	5.0	12.2 0
(55, 54)	14.59	87	31.3	44.0	75.3	.42	5.16	3.02	52.0	30.4	18.8	16.5	73	348	11.6	5.1	12.5 0
SUBNETWORK=		1270.00	2787	43.28	126.24	169.52			3.45	2.72	1.99	1.79	2.6		7.5	695.4	17.0 292
													PER				
													TRIP				

Figure 4.1 Sample of NETSIM Cumulative Link and Subnetwork Statistics

LINK	BUS TRIPS	PERSON TRIPS	TRAVEL TIME (MINUTES)	MOVING TIME (MINUTES)	DELAY TIME (MINUTES)	M/T	SPEED (MPH)	NUMBER OF STOPS
(8012, 12)	4	240	4.8	1.1	3.7	.22	4.4	4
(8014, 14)	4	240	4.1	1.3	4.8	.22	4.5	3
(8023, 23)	4	120	5.0	.9	4.1	.18	5.4	3
(8043, 43)	10	600	4.9	1.1	3.8	.23	7.0	3
(8054, 54)	7	420	7.4	.8	4.7	.10	3.1	3
(8052, 52)	8	300	3.8	1.9	3.7	.34	10.1	4
(8041, 41)	10	600	7.4	1.3	2.9	.24	7.2	3
(8021, 21)	2	120	10.4	1.3	5.9	.21	6.2	3
(12, 22)	4	240	8.2	1.9	6.3	.23	4.8	4
(22, 32)	5	300	5.4	1.4	4.2	.23	7.4	3
(32, 42)	5	300	9.3	1.4	7.9	.17	5.0	7
(42, 52)	3	180	4.0	1.9	4.1	.32	9.5	4
(52, 62)	3	300	12.3	1.5	9.3	.25	2.4	3
(24, 34)	4	240	5.9	2.4	4.4	.20	4.1	3
(34, 44)	5	300	12.0	2.4	9.5	.09	2.5	2
(44, 54)	5	300	1.9	.2	1.7	.09	2.3	2
(21, 22)	2	120	3.0	.8	1.6	.34	10.2	2
(22, 23)	2	120	3.0	.8	2.2	.27	7.7	3
(23, 24)	2	180	3.4	.2	3.2	.07	2.0	2
(24, 25)	2	120	2.0	.2	1.9	.08	2.2	2
(25, 26)	3	180	3.9	1.1	2.8	.28	7.8	3
(26, 27)	2	120	4.8	.7	4.1	.13	4.3	2
(41, 42)	9	540	9.3	.7	8.6	.07	2.1	10
(42, 43)	8	480	6.4	.6	5.8	.10	2.8	9
(43, 44)	9	540	14.9	3.5	11.4	.23	6.9	9
(44, 45)	7	420	20.2	2.7	17.5	.13	3.9	7
(45, 46)	9	540	8.4	.7	7.6	.08	2.4	9
(46, 47)	9	540	7.8	.7	7.1	.09	2.4	9
(47, 48)	9	540	14.3	3.1	13.2	.19	3.4	9
(48, 49)	9	540	14.4	3.1	11.3	.21	4.3	9

Figure 4.2 Sample of NETSIM Bus Statistics

MOE, total delay (veh.min), average delay (sec/veh), volume, total vehicle miles, link occupancy, and number of traffic signal cycles that failed.

The second step was to get the average values for the rest of the MOE that were not included in step one. These included, percent of vehicles stopped, and percent of delay to total time.

The third step was to do the same as the first step but for the input data. The aggregation included the number of parking maneuvers per hour.

The fourth step was similar to the second step except that the averaging was performed to input variables and not to output MOE. These input data variables included percent of right turn, percent of left turn and the average distance between intersections which was used to estimate the average number of signals per mile.

The last step was to include the input data that were neither averaged nor aggregated to their corresponding input and output data. These data included the number of buses per hour that would run over these streets, the percent of heavy vehicles and the number of lanes.

A classification scheme was used to differentiate between streets based on different criteria. In other words one-way streets were separated from two-way streets. Failure to include the direct effect of parking maneuvers on delay in the equations in any form resulted in another sub-classification. Delay equations were developed for these two separate cases with parking and without parking.

4.3 Multiple Regression Analysis

In the last section, the data were aggregated, averaged and classified and were prepared for the regression analysis. In order to perform the multiple regression analysis, a computer software package called STATGRAFICS was used. The STATGRAFICS statistical Graphics System is a unique PC software package integrating a wide variety of statistical functions with high resolution colour graphics [40]. STATGRAFICS is a powerful statistical software package which offers a comprehensive data analysis capability.

The multiple regression analysis consumed a large amount of time, as a lot of trial and error steps were involved. The procedure was not easy, as it was not known which variables would fit in the regression equation and in which form would they be expressed. Besides, it was an important objective to include most of the variables that are important measures in TSM. Additionally, their presence in the equations should be statistically significant as much as possible. For each estimated equation, the R^2 which is known as the multiple coefficient of determination was checked. It is known that the higher the R^2 the better the equation, as R^2 measures the goodness of fit of the equation. That is, it gives the proportion or the percentage of the total variation in the dependent variable explained by the independent variables [55].

To test the overall significance of the estimated multiple regression the Analysis-Of-Variance (AOV) for the whole equation was performed. Besides, the analysis of variance for each independent variable of the main variables was performed to test the extent to which different variables affect the dependent variable, namely delay. In empirical investigations, it is important to perform (AOV) tests as it is useful in deciding whether it is worth adding another independent variable to the model knowing that several other variables are already present [55].

4.4 Model Development

4.4.1 Two-way Streets

Developing separate delay equations for two-way and one-way streets gave a chance to test the effect of each configuration on total delay. In the following sections, the estimated equations for each category will be given separately.

4.4.1.1 Two-way Streets Without Parking

The following equations were developed using regression analysis for two way streets without parking (i.e., curb parking is prohibited completely or during peak hours only).

General Equations

It was mentioned in a previous section that a degree of flexibility was incorporated in the developed equations. This flexibility is represented in the form of several equations. Each equation can be used on its own, depending on the different data available and the main concerns of the user.

The different equations for two-way streets without parking will be given together, as the difference among them is minimal. For each independent variable in the following equations the F-Ratio is given between brackets. The F-test represents the AOV of the independent variables. At the end of each equation, the R^2 is mentioned. The F-Ratio for the whole equation, its level of significance and the number of observations fitted will be given.

$$\begin{aligned}
 DEL = & -110.4717 + 12.7791 \text{ OCCUP} + 0.0989 \text{ STOP.SIGPM} \\
 & \quad (5927.27) \quad (65.84) \\
 & -7.9541 \text{ NLAN} \quad (4.01)
 \end{aligned} \tag{4.1}$$

$$\begin{aligned}
 R^2 &= 0.9721 & \text{No. of obs.} &= 173 \\
 \text{F-Ratio} &= 1999.04 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

$$\begin{aligned}
 DEL = & -120.0844 + 12.7906 \text{ OCCUP} + 0.09658 \text{ STOP.SIGPM} \\
 & \quad (5996.20) \quad (66.60) \\
 & -8.1094 \text{ NLAN} + 0.6847 \text{ LT} \\
 & \quad (4.06) \quad (2.97)
 \end{aligned} \tag{4.2}$$

$$\begin{aligned}
 R^2 &= 0.9724 & \text{No. of obs.} &= 173 \\
 \text{F-Ratio} &= 1517.46 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

$$\begin{aligned}
 DEL = & -112.3424 + 12.7937 \text{ OCCUP} + 0.09849 \text{ STOP.SIGPM} \\
 & \quad (5918.02) \quad (65.73) \\
 & -7.8740 \text{ NLAN} + 0.0445 \text{ (HV)}^4 \\
 & \quad (4.01) \quad (0.74)
 \end{aligned} \tag{4.3}$$

$$\begin{aligned}
 R^2 &= 0.9721 & \text{No. of obs.} &= 173 \\
 \text{F-Ratio} &= 1497.13 & \text{Prch. Value} &= 0.0000
 \end{aligned}$$

$$DEL = -110.0177 + 12.7671 OCCUP + 0.0979 STOP.SIGPM$$

(5918.93) (65.74)

$$-8.4935 NLAN + 0.0032 (NBUS)^2 \quad (4.4)$$

(4.01) (0.76)

$$R^2 = 0.9721 \quad \text{No. of obs.} = 173$$

$$F\text{-Ratio} = 1497.36 \quad \text{Prob. Value} = 0.0000$$

$$DEL = -122.4006 + 12.8069 OCCUP + 0.0960 STOP.SIGPM$$

(5992.63) (66.56)

$$-8.0255 NLAN + 0.7026 LT + 0.0489 (HV)^2 \quad (4.5)$$

(4.06) (3.12) (0.75)

$$R^2 = 0.9724 \quad \text{No. of obs.} = 173$$

$$F\text{-Ratio} = 1213.40 \quad \text{Prob. Value} = 0.0000$$

$$DEL = -119.9994 + 12.7772 OCCUP + 0.0953 STOP.SIGPM$$

(5997.32) (66.61)

$$-8.7418 NLAN + 0.7156 LT + 0.0037 (BUS)^2 \quad (4.6)$$

(4.06) (2.97) (1.03)

$$R^2 = 0.9724 \quad \text{No. of obs.} = 173$$

$$F\text{-Ratio} = 1214.40 \quad \text{Prob. Value} = 0.0000$$

$$DEL = -112.1005 + 12.7821 OCCUP + 0.0973 STOP.SIGPM$$

(5917.35) (65.73)

$$-8.4792 NLAN + 0.0037 (NBUS)^2 + 0.0511 (HV)^2 \quad (4.7)$$

(4.01) (0.76) (0.96)

$$R^2 = 0.9721 \quad \text{No. of obs.} = 173$$

$$F\text{-Ratio} = 1197.76 \quad \text{Prob. Value} = 0.0000$$

$$\begin{aligned}
 DEL = & -122.6770 + 12.7943 OCCUP + 0.0944 STOP.SIGPM \\
 & \quad (6004.31) \quad (66.69) \\
 & -8.7349 NLAN + 0.7415 LT + 0.0043 (NBUS)^2 \\
 & \quad (4.06) \quad (3.12) \quad (1.33) \\
 & + 0.0568 (HV)^2 \quad (4.8) \\
 & \quad (0.75)
 \end{aligned}$$

$$\begin{aligned}
 R^2 &= 0.9725 & \text{No. of obs.} &= 173 \\
 F\text{-Ratio} &= 1013.38 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

- Where **DEL** : total delay for the street (veh.min) for 15 minutes period;
- OCCUP** : street average occupancy in terms vehicles or average number of vehicles traveling over that particular street at an instant time;
- STOP** : percent of vehicles stopped at least once on each link on the street and averaged for the entire street;
- SIGPM** : average number of signals per mile on that street;
- NLAN** : number of moving lanes for that street;
- LT** : average percent of left turn for the entire street;
- HV** : percent of heavy vehicles which consist of trucks and buses (other than transit buses);
- NBUS** : number of buses per hour running on that street.

Range of Application

The range of values for which these equations are applicable depends on the range of the independent variables from which these were estimated:

$$5.3 \leq OCCUP \leq 81.9$$

$$34.5 \leq STOP \leq 87.25$$

$$5 \leq SIGPM \leq 19$$

$$1 \leq NLAN \leq 2$$

$$2 \leq LT \leq 28$$

$$0 \leq HV \leq 10$$

$$0 \leq BUS \leq 40$$

It should be noted that these equations should not be applied outside these ranges of values and especially **OCCUP**, **STOP**, and **SIGPM** since these variables have a large weight in these equations. On the other hand, applying these equations outside these ranges for the rest of the variables could be done within reasonable deviation because the error in the estimation will be minimal. In general it is not recommended to use these equations outside the ranges mentioned above.

Equation Analysis

Looking at the eight equations, together it can be observed that the value of R^2 (which is a measure of goodness of fit) are above 97%. This very high R^2 indicates that 97% of the variability of the dependent variable, which is delay, is explained by the different independent variables. It can also be observed that F-Ratio for the whole equation is very high and its significance is as high as 100%. In other words, the probability of incorrectly rejecting the null hypothesis which states that this equation does not explain the variability of the dependent variable (delay) is 0.00.

Considering each independent variable separately and comparing the critical F-Ratio for each variable (the number written between brackets below each independent variable) with the critical F-Ratios given in Table 4.1, the following observations can be made.

Table 4.1 Critical F-Ratios at Different Probability Values

Prob. Value	F (1, 120)
0.25	1.34
0.10	2.75
0.05	3.92
0.01	6.85

1. The F-Ratio for **OCCUP**, which is the mean occupancy of the link with vehicles, in all the equations was very high. It was significant at 0.00 level of significance. This result is logical as **OCCUP** is a measure of density. And it is logical that the higher the density (**OCCUP**) the higher the delay on that street.
2. The second term in all the estimated equations was the product of **STOP** and **SIGPM**. The F-Ratio for this term was significant for 0.00 level of significance. **STOP**, which is the percentage of vehicles that stopped at least once on that link, is an indication of the efficiency of the traffic signals on that street. **SIGPM** is the average number of signals per mile on that street and it is well known that the more signals per mile on a particular street the more time vehicles lose. This effect is taken into consideration implicitly in the HCM (chapter eleven) in estimating the average speed on the arterial [18].
3. The third term in the equations was the number of lanes on that street. This variable was significant at 5% level of significance. The effect of the number of lanes on delay is obvious. The higher the number of lanes the less delay experienced and that

is why the sign of this term is negative. It was thought that the reason for that decrease in the significance of that important variable could be explained due to the illegal stopping and parking.

4. **LT**, the fourth variable that appeared in equations (4.2), (4.5), (4.6) and (4.8), is the average percentage of left turn for the entire street. The F-Ratio of this variable was quite high and was significant at a probability value of less than 10%.
5. **HV**, the fifth variable that is present in equations (4.3), (4.7), and (4.8), is the average percentage of heavy vehicles of the total traffic and it includes trucks and buses (other than transit buses). Comparing the F-Ratio of this variable with values in Table 4.1, it is clear that this variable is not statistically significant for a probability of 25%. The reason for this is that percentage of heavy vehicles does not have that strong effect on delay on the intersection level and may be the aggregation decreased that effect. However, the reasons for keeping this variable in these equations, despite its lack of significance at the level of significance of 25%, are given below:
 - (a) it is not wrong from a statistical point of view to keep a variable with low significance in an equation as long it does not have an adverse effect on that equation [55];
 - (b) it is one of the important actions in TSM as its presence in the model is important;
 - (c) although it did not have a significance at a probability value 25%, it does have a significance at a probability value of 40%, which means that 60% of variability of this variable is explained with these constants associated with it in these equations.
6. The last variable that appeared in these equations was **NBUS** which is the number of transit buses per hour that are run on that street. It was present in equations (4.4), (4.6), and (4.8). **NBUS** was not significant for a probability value of 25% in equations (4.4) and (4.6), and it was significant in equation (4.8) for 25% probability value. Thinking of the reasons that caused that level of significance, it was concluded that this is due to the presence of protected bus stops which minimize the effect of

bus blockage. Almost 50% of the bus stops in the simulated networks were protected. Also aggregation may be another reason. It should be noted that the effect of bus blockage is higher than the effect of heavy vehicles on traffic [53] and the results of the estimated equations include that higher effect in the form of higher significance.

It should be mentioned that the effect of transit buses is not only the effect of the number of buses but also several other factors which are discussed in the next section [53]. Finally, it is recommended to use equation (4.5) in case of streets which do not include transit buses, and equation (4.8) in case of the presence of transit buses. They are recommended because of their high R^2 value, in addition to their coverage of most of the variables. Figures 4.3 and 4.4 show the predicted equations versus the observed points for equations (4.5) and (4.8).

Equations Including The Effect of Location of The Bus-stop

The effect of transit bus blockage on capacity, which in turn has an effect on total street delay, does not depend only on the number of buses per hour, but also on the area of the city, the street width and the location of the bus stop [53]. For that reason planners and traffic engineers take into consideration the location of bus stops when planning for transit routes. There are three general types of location for bus stops:

- Near-side bus stops (just prior to intersection);
- Far-side bus stops (just beyond the intersection);
- Mid-block bus stops.

In general, traffic engineers prefer far-side bus stops because they cause minimal blockage to right-turn and through movements [35,53]. On the other hand the near-side bus stop has a greater adverse effect on traffic than the far-side bus stop. However, the near-side bus stop has an advantage for bus operations which is combining the delay of loading and unloading and the delay of the red signal indication. This is achieved by adjusting the bus arrival at the beginning of the red signal. The third type of bus stop is located at mid-block. It is used when the entire width of the street is needed for storing and moving the traffic. Its influence on the traffic is not constant compared to the other two types and depends on the circumstances of each individual case [53].

Figure 4.3 Estimated Equation vs Observed Points (Eq. 4.5)

(X 100)

Figure 4.4 Estimated Equation vs Observed Points (Eq. 4.8)

In order to consider the effect of the location of the bus stop and number of buses, the streets that had buses running on them were separated from the rest of the two-way streets. Using multiple regression analysis, the following delay equation was estimated:

$$\begin{aligned}
 DEL = & -103.6372 + 12.7999 OCCUP + 0.0946 STOP.SIGPM \\
 & (3579.41) \quad (44.10) \\
 & - 8.5559 NLAN + BSTOP \{BSTOP=-12.8154 \text{ for M}\} \\
 & (2.67) \quad (2.45) \quad \{BSTOP=-1.0456 \text{ for F}\}
 \end{aligned} \tag{4.9}$$

$$\begin{aligned}
 R^2 &= 0.9755 & \text{No. of obs.} &= 92 \\
 F\text{-Ratio} &= 726.216 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

Where:

- BSTOP** : Dummy variable which gives the effect of the location of the bus-stop;
M : Mid-block bus-stop;
F : Far-side bus-stop.

Range of Application

The range of values for the application of this equation is given below:

$$\begin{aligned}
 5.86 &\leq OCCUP \leq 81.4 \\
 34.5 &\leq STOP \leq 87.25 \\
 5 &\leq SIGPM \leq 19
 \end{aligned}$$

Equation Analysis

It can be observed that the value of R^2 is 0.9755, which is a very high value of the coefficient of determination. It can also be observed that the equation is significant for a probability value of 0.00.

Analyzing the different independent variables in the same manner of the previous section, the following remarks are given.

Table 4.2 Critical F-Ratios at Different Probability Values

Prob. Value	F (1, 60)	F (2, 60)
0.25	1.35	1.42
0.10	2.79	2.39
0.05	4.00	3.15
0.01	7.08	4.89

1. The variables **OCCUP** and **STOP.SIGPM** are significant for a probability value of 0.00 .
2. The variable **LT** was significant at approximately 10% significance level.
3. The last variable in the equation shown was a dummy variable which took into consideration the effect of the location of bus stop. It should be noted that the effect of near-side bus stop on delay is considered inclusively in the constant of the equation. On the other hand, the term **BSTOP** consists of two constants. Each constant considers the effect of one of the two other types of bus stops relatively to the near-side bus stop. In other words the delay calculated using equation (4.9) without the term of **BSTOP** takes into consideration the effect of near-side bus stop and subtracting either one of the two constants gives the effect of the other type. The negative sign of far-side bus stop follows the general trend mentioned previously. The negative sign of mid-block bus stop relative to the near-side bus stop makes sense especially when this negative sign will be positive in a later case which proves that the effect of the mid-block is not constant and it depends on the individual circumstances.

Finally, the **BSTOP** variable is significant at 10% probability value. It should be noted that the degree of freedom of this variable is two while all the other variables were one. Figure 4.5 shows the estimated equation versus the observed points.

4.4.1.2 Two-way Streets With Parking

One of the important measures in TSM, is prohibiting parking either partially or completely. Failure to include this important variable in the delay equations resulted in developing separate equations for streets having parking allowed in peak hours. It should be noted that the observations used in these equations are different from those used to develop the previous equations. The following equations have the same general terms.

$$\begin{aligned}
 DEL = & -119.9469 + 12.0429 OCCUP + 0.1373 STOP.SIGPM \\
 & \quad (4180.74) \quad (78.04) \\
 & - 7.2482 NLAN \quad (4.10) \\
 & \quad (3.73)
 \end{aligned}$$

$$\begin{aligned}
 R^2 &= 0.9793 & \text{No. of obs.} &= 91 \\
 F\text{-Ratio} &= 1420.72 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

$$\begin{aligned}
 DEL = & -121.3686 + 12.0536 OCCUP + 0.1365 STOP.SIGPM \\
 & \quad (4137.65) \quad (77.24) \\
 & - 7.3806 NLAN + 0.1183 LT \quad (4.11) \\
 & \quad (3.33) \quad (0.10)
 \end{aligned}$$

$$\begin{aligned}
 R^2 &= 0.9791 & \text{No. of obs.} &= 91 \\
 F\text{-Ratio} &= 1054.58 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

$(\times 100)$

Figure 4.5 Estimated Equation vs Observed Points (Eq. 4.9)

$$\begin{aligned}
 DEL = & -120.1702 + 12.0404 OCCUP + 0.1374 STOP.SIGPM \\
 & (4132.87) \quad (77.15) \\
 & -7.2221 NLAN + 0.0347 HV \\
 & (3.33) \quad (0.00)
 \end{aligned} \tag{4.12}$$

$$\begin{aligned}
 R^2 &= 0.9791 & \text{No. of obs.} &= 91 \\
 F\text{-Ratio} &= 1053.34 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

$$\begin{aligned}
 DEL = & -121.3899 + 12.0532 OCCUP + 0.1366 STOP.SIGPM \\
 & (4089.54) \quad (76.34) \\
 & -7.3769 NLAN + 0.1178 LT + 0.0042 HV \\
 & (3.29) \quad (0.10) \quad (0.00)
 \end{aligned} \tag{4.13}$$

$$\begin{aligned}
 R^2 &= 0.9788 & \text{No. of obs.} &= 91 \\
 F\text{-Ratio} &= 833.855 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

Range of Application

The range of values for the different variables from which these equations were estimated is as follows:

$$\begin{aligned}
 4.7 &\leq OCCUP \leq 54 \\
 3.5 &\leq STOP \leq 85.75 \\
 5 &\leq SIGPM \leq 14 \\
 5 &\leq LT \leq 30
 \end{aligned}$$

The rest of the variables are as mentioned in the previous section and are mainly the range of the input data.

Equation Analysis

It is clear that the effect of buses is not shown in these equations. The reason for this is that at the time of creating the input data, it was chosen to have a very small number of streets that experience both buses running and parking maneuvers. As it is rare to find two-way streets where curb parking is permitted while buses are running in peak hours. For that reason, there were only a very small number of observations that did not

give any significance as a separate variable in the equations.

The results of the regression analysis show a coefficient of determination above 97%. This indicates that 97% of the variability of delay could be explained by the independent variables.

Comparing the F-Ratios for the different independent variables and the critical F-Ratios given in Table 4.3, the following observations are offered:

Table 4.3 Critical F-Ratios at Different Probability Values

Prob. Value	F (1, 60)
0.25	1.35
0.10	2.79
0.05	4.00
0.01	7.08

1. The variables **OCCUP**, and **STOP.SIGPM** were significant at 0.00% level of significance which was the same result attained in the previous equations.
2. The variable **NLAN** was significant at 10% significance level which is less than two-way streets without parking which was significant at 5% significance level. That reduction in the significance can be explained due to the disturbing effect of parking on the moving lanes adjacent to the parking lane which might have caused some unclear effect of **NLAN**.
3. The variable **LT** was not significant even at 25% level of significance although it was significant at 10% level of significance in case of two-way without parking. It was thought that the reason was due to the randomness of the input data which might

have turned out that streets with high delay values had low percentages of left turn and vice-versa.

4. The last variable shown in these equations was **HV**. It was insignificant at 25% level of significance, and its F-Ratio was almost zero. For that reason, and due to the importance of this variable another equation including this variable was estimated. The new equation included **HV** at 0.00% level of significance and is as follows:

$$\begin{aligned}
 DEL = & -102.0392 + 0.0851 \text{ OCCUP.HV} + 0.0195 (STOP)^2 \\
 & \quad (449.67) \quad \quad (489.62) \\
 & + 1.2037 \text{ OCCUP.SIGPM} \quad \quad \quad (4.14) \\
 & \quad (611.81)
 \end{aligned}$$

$$\begin{aligned}
 R^2 &= 0.9011 & \text{No of obs.} &= 91 \\
 F\text{-Ratio} &= 517.0350 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

The estimated equation showed quite a high R^2 and it was significant at 0.00 significance level. On the other hand, the different terms were also significant at 0.00% significance level. The only problem in this equation was its very high sensitivity to **OCCUP** which means that any slight error in estimating the **OCCUP** will cause a very large error in estimating delay. For that reason it was decided to use this equation to estimate a factor that can be used in estimate the effect of **HV** on delay. Sensitivity analysis was used within the applicable range of the different variables. The following table was estimated:

Table 4.4 Delay Reduction Factor For Different Percentages of Heavy Vehicles

Percent Decrease in HV	Delay Reduction Factor
1 %	0.99
5 %	0.95
10 %	0.91

Table A.1 in Appendix A shows the detailed calculations for estimating the delay reduction factor.

Equation (4.10) is recommended for use in case of two-way streets with parking. Figure 4.6 shows the estimated equation vs the observed points.

4.4.2 One-way Streets

It was mentioned in the previous section that different delay equations were developed for two-way and one-way streets. The one-way street delay equations have a format similar to that for two-way street equations.

4.4.2.1 One-way Streets Without Parking

Similar to the two-way streets, separate equations were developed for one-way with parking and without parking. These they are given as follows:

General

$$\begin{aligned}
 DEL = & -266.21039 + 15.0448 OCCUP + 0.2043 STOP.SIGPM \\
 & \quad (5925.48) \quad (79.92) \\
 & -12.7617 NLAN \quad (4.15) \\
 & \quad (3.29)
 \end{aligned}$$

$$\begin{aligned}
 R^2 &= 0.9914 & \text{No. of obs.} &= 53 \\
 F\text{-Ratio} &= 2002.90 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

$$\begin{aligned}
 DEL = & -277.9530 + 15.0324 OCCUP + 0.1940 STOP.SIGPM \\
 & \quad (5900.32) \quad (79.58) \\
 & -13.0654 NLAN + 1.4299 LT \quad (4.16) \\
 & \quad (3.28) \quad (0.79)
 \end{aligned}$$

$$\begin{aligned}
 R^2 &= 0.9914 & \text{No. of obs.} &= 53 \\
 F\text{-Ratio} &= 1495.99 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

$$DEL = -271.6212 + 15.0806 OCCUP + 0.2032 STOP.SIGPM$$

(4665.48) (61.35)

$$-11.9154 NLAN + BSTOP \{BSTOP= 11.3355 \text{ for M}\} \quad (4.20)$$

(1.37) (0.33) $\{BSTOP= -2.1467 \text{ for F}\}$

$$R^2 = 0.9908 \quad \text{No. of obs.} = 45$$

$$F\text{-Ratio} = 945.771 \quad \text{Prob. Value} = 0.0000$$

Range of Application

The range of application for equation (4.20) is :

$$17.5 \leq OCCUP \leq 130.3$$

$$37 \leq STOP \leq 88.5$$

$$5 \leq SIGPM \leq 19$$

Equation Analysis

The R^2 for equation (4.18) is very high and in the order of 99%. Again the entire equation is significant at 0.00% level of significance. The analysis of variance for the individual variables were performed. Comparing the calculated F-Ratios and the critical ones from Table 4.7, the following comments are presented.

Table 4.7 Critical F-Ratios for Different Probability Values

Prob. Value	F (1, 40)	F (2, 40)
0.25	1.36	1.44
0.10	2.84	2.44
0.05	4.08	3.23
0.01	7.31	5.18

$$\begin{aligned}
 DEL = & -264.6713 + 15.0962 OCCUP + 0.2053 STOP.SIGPM \\
 & (5898.78) \quad (79.55) \\
 & - 16.8420 NLAN + 0.0097 (NBUS)^2 \quad (4.17) \\
 & (3.27) \quad (0.80)
 \end{aligned}$$

$R^2 = 0.9914$ No. of obs. = 53
 F-Ratio = 1495.60 Prob. Value = 0.0000

$$\begin{aligned}
 DEL = & -277.1542 + 15.0865 OCCUP + 0.1943 STOP.SIGPM \\
 & (5887.02) \quad (79.39) \\
 & -17.4633 NLAN + 1.5333 LT + 0.0104 (NBUS)^2 \quad (4.18) \\
 & (3.27) \quad (0.79) \quad (0.91)
 \end{aligned}$$

$R^2 = 0.9914$ No. of obs. = 53
 F-Ratio = 1194.27 Prob. Value = 0.0000

Range of Application

The range of values for the application of equations (4.15) through (4.18) is given as follows:

$$\begin{aligned}
 17.5 & \leq OCCUP \leq 130.3 \\
 37 & \leq STOP \leq 88.5 \\
 5 & \leq SIGPM \leq 19 \\
 2 & \leq NLAN \leq 4 \\
 5 & \leq LT \leq 24 \\
 0 & \leq HV \leq 10 \\
 0 & \leq NBUS \leq 40
 \end{aligned}$$

Equation Analysis

Applying the procedure for the assessment of the equations, the following steps were performed. First, the overall performance of the equations is identified through the coefficient of determination and the analysis of variance for the entire equation. Secondly, the analysis of variance for each independent variable is discussed. Thirdly, brief

comments on the different reasons that might have caused these results are given.

The results of the regression analysis indicate that the goodness of fit of these equations is very high. Less than 1% of the variability of delay cannot be explained with these equations.

The AOV indicates that the model is significant at 0.00 level of significance.

It is clear that all the four equations mentioned above do not include the effect of HV. Failure to include it with the expected sign was the main reason for its absence. The explanation for this and the solution are given at the end of this section.

Comparing the F-Ratio (the figure between brackets) for the different independent variables and the critical F-Ratios given in Table 4.5, results in the following comments.

Table 4.5 Critical F-Ratio at Different Probability Values

Prob. Value	F (1, 40)
0.25	1.36
0.10	2.84
0.05	4.08
0.01	7.31

1. The variable **OCCUP** is still significant at 0.00 level of significance. Its F-Ratio exceeds the critical value by a large margin.
2. The second term in the equations **STOP.SIGPM** is also significant at 0.00 level of significance. This strong significance of both the first and second terms were expected as they were already significant to that level of significance in two-way

streets and that change in type of streets does not effect the nature of these variables.

3. The third variable which is **NLAN** was significant at 10% level of significance. It should be noticed that the level of significance was less than in case of two-way streets. This decrease in the significance might be explained due to the presence of exclusive bus lanes which has less positive effect on delay because it does not carry the same traffic as an ordinary lane.
4. **LT**, the fourth variable that appeared in these equations was not significant at 25% level of significance. It showed to be significant at about 40% level of significance. The reason for that insignificance was the absence of the opposing traffic in one-way streets which is the major cause of delay caused by left turn, besides **LT** was not a key variable in two-way equations (i.e., it was significant at 0.00% level of significance).
5. **NBUS** was not significant at 25% level of significance. This result was expected as it was approximately significant at 25% level of significance in equation (4.8), as well as one-way streets include exclusive bus lanes which have less effect on delay.

As mentioned earlier, failure to include **HV** in the equations with the expected sign resulted in estimating a new equation which includes that variable .

$$\begin{aligned}
 DEL = & -249.6205 + 0.0696 \text{ OCCUP.HV} + 0.0447 (STOP)^2 \\
 & \quad (97.51) \quad \quad \quad (47.16) \\
 & + 1.2311 \text{ OCCUP.SIGPM} \quad \quad \quad (4.19) \\
 & \quad (65.43)
 \end{aligned}$$

$$\begin{array}{ll}
 R^2 = 0.7493 & \text{No. of obs.} = 53 \\
 F\text{-Value} = 70.0329 & \text{Prob. Value} = 0.0000
 \end{array}$$

Again due to the high sensitivity of this equation to **OCCUP** and the possibility of receiving major errors in estimating delay as a result of a minor error in estimating **OCCUP**, this equation was used only to develop a factor that gives the effect of **HV** on

delay. Table 4.6 shows the estimated factor. Detailed calculations are shown in Table A.2 in Appendix A.

Table 4.6 Delay Reduction Factor for Different Percentages of HV

Percent decrease in HV	Delay Reduction Factor
1 %	0.99
5 %	0.97
10 %	0.94

Equations (4.15) and (4.18) are recommended for use in case of one-way streets without parking. The first is used in case where no detailed information is available. The second is used where information about transit buses and left turn is available. These estimated equations are shown in Figures 4.7 & 4.8.

Equations Including Location of Bus-stop

The procedure used in estimating the equation including the effect of the location of bus stop was exactly the same as in the two-way case. One-way streets including transit buses were separated, multiple regression analysis was used and the following equation was estimated:

Figure 4.7 Estimated Equation vs Observed Points (Eq. 4.15)

Figure 4.8 Estimated Equation vs Observed Points (Eq. 4.18)

$$DEL = -271.6212 + 15.0806 OCCUP + 0.2032 STOP.SIGPM$$

(4665.48) (61.35)

$$-11.9154 NLAN + BSTOP \{BSTOP= 11.3355 \text{ for M}\} \quad (4.20)$$

(1.37) (0.33) {BSTOP= - 2.1467 for F}

$$R^2 = 0.9908 \quad \text{No. of obs.} = 45$$

$$F\text{-Ratio} = 945.771 \quad \text{Prob. Value} = 0.0000$$

Range of Application

The range of application for equation (4.20) is :

$$17.5 \leq OCCUP \leq 130.3$$

$$37 \leq STOP \leq 88.5$$

$$5 \leq SIGPM \leq 19$$

Equation Analysis

The R^2 for equation (4.18) is very high and in the order of 99%. Again the entire equation is significant at 0.00% level of significance. The analysis of variance for the individual variables were performed. Comparing the calculated F-Ratios and the critical ones from Table 4.7, the following comments are presented.

Table 4.7 Critical F-Ratios for Different Probability Values

Prob. Value	F (1, 40)	F (2, 40)
0.25	1.36	1.44
0.10	2.84	2.44
0.05	4.08	3.23
0.01	7.31	5.18

1. The variables **OCCUP**, **STOP** and **SIGPM** showed to be significant at 0.00 level of significance as usual.
2. **NLAN** was significant at 25% level of significance. This result supports the reasons mentioned in the previous section about the decrease in the level of significance of this variable. The effect of exclusive lanes on the significance is clearer as all these streets have buses running on them.
3. The last variable in this equation was the dummy variable **BSTOP**. It did not show any significance at 25% level of significance. Again this result is realistic as it is known that the effect of bus blockage and bus stops decreases with the increase in the street width and can even be negligible in some cases of streets with high percentages of turning movements. And although the variable **BSTOP** was not significant, the signs estimated by the model were logical. As for the far-side bus stop, the sign was correct (i.e, negative) in comparison to the effect of near-side bus stop. The mid-block bus stop had a realistic positive sign which indicates that its effect on delay is higher than near-side bus stop. As mentioned before its effects depends on the circumstances of each individual case [53].

Figure 4.9 shows the estimated equation vs the observed points.

4.4.2 One-way Streets With Parking

Similar to two-way streets, separate delay equations were estimated for one-way streets with parking and without parking.

General Equations

$$\begin{aligned}
 DEL = & -196.3250 + 14.6415 OCCUP + 0.1652 STOP.SIGPM \\
 & \quad (2399.95) \quad (32.69) \\
 & - 20.4013 NLAN \\
 & \quad (4.93)
 \end{aligned}
 \tag{4.21}$$

$$\begin{array}{ll}
 R^2 = 0.9807 & \text{No of obs.} = 49 \\
 F\text{-Ratio} = 812.525 & \text{Prob. Value} = 0.0000
 \end{array}$$

Figure 4.9 Estimated Equation vs Observed Points (Eq. 4.20)

$$\begin{aligned}
 DEL = & -226.4519 + 14.6840 OCCUP + 0.1589 STOP.SIGPM \\
 & \quad (2557.24) \quad (34.84) \\
 & - 20.2233 NLAN + 2.2310 LT \quad (4.22) \\
 & \quad (5.25) \quad (3.95)
 \end{aligned}$$

$$\begin{aligned}
 R^2 &= 0.9819 & \text{No. of obs.} &= 49 \\
 F\text{-Ratio} &= 650.319 & \text{Prob. Value} &= 0.0000
 \end{aligned}$$

Range of Application

The range of the different variables for the application of the equations are as follows :

$$\begin{aligned}
 17.3 &\leq OCCUP \leq 109.9 \\
 39 &\leq STOP \leq 82.75 \\
 5 &\leq SIGPM \leq 19 \\
 4 &\leq LT \leq 27
 \end{aligned}$$

Equation Analysis

The R^2 and the overall significance for both equations (4.21) and (4.22) are as high as for all the previous equations. It is clear that **HV** is missing from these equations. The reasons are the same as noted in the previous sections. Delay reduction factors similar to those estimated for the previous cases are given. The variable **NBUS** is also not present in these equations. The reasons for that are the same as in two-way streets with parking. The very small number of streets simulated including transit buses and parking, caused that this variable did not show any significance in these equations. A comparison of the F-Ratios for the different independent variables with the critical F-Ratios given in Table 4.8, results in the following observations:

Table 4.8 Critical F-Ratios for Different Probability Values

Prob. Value	F (1, 40)
0.25	1.36
0.10	2.84
0.05	4.08
0.01	7.31

1. **OCCUP** and **STOP.SIGPM** were both significant at 0.00% level of significance.
2. **NLAN** was significant for 5% level of significance
3. **LT** was significant at 10% level of significance.

In order to estimate the delay reduction factors, the following equation was estimated first:

$$\begin{aligned}
 DEL = & -243.1664 + 0.0922 \text{ OCCUP.HV} + 0.0467 (STOP)^2 \\
 & \quad (24.77) \quad (25.87) \\
 & + 1.2559 \text{ OCCUP.SIGPM} \quad (4.23) \\
 & \quad (140.59)
 \end{aligned}$$

$$\begin{aligned}
 R^2 = & 0.7968 & \text{No. of obs.} = & 49 \\
 F\text{-Ratio} = & 63.7429 & \text{Prob. Value} = & 0.0000
 \end{aligned}$$

Again due the high sensitivity of equation (4.23) to **OCCUP**, it was used only to estimate the following reduction factors shown in Table 4.9.

Table 4.9 Delay Reduction Factors for Different Percentages of HV

Percent Decrease in HV	Delay Reduction Factor
1 %	0.99
5 %	0.96
10 %	0.93

Calculations for estimating the delay reduction factor are shown in Appendix A.

Figure 4.10 shows the recommended equation in case of one-way streets with parking (i.e., equation 4.22).

4.5 General Analysis of The Developed Equations

The results of the regression analysis including all the statistical tests were given in the previous sections. Individual analyses were performed for each group of equations. In this section, a general analysis for all the equations is given as follows:

1. Total delay in veh.min fitted better than average delay per vehicle in sec/veh in all the different cases. The R^2 for total delay equations were over 97% while for average delay per vehicle (sec/veh) were in the range of 72% - 82%. Some of the important variables representing TSM measures, gave wrong signs when included in the equations of delay sec./veh. On the other hand these same variables gave right signs with delay in veh.min and also had high significance. That gives the reason as to why total delay in veh.min was used and not delay in sec/veh.
2. The R^2 for one-way streets were slightly higher than for two-way streets. The R^2 for one-way streets were over 98% and the R^2 for the two-way streets were in the range of 97%-98%.

Figure 4.10 Estimated Equation vs Observed Points (Eq. 4.22)

2

of/de

2

3. Street occupancy or street mean density with vehicles, was a key variable that had a very strong significance on total delay. The probability value or the significance level was 0.00%.
4. The product of average percent of vehicles stopped at least once for the entire street and the average number of signals per mile for that street were significant at 0.00 level of significance.
5. The number of lanes per street had a high significance on delay. It was statistically significant for a probability value of 10%.
6. The effect of the number of lanes on the total delay had a higher level of significance for two-way streets without parking than two-way streets with parking.
7. The average percent of vehicles turning left for the entire street was significant for a probability value of less than 10% for two-way streets without parking and one-way streets with parking. On the other hand, this average percent was not significant for that probability value for two-way streets with parking and one-way streets without parking.
8. The variable number of buses per hour was not significant for a probability value of 25%.
9. The average percent of heavy vehicles for the entire street was not significant for a probability value of 25%.
10. The location of bus-stop as a dummy variable was significant for a significance level of 10% for two-way streets, while it was not significant for a significance level of 25% for one-way streets. This result is logical as the effect of transit buses blockage and the location of the bus-stop would have a stronger effect in two-way streets as the number of lanes in each direction are less than in case of one-way streets. This causes the effect of any kind of friction to become more pronounced.

CHAPTER 5

NETWORK-LEVEL ANALYSIS

5.1 Introduction

Characterizing road traffic in CBD networks by a small number of network-level descriptors and the quantification of their interrelationships is an important problem that has not received sufficient attention to date in traffic research. The importance of this problem is due to the following reasons:

- (a) there is a need to evaluate and compare the overall quality of service of traffic control strategies and designs at the network level;
- (b) comparisons can be made among cities or among networks of the same city over a number of time periods; and
- (c) possible deficiencies can be identified in current operations that result in overall conditions that are below the norm [18].

This problem was investigated by Mahmassani, Williams, and Herman [19,44]. The investigation was performed through a number of simulation experiments using **NETSIM**. They examined several concepts and theories related to the network-level traffic flow. One of the theories they examined was the fundamental theory of flow that relates speed, concentration and flow.

The results indicated that the main relations between speed, concentration, and flow can be applied at the network-level in a similar way they are applied to the arterial level. Tables 5.1 and 5.2 present a sample of their results, which is a comparison of the average network flow with the product of average speed and concentration.

Later, extensive field work was done to validate the work. Time-lapse aerial photography over the CBD of Austin and Dallas, as well as ground experiments were

**Table 5.1 Comparison of Average Network Flow with Product
of Average Speed and Concentration, [19]**

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K (vehicles per lane- mile)	V (miles per hour)	KV (vehicles per lane- hour)	Q (vehicles per lane- hour)	Percent Dif- ference [($KV - Q$)/ Q]
9.90	16.836	166.7	168.6	-1.1
19.80	15.418	305.3	309.6	-1.4
41.58	10.904	453.4	453.0	0.1
61.38	7.592	466.0	473.1	-1.5
81.18	5.751	466.9	454.5	2.7
100.65	2.881	290.0	300.3	-3.4

**Table 5.2 Comparison of Average Network Flow with Product
of Average Speed and Concentration, [56]**

V^a	K^b	KV^c	Q^d	Difference ^e (%)
12.21	61.38	749	762	1.7
12.74	61.38	782	792	1.3
11.32	69.30	784	810	3.3
12.64	53.46	676	690	2.1
13.20	45.54	601	618	2.8
12.54	27.72	348	354	1.7
12.87	27.72	357	360	0.8
12.24	35.64	436	438	0.5
12.71	35.64	453	456	0.7
11.19	61.38	687	696	1.3
12.90	19.80	255	258	1.2
11.84	45.54	539	546	1.3
12.32	45.54	561	570	1.6
10.70	69.30	742	750	1.1
9.93	79.20	786	798	1.5
11.30	53.46	604	600	-0.7

performed. The field tests verified the simulation work and the results indicated that the average flow in a street network may indeed be expressed as the product of space mean speed and concentration [56,57].

Due to the availability of this kind of data for twenty four network, it was encouraging to verify this previous work using the existing data. In addition, the authors [43], recommended further simulation experiments including network topological irregularities which were present in these data.

5.2 The Traffic Flow Theory for Network-Level

The first step was to estimate the three fundamental variables; speed, concentration, and flow for the network level. Average speed, V in mph for the network-level was given in the outputs as a MOE for the entire network. It was originally calculated as the ratio of total vehicle-mile to the total vehicle-hours over the network.

The average concentration, K in vehicle/lane.mile which is the average of the number of vehicles per unit lane length, was estimated by dividing the constant number of vehicles on the network (i.e.the total network occupancy) by the total lane miles L for the entire network [18,43,55].

The average network flow Q in vehicles/hour which is considered the average number of vehicles per unit time lane (hour) that passes through an "average" point of the network [18,43] was estimated as follows. The traffic volumes for each link over the entire network were averaged after dividing it by the number of lanes.

After estimating the variables K , V and Q , for the networks, the product $K.V$ was then compared with the independently estimated Q . The results of the comparison for the twenty four simulated networks are shown in Table 5.3. It is clear that the two quantities are quite close, and the differences are minimal except for about five points which are

**Table 5.3 Comparison of Average Network Flow with Product
of Average Speed and Concentration**

V mph	K (veh/lane.mile)	K.V	Q (veh/lane.hour)	Difference (%)
10.80	26.80	291.18	289.47	0.6
9.20	30.50	281.48	280.60	0.3
10.70	26.50	266.45	281.25	5.6
12.40	18.09	216.40	224.30	3.7
9.70	26.44	261.33	256.48	1.9
10.90	26.46	263.18	288.50	9.6
7.50	39.65	314.33	297.38	5.4
13.60	19.18	253.80	260.81	2.8
8.30	35.77	305.34	296.94	2.8
13.00	22.58	288.04	293.50	1.9
6.80	43.34	307.05	294.70	4.0
12.50	18.70	228.49	233.80	2.3
7.60	39.54	320.13	300.50	6.1
9.90	30.50	315.80	301.96	4.4
6.84	39.15	297.20	267.66	9.9
11.00	27.11	303.26	298.18	1.7
8.60	35.34	303.56	303.88	1.8
4.33	51.24	243.55	221.89	8.9
10.20	25.65	282.07	261.61	7.3
6.30	47.11	326.17	296.78	9.0
3.10	70.68	218.18	219.10	0.4
9.30	34.66	334.39	322.38	3.6
5.30	43.66	237.90	231.39	2.7
5.10	54.98	289.75	280.38	3.2

slightly higher than results shown in Tables 5.1 and 5.2. However, it should be kept in mind that the authors in their work used networks that have a high degree of regularity and uniformity, while in this research the networks included a high degree of randomness and irregularity.

Plotting the famous curves that explain the fundamental theory of flow, Figures 5.1, 5.2, and 5.3 resulted. Figure 5.1 shows the relation between concentration K and speed V . It is clear that as the concentration increases the speed decreases. The straight line in Figure 5.1 represents the equation that relates speed to concentration. It was estimated by using regression analysis. The assumption of the linear relation between speed and concentration is based on the widely used models of Greenshields and Greenberg [58,59,60].

This linear relationship can be expressed as follows:

$$V = 16.2054 - 0.2103 K \quad (5.1)$$

$$R^2 = 0.8939 \quad \text{No. of obs.} = 24$$

$$F = 211.73 \quad \text{Prob. Value} = 0.00$$

Where

V : speed in mph

K : average concentration vehicle/lane.mile

The results of the regression analysis indicate that the linear relation between K and V is significant, as the R^2 is very high and the F-Ratio is significant for 0.00 level of significance.

Figures 5.2 and 5.3 show the relation between speed-flow and concentration-flow. It is clear that the points are considerably scattered around the best fit curve. The points in Figures 5.2 and 5.3 can be considered quite good in expressing the tendency of the expected curves especially when compared to the real data curves shown in Figures 5.4 and 5.5.

**Figure 5.1 Speed-Concentration Relationship
for Network-Level**

**Figure 5.2 Speed-Flow Relationship
for Network-Level**

**Figure 5.3 Flow-Concentration Relationship
for Network-Level**

Figure 5.4 Speed-Flow Relationship, [18,60]

Figure 5.5 Concentration-Flow Relationship, [36,61]

5.3 Network-Level Delay Model

The development of a network delay model is very useful for TSM planners and traffic engineers. Such a tool is useful for the planner to decide whether to apply a planned control strategy or not, based on the overall performance of the network. Such a tool is very useful in cost-effectiveness studies, when there is a comparison among a number of plans. Furthermore, such a tool can be useful in long-range planning, when future traffic projections are available and there is a need to estimate delay.

In order to develop that model, the MOE obtained from NETSIM's second run were used. The following delay equation was estimated using regression analysis. The reasons for having only one independent variable relates to the statistical significance and the expected sign of variables. Figure 5.6 shows the estimated equation versus the observed points.

$$DEL = -34.0340 + 0.2500 OCCUP \quad (5.2)$$

$$R^2 = 0.9825$$

$$\text{No. of obs.} = 24$$

$$F = 1404.76$$

$$\text{Prob. Value} = 0.00$$

where:

DEL : Total delay for the entire network in veh.hr;

OCCUP : Total network occupancy with vehicles.

The range of values for the application of this equation is given below.

$$257.9 \leq OCCUP \leq 971.8$$

Assessment of The Equation

- The coefficient of determination of the equation is as high as for the street level. This figure indicates that over 98% of the variability of total delay is explained with the independent variable *OCCUP*.
- *OCCUP* was significant at 0.00 level of significance.

Figure 5.6 Estimated Equation vs Observed Points

CHAPTER 6

MODEL VALIDATION

6.1 Field Data Collection

Due to difficulty in obtaining a complete delay study (i.e., includes information about all the prevailing roadway, traffic, and signalization conditions) for a number of downtown streets from any transportation firm in Ottawa or from the literature for the model validation, a field data acquisition was needed.

In order to complete any traffic study three main activities should be carried out with care and accuracy. These are:

1. data collection;
2. data reduction;
3. data analysis.

Figure 6.1 shows of the three main activities performed to complete the field data collection.

6.1.1 Data Collection

Field data were collected through the use of the videotape. The advanced level of development of videotape technology is making this method of data collection easier and cheaper than all other methods. The videotape has several advantages over other methods of data collection:

- videotape is a continuous real-time record of flow;
- a single videotape cartridge can record two hours or more of field data;
- videotape does not require costly development and is immediately available for use;
- videotape can be retained as a permanent record or may be used as appropriate; and

Figure 6.1 Field Data Collection Procedure

- Videotape of data can be reviewed many times, and at reduced speed if desired [36].

The first step in the data collection process was to choose the streets to be studied and the locations where the video camera was to be placed to tape traffic on the selected streets. The camera had to be mounted on the highest floor possible in the buildings chosen in order to view the longest possible section of the street.

The view of the camera had to capture at least two links, as this was considered the minimum length that can represent a street. Due to the capability of the camera used, which included a zoom lens F1.2, Macro 11 ~ 88, ϕ 52, it was not possible to get an accurate view for more than two links.

The streets selected for data collection are located in the CBD of Ottawa. Several field investigations had to be done to find suitable buildings. These buildings had to be high rise type with an unobstructed view of two links of selected streets. Following the choice of suitable buildings, permission had to be obtained to tape from these buildings.

Figures 6.2 and 6.3 show the selected streets and the location of the chosen buildings. Table 6.1 gives the following: the direction of the traffic taped; the date; and the time of taping.

Table 6.1 Location, Dates, and Times For Field Studies

Street	Direction	Date	Time
Laurier	EB	11/4/90	4:00-5:00 p.m
Queen	EB	12/4/90	8:30-9:30 a.m
Kent	NB	17/4/90	3:30-4:30 p.m

The following observations can be drawn from Table 6.1 .

- All the videotaping was performed for peak hour conditions. Either a.m. peak or p.m. peak traffic was captured and this is due to the scope of this research.

Figure 6.2 Streets and Buildings Selected for Videotaping

Figure 6.3 Streets and Buildings Selected for Videotaping

- All the videotaping was performed on either Tuesdays or Wednesdays, given that the middle part of the week is the best time for urban traffic studies, since the variation in commuter traffic is minimal [18].
- The selected streets with their dissimilar configurations and characteristics cover a considerable number of cases for model validation. Laurier Avenue is a two-way street without parking. Queen Street is a two-way street with transit buses running on it, parking is prohibited from 7:30-9:00 a.m. and permitted after 9:00 a.m. Kent Street is one-way street with parking allowed during peak hours.

6.1.2 Data Reduction

The next step taken was to extract the needed data from the tapes and to record them manually onto paper. The tapes were viewed on a video monitor in the main video control room at Carleton University. The advantage of using this room was that it includes video machines with 4-heads which give the capability of receiving an accurate vision while pausing. In addition, there are a large number of television screens, both coloured and black and white types. Having both kinds of screens is useful as the black and white screens give brighter and more accurate pictures when pausing. It should be noted that street occupancy counts are made when the tape is in the pause position. On the other hand coloured monitors are very useful for estimating the total travel time on the street by tracing vehicles through their colour and using a stop watch to measure the time spent to traverse the street.

It is important to note that the delay equations for the street level were based on 15-minute simulations. In other words the total delay in veh.min estimated by these equations is the total delay experienced on a street for a 15-minute period. In practical terms, every one hour tape produced four data sets for each street. But because of the very limited time allowed by the building managers to use their properties for videotaping purposes, the one hour allowance to tape traffic did not result in one complete hour of videotaping per street. Consequently, only three data sets were produced.

The data extracted from the tapes for each 15 minutes included:

- street occupancy (i.e., the occupancy of vehicles on the links observed);

- traffic flow for each link (i.e., number of vehicles that pass any point on this link in fifteen minutes);
- number of stopped vehicles for each link;
- number of vehicles turning left for each link;
- number of heavy vehicles on each link;
- number of transit buses per fifteen minutes; and
- total travel time (in seconds) for a vehicle to travel the two links.

In order to estimate the street occupancy as accurately as possible the video machine was paused every fifteen seconds and vehicles occupying that section of the street were counted. This means that sixty readings for the street occupancy were taken for each fifteen minutes. Occupancy observations Tables are shown in Appendix B.

The total travel time was estimated by tracing a vehicle from the beginning of the two links till the end, and using a stop watch to measure the time in seconds. In order to assure a reasonable accuracy in estimating the travel time, the total travel time for fifteen vehicles was estimated for each period. The estimated travel time tables are given in Appendix B. The rest of the data mentioned were extracted by viewing the tape several times for better accuracy, and for each run of the tape, one of these data was obtained.

6.1.3 Data Analysis

In this section the use of data for model validation is described. The street occupancy was estimated by averaging the one minute occupancy over the fifteen minute period. The percent of vehicles stopped was estimated for each link by dividing the number of vehicle stopped in fifteen minutes by the traffic flow for the same period. Then this percentage was averaged for the entire street. Both the percent of vehicles turning left and the percent of heavy vehicles were estimated in a manner similar to that of the percent of vehicles stopped. The average travel speed, which is a form of space mean speed, was estimated based on the following relation [36,58]:

$$U = n.d/\Sigma t_i \quad (6.1)$$

Where: U : average travel speed (fps);

n : number of total travel time observed;

d : distance traversed (ft);

t_i : total travel time of the i^{th} vehicle (sec).

This relation was applied for the entire street as: $n = 15$, d = length of street section or of two links, t_i = total travel time measured every minute. It was estimated that each hour of videotape would require ten hours of reduction/analysis time.

6.2 Model Validation

In order to estimate delay for actual streets, the definition of delay had to be the same as used in model developed. The definition of delay for the equations developed was based on NETSIM's definition of delay. NETSIM defines total delay time as the difference between total travel time and ideal travel time for each link, based on mean free flow speed [8,16,17]. Putting this definition in practical terms, the following expressions are given:

$$\blacksquare \quad \text{ITT} = [(N\text{VEH} * L) / U_f] / 60 \text{ (sec/min)} \quad (6.2)$$

$$\blacksquare \quad \text{TTT} = [(N\text{VEH} * L) / U] / 60 \text{ (sec/min)} \quad (6.3)$$

Where ITT : Ideal Travel Time in veh.min;

TTT : Total Travel Time in veh.min;

NVEH : Traffic flow for 15 minutes in veh;

U_f : Mean Free Flow Speed in fps; and

U : Average Travel Speed in fps.

$$\blacksquare \quad \text{Total Delay Time for Each Link} = \text{TTT} - \text{ITT} \quad (6.4)$$

$$\blacksquare \quad \text{Total Delay Time for The Entire Street}$$

$$= \Sigma \text{ Total Delay Time on Each Link}$$

The details of the calculations for all the streets are given in Appendix B.

The equations that were tested using the field data are given as follows:

Equation (4.5) was applied for Laurier Avenue, as this equation is recommended

to be used for two-way streets without parking and without transit buses.

Equation (4.8) was applied in the first two periods (first two 15 minutes) for Queen Street, because it is recommended to be used for two-way streets without parking and with transit buses.

In the third fifteen minutes period for Queen Street, parking was allowed. Therefore, equation (4.10) was applied.

In case of Kent Street, equation (4.22) was used as it is the recommended equation for one-way streets including parking.

The results of the comparison between field and estimated delay are given in Table 6.2. Besides Figures 6.5, 6.6, and 6.7 show the difference between field and estimated delay for each street separately.

It can be seen that the results are very good as the differences are within 10 % and less, except for the third period of Queen Street. This might be explained due to the assumption that a few of two-way streets have both parking and transit buses which might have had caused that decrease in the efficiency of equation (4.10) when used to estimate cases including transit buses.

Looking at Table 6.2 and Figure 6.6, it is clear that the 1st reading for Kent Street is missing. The reason is that the street occupancy estimated for the first period was less than the minimum range of application of equation (4.22).

Table 6.2 Comparison of Field Delay and Delay Estimated by Model

Street	Period (15 min)	Field Delay	Model Delay	Difference (%)	Equation Used
Laurier	1 st	165.08	150.05	9.1%	(4.5)
	2 nd	95.86	89.94	6.2%	(4.5)
	3 rd	93.65	88.03	6.0%	(4.5)
Queen	1 st	107.57	98.59	9.1%	(4.8)
	2 nd	208.02	202.10	2.8%	(4.8)
	3 rd	284.77	245.07	13.9%	(4.10)
Kent	2 nd	318.90	326.02	2.2%	(4.22)
	3 rd	390.91	393.91	1.0%	(4.22)

Figure 6.4 Comparison of Field Delay Estimate and Delay Estimated by The Model for Laurier Avenue

Figure 6.5 Comparison of Field Delay Estimate and Delay Estimated by The Model for Queen Street

Figure 6.6 Comparison of Field Delay Estimate and Delay Estimated by The Model for Kent Street

CHAPTER 7

MODEL COMPARISON WITH OTHER

MODELS AND APPLICATIONS

7.1 Introduction

In this chapter the developed model will be compared with the delay models **NETSIM** and the **HCM**. The data used in the comparison are the field data obtained from the videotape. Additionally, several illustrative examples are given to show the different applications of the model developed in this research.

7.2 Comparison of the Developed Model and NETSIM

NETSIM is an accurate computer micro-simulation program that has been used as a powerful tool on a very wide basis. Both researchers and engineers felt that total delay estimated by **NETSIM** is trustworthy. It was thought that it would be useful to compare delay estimated by the model developed and **NETSIM** using field data.

Laurier Avenue, in Ottawa CBD was chosen for this study. Geometric characteristics of the street were obtained from the data provided by the Regional Municipality of Ottawa Carleton (RMOC). The signalization characteristics for the two intersections within the street were also obtained from RMOC. The cycle length, the green split and the phasing were provided. Traffic on Laurier was considered the main direction (i.e, it was given the green phase first).

The traffic characteristics for the chosen street were estimated from the videotape. Peak-hour volumes for the entry links, percentage of heavy vehicles and percentage of turning movements were extracted from the videotape.

The results of the comparison are shown in Figure 7.1. The difference between delay estimated by NETSIM and delay estimated by the model is quite small (7.5%). This result reinforces the validation of the developed equations, since delay estimates by NETSIM have been proved to be valid and accurate [24,25].

7.3 Comparison of the Developed Model and the HCM

The HCM is one of the most comprehensive documents in traffic engineering. Its delay model for signalized intersections is considered very famous. The HCM does not include an explicit delay model for CBD streets, but it has a procedure in chapter eleven which can be used to estimate average travel speed for urban arterials (which can be considered CBD streets). This procedure, in turn, could be used to estimate delay for the entire street. For such an application, the definition of delay is assumed to be the difference between total travel time and ideal travel time based on a designated free flow speed.

In order to estimate the total travel time for the arterial, the average travel speed should be estimated first. It is estimated by the following relationship:

$$Art\ spd = \frac{3600 (L)}{(running\ time/mile) (L) + (total\ of\ intersection\ approach\ delay)}$$

Where:

L = arterial segment length (miles)

Figure 7.1 Comparison of Delay Estimated by NETSIM and
Delay Estimated by The Model

*Running time
per mile*

= total of the running time per mile on all segments in the arterial or section, in sec and it is estimated from Table 7.1 based on the classification of the arterial. This classification is based on the arterial location, the arterial environment, the roadside development and other factors. In this case the arterial class was 3 and the free flow speed used was 30 mph which is the same as used in several locations in this research.

*Total intersection
approach delay* =

total of the approach delay at all intersections within the defined arterial or section.

The 3600 is the conversion factor to compute the ART SPD in mph.

The total delay for each lane group is calculated from the following relationships:

$$d = (d_1 + d_2) PF$$

$$d_1 = 0.38 C \left[\frac{\left(1 - \frac{g}{C}\right)^2}{\left(1 - \left(\frac{g}{C}\right)X\right)} \right] \quad (7.1)$$

(7.2)

$$d_2 = 173 X^2 \left[(X-1) + \sqrt{(X-1)^2 + 16 \frac{X}{C}} \right] \quad (7.3)$$

where: d = average stopped delay per vehicle for the subject lane group (sec/veh);

d_1 = uniform delay per vehicle for the subject lane group (sec/veh);

d_2 = incremental delay per vehicle for the subject lane group (sec/veh);

g = green ratio for the subject lane group (sec);

C = cycle length (sec);

X = V/c ratio for the subject lane group;

c = capacity of the subject lane group (vph); and

Table 7.1 Segment Running Time Per Mile, [18]

ARTERIAL CLASS	I			II			III		
	45	40	35	35	30	35	30	35	25
Free Flow Speed (mph)									
Average Segment Length (miles)									
0.05									
0.10									
0.15									
0.20									
0.25									
0.30									
0.40									
0.50									
1.00									

RUNNING TIME PER MILE (SEC/MI)

NOTES

1. It is best to have an estimate of free flow speed. If one is lacking, however, then use the above table assuming the following default values

For Class I Free Flow Speed (mph)

II 40
III 35
30

2. For very long segment lengths on Class I arterials (1 mi or longer), free flow speeds may be used to compute running time per mile. These times are shown in the entries for a 1.0 mile segment length.
3. If a Class I arterial has a segment length lower than 0.20 mi, the user should (a) reevaluate the classification and if it should remain as a distinct segment, then (b) use the values for 0.20 mi.
4. Likewise, Class II and Class III arterials with segment lengths greater than 0.25 mi should first be reevaluated (i.e., is the classification correct?) If necessary the above values can be extrapolated.
5. Although this table does not show segment running time dependent on traffic flow rate, it is logical that there is such a dependence. However, the dependence of intersection delay on traffic flow rate is much stronger and thus dominates in the computation of arterial travel speed.

PF = platoon adjustment factor for the arrival type in the subject lane group.

It should be noted that the delay estimated using equations (7.2) and (7.3) is for an assumed random arrival condition, although, in most cases, arrivals are not random but platooned as a result of signal progression and other factors. The HCM defines five arrival types and one should be specified for each lane group. The arrival type for each lane group is estimated by calculating the platoon ratio which is defined as the proportion of vehicles arriving on green divided by the g/C and using Table 7.2. The platoon adjustment factors are obtained from Table 7.3.

Data for Queen and Kent streets were used for this comparison. The proportion of vehicles arrived on green were calculated by subtracting the proportion of vehicles stopped, estimated for model validation, from one for each approach. The g/C ratio was obtained with all the other data from the RMOC. Knowing the platoon ratio, the type of arrival was estimated. The rest of the input data were obtained from the RMOC and estimated from the videotape.

Using the HCM computer software, the average travel speed for each arterial (street) was estimated. Consequently, the total travel time was calculated based on the total length of the street and the average travel speed. The ideal travel time was also estimated, based on the total length and the free flow speed, which was assumed to be 30 mph. Subtracting the ideal travel time from the total travel time, the average delay per vehicle for the entire street was estimated.

In order to compare both delays, they had to have the same form. The delay estimated by the HCM had to be multiplied by the total traffic flow of the two intersections within each street. On the other hand the delay estimated by the model had to be multiplied by four to give the delay experienced for the whole hour. The delay experienced in the peak fifteen minutes was the one multiplied by four for the comparison, as delay estimated by the HCM is based on peak hour flow (demand volume is divided by the peak hour factor).

The results are shown in Figure 7.2. For Queen Street the difference between the HCM delay and the model delay was 7.9% as a percent of the HCM delay. For Kent Street, the difference between the HCM delay and the model delay was 5.5% as a

<i>Arrival Type</i>	<i>Range of Platoon Ratio, R_p</i>
1	0.00 to 0.50
2	0.51 to 0.85
3	0.86 to 1.15
4	1.16 to 1.50
5	≥ 1.51

Table 7.2 Platoon Ratio as A Measure of Arrival Type, [18]

Type of Signal	Lane Group Types	v/c Ratio, X	Arrival Type ^a				
			1	2	3	4	5
Pretimed	TH, RT	≤ 0.6	1.85	1.35	1.00	0.72	0.53
		0.8	1.50	1.22	1.00	0.82	0.67
		1.0	1.40	1.18	1.00	0.90	0.82
Actuated	TH, RT	≤ 0.6	1.54	1.08	0.85	0.62	0.40
		0.8	1.25	0.98	0.85	0.71	0.50
		1.0	1.16	0.94	0.85	0.78	0.61
Semiactuated	Main St. TH, RT ^b	≤ 0.6	1.85	1.35	1.00	0.72	0.42
		0.8	1.50	1.22	1.00	0.82	0.53
		1.0	1.40	1.18	1.00	0.90	0.65
Semiactuated	Side St. TH, RT ^b	≤ 0.6	1.48	1.18	1.00	0.86	0.70
		0.8	1.20	1.07	1.00	0.98	0.89
		1.0	1.12	1.04	1.00	1.00	1.00
	All LT ^c	all	1.00	1.00	1.00	1.00	1.00

* See Table 7-2

^b Semiactuated signals are typically timed to give all extra green time to the main street. This effect should be taken into account in the allocation of green times.

^c This category refers to exclusive LT lane groups with protected phasing only. When LTs are included in a lane group encompassing an entire approach, use factor for the overall lane group type. Where heavy LTs are intentionally coordinated, apply factors for the appropriate through movement.

Table 7.3 Platoon or Progression Adjustment Factor, PF_p , [18]

Figure 7.2 Comparison of Delay Estimated by HCM and Delay Estimated by The Model

percent of the HCM delay. It is clear that differences between the two models is quite small. These results support the validation of the developed model.

7.4 Model Applications

The developed equations can be used to evaluate a large number of TSM actions such as:

- Traffic signal improvements;
- Curb parking prohibition in CBD during peak-hours;
- Truck peak-hour deliveries restriction in CBD;
- Transit bus service improvements;
- Vehicles Miles of Travel (VMT) reductions such as toll bridges and high occupancy vehicles projects;
- Road widening;
- Bus-stop location changes

7.5 Illustrative Examples of Model Application

In this section a number of illustrative examples will be discussed. The purpose of giving these examples is to show, in more detail, how the TSM users can apply these delay equations.

Example 1:

1. Description- Given a two-way, four lane street in a downtown area. The section of the street considered consists of five intersections. The following information items are given for the southbound direction: parking is prohibited; street occupancy = 28 veh; and the number of signals per mile = 9. Detailed information is given for each intersection in the following Table 7.5.1

Table 7.5.1 Field Observations

Intersection	% LT	% HV	% Veh. Stopped
(1)	10	12	50
(2)	18	8	45
(3)	12	10	70
(4)	13	7	82
(5)	27	13	88
Average	16	10	67

It should be noted that all the information given was estimated through the use of the videotape technique. It is required to estimate the total delay experienced on that street during peak hours.

2.Solution- The average values for the % LT, % HV and the percent of vehicles stopped were calculated and are shown in the last line in Table 7.5.1.

Equation (4.5) recommended for two-way streets without parking and transit buses, is applied through the following calculation.

$$\begin{aligned}
 \text{DEL} &= -122.4006 + 12.8069 \text{ OCCUP} + 0.0960 \text{ STOP.SIGPM} \\
 &\quad - 8.0255 \text{ NLAN} + 0.7026 \text{ LT} + 0.0489 (\text{HV})^2 \\
 \text{DEL} &= -122.4006 + 12.8069 (28) + 0.0960 (67)(9) \\
 &\quad - 8.0255 (2) + 0.7026 (16) + 0.0489 (10)^2 \\
 &= 294.16 \text{ veh.min}
 \end{aligned}$$

This example shows how easy a TSM user can apply the developed equations and receive an accurate estimate for the total delay for the entire system with minimum

expense and time.

Example 2:

1. Description- If for the same information given in example 1, the planner decided to prohibit left turns on that section of the street, what would be the saving in the total travel time during the peak hour?

2. Solution- Applying equation (4.5) again but with $LT = 0$, the following calculations are given:

$$\begin{aligned} DEL &= -122.4006 + 12.8069 (28) + 0.096 (67)(9) \\ &\quad - 8.0255 (2) + 0.0489 (10)^2 \\ &= 282.92 \text{ veh.min} \end{aligned}$$

Saving in total travel time = $294.16 - 282.92 = 11.24 \text{ veh.min}$

Percent decrease of delay based on the original case = 4%

Prohibiting LT is an important TSM measure, especially on congested CBD streets where excessive delay is experienced due to that permissive LT. Applying the equations the effect of this important TSM measure on total delay, was estimated in a very simple manner.

Example 3:

1. Description- Given a one-way, three lane street, the section considered consists of five links with a total length of 3300 ft. A travel time study was performed and the average travel speed was estimated 9.75 mph. Traffic volume was counted for the peak hour amounted to 1200 vph. The following averages were estimated for the entire street:

- percent of left turn = 11%;
- percent of heavy vehicles = 10%;
- the number of buses per hour = 10; and
- the percent of vehicles stopped = 78% (this figure was one of the outputs of a signal optimization program used for the street progression).

The traffic engineer wants to implement a new regulation by banning heavy vehicles from using the street in peak hours. The saving of total travel time is needed for

the cost-effectiveness study of the project.

2. Solution- The first step is to calculate the occupancy from the information given. It is assumed that each lane carries the same volume (i.e., 400 vph). Based on this, the density could be estimated using the fundamental theory of flow which states:

$$Q \text{ (flow)} = K \text{ (density)} \cdot V \text{ (speed)}$$

$$K = 400 / 9.7 = 41.24 \text{ veh/mile/lane}$$

Knowing the total length of the section of the road, the occupancy per lane was estimated.

$$\text{OCCUP (per lane)} = (41.24)(3300) / (5280) \text{ ft/mile}$$

$$\text{Therefore OCCUP (per lane)} = 25.775$$

$$\text{Total OCCUP} = 77.33 \text{ (for the whole street)}$$

Applying equation (4.18) which is recommended for one-way streets without parking the following calculations are given:

$$\begin{aligned} \text{DEL} = & -277.1542 + 15.0865 \text{ OCCUP} + 0.1943 \text{ STOP.SIGPM} \\ & - 17.4633 \text{ NLAN} + 1.5333 \text{ LT} + 0.0104 (\text{NBUS})^3 \end{aligned}$$

$$\begin{aligned} \text{DEL} = & -277.1542 + 15.0865 (77.33) + 0.1943 (8)(78) \\ & - 17.4633 (3) + 1.5333 (11) + 0.01 (10)^2 \\ = & 976.20 \text{ veh.min} \end{aligned}$$

Table 4.6 is used to obtain the delay reduction factor for decrease 10% of HV. The delay reduction factor is multiplied by the delay estimated from the last step to obtain the total delay for zero percent HV.

$$\text{DEL} = (976.20)(0.94) = 917.63 \text{ veh.min}$$

$$\text{Saving in total travel time} = 976.20 - 917.63 = 58.57 \text{ veh.min (6\% reduction in delay)}$$

With information available in most municipalities (volumes and travel speeds), the effect of banning heavy vehicles in peak hours was tested simply but efficiently for the entire street.

Example 4:

1. Description- Given a two-way four lane street with nine signals per mile, parking was allowed and occupied one lane in each direction (i.e, number of moving lanes = 1 per direction). Due to severe delays experienced on that street, parking was prohibited during

peak hours. The planner wants to estimate the benefits of prohibiting parking in terms of savings in the total travel time. Using a videotape technique the following information for before and after conditions is obtained.

Before: OCCUP = 22.6 veh, percent of vehicles stopped = 81 %

After: OCCUP = 22.33 veh, percent of vehicles stopped = 79 %

2. Solution- Using equation (4.10) for two-way streets with parking, the total delay for the before case was estimated.

$$\text{DEL} = -119.9469 + 12.0429 \text{ OCCUP} + 0.1373 \text{ STOP.SIGPM} \\ - 7.2482 \text{ NLAN}$$

$$\text{DEL} = 245.07 \text{ veh.min}$$

Using equation (4.5) which is used for two-way streets without parking, the total delay for the after case was estimated.

$$\text{DEL} = -122.4006 + 12.8069 (22.33) + 0.095 (9)(79) \\ - 8.0255 (2) = 215.07 \text{ veh.min}$$

Saving in the total travel time = $245.07 - 215.07 = 30.00 \text{ veh.min.}$

It is clear that the results are logical. Delay decreased as the parking was prohibited. In addition, the model proved to be sensitive, since it was capable of capturing the decrease in total delay, although differences in OCCUP and STOP between before and after were minimal.

Example 5:

1. Description- A new bus transit route is proposed to run on a two-way, four lane street. In order to ensure that the transit route operates efficiently, it is suggested to close a freeway ramp that discharges traffic onto that street during peak hours. It was estimated that 10% of the number of vehicles on that street are discharged from the ramp. It is required to estimate the effect of this proposed transit route and the ramp closure on the total delay experienced on that street. The following information for traffic on the heavier direction is given:

The average street occupancy = 41.7 veh (this figure was estimated from the traffic volume and the average travel speed); the average percent of left turn = 22%; the number of signals per mile = 7; the number of proposed buses = 30 bus/hour; and the percent of

heavy vehicles = 2%. The percent of vehicles stopped is assumed = 75%.

2. Solution- Equation (4.5) is used to estimate the total delay for the original case (before running the bus route and closing the freeway ramp), to estimate delay. On the other hand, equation (4.8) is to be applied to estimate the total delay for the new case.

Original case:

$$\begin{aligned} \text{DEL} &= -122.4006 + 12.8069 \text{ OCCUP} + 0.0960 \text{ STOP.SIGPM} \\ &\quad - 8.0255 \text{ NLAN} + 0.7026 \text{ LT} + 0.0489 (\text{HV})^2 \\ \text{DEL} &= -122.4006 + 12.8069 (41.7) + 0.0960 (75)(7) \\ &\quad - 8.0255 (2) + 0.7026 (22) + 0.0489 (4) \\ &= 461.65 \text{ veh.min} \end{aligned}$$

New case; OCCUP = (41.7)(0.90) = 37.53 VEH

$$\begin{aligned} \text{DEL} &= -122.6770 + 12.7943 \text{ OCCUP} + 0.0944 \text{ STOP.SIGPM} \\ &\quad - 8.7349 \text{ NLAN} + 0.7415 \text{ LT} + 0.0568 (\text{HV})^2 \\ &\quad + 0.0043 (\text{NBUS})^2 \\ &= -122.6770 + 12.7943 (37.53) + 0.0944 (75)(7) \\ &\quad - 8.0255 (2) + 0.7415 (22) + 0.0568 (4) \\ &\quad + 0.0043 (30)^2 \\ &= 411.41 \text{ veh.min} \end{aligned}$$

$$\begin{aligned} \text{Saving in total travel time (benefit)} &= 461.65 - 411.41 \\ &= 50.24 \text{ veh.min} \end{aligned}$$

Example 6:

1. Description- Planners of a certain city are planning to apply a high occupancy vehicle (HOV) project to the downtown area and they want to estimate the decrease in the total delay experienced, if they are expecting a reduction of 5% in the number of automobile trips (the estimation of the 5% is based on analogy and experience). The total occupancy of the network in peak hour is 700 veh.

2. Solution- Total delay for the network based on existing conditions is estimated using equation (5.2):

$$\text{DEL} = -34.0340 + 0.25 \text{ OCCUP}$$

$$\text{DEL} = -34.0340 + 0.25 (700) = 140.97 \text{ veh.hour}$$

The total delay for the network after applying the HOV project (OCCUP is decreased by 5%).

$$\text{DEL} = -34.0340 + 0.25 (700)(0.95) = 132.22 \text{ veh.hour}$$

$$\text{Decrease in the total delay} = 140.97 - 132.22 = 8.75 \text{ veh.hour.}$$

Equation (5.2) is very useful for TSM users, in terms of its simplicity and data requirements. The alternative of this equation is simulation models, such as NETSIM and TRANSYT. The huge differences in terms of the data requirements and simplicity of application between the developed equation and the simulation models mentioned above are clear. The first needs information about occupancy, which could be estimated through a cordon volume study. On the other hand, both NETSIM and TRANSYT need detailed information about the signalization conditions, geometric conditions and the traffic conditions for each link.

Example 7:

1. Description- Future projections of traffic volumes indicate an increase of 10% in traffic volumes in the next five years. An estimate of the increase in network congestion in terms of total delay is needed for the case of no improvements are made. The total network occupancy equals 750 veh.

2. Solution- Difference between total delay estimate based on the present occupancy and the projected occupancy will give the increase in the network congestion. It is assumed that the increase in the traffic volume of 10% will result in an increase of 10% of the occupancy.

$$\text{DEL} = -34.0340 + 0.25 (750) = 153.47 \text{ veh. hour}$$

$$\text{DEL} = -34.0340 + 0.25 (750)(1.10) = 172.22 \text{ veh. hour}$$

$$\text{Increase in total delay as percent of the present total delay} = 12.22 \%$$

It is clear from this application that the developed equation is a very useful planning tool.

APPENDIX A

The following Tables are the result of sensitivity analysis applied to equations (4.14), (4.19), and (4.22).

Table A.1 HV Delay Reduction Factor Calculations

OCCUP	STOP	SIGPM	Reduction factor for % HV decrease		
			1%	5%	10%
21	35	05	N.A.	N.A.	N.A.
37	35	05	0.980	0.900	0.820
54	35	05	0.980	0.910	0.840
21	35	10	0.990	0.950	0.910
37	35	10	0.992	0.960	0.920
54	35	10	0.992	0.960	0.926
21	35	14	0.994	0.970	0.939
37	35	14	0.994	0.972	0.945
54	35	14	0.994	0.973	0.957
21	52	05	0.982	0.910	0.820
37	52	05	0.984	0.920	0.846
54	52	05	0.990	0.950	0.880
21	52	10	0.992	0.960	0.919
37	52	10	0.992	0.962	0.927
54	52	10	0.992	0.963	0.929
21	52	14	0.994	0.970	0.945
37	52	14	0.994	0.973	0.948
54	52	14	0.995	0.974	0.949
21	69	05	0.994	0.930	0.867
37	69	05	0.986	0.931	0.871
54	69	05	0.987	0.932	0.873
21	69	10	0.993	0.965	0.931
37	69	10	0.993	0.965	0.933
54	69	10	0.993	0.965	0.933
21	69	14	0.996	0.978	0.956
37	69	14	0.996	0.977	0.955
54	69	14	0.996	0.976	0.950
21	85	05	0.990	0.902	0.802
37	85	05	0.989	0.892	0.802
54	85	05	0.988	0.888	0.888
21	85	10	0.994	0.942	0.942
37	85	10	0.993	0.939	0.939
54	85	10	0.993	0.937	0.937
21	85	14	0.996	0.956	0.956
37	85	14	0.996	0.955	0.955
54	85	14	0.996	0.954	0.954
Averages			0.990	0.950	0.910

25%. This result indicates that their effect on the total delay for the entire street are not strong enough to be presented separately.

8. The delay reduction factor for two-way streets was slightly higher than that in case of one-way streets. The result is logical, since two-way streets are more sensitive than one-way streets to vehicle friction.
9. The location of bus stops had a significant effect on total delay for two-way streets. On the other hand, its significance decreased in case of one-way streets, which could be explained due to the presence of the exclusive bus lanes.
10. The videotape technique is a very useful in collecting field data for urban streets. It is a cheap and very efficient procedure.
11. The results of the validation proved that the model's delay estimates are quite accurate. The average difference between field delay estimate and the delay calculated by the model, as a percentage of field delay, was 7.9% for two-way streets and 2% for one-way streets.
12. The results of the comparison with NETSIM and the HCM showed that the delay estimated by the model is very close to both models. This result reinforces the validation of the model and its accuracy in delay estimation.
13. It is clear that the model is a very useful tool for TSM users. It covers most of the important TSM actions. The model is easy to use and its data requirement is considerably small. The model is capable of estimating delay for the entire street without going through details of the signalization characteristics.

II) Network - Level

1. The results of the analysis indicated that the fundamental relations between speed, concentration, and flow can be applied at the network-level in a similar way they are applied to the street-level.
2. The network mean content with vehicles (occupancy) proved to be significant for 0.00% level of significance. It was the only variable that had a significant presence with expected sign.
3. The high significance of occupancy for both street-level and the network-level

proves that this variable has the strongest effect on total delay (veh.min or veh.hour) for urban CBD street network systems.

4. The delay model developed for the entire network is a useful tool not only for TSM planning process but also for long-range planning process.

8.2 Recommendations for Further Work

1. More work should be done to refine the effect of transit buses running on streets with parking. As it was noted earlier, owing to a small number of cases which were considered, difficulties were encountered in terms of level of significance and sign of the coefficient.
2. More work needs to be conducted to improve the presence of the heavy vehicles variable in the regression equations.
3. More simulation work needs to be conducted to ensure that the estimated equations are in the best form that can be reached.
4. More field data should be collected for checking the accuracy of the developed models in delay estimation under different traffic conditions.
5. The developed equations have their ranges of application. Due to the scope of this research these ranges are limited to peak hour conditions. It would be beneficial to extend these ranges of application to cover off-peak conditions as well.
6. The development of a computer software that could extract and estimate the following information directly from the videotape would be a valuable tool for field data collection: street occupancy; volume; average speed; and delay.
7. It would be useful to investigate the possibility of adding the effect of bus stop type, whether it is protected (with bay) or unprotected as a separate term.
8. The inclusion of vehicle occupancy (persons) to the developed models needs to be investigated, since High Occupancy Vehicles options are considered important TSM actions.
9. Improving transit bus level of service is one of the main objectives of TSM

planning process. Among other means, exclusive bus lanes and bus preemption are examples of TSM measures implemented to achieve this objective. In order to assess such measures, developing a separate delay model for transit buses should be developed.

10. As mentioned in Chapter 1, TSM actions are initiated to accomplish several goals concerning congestion, safety, energy and environment. In this thesis, a number of models that can be used to assess delay (congestion) were developed. Subsequently, there is a need to review the literature for tools that can be used for assessing the impacts of actions concerning energy, emissions and safety. Following that, these tools or models, if they exist, should be evaluated as to whether they could be used for TSM planning process efficiently. If not, more research should be performed to develop new models which are compatible with the delay models developed in this research, i.e., easy to use and moderate in their data requirements.

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APPENDIX A

The following Tables are the result of sensitivity analysis applied to equations (4.14), (4.19), and (4.22).

Table A.1 HV Delay Reduction Factor Calculations

OCCUP	STOP	SIGPM	Reduction factor for % HV decrease		
			1%	5%	10%
21	35	05	N.A.	N.A.	N.A.
37	35	05	0.980	0.900	0.820
54	35	05	0.980	0.910	0.840
21	35	10	0.990	0.950	0.910
37	35	10	0.992	0.960	0.920
54	35	10	0.992	0.960	0.926
21	35	14	0.994	0.970	0.939
37	35	14	0.994	0.972	0.945
54	35	14	0.994	0.973	0.957
21	52	05	0.982	0.910	0.820
37	52	05	0.984	0.920	0.846
54	52	05	0.990	0.950	0.880
21	52	10	0.992	0.960	0.919
37	52	10	0.992	0.962	0.927
54	52	10	0.992	0.963	0.929
21	52	14	0.994	0.970	0.945
37	52	14	0.994	0.973	0.948
54	52	14	0.995	0.974	0.949
21	69	05	0.994	0.930	0.867
37	69	05	0.986	0.931	0.871
54	69	05	0.987	0.932	0.873
21	69	10	0.993	0.965	0.931
37	69	10	0.993	0.965	0.933
54	69	10	0.993	0.965	0.933
21	69	14	0.996	0.978	0.956
37	69	14	0.996	0.977	0.955
54	69	14	0.996	0.976	0.950
21	85	05	0.990	0.902	0.902
37	85	05	0.989	0.892	0.892
54	85	05	0.988	0.888	0.888
21	85	10	0.994	0.942	0.942
37	85	10	0.993	0.939	0.939
54	85	10	0.993	0.937	0.937
21	85	14	0.996	0.956	0.956
37	85	14	0.996	0.955	0.955
54	85	14	0.996	0.954	0.954
Averages			0.990	0.950	0.910

Table A.2 HV Reduction Factor Calculations

OCCUP	STOP	SIGPM	Reduction factor for % HV decrease		
			1%	5%	10%
20	40	06	N.A.	N.A.	N.A.
60	40	06	0.985	0.927	0.864
100	40	06	0.988	0.942	0.890
130	40	06	0.989	0.945	0.896
20	40	12	0.988	0.944	0.894
60	40	12	0.994	0.971	0.944
100	40	12	0.995	0.974	0.949
130	40	12	0.995	0.975	0.951
20	40	18	0.995	0.974	0.950
60	40	18	0.996	0.982	0.965
100	40	18	0.997	0.983	0.967
130	40	18	0.997	0.984	0.968
20	64	06	0.984	0.921	0.854
60	64	06	0.989	0.947	0.900
100	64	06	0.990	0.951	0.906
130	64	06	0.990	0.952	0.908
20	64	12	0.994	0.970	0.940
60	64	12	0.995	0.975	0.952
100	64	12	0.995	0.976	0.953
130	64	12	0.995	0.976	0.953
20	64	18	0.996	0.982	0.964
60	64	18	0.997	0.984	0.967
100	64	18	0.997	0.987	0.968
130	64	18	0.996	0.984	0.969
20	88	06	0.994	0.969	0.940
60	88	06	0.991	0.957	0.914
100	88	06	0.990	0.953	0.911
130	88	06	0.990	0.952	0.908
20	88	12	0.996	0.983	0.965
60	88	12	0.995	0.979	0.959
100	88	12	0.994	0.978	0.957
130	88	12	0.994	0.978	0.957
20	88	18	0.997	0.987	0.974
60	88	18	0.996	0.986	0.972
100	88	18	0.996	0.985	0.971
130	88	18	0.996	0.985	0.970
Averages			0.990	0.970	0.940

Table A.3 HV Delay Reduction Factor Calculations

OCCUP	STOP	SIGPM	Reduction Factor for % HV Decrease		
			1%	5%	10%
30	40	06	N.A.	N.A.	N.A.
70	40	06	0.984	0.918	0.848
110	40	06	0.986	0.929	0.867
30	40	12	0.990	0.950	0.911
70	40	12	0.993	0.965	0.932
110	40	12	0.993	0.967	0.936
30	40	18	0.995	0.974	0.949
70	40	18	0.996	0.978	0.956
110	40	18	0.996	0.979	0.958
30	60	06	0.983	0.916	0.845
70	60	06	0.987	0.933	0.875
110	60	06	0.987	0.937	0.881
30	60	12	0.992	0.960	0.930
70	60	12	0.994	0.968	0.938
110	60	12	0.994	0.969	0.940
30	60	18	0.996	0.978	0.956
70	60	18	0.996	0.979	0.959
110	60	18	0.996	0.979	0.960
30	80	06	0.990	0.950	0.910
70	80	06	0.990	0.948	0.900
110	80	06	0.989	0.946	0.897
30	80	12	0.995	0.973	0.948
70	80	12	0.994	0.972	0.945
110	80	12	0.994	0.971	0.944
30	80	18	0.996	0.982	0.964
70	80	18	0.996	0.980	0.962
110	80	18	0.996	0.980	0.962
Averages			0.990	0.960	0.930

APPENDIX B

In this appendix detailed calculations for model validation are given. Calculations for each 15 minute of the selected streets will be given separately.

1. Laurier Avenue East-bound (4:00- 4:15 p.m.)

In order to estimate the street occupancy accurately, the videotape was paused every 15 seconds. Table B.1 shows the street occupancy observations for each 15 seconds as well as the average occupancy for each minute.

Table B.1 Occupancy Observations

Minute	15 seconds occupancy				Average
	1st	2nd	3rd	4th	
1st	13	15	21	13	16
2nd	14	21	17	13	16
3rd	13	15	16	15	15
4th	16	14	18	16	16
5th	13	17	20	18	17
6th	22	21	21	23	22
7th	23	27	21	16	22
8th	20	19	18	14	18
9th	14	19	16	14	16
10th	15	20	14	16	16
11th	19	12	12	11	14
12th	13	15	16	22	17
13th	13	15	14	13	14
14th	15	14	15	16	15
15th	13	14	13	13	13

Average occupancy = 16.5

Using a stop watch and tracing vehicles along the entire section, fifteen observations for total travel time were obtained. These observations are shown in Table B.2.

Table B.2 Field Measurements for Total Travel Time (sec)

85	85	70
80	100	55
85	110	50
100	105	70
85	70	65

Average travel time = 81 sec

Data extracted from the videotape for the first 15 minutes is given for each link separately as follows.

■ **1st link (between Kent & bank)**

15 minute traffic flow = 166 veh

number of vehicles stopped in 15 min = 158 veh

number of heavy vehicles in traffic = 4 veh

percent of vehicles stopped = $(158/166) \cdot 100 = 95\%$

percent of heavy vehicles in traffic = $(4/166) \cdot 100 = 2.4\%$

■ **2nd link (between Bank St. & O'Connor)**

15 minute traffic flow = 204 veh

number of vehicles stopped in 15 min = 169 veh

number of heavy vehicles in traffic = 5 veh

percent of vehicles stopped = 83%

percent of heavy vehicles in traffic = 2.5%

average percent of vehicles stopped on the two links =

$(95\% + 83\%) / 2 = 89\%$

average percent of heavy vehicles in traffic for both links = $(2.4\% + 2.5\%) / 2 = 2.45\%$

Length of 1st link = 623 ft

Length of 2nd link = 577 ft

Applying equations (6.2) and (6.3), and using the data extracted from the videotape, the following calculations are given.

$$ITT = [(NVEH * L)/U_r] / 60$$

Where ITT : ideal move time

$$NVEH1 = 166 \text{ veh} \text{ \& } NVEH2 = 204$$

$$U_r = 30 \text{ mph} = 44 \text{ fps}$$

$$L1 = 623 \text{ ft}$$

$$L2 = 577 \text{ ft}$$

Therefore

$$ITT (1) = [(166 * 623)/44] / 60 = 39.17 \text{ veh.min}$$

$$ITT (2) = [(204 * 577)/44] / 60 = 44.59 \text{ veh.min}$$

Since average travel time = 81 sec

Therefore, average travel speed = $[623 + 577] / 81 = 14.81 \text{ fps}$

$$TTT = [(NVEH * L)/U] / 60$$

Where TTT: total travel time

L1 & L2 same as for ITT

$$U = 14.81 \text{ fps}$$

$$TTT (1) = [(166 * 623)/14.81] / 60 = 116.38 \text{ veh.min}$$

$$TTT (2) = [(204 * 577)/14.81] / 60 = 132.46 \text{ veh.min}$$

Total delay for each link = TTT - ITT

$$\text{Total delay for link 1} = 116.38 - 39.17 = 77.21 \text{ veh.min}$$

$$\text{Total delay for link 2} = 132.46 - 44.59 = 87.87 \text{ veh.min}$$

$$\text{Total delay for the section} = 77.21 + 87.87 = 165.08 \text{ veh.min}$$

Applying equation (4.5) for Laurier Avenue, since it is recommended for two-way streets without parking and transit, the following calculations are noted.

$$DEL = -122.4006 + 12.8069 \text{ OCCUP} + 0.0960 \text{ STOP.SIGPM}$$

$$- 8.0255 \text{ NLAN} + 0.7026 \text{ LT} + 0.0489 (\text{HV})^2$$

$$DEL = -122.4006 + 12.8069 (16.5) + 0.096 (89)(9)$$

$$- 8.0255 (2) + 0.7026(0) + 0.0489 (2.45)^2$$

$$= 150.05 \text{ veh.min}$$

Difference between field delay estimate and the delay calculated by the model as a percentage of field delay

$$= [(165.08 - 150.05) / 165.02] * 100 = 9.1 \%$$

2. Laurier Avenue East-bound (4:15- 4:30 p.m.)

Pausing the videotape every fifteen seconds to count the street occupancy, sixty observations were obtained. These observations are shown in Table B.3.

Table B.3 Occupancy Observations

Minute	15 second occupancy				Average
	1st	2nd	3rd	4th	
1st	15	15	16	12	15
2nd	08	07	06	06	07
3rd	14	12	14	15	14
4th	19	18	18	14	17
5th	12	15	12	11	13
6th	16	10	15	19	15
7th	14	13	11	16	14
8th	16	14	16	15	16
9th	13	11	15	13	13
10th	12	11	12	10	11
11th	12	12	15	14	13
12th	12	13	10	11	12
13th	09	08	12	08	09
14th	11	10	10	14	11
15th	14	15	16	15	15

Average occupancy = 13

Fifteen measurements for total travel time are given in Table B.4.

Table B.4 Field Measurements for Total Travel Time (sec)

75	55	70
70	75	75
75	60	55
70	60	90
65	55	66

Average travel time = 67.75 sec

The following data was extracted from the videotape for each link separately.

■ **1st link (between Kent St.- Bank St)**

15 minute traffic flow = 131 veh
 number of vehicles stopped = 79 veh
 number of heavy vehicles in traffic = 4 veh
 percent of vehicles stopped = 60.3%
 percent of heavy vehicles in traffic = 3%

■ **2nd link (between Bank St.- O'Connor St.)**

15 minute traffic flow = 154 veh
 number of vehicles stopped = 126 veh
 number of heavy vehicles in traffic = 4 veh
 percent of vehicles stopped = 82%
 percent of heavy vehicles = 2.6%

Average percent of vehicles stopped for the entire section = $(60.3\% + 82\%)/2 = 71.2\%$

Average percent of heavy vehicles for the entire section
 $= (3\% + 2.6\%)/2 = 2.8\%$

Length of the 1st link = 623 ft

Length of the 2nd link = 577 ft

Number of signals per mile = 9

Applying equations (6.2) and (6.3), the following calculations are given.

$U_f = 30 \text{ mph} = 44 \text{ fps}$

$ITT (1) = [(131 \times 623)/44]/60 = 30.91 \text{ veh.min}$

$ITT (2) = [(154 \times 577)/44]/60 = 33.66 \text{ veh.min}$

average travel speed = $(623 + 577)/67.75 = 17.71 \text{ fps}$

$TTT (1) = [(131 \times 623)/17.71]/60 = 76.81 \text{ veh.min}$

$TTT (2) = [(154 \times 577)/17.71]/60 = 83.62 \text{ veh.min}$

Total delay on link (1) = $76.81 - 30.91 = 45.90 \text{ veh.min}$

Total delay on link (2) = $83.62 - 33.66 = 49.96 \text{ veh.min}$

Total delay for the entire section = $45.90 + 49.96 = 95.86 \text{ veh.min}$

Again equation (4.5) is applied to estimate delay for Laurier Avenue.

$$\begin{aligned} \text{DEL} &= -122.4006 + 12.8069 (13) + 0.096(71.2)(9) \\ &\quad - 8.0255 (2) + 0.7026 (0) + 0.0489 (2.8)^2 \\ &= 89.94 \text{ veh.min} \end{aligned}$$

Difference between field delay estimate and delay calculated by the model as a percentage of field delay = 6.2%

3. Laurier Avenue East-bound (4:30 - 4:45 p.m.)

Pausing the videotape every fifteen seconds, Table A.5 resulted.

Table A.5 Occupancy Observations

Minute	15 seconds occupancy				Average
	1st	2nd	3rd	4th	
1st	14	13	13	12	13
2nd	12	09	11	16	12
3rd	19	21	17	11	17
4th	07	06	04	04	05
5th	09	17	16	18	15
6th	15	16	14	15	15
7th	14	11	12	15	13
8th	10	11	05	06	08
9th	12	13	13	13	13
10th	18	16	13	13	15
11th	14	14	13	14	14
12th	10	11	13	10	11
13th	13	14	17	12	14
14th	13	11	14	14	13
15th	14	12	11	12	12

Average = 12.67

Using a stop watch and tracing vehicles, fifteen measurements for the total travel time were obtained. See Table B.6.

Table B.6 Field Measurement for Total Travel Time (sec)

74	60	60
65	74	63
48	50	65
56	50	100
60	95	70

Average travel time = 66 sec

Watching the videotape several times, the following data was extracted for each link.

■ **1st link (between Kent St.- Bank St.)**

15 minute traffic flow = 134 veh

number of vehicles stopped = 82 veh

number of heavy vehicles = 3 veh

percent of stopped vehicles = 61 %

percent of heavy vehicles = 2.2%

■ **2nd link (between Bank St.- O'Connor St.)**

15 minute traffic flow = 157 veh

number of vehicles stopped = 137 veh

number of heavy vehicles = 4 veh

percent of vehicles stopped = 87 %

percent of heavy vehicles = 2.6%

Average percent of vehicles stopped for the entire section = $(61\% + 87\%)/2 = 74\%$

Average percent of heavy vehicles = $(2.2\% + 2.6\%)/2 = 2.4\%$

Length of 1st link = 623 ft

Length of 2nd link = 577 ft

Number of signals per mile = 9

Applying equations (6.2) and (6.3) to estimate field total delay, the following calculations are stated.

$$\text{ITT (1)} = [(134 * 623)/44]/60 = 31.62 \text{ veh.min}$$

$$\text{ITT (2)} = [(157 * 577)/44]/60 = 34.31 \text{ veh.min}$$

$$\text{Average travel speed} = (623 + 577)/66 = 18.18 \text{ fps}$$

$$\text{TTT (1)} = [(134 * 623)/18.18]/60 = 76.53 \text{ veh.min}$$

$$\text{TTT (2)} = [(157 * 577)/18.18]/60 = 83.05 \text{ veh.min}$$

$$\text{Total delay on link (1)} = 76.53 - 31.62 = 44.91 \text{ veh.min}$$

$$\text{Total delay on link (2)} = 83.05 - 34.31 = 48.74 \text{ veh.min}$$

$$\text{Total delay for the entire section} = 44.91 + 48.74 = 93.65 \text{ veh.min}$$

Applying equation (4.5) (recommended for two-way streets without parking and transit buses), the following calculations are given.

$$\begin{aligned} \text{DEL} &= -122.4006 + 12.8069 (12.67) + 0.0960 (74)(9) \\ &\quad - 8.0255 (2) + 0.7026 (0) + 0.0489 (2.4)^2 \\ &= 88.03 \text{ veh.min} \end{aligned}$$

Difference between field delay estimate and delay estimated using the model as a percentage of field delay = 6.0%

4. Queen Street East-bound (8:30 - 8:45 a.m.)

Pausing the videotape every fifteen second, the occupancy observations shown in Table B.7 resulted.

Table B.7 Occupancy Observations

Minute	15 second occupancy				Average
	1st	2nd	3rd	4th	
1st	10	13	07	06	09
2nd	07	05	05	07	06
3rd	11	12	16	13	13
4th	09	13	12	07	10
5th	06	10	06	06	07
6th	08	12	10	06	09
7th	10	17	19	18	16
8th	14	15	15	15	15
9th	17	13	14	16	15
10th	18	16	15	19	17
11th	18	15	17	18	17
12th	15	15	20	14	16
13th	18	22	21	19	20
14th	15	14	12	11	13
15th	16	22	23	23	21

Average = 13.6

Fifteen measurements for total travel time were extracted from the videotape, by tracing the vehicles from the beginning of the section to the end and using a stop watch. These measurements are shown in Table B.8.

Table B.8 Field Measurement for Total Travel Time (sec)

60	50	75
50	80	70
60	70	60
85	80	65
55	65	65

Average total travel time = 66 sec

Watching the videotape several times, the following data was extracted for each link.

■ **1st link (between Kent St. and Bank St.)**

15 minute traffic flow = 180 veh
 number of vehicles stopped = 127 veh
 number of heavy vehicles in traffic = 9 veh
 percent of vehicles stopped = 70.56%
 percent of heavy vehicles = 5%

■ **2nd link (between Bank St. - O'Connor St.)**

15 minute traffic flow = 150 veh
 number of vehicles stopped = 104 veh
 number of heavy vehicles = 7 veh
 percent of vehicles stopped = 69.33%
 percent of heavy vehicles = 4.67%

Average percent of vehicles stopped for the entire section = $(70.56\% + 69.33\%)/2 = 69.95\%$

Average percent of heavy vehicles for the entire section
 $= (5\% + 4.67\%)/2 = 4.84\%$

Number of transit buses per hour = 24 bus/hour

Length of 1st link = 623 ft

Length of 2nd link = 577 ft

number of signals per mile = 9

Applying equations (6.2) and (6.3), the following calculations are given.

$$ITT (1) = [(180 * 623)/44]/60 = 42.48 \text{ veh.min}$$

$$ITT (2) = [(150 * 577)/44]/60 = 32.78 \text{ veh.min}$$

$$\text{average travel speed} = (623 + 577)/66 = 18.18 \text{ fps}$$

$$TTT (1) = [(180 * 623)/18.18]/60 = 102.81 \text{ veh.min}$$

$$TTT(2) = [(150 * 577)/18.18]/60 = 79.35 \text{ veh.min}$$

$$\text{Total delay on link (1)} = 102.81 - 42.48 = 60.33 \text{ veh.min}$$

$$\text{Total delay on link (2)} = 79.35 - 32.78 = 46.57 \text{ veh.min}$$

$$\begin{aligned} \text{Total delay for the entire section} &= 60.33 + 46.57 \\ &= 106.90 \text{ veh.min} \end{aligned}$$

Since Queen St. is a two-way street without parking and transit buses are running on it, equation (4.8) should be applied.

$$\begin{aligned} \text{DEL} &= -122.6770 + 12.7943 \text{ OCCUP} + 0.0944 \text{ STOP.SIGPM} \\ &\quad - 8.7349 \text{ NLAN} + 0.7415 \text{ LT} + 0.0043 (\text{NBUS})^2 \\ &\quad + 0.0568 (\text{HV})^2 \end{aligned}$$

$$\begin{aligned} \text{DEL} &= -122.6770 + 12.7943 (13.6) + 0.0944 (69.95)(9) \\ &\quad - 8.7349 (2) + 0.7415(0) + 0.0043 (24)^2 \\ &\quad + 0.0568 (4.84)^2 \\ &= 97.10 \end{aligned}$$

Percent difference between field delay estimate and delay estimated by the model based on field delay = 9.1%

5. *Queen Street East-bound (8:45 - 9:00 a.m.)*

Pausing the videotape every fifteen seconds to count the street occupancy, Table B.9 resulted.

Table B.9 Occupancy Observations

Minute	1st	15 second occupancy 2nd	3rd	4th	Average
1st	20	19	17	16	18
2nd	24	30	33	25	28
3rd	23	21	16	12	18
4th	13	14	11	14	13
5th	13	10	09	12	11
6th	17	26	25	24	23
7th	22	19	17	14	18
8th	19	26	29	34	27
9th	32	29	26	21	27
10th	23	26	20	27	24
11th	25	22	20	25	23
12th	22	18	17	23	20
13th	21	25	19	23	22
14th	20	18	17	25	20
15th	24	22	20	26	23

Average occupancy = 21

Using a stop watch and tracing vehicles along the entire section, fifteen observations for total travel time were obtained. The observations are shown in Table B.10.

Table B.10 Field Measurement for Total Travel Time (sec)

130	98	125
70	100	105
85	115	115
145	70	100
130	112	105

Average travel time = 107 sec

Watching the videotape several times, the following data was extracted for each link.

■ **1st link (between Kent St.- Bank St.)**

15 minute traffic flow = 168 veh

number of vehicles stopped = 148 veh

number of heavy vehicles = 6 veh

percent of vehicles stopped = 88.1%

percent of heavy vehicles = 3.6%

■ **2nd link (between Bank St.- O'Connor St.)**

15 minute traffic volume = 144 veh

number of vehicles stopped = 113 veh

number of heavy vehicles = 4 veh

percent of vehicles stopped = 78.5 %

percent of heavy vehicles = 2.8 %

Average percent of vehicles stopped for the entire street = $(88.1\% + 78.5\%)/2 = 83.3\%$

Average percent of heavy vehicles = $(3.6\% + 2.8\%)/2 = 3.2\%$

Number of transit buses per hour = 24 bus

Length of 1st link = 623 ft

Length of 2nd link = 577 ft

Number of signals per mile = 9

Applying equations (6.2) and (6.3) to estimate field total delay, the following calculations are given.

$$ITT (1) = [(168 * 623)/44]/60 = 39.65 \text{ veh.min}$$

$$ITT (2) = [(144 * 577)/44]/60 = 31.47 \text{ veh.min}$$

$$\text{Average travel speed} = (623 + 577)/107 = 11.21 \text{ fps}$$

$$TTT (1) = [(168 * 623)/11.21]/60 = 155.61 \text{ veh.min}$$

$$TTT (2) = [(144 * 577)/11.21]/60 = 123.53 \text{ veh.min}$$

$$\text{Total delay on link (1)} = 155.61 - 39.65 = 115.96 \text{ veh.min}$$

$$\text{Total delay on link (2)} = 123.53 - 31.47 = 92.06 \text{ veh.min}$$

$$\text{Total delay for the entire section} = 115.96 + 92.06 = 208.02 \text{ veh.min}$$

Applying equation (4.8), the following calculations are stated.

$$\begin{aligned}
 \text{DEL} &= -122.6770 + 12.7940 (21) + 0.0944 (83.3)(9) \\
 &\quad - 8.7349 (2) + 0.0043 (24)^2 + 0.0568 (3.2)^2 \\
 &= 202.35 \text{ veh.min}
 \end{aligned}$$

Percent difference between field delay estimate and delay estimated using the model, based on the field delay = 2.8%

6. Queen Street East-bound (8:45 - 9:00)

Pausing the videotape every fifteen seconds, occupancy observations were obtained. These observations are shown in Table B.11.

Table B.11 Occupancy Observations

Minute	15 second occupancy				Average
	1st	2nd	3rd	4th	
1st	22	26	24	24	24.0
2nd	25	23	21	27	24.0
3rd	26	25	19	20	22.5
4th	23	22	26	21	23.0
5th	22	23	22	26	23.3
6th	25	21	18	22	21.5
7th	22	27	20	27	24.0
8th	25	29	30	23	26.7
9th	26	21	28	21	24.0
10th	25	24	19	27	23.7
11th	24	18	20	23	21.3
12th	22	21	18	20	20.3
13th	23	22	17	22	21.0
14th	20	21	22	15	19.5
15th	18	19	22	20	19.7

Average occupancy = 22.6

Using a stop watch and tracing vehicles along the entire section, fifteen observations for total travel time were obtained. These observations are shown in Table B.12.

Table B.12 Field Measurements for Total Travel Time (sec)

105	145	135
105	150	110
100	150	120
121	130	105
130	110	115

Average travel time = 122.1 sec

Watching the videotape several times, the following data was extracted for each link.

■ **1st link (between Kent St. - Bank St.)**

15 minute traffic flow = 186 veh
 number of vehicles stopped = 175 veh
 number of heavy vehicles = 7 veh
 percent of vehicles stopped = 94%
 percent of heavy vehicles = 3.7%

■ **2nd link (between Bank St.- O'Connor)**

15 minute traffic flow = 174 veh
 number of vehicles stopped = 119 veh
 number of heavy vehicles = 3 veh
 percent of vehicles stopped = 68%
 percent of heavy vehicles = 1.7%

Average percent of vehicles stopped for the entire section = $(94\% + 68\%)/2 = 81\%$

Average percent of heavy vehicles = $(3.7\% + 1.7\%)/2$
 = 2.7%

Number of transit buses per hour = 24 bus

Length of 1st link = 623 ft

Length of 2nd link = 577 ft

Number of signals per mile = 9

Applying equations (6.2) and (6.3) to estimate field total delay, the following calculations are given.

$$ITT (1) = [(186 * 623)/44]/60 = 43.89 \text{ veh.min}$$

$$ITT (2) = [(174 * 577)/44]/60 = 38.03 \text{ veh.min}$$

$$\text{Average travel speed} = (623 + 577)/122.1 = 9.83 \text{ fps}$$

$$TTT (1) = [(186 * 623)/9.83]/60 = 196.47 \text{ veh.min}$$

$$TTT (2) = [(174 * 577)/9.83]/60 = 170.22 \text{ veh.min}$$

$$\text{Total delay on link (1)} = 196.47 - 43.89 = 152.58 \text{ veh.min}$$

$$\text{Total delay on link (2)} = 170.22 - 38.03 = 132.19 \text{ veh.min}$$

$$\text{Total delay for the entire section} = 152.58 + 132.19 = 284.77 \text{ veh.min}$$

In order to calculate delay using the model, equation (4.10) was used. This equation is recommended for use in case of two-way streets with parking.

$$\begin{aligned} \text{DEL} = & -119.9469 + 12.0429 \text{ OCCUP} + 0.1373 \text{ STOP.SIGPM} \\ & - 7.2482 \text{ NLAN} \end{aligned}$$

$$\begin{aligned} \text{DEL} = & -119.9469 + 12.0429 (22.6) + 0.1373 (81)(9) \\ & - 7.2482 (1) = 245.07 \text{ veh.min} \end{aligned}$$

Percent difference between field delay estimate and delay estimated using the model, based on field delay = 13.9%

7. Kent Street North-bound (4:00 - 4:15)

Pausing the videotape every fifteen second, occupancy observations were obtained. These observations are shown in Table B.13.

Table B.13 Occupancy Observations

Minute	1st	15 second occupancy 2nd	3rd	4th	Average
1st	16	19	13	20	17
2nd	15	14	14	17	15
3rd	13	10	09	12	11
4th	15	16	16	17	16
5th	14	13	10	11	12
6th	19	18	23	16	19
7th	25	32	27	28	28
8th	32	36	33	31	33
9th	36	36	32	36	35
10th	34	37	31	34	34
11th	33	31	37	39	35
12th	37	39	42	34	38
13th	35	40	33	40	37
14th	39	41	35	37	38
15th	36	33	31	36	34

Average occupancy = 27

Fifteen estimates for total travel time were obtained by tracing vehicles along the entire section and using a stop watch. These observations are shown in Table B.14

Table B.14 Field Measurements for Total Travel Time (sec)

40	53	75
50	60	120
45	80	135
55	110	135
40	75	105

Average travel time = 78.55 sec

Watching the videotape several times, the following data was extracted for each link.

■ **1st link (between Laurier Avenue - Slater St.)**

15 minute traffic flow = 289 veh

number of vehicles stopped = 194 veh

number of heavy vehicles = 16 veh

number of vehicles turning left = 0

percent of vehicles stopped = 67%

percent of heavy vehicles = 5.5%

■ 2nd link (between Slater St. - Albert St.)

15 minute traffic flow = 301 veh

number of vehicles stopped = 226 veh

number of heavy vehicles = 9 veh

number of vehicles turning left = 37

percent of vehicles stopped = 75%

percent of heavy vehicles = 3%

percent of vehicles turning left = 12.2%

Average percent of vehicles stopped for the entire section = $(67\% + 75\%) / 2 = 71\%$

Average percent of vehicles turning left = $(0 + 12.2\%) / 2$
 $= 6.1\%$

Length of 1st link = 328 ft

Length of 2nd link = 270 ft

Number of signals per mile = 18

Applying equations (6.2) and (6.3) to estimate field total delay, the following calculations are given

$$ITT (1) = [(289 * 328) / 44] / 60 = 35.91 \text{ veh.min}$$

$$ITT (2) = [(301 * 270) / 44] / 60 = 30.78 \text{ veh.min}$$

$$\text{Average travel speed} = (328 + 270) / 78.55 = 7.61 \text{ fps}$$

$$TTT (1) = [(289 * 328) / 7.61] / 60 = 207.60 \text{ veh.min}$$

$$TTT (2) = [(301 * 270) / 7.61] / 60 = 177.99 \text{ veh.min}$$

$$\text{Total delay on link (1)} = 207.60 - 35.91 = 171.69 \text{ veh.min}$$

$$\text{Total delay on link (2)} = 177.99 - 30.78 = 147.21 \text{ veh.min}$$

$$\text{Total delay for the entire section} = 171.69 + 147.21 = 318.9 \text{ veh.min}$$

Since equation (4.22) is recommended for use in case of one-way streets with

parking, it was used to estimate delay for Kent Street.

$$\text{DEL} = -226.4519 + 14.6840 \text{ OCCUP} + 0.1589 \text{ STOP.SIGPM}$$

$$- 20.2233 \text{ NLAN} + 2.2310 \text{ LT}$$

$$\text{DEL} = -226.4519 + 14.6840 (27) + 0.1589 (71)(18)$$

$$- 20.2233 (3) + 2.2310 (6.1)$$

$$= 326.03 \text{ veh.min}$$

Difference between field delay estimate and delay calculated by the model as a percentage of field delay

$$= 2.2\%$$

8. Kent Street North-bound (4:15 - 4:30)

The videotape was paused every fifteen seconds. Occupancy observations shown in Table B.15 were obtained.

Table B.15 Occupancy Observations

Minute	15 second occupancy				Average
	1st	2nd	3rd	4th	
1st	34	28	39	39	35.0
2nd	36	27	31	36	32.5
3rd	35	34	37	32	34.5
4th	36	37	31	35	34.8
5th	32	31	35	31	32.3
6th	29	28	22	32	27.8
7th	35	36	31	26	32.0
8th	35	31	36	33	33.8
9th	34	35	29	27	31.3
10th	25	29	27	26	26.8
11th	30	28	27	29	28.5
12th	31	31	24	27	28.2
13th	26	24	23	18	22.8
14th	22	21	16	19	19.5
15th	20	20	17	17	18.5

Average occupancy = 29.22

The field total travel time measurements are shown in Table B.16.

Table B.16 Field Measurement for Total Travel Time (sec)

85	65	55
105	100	65
130	85	115
110	85	100
60	64	70

Average travel time = 86.26 sec

The data extracted from the videotape for each link is:

■ **1st link (between Laurier Avnu. - Slater St.)**

15 minute traffic flow = 319 veh

number of vehicles stopped = 259 veh

number of vehicles turning left = 0

percent of vehicles stopped = 81.2%

percent of vehicles turning left = 0

■ **2nd link (between Slater St.- Albert St.)**

15 minute traffic flow = 327 veh

number of vehicles stopped = 283 veh

number of vehicles turning left = 33 veh

percent of stopped vehicles = 86.6%

percent of vehicles turning left = 10%

Average percent of vehicles stopped for the entire section = $(81.2\% + 86.6\%)/2$
= 83.9%

Average percent of vehicles turning left = $(0 + 10\%)/2 = 5\%$

Length of 1st link = 328 ft

Length of 2nd link = 270 ft

Number of signals per mile = 18

Applying equations (6.2) and (6.3) to estimate field total delay, the following calculations are stated.

$$ITT (1) = [(319 * 328)/44]/60 = 39.63 \text{ veh.min}$$

$$ITT (2) = [(327 * 270)/44]/60 = 33.44 \text{ veh.min}$$

$$\text{Average travel speed} = (328 + 270)/86.26 = 6.93 \text{ fps}$$

$$TTT (1) = [(319 * 328)/6.93]/60 = 251.64 \text{ veh.min}$$

$$TTT (2) = [(327 * 270)/6.93]/60 = 212.34 \text{ veh.min}$$

$$\text{Total delay on link (1)} = 251.64 - 39.63 = 212.01 \text{ veh.min}$$

$$\text{Total delay on link (2)} = 212.34 - 33.44 = 178.90 \text{ veh.min}$$

$$\begin{aligned} \text{Total delay for the entire section} &= 212.01 + 178.90 = \\ &390.91 \text{ veh.min} \end{aligned}$$

Applying equation (4.22), the following calculations are given.

$$\begin{aligned} \text{DEL} &= -226.4519 + 14.6840 (29.22) + 0.1589 (83.9)(18) \\ &\quad - 20.2233 (3) + 2.2310 (5) \\ &= 393.07 \text{ veh.min} \end{aligned}$$

Difference between field delay estimate and delay calculated using the model as a percentage of field delay = 1%

END

1 2 0 4 9 1

FIN