USE OF MICROSCOPIC TRAFFIC SIMULATION AND FIELD DATA TO INVESTIGATE SATURATED AND FREE FLOW TRAFFIC CONDITIONS AT ARTERIAL SIGNALS

by

SARAVANAN GURUPACKIAM

A DISSERTATION

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ABSTRACT

Over the years microscopic traffic simulation has evolved as the premier tool to analyze complex and congested transportation networks. However, despite the robustness and wide spread use of traffic microsimulation, some gaps and limitations still exist that can affect the accuracy of these models' results. Moreover the change of traffic characteristics and driver behavior during the transition from an undersaturated to a saturated condition is not completely understood. This dissertation addresses these two issues.

The road network chosen for the microscopic simulation and field data collection is a six lane main traffic artery located in Tuscaloosa, Alabama, USA. The entire research work contains three related research efforts, each conducted along the topic of this dissertation. The first research thrust focused on the sensitivity and accuracy of microscopic traffic simulation. Specifically it investigated the sensitivity of measures of effectiveness (MOEs) to simulation initialization time, required number of repetitions, and major contributors of variation in MOEs. The second research thrust dealt with field investigation of operational parameters including gap acceptance and lane changing during different levels of traffic flow. The final research effort explored the variations in simulation results using existing embedded/default values of lane change parameters (lane change duration and look ahead distance), versus using values obtained from field observation for both free flow and saturated traffic conditions. From all the research efforts, the following broad conclusions were drawn,

- Traffic flows at signals that are approaching saturation are still complex to analyze, and the interactions between traffic parameter are not well understood.
- When traffic flow on a typical arterial approaches saturation, drivers take higher risks (eg: drivers accept smaller gaps).
- A statistical analysis of gap acceptance and lane changing confirmed what is suspected intuitively.
- Existing traffic microsimulation tools simplify some of the traffic parameters in simulation models. These parameters may be recoded or recalibrated for better accuracy of simulation results.
- In traffic microsimulation an increased number of simulation runs certainly helps in stabilizing the variability of the MOE and it is advisable to use a longer simulation time (eg. 60 minutes) to reduce the variation of MOEs.

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
Accel	Acceleration
Adj	Adjustment
AIMSUN	Advanced Interactive Microscopic Simulator for Urban and Non-urban Networks
BTS	Bureau of Transportation Statistics
CCTV	Closed Circuit Television
CORSIM	CORridor-microscopic SIMulation program
Decel	Deceleration
Diff	Difference
Dist	Distance
EB	East Bound
FHWA	Federal Highway Authority
H_O	Null Hypothesis
H_A	Alternative Hypothesis
HCM	Highway Capacity Manual
HCS	Highway Capacity Software
hrs	Hours
ITS	Intelligent Transportation Systems

LC	Lane Change
LCD	Lane Change Duration
LOS	Level of Service
min	Minute
MOE	Measures of Effectiveness
mph	Miles Per Hour
ms	Millisecond
MUTCD	Manual on Uniform Traffic Control Devices
NB	North Bound
NCHRP	National Cooperative Highway Research Program
NGSIM	Next Generation Simulation
s	Sample Standard Deviation
SB	South Bound
sec	Seconds
STDEV	Standard Deviation
T	Time
TDOT	Tuscaloosa Department of Transportation
TMC	Traffic Management Center
TRB	Transportation Research Board
UA	University of Alabama
UAB	University of Alabama at Birmingham
UAH	University of Alabama at Huntsville
UMall	University Mall

UTCA	University Transportation Center for Alabama
VHT	Vehicle Hours Traveled
v-m	Vehicle Minutes
VMT	Vehicle Miles Traveled
vph	Vehicles per Hour
WB	West Bound

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CHAPTER 1

INTRODUCTION

1.1 Objective

The main objective of this research is to investigate the changes in operational parameters and performance measures between saturated and free flow traffic conditions using microscopic traffic simulation and field data.

1.2 Background - Congestion

Congestion is a familiar term for almost everyone who rides a vehicle. It refers to the condition on a transportation facility such as road that occurs when the traffic demand exceeds the capacity of the facility. Congestion often means stopped or stop-and-go traffic. It is usually characterized by reduced speeds, increased queuing, and longer travel time. This is a problem of great magnitude and is prevalent in most urban areas throughout the world. In addition to the delay in personal and commercial trips, traffic congestion also causes waste of fuel and excessive emissions of particulate matter and greenhouse gases.

In America, demand for highway travel continues to grow as population increases, particularly in metropolitan areas. This can be attributed to increased car ownership and vehicle miles traveled. Construction of new highway capacity to accommodate this growth in travel has not kept pace. The increase in route miles of highways is non-commensurate with the increase in vehicle miles traveled. As per the Bureau of Transportation Statistics (BTS) the number of registered vehicles over the past decade has increased by 19.2 % and the vehicle miles traveled

has increased by 21.3 %; whereas the highway miles have increased only by 2.5 % during the same period. Consequently, urban areas are experiencing increased vehicle-hours of delay, resulting in very heavy congestion cost.

The Urban Mobility Report 2007 by the Texas Transportation Institute indicates that about 437 urban areas in America are experiencing problems due to congestion and it is getting worse in regions of all sizes. The report calculated congestion cost by estimating values for travel time delay and excess fuel consumption. It indicates that congestion caused urban Americans to travel 4.2 billion hours more and to purchase an extra 2.9 billion gallons of fuel for a congestion cost of $78 billion. Figure 1.1 shows the increase in travel delay and congestion cost over the past three decades. During this span, it can be observed that the congestion cost has increased nearly 5 times over.

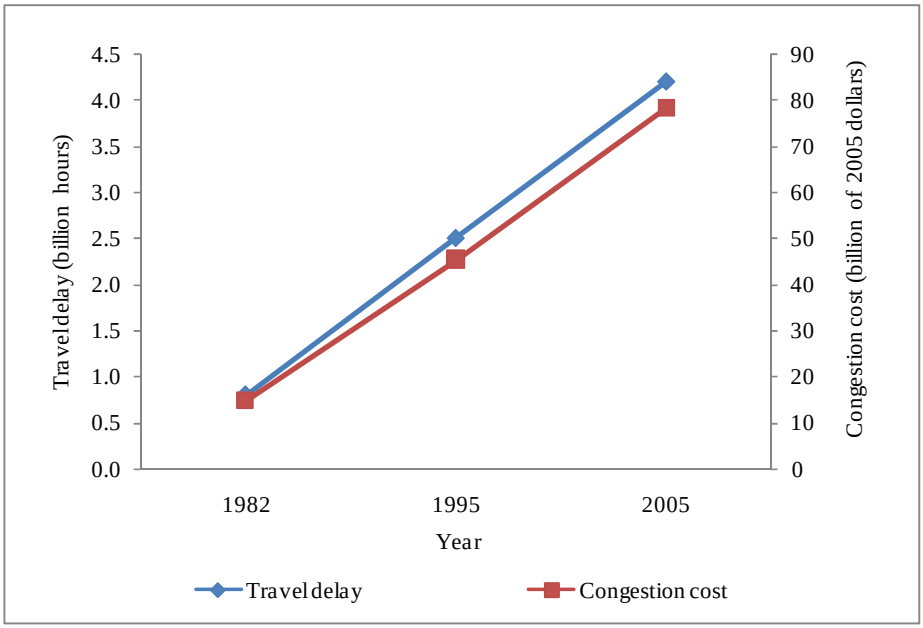

Figure1.1 Trend of Travel Delay and Congestion Cost in America (Source: The 2007 Urban Mobility Report, Texas Transportation Institute, September 2007)

1.3 Traffic Signals

A traffic signal is a traffic control device used for roadway operations, particularly at intersections. The Manual on Uniform Traffic Control Devices (MUTCD) specifies a series of warrants for installing traffic signals. These warrants are mainly based on the traffic flow on both streets of an intersection. The main function of a traffic signal is to allow shared use of road space by separating conflicting movements in time. A typical four legged intersection has 32 conflicting points as shown in Figure 1.2.

As traffic volumes rise, and time gaps in the traffic streams decrease, drivers become impatient and take greater risks. With 32 conflict points in an intersection. delays, congestion and crashes increase rapidly. Installation of a traffic signal can isolate conflicting flows and improve both safety and operations. But as flows approaches the absolute capacity of the intersection, traffic signal control begins to break down and it no longer efficient.

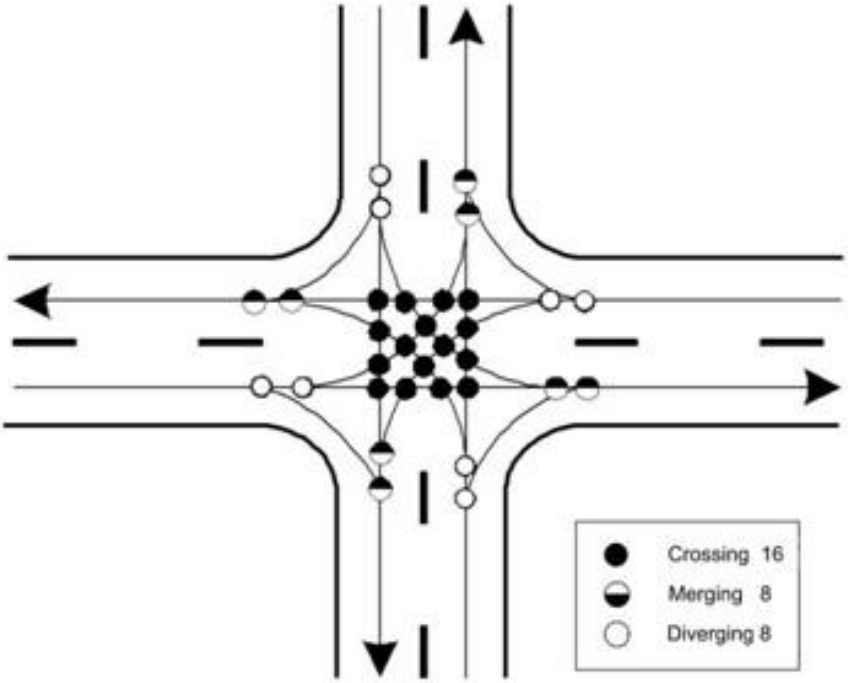

Figure 1.2 Conflicting Points in a Typical Four Legged Intersection (Source: FHWA)

Traffic signals are a common feature in urban street networks which contain many intersections and each experience considerable traffic flow. Signal timings are determined depending upon the volume of traffic flow across the streets and their classification. For instance, traffic along the major roadway receives relatively more time for traffic flow when compared to minor roads. An efficient traffic signal will be designed for maximum throughput and minimum delay. Hence it is imperative that signals have an appropriate timing plan to optimize their performance. But during heavily congested traffic conditions even the best performing traffic signals fail.

1.4 Measures of Effectiveness (MOEs)

In general Measures of Effectiveness (MOEs) refer to quantitative measures that give some insight into how effectively a unit is performing. In the field of transportation engineering the primary measures are speed, travel time, delay, etc. They are used to evaluate the performance of transportation facilities such as signalized intersections, roundabouts, urban streets, and freeways. They are also used to qualitatively assess a transportation facility. For instance level of service (LOS) is the qualitative measure used to describe signalized intersection performance. In this instance the MOE is delay. Table 1.1 shows different categories of LOS based on control delay, which is the component of delay that results from the type of control at the intersection. LOS A is usually referred to as free flow condition and LOS F is referred to as saturated condition.

Table 1.1 LOSs of Intersections based on Delay

Control delay/vehicle (sec)	LOS
≤ 10	A
> 10 and ≤ 20	B
> 20 and ≤ 35	C
> 35 and ≤ 55	D
> 55 and ≤ 80	E
> 80	F

1.5 Why Flow Breaks Down

Traffic demands vary throughout the day and congestion frequently occurs during the morning and afternoon peak periods due to the large number of people commuting to or from work. During the peak hours, traffic demand at a signalized intersection increases and often exceeds the capacity of the intersection. Capacity is exceeded when traffic volumes increase to the point that the green signal indication is insufficient to clear the vehicles waiting to pass through the intersection, and queues develop. When these conditions continue, the intersection is said to be oversaturated. During oversaturated conditions the queues developed at a downstream intersection may grow to such extended lengths that they may “spill back” to the upstream intersection. Consequently the flow from the upstream intersection is obstructed. This is especially the case when the intersections are closely spaced. The worst scenario is a grid-lock when the traffic flow is jammed from all legs of an intersection.

There are several types of cases, when the through movement of traffic at a signalized intersection is blocked by extended queues of vehicles. For example, when a left turn bay over fills and blocks through traffic or through traffic over fills and blocks entrance into the left turn bay. All of these cases are examples of when a free flow of traffic is obstructed at a traffic signal. It is important to improve traffic signal operation to decrease congestion, but we are still struggling to understand the change in traffic characteristics from free flow to saturated traffic condition.

1.6 Possible Routes to Solution

There are tools available that address different aspects of traffic signal operation as flow approaches saturation, but there is no single tool that adequately addresses the entire situation.

Several of the most prominent tools are discussed in the next few pages of this dissertation to set the stage for this dissertation research project.

1.6.1 Highway Capacity Manual (HCM)

The Highway Capacity Manual (HCM), a publication of the Transportation Research Board (TRB) of the National Academies provides transportation practitioners and researchers with a consistent system of techniques for the evaluation of the quality of service on highway and street facilities. The manual contains research findings on capacity and quality of service, and presents methods for analyzing the operations of streets and highway facilities. While the HCM is widely used throughout the United States for evaluating transportation facilities there are certain limitations to the methodologies adopted in the manual particularly when the traffic flow breaks down as discussed earlier in this chapter. For instance the methodology to analyze a signalized intersection does not take into account the potential impact of downstream congestion on intersection operation and the impacts of turn-pocket overflows on through traffic and intersection operation. In order to overcome these limitations of the HCM during saturated conditions, transportation practitioners often resort to alternative traffic analysis tools such as traffic simulation.

1.6.2 Traffic Simulation

Simulation can be defined as a representation of a part of the real world using a computer model. Traffic simulation involves simulating a traffic flow in a transportation network using computer models that govern the movement of vehicles over a certain period of time. When the behavior of each vehicle in a simulated traffic stream including acceleration, deceleration, lane changing, gap acceptance, and turning movement is taken into consideration the simulation is referred to as microscopic traffic simulation. Thus with a microscopic traffic simulation the

limitations of HCM such as impacts of downstream congestion and turn pocket overflows can be overcome. However, despite the widespread use of traffic simulation and its advantages, there are still conflicting thoughts and practices on how simulation should be used, and research is underway for better use of simulation. For instance when the traffic volume on a particular roadway reaches capacity (i.e., the volume to capacity ratio equals one) there is no progression of traffic and the simulation model breaks down resulting in unrealistic MOEs. Hence it is important to understand the change in traffic characteristics when traffic approaches saturated conditions which is one of the research objectives in this dissertation research.

1.7 Prior and Ongoing Applicable Research

As discussed earlier even though microscopic traffic simulation has its own advantages when compared to the HCM, research on better use and accuracy of simulation models (particularly for saturated traffic conditions) has been going on for many years. The Federal Highway Administration (FHWA) developed the Traffic Analysis Toolbox as a guide, primarily to assist traffic engineers, planners, and traffic operations professionals in the selection of the correct type of traffic analysis tool including microscopic traffic simulation for operational improvements. The Toolbox includes a separate chapter with guidelines for applying traffic microsimulation modeling software. Apart from the Toolbox, many other studies have been done by researchers focusing on different aspects of traffic microsimulation such as data requirements, calibration, validation, etc.

One FHWA product relating to this dissertation research is the Next Generation Simulation model (NGSIM). It is actually a public-private partnership between FHWA and commercial microsimulation software developers, the academic research community, and the traffic microsimulation community. To help achieve wider acceptance of the use of

microsimulation systems and to ensure that the tools provide accurate results, FHWA's Traffic Analysis Tools Program launched the Next Generation Simulation (NGSIM) program. One of the main objectives of NGSIM is to develop a core of open behavioral algorithms in support of traffic simulation with a primary focus on microscopic modeling. It includes supporting documentation and validation data sets that describe the interactions of multi-modal travelers, vehicles and highway systems, and interactions presented to them from traffic control devices, delineation, congestion and other features of the environment.

The National Cooperative Highway Research Program (NCHRP) is a major vehicle for highway research. It is administered by the American Association of State Highway and Transportation Officials (AASHTO) and TRB, in cooperation with FHWA. It was created in 1962 as a means to conduct research in acute problem that affect highway planning, design, construction, operation, and maintenance nationwide. NCHRP project 3-85 is pertinent to this project. It is intended to enhance the guidance in the HCM for selection and use of alternative traffic analysis tools for a wide range of transportation facilities such as intersections, arterials, highways, freeways, etc. This project is currently in progress. The major tasks of this project include reviewing the current state of practice, identifying the limitations of the HCM, and performing comparisons of alternative tools such as traffic simulation. This project will make recommendations for many topics including improving traffic signal operations on saturated arterials (the topic of this dissertation).

Research on driver behavior parameters such as gap acceptance and lane changing is attracting increased attention from researchers. Gap acceptance and lane changing depend on driver parameters like aggressiveness, urgency, and impatience [Goswami and Bham, 2007]. Drivers behave differently under diverse traffic, geometric, and environmental conditions.

Similarly, the same driver can behave differently under varying traffic and control conditions [Goswami and Bham, 2007]. In a microscopic traffic simulation each vehicle is modeled, so the study of driver behavior has gained importance. Currently the main focus of research in driver behavior with respect to traffic microsimulation is to find the distribution of gap acceptance and lane changing. Gap acceptance and lane changing models are integral parts of a traffic microsimulation model.

1.8 Need for This Study

Despite the robustness and wide spread use of traffic microsimulation models, some gaps and limitations still exist that can affect the accuracy of the results produced by the models. For instance the effects of initialization times, required number of repetitions, and variability between MOEs are not completely understood during saturated traffic conditions. The existing studies do not clearly present the sensitivity of simulation results to different initialization times under varying traffic conditions. Similarly, the required number of repetitions and the variability between MOEs during saturated traffic conditions are not clearly documented in previous research. This study specifically addresses these issues in microscopic traffic simulation.

Another major reason for this study is to enhance the understanding and modeling of driver behavior parameters such as lane changing and gap acceptance under different traffic conditions. The existing traffic microsimulation models contain some potential weaknesses that are identified in this study, and suitable recommendations are proposed to improve the performance of the models. For instance most of the microscopic traffic simulation models neglect the detailed modeling of the lane changing action itself and model it as an instantaneous event (a black box approach). This study addressed this issue by determining the range and distribution of lane change durations with respect to different levels of congestion. Similarly a

gap acceptance distribution is also obtained and proposed for addition to existing microscopic simulation models while simulating arterials with congested traffic conditions.

1.9 University Transportation Center for Alabama (UTCA)

The University Transportation Center for Alabama conducts transportation education, research and technology transfer activities using faculty members and students from The University of Alabama, The University of Alabama at Birmingham and the University of Alabama in Huntsville (UA, UAB and UAH). The research described in this dissertation was conducted as part of UTCA project 07112 “Characterization of Arterial Traffic Congestion”. The objective of the UTCA project was to investigate the changes between undersaturated and saturated traffic conditions at arterial signals in terms of performance measures (delay) and operational parameters (gap acceptance and lane changing). The project directly supported the UTCA theme of management of traffic flow and mitigation of congestion. The proposal also complements NCHRP project 3-85.

1.10 Deliverables

This dissertation research has shed new light on the understanding of saturated versus free flow traffic conditions using microscopic traffic simulation as well as field data. The entire research work contains three related research efforts, each conducted along the topic of this dissertation. Each research step provided information to improve knowledge of traffic characteristics at busy traffic signals, or otherwise contributed to enhancing accuracy of microscopic traffic simulation under saturated conditions. Each research step was documented through an article written in journal format, and consequently submitted for peer review and possible publication.

1.10.1 First Research Step

The first research thrust in this project resulted in an article titled “Rethinking Initialization Times, Number of Repetitions, and Variability in MOEs when Simulating Oversaturated Traffic Conditions”. It focuses on the accuracy of simulation models used to study traffic characteristics during the transition from free flow to saturated flow at arterial traffic signals. The objectives of this article are:

- To investigate the sensitivity of key traffic parameters to simulation initialization time.
- To study the possible relationship between levels of congestion and the number of simulation runs needed to obtain statistically optimum MOE metrics.
- To identify the source of variation in MOEs between simulation runs with different random seeds, and the relationship of the variation to different levels of congestion.

1.10.2 Second Research Step

The second research thrust in this dissertation resulted in an article on “Observed Variations in Gap Acceptance and Lane Changing in Relation to Traffic Flow Rates in Urban Streets”. This article deals with field investigation of operational parameters such as gap acceptance and lane changing during different levels of traffic flow. Lane changing refers to drivers changing roadway lanes without interfering with vehicles in the destination lane. Gap acceptance is the process by which drivers accept gaps (in seconds) in a traffic stream when changing lanes. The major tasks in this article were to collect gap acceptance and lane changing data on a study road, to obtain descriptive statistics for the data and to fit statistical distributions for them. This study made use of the facilities in the UTCA Intelligent Transportation Systems/Traffic Management Center (ITS/TMC) lab which was developed as a joint venture between the University of Alabama and the Tuscaloosa Department of Transportation (TDOT).

This lab has live feed on traffic flows from 40 closed circuit television (CCTV) cameras throughout the city. The video recordings from one of the CCTV cameras along the study road were used to collect the lane change and gap acceptance data needed for this article.

1.10.3 Third Research Step

The third and final research effort resulted in an article called “Sensitivity Analysis of Lane Change Parameters to Saturated and Free Flow Traffic Conditions in Microscopic Traffic Simulation Models”. This article is an extension of the second article. It explores the variations in simulation results using existing embedded/default values of lane change parameters, versus using parameter values obtained from field observation for both free flow and saturated traffic conditions. An experimental design was created to perform simulations on the study road so that it covered the range of lane change parameters and different levels of congestion (saturated and free flow).

1.11 Organization of the Dissertation

This dissertation is organized into five chapters. After chapter I (Introduction), the next three chapters include the three articles discussed in the last few paragraphs. The final chapter contains summary of conclusions and recommendations based on the overall research program and the specific research efforts contained in the three articles.

CHAPTER 2

RETHINKING INITIALIZATION TIMES, REPETITIONS, AND MOE VARIABILITY IN SIMULATION OF SATURATED TRAFFIC SIGNALS

2.1 Abstract

This study is an attempt to understand the effects of initialization time and number of repetitions on the accuracy of simulation results. The road network chosen for the entire study was a six lane main traffic artery located in Tuscaloosa, Alabama, USA. SimTraffic was used to simulate typical weekday peak hour traffic conditions. This study consisted of a series of investigation steps. The first step investigated the sensitivity of MOEs to simulation initialization time. Accordingly, simulation runs were performed for initialization times of 7, 10 and 13 minutes. For each initialization time, 50 random seeds were used to compute MOEs and then a Student's t-Test (two sample assuming unequal variance) and hypothesis testing were performed to compare MOEs obtained for different initialization times for the intersections and for each approach in the intersections considered in the study. In order to investigate the required number of repetitions, the trend of standard deviation of MOEs with increasing simulation runs was studied along with the LOS of the approaches. The final step to identify the major contributors of variation in MOEs was achieved by performing simulation runs with one of the three parameters (lane changing, gap acceptance, and car following) held constant while the other two were allowed to vary. It was expected that this procedure would help to isolate the source of variability and the effects of congestion.

The results showed that the MOEs of a simulated saturated intersection are indeed sensitive to initialization times of simulation. It was also found that with increased number of simulation runs the variation within MOEs reached a steady state and that the required number of simulation runs to attain this state changed with respect to the level of congestion. The length of simulation time makes a considerable difference in the simulation results. Lane change parameters play a major role as a source of variation of MOEs (delay/vehicle) obtained for different random seeds, but this is not conclusive for congested conditions (LOSs E and F).

2.2 Introduction

Traffic congestion is a common concern while considering urban streets. During peak hours, traffic demand at a signalized intersection increases and often exceeds the capacity of the intersection. Capacity is exceeded when traffic volumes increase to the point that the green signal indication is insufficient to clear the vehicles waiting to pass through the intersection, and the intersection is said to be saturated. Despite considerable efforts among both researchers and practitioners, improving traffic operations in saturated conditions remains a challenge.

This study is a part of on-going research project 07112 by the University Transportation Center of Alabama (UTCA), “Characterization of Arterial Traffic Congestion”. The broad objective of this research is to investigate the changes between undersaturated and saturated traffic conditions in terms of performance measures (delay) and operational parameters (gap acceptance and lane changing). This paper contributes to the research by exploring the sensitivity of performance measures of undersaturated and saturated traffic conditions to simulation initialization time (seeding time).

Traffic simulation has been widely used as a traffic analysis tool in transportation analyses. The advantages of using simulation are many but foremost is its ability to analyze the

operation of complex and congested transportation systems when other analytical techniques fail. However, despite the widespread use of traffic simulation software, there are conflicting thoughts and practices on how simulation should be used, and research is underway to better understand how simulation should be used. It is expected that this paper will also contribute towards better use of simulation and will help traffic engineers and practitioners understand how initialization time plays an important role in simulating undersaturated and saturated traffic conditions.

2.3 Background

The Traffic Analysis Toolbox indicates that simulation initialization time is the time needed for vehicles to fill an entire network and reach an equilibrium state. The number of vehicles present at any time on the network is used to determine whether the model has reached equilibrium, allowing the tallying of network performance statistics to begin. Once the number of vehicles present on the network ceases to increase by a minimum specified amount, then the warm-up period is deemed to have been concluded [Traffic Analysis Toolbox, Volume III, 2004]. For congested networks, the analyst chooses a warm-up period that is equal to at least twice the estimated travel time at free-flow conditions to traverse the length of the network [Traffic Analysis Toolbox, Volume III, 2004].

In practice, the recommendations of the Traffic Analysis Toolbox are used in many studies. There are also instances where a default value of initialization time embedded in the software is used without checking whether the network has actually attained equilibrium (e.g., SimTraffic uses a default value of 3 minutes). Other practitioners may use a conservative rule of thumb, for instance a 10 minute seeding time for a 2 mile network.

The recommendations of two commonly used simulation models are discussed below.

SimTraffic [User Guide, Version 6.0, 2003], recommends that the seeding time should be long enough for a vehicle to traverse the entire network between the two most distant points including all stops. The seeding time should also be longer than the longest cycle length at any intersection in the network. After the seeding time, the number of vehicles entering the network per minute should be about the same as the number of vehicles exiting the network per minute. The vehicle counts in the status window during seeding and recording will show how many vehicles are entering and exiting the network.

CORSIM [User's Guide, Version 6.0, 2005] gives the user two options:

- The user sets the initialization time.
- CORSIM monitors the run, and it will end initialization time when it senses that equilibrium has been reached.

When choosing the initialization period, most of the previous studies have followed the recommendations of the Traffic Analysis Toolbox. However, no studies have been identified that showed a comparison of performance measures obtained for different initialization times.

The literature outlines several methods for determining the minimum number of simulation runs. The Traffic Analysis Toolbox provides the following equation to determine that number,

$$CI_{1-\alpha\%} = 2 * t_{(1-\alpha/2),N-1} \frac{s}{\sqrt{N}} \quad \text{.....} \quad (2.1)$$

where,

$CI_{1-\alpha\%}$ = confidence interval for the true mean

where α = the probability of the true mean not lying within the confidence interval;

$t_{(1-\alpha/2),N-1}$ = Student's t-statistic for the probability of a two-sided error summing to alpha with N-1 degrees of freedom;

N = the number of repetitions; and

s = standard deviation of the model results (initially ‘s’ is directly estimated based on past experience or after executing a few model run repetitions).

When solving this equation for N, it is necessary to iterate until the estimated number of repetitions matches the number of repetitions assumed when looking up the t statistic. For example, if the standard deviation in the delay is 1.5 sec and the desired confidence interval is 3.0 sec at a 95-percent confidence level, then it will take eight repetitions to estimate the mean delay to within ± 1.5 sec.

A study done by Chu et al (2003) used the following equation to determine the required number of simulation runs,

$$N = (t_{\alpha/2} \cdot \frac{\delta}{\mu * \varepsilon})^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(2.2)}$$

where,

μ and δ are the mean and standard deviation of the performance measure based on the already conducted simulation runs;

ε is the allowable error specified as a fraction of the mean μ;

$t_{\alpha/2}$ is the critical value of the t-distribution at the confidence interval of 1- α.

Assuming the following for a sample calculation:

- Mean of the performance measure for initial four runs, $\mu = 30$
- Standard deviation of the performance measure, $\delta = 1.5$
- Allowable error, $\varepsilon = 5\%$
- Desired confidence level, $\alpha = 95\%$

From the student t-Table, $t_{0.975} = 3.182$,

Substituting the above values in the equation for N, we get the number of repetitions, N = 11 runs.

The authors also specified that all performance measures of interest must be involved in this calculation, and the highest value is the required number of runs.

The following equation was used by Tian et al (2002) to arrive at the required number of simulation runs,

$$n = \left(\frac{z_{\alpha/2} s}{E} \right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.3)$$

where,

n = required number of simulation runs;

s = sample standard deviation (based on 30 runs from this study);

$z_{\alpha/2}$ = the threshold value for a 100(1-a) percentile confidence interval. With a 95% confidence interval, $z_{\alpha/2} = 1.96$;

E = the allowed error range.

Assuming the following values for a sample calculation:

- Sample standard deviation (based on 30 runs), s = 1
- Allowed error range = 5%
- Desired confidence level = 95%

We have $z_{0.975} = 1.96$,

Substituting the values in the equation for n, the required number of simulation runs, n = 59 runs.

The authors also compared how the required numbers of runs change for under-capacity and over-capacity conditions. Based on the variations between MOEs, they suggested that 2 or 5 runs may be sufficient for under-capacity conditions and at least 40 runs may be necessary for

over-capacity conditions depending on the microscopic model used. However, the authors mentioned that those recommendations were based on the simplest case tested in their study. The study cases consisted of a single-lane approach with 100% through traffic and a single-lane approach with a right-turn pocket.

2.4 Objectives

There are three objectives to this study, each designed to ensure the accuracy of simulation models used for study of traffic characteristics during the transition from undersaturated to saturated flow at traffic signals:

(1) To investigate the sensitivity of traffic parameters to simulation initialization time.

(2) To study the possible relationship between levels of congestion and the number of simulation runs needed to produce statistically optimum MOE metrics.

(3) To identify the source of variation in MOEs between simulation runs with different random seeds and the relationship between variation in MOEs and different levels of congestion.

2.5 Simulation Model

Synchro/SimTraffic 6.0 was used for this study. Synchro is a complete software package for modeling and optimizing traffic signal timings. SimTraffic is versatile, easy-to-use traffic simulation software. The program makes traffic simulation and animation easy and fast enough for everyone to use, and it includes vehicle and driver performance characteristics developed by the Federal Highway Administration for use in traffic modeling. A Synchro network file of the study corridor with signal timing, phasing data, and road geometrics was readily available from a signal timing study done by Sain Associates, a consulting engineering firm, in 2006.

2.6 Data

The data used in this study include the PM peak hour volume data (turning movements), signal timing, phasing data, and road geometrics of McFarland Boulevard (US-82) in Tuscaloosa, Alabama. The study stretch extended from Skyland Boulevard to 13th Street along McFarland Boulevard. There were nine intersections, including three major and six minor, all of which were controlled by actuated and coordinated signals. Figure 2.1 shows the simulation network.

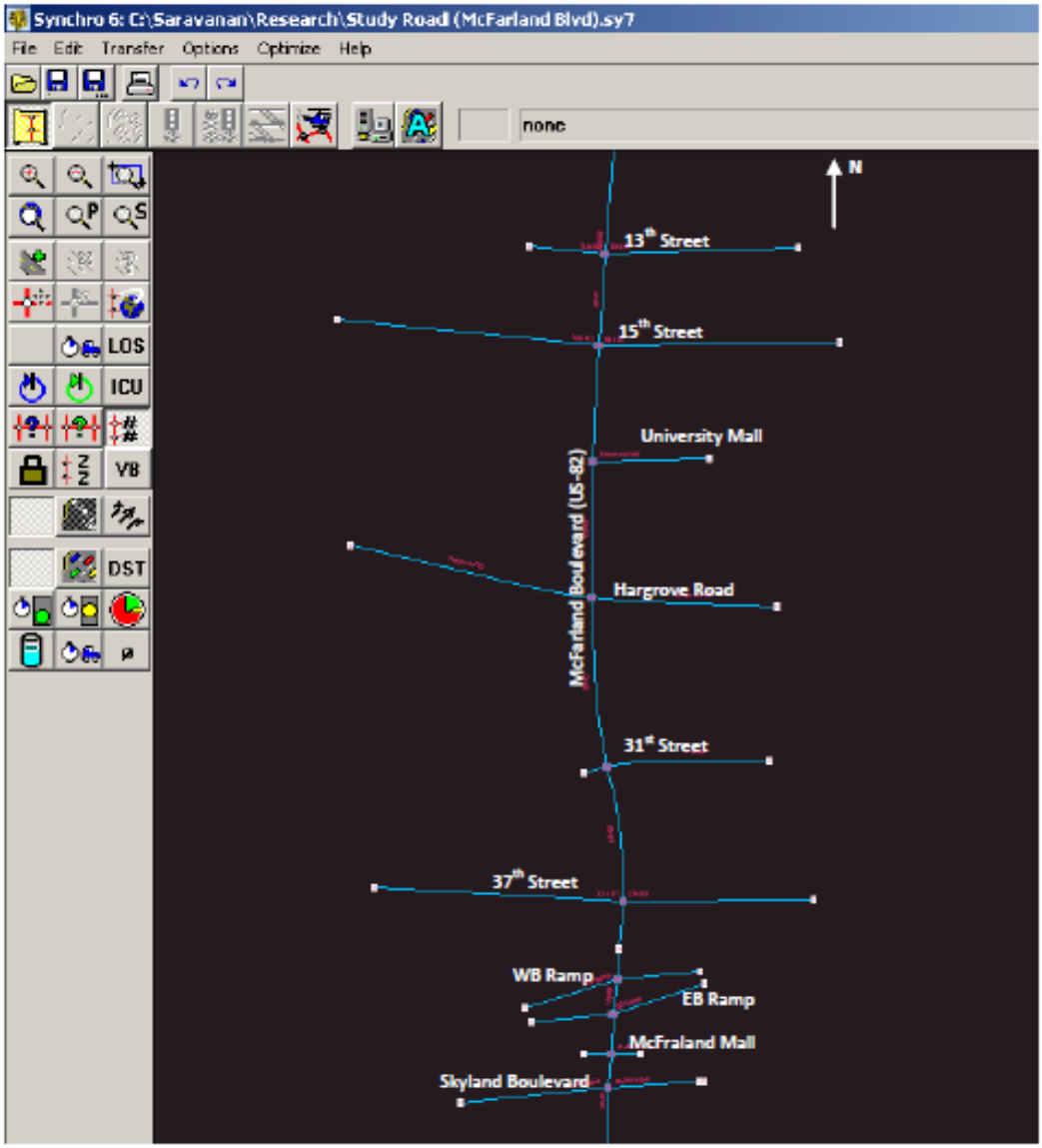

Figure 2.1 Simulation Network

2.7 Simulation Runs

The Synchro file containing the traffic volume, geometrics, timing, and phasing details of the study road network provided input for the SimTraffic model. The authors learned that results of the simulation (i.e., MOEs) are valid only if there is sufficient initialization time for the network to load completely and reach equilibrium [Traffic Analysis Toolbox, Volume III, 2004]. Accordingly, various initialization times were tested starting with a default value of 3 minutes, with increments of one minute for each subsequent attempt. It was found that the study road network started attaining equilibrium for an initialization time of 7 minutes. Once this was established, two longer initialization times (10 and 13 minutes) were selected to investigate the effects of these initialization times. The simulation runs used the same default values for all parameters with the exception of different initialization times, namely, 7, 10, and 13 minutes. For each initialization time, 50 simulation runs were performed with each run having a different random seed number. Hence, a total of 150 simulation runs were performed for each intersection. For each simulation run, the MOEs were obtained as a text file and exported to MS Excel for analysis.

2.8 Investigation Steps

The entire research consisted of five steps as discussed below:

2.8.1 Step I

In this attempt, a Student's t-Test (two sample assuming unequal variance) was performed to compare MOEs (average speed, travel time and total delay) obtained for different initialization times. After that hypothesis testing was used to determine which MOEs of the intersections passed/failed the t-Test. The null and alternate hypotheses were defined as follows:

H_0: There is no difference between MOE obtained for different initialization times for the intersection under consideration.

H_A: There is difference between MOE obtained for different initialization times for the intersection under consideration.

An example of hypothesis testing is comparing the total delay obtained for a 7 minute initialization time for 50 different random seed runs for an intersection under consideration against delay obtained with a 10-minute initialization time for the same number of runs. Likewise, the same comparison was carried out for 7 and 13 minutes, and 10 and 13 minutes. The t-Statistic was compared with the t-Critical value for a 95% confidence level. If the absolute value of the t-Statistic was greater than that of the t-Critical, then H_0 was rejected (i.e., the MOE differ for distinct initialization times considered), otherwise, H_0 was not rejected.

The total delay/vehicle obtained from simulation runs was used to calculate the LOS individually for each intersection considered in the study. The mean total delay/vehicle for 50 runs was obtained for all the intersections and then Table 2.1 the corresponding LOS values were obtained from Table 2.1. Table 2.2 displays the LOSs of intersections in the study road. The LOS obtained was used for relating the hypothesis testing results. Table 2.3 shows the hypothesis testing results of the studied intersections along with their LOSs.

Table 2.1 Criteria to Determine LOS

Total delay/vehicle (sec)	LOS
≤ 10	A
> 10 and ≤ 20	B
> 20 and ≤ 35	C
> 35 and ≤ 55	D
> 55 and ≤ 80	E
> 80	F

Table 2.2 LOSs of Intersections on Study Road

Intersection	SimTraffic	
	Total Delay/Veh	LOS
WB Ramp and US-82	11.4	B
University Mall and US-82	20.7	C
Skyland Blvd and US-82	42.4	D
McFarland Mall and US-82	14.9	B
Hargrove RD and US-82	69.2	E
EB Ramp and US-82	16.3	B
31st ST and US-82	10.1	B
15th ST and US-82	52.1	D
37th ST and US-82	25.1	C

Table 2.3 Hypothesis Testing Results for Speed, Travel Time, and Total Delay

Intersections	Hypothesis Testing Results for Speed			LOS
	7 mins vs 10 mins	7 mins vs 13 mins	10 mins vs 13 mins	
WB Ramp and US-82	Not Reject H_0	Not Reject H_0	Reject H_0	B
University Mall and US-82	Not Reject H_0	Not Reject H_0	Reject H_0	C
Skyland Blvd and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	D
McFarland Mall and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	B
Hargrove RD and US-82	Reject H_0	Reject H_0	Not Reject H_0	E
EB Ramp and US-82	Not Reject H_0	Reject H_0	Reject H_0	B
31st ST and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	B
15th ST and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	D
37th ST and US-82	Not Reject H_0	Not Reject H_0	Reject H_0	C
Hypothesis Testing Results for Travel Time				
WB Ramp and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	B
University Mall and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	C
Skyland Blvd and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	D
McFarland Mall and US-82	Reject H_0	Not Reject H_0	Not Reject H_0	B
Hargrove RD and US-82	Reject H_0	Reject H_0	Not Reject H_0	E
EB Ramp and US-82	Not Reject H_0	Reject H_0	Reject H_0	B
31st ST and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	B
15th ST and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	D
37th ST and US-82	Reject H_0	Reject H_0	Not Reject H_0	C
Hypothesis Testing Results for Total Delay				
WB Ramp and US-82	Not Reject H_0	Not Reject H_0	Reject H_0	B
University Mall and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	C
Skyland Blvd and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	D
McFarland Mall and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	B
Hargrove RD and US-82	Reject H_0	Reject H_0	Not Reject H_0	E
EB Ramp and US-82	Not Reject H_0	Not Reject H_0	Reject H_0	B
31st ST and US-82	Not Reject H_0	Not Reject H_0	Not Reject H_0	B
15th ST and US-82	Not Reject H_0	Reject H_0	Reject H_0	D
37th ST and US-82	Reject H_0	Not Reject H_0	Reject H_0	C

From Table 2.3, the number of null hypothesis rejections between 7 and 10 mins, 7 and 13 mins, and 10 and 13 mins were found to be 5, 7, and 9 respectively. It can also be observed that the hypothesis testing results are different for different MOEs. Though the numbers of null hypothesis rejections are few, the rejections could be observed among intersections with operations ranging from LOS B to E. It can be said that irrespective of the LOS of the intersection, the MOEs are sensitive to initialization times of simulation. It was also observed that it was more appropriate to analyze each approach separately than the intersection as a whole. This is because there are a myriad of variations in any MOE between approaches within the same intersection. Hence to further this work, the intersections in the study road were analyzed by each approach.

2.8.2 Step II

This step determined MOE (delay/vehicle) by intersection approach and related it to LOS. From the previous step, it was learned that it is better to analyze an intersection by approach than the intersection as a whole. Hence, in this attempt, the MOE total delay/vehicle was obtained for all the approaches of five intersections in the study road; namely Hargrove Road and US-82, 15th Street and US-82, Skyland Boulevard and US-82, 13th Street and US-82 and 37th Street and US-82. Since delay is a more representative MOE for an intersection than speed or travel time, it was chosen for this attempt. The main objective of this step is to determine the sensitivity of total delay/vehicle to simulation initialization times (7, 10, and 13 minutes) and its relation to levels of congestion. The t-Test (two samples assuming unequal variance) was performed to compare the MOE (total delay/vehicle) obtained for all the approaches for five intersections for different initialization times. The null and alternate hypothesis for the hypothesis testing are given below,

H_0: There is no difference between MOE obtained for different initialization times for the approach under consideration.

H_A: There is difference between MOE obtained for different initialization times for the approach under consideration.

The hypothesis testing results related to LOS were obtained from SimTraffic according to the procedure explained in Step II. Table 2.4 shows the results of hypothesis testing.

Table 2.4 Hypothesis Testing Results for Total Delay/Vehicle (Approach-wise)

Intersection	Approach	Reject or Not Reject H_0			LOS
		7 mins vs. 10 mins	7 mins vs. 13 mins	10 mins vs. 13 mins	
Skyland Boulevard Intersection	EB	Not Reject H_0	Not Reject H_0	Not Reject H_0	D
	WB	Not Reject H_0	Not Reject H_0	Not Reject H_0	D
	NB	Not Reject H_0	Reject H_0	Not Reject H_0	D
	SB	Not Reject H_0	Reject H_0	Not Reject H_0	D
15th Street Intersection	EB	Not Reject H_0	Not Reject H_0	Reject H_0	E
	WB	Not Reject H_0	Not Reject H_0	Not Reject H_0	D
	NB	Not Reject H_0	Reject H_0	Reject H_0	D
	SB	Not Reject H_0	Reject H_0	Not Reject H_0	D
Hargrove Road Intersection	EB	Reject H_0	Reject H_0	Not Reject H_0	F
	WB	Not Reject H_0	Reject H_0	Not Reject H_0	F
	NB	Not Reject H_0	Reject H_0	Reject H_0	D
	SB	Reject H_0	Reject H_0	Not Reject H_0	D
13th Street	EB	Not Reject H_0	Not Reject H_0	Not Reject H_0	E
	WB	Not Reject H_0	Not Reject H_0	Not Reject H_0	D
	NB	Not Reject H_0	Not Reject H_0	Not Reject H_0	B
	SB	Not Reject H_0	Not Reject H_0	Not Reject H_0	C
37th Street	EB	Not Reject H_0	Reject H_0	Reject H_0	D
	WB	Not Reject H_0	Not Reject H_0	Not Reject H_0	D
	NB	Not Reject H_0	Reject H_0	Reject H_0	B
	SB	Reject H_0	Not Reject H_0	Not Reject H_0	C

The observations were similar to those obtained for the t-Test in Step I in that the MOE total delay/vehicle is sensitive to the initialization times of simulation. From Table 2.4 the number of null hypothesis rejections between 7 and 10 mins, 7 and 13 mins, and 10 and 13 mins were found to be 3, 10, and 5 respectively. The hypothesis testing of total delay/vehicle for approaches between 7 and 13 minutes of initialization had the most null hypothesis rejections. In

other words, the larger the difference in initialization times the larger the number of rejections of the null hypothesis. It was also observed that there was no clear relationship between rejection of the null hypothesis and the level of congestion of the approaches. Approaches with various levels of service (B, C, D, E, and F) had null hypothesis rejections. Hence, irrespective of the LOSs of approaches, the MOEs are sensitive to simulation initialization times. This is in agreement with similar findings from Step I.

2.8.3 Step III

This step involved observing the trend in standard deviation of the MOE with an increase in the number of simulation runs and relating the findings to LOS.

In some disciplines like hydrology and water resources, an extreme number of simulation repetitions (500 to 1,000) are used to stabilize variability. Although that is not a common practice in transportation modeling, such a step was attempted in this research. The purpose was to observe the trend in the standard deviation of the MOE with a very large increase in the number of simulation runs.

In this attempt, 400 simulation runs were made. This was the maximum that the software supported. Data were recorded for the 8th, 25th, 50th, 100th, 200th and 400th repetitions. The standard deviations of the MOEs were compared for all approaches considered in this study. It was expected that an increase in the number of simulation runs would decrease the variation in MOE. The data were plotted for analysis (Figure 2.2). This figure is quite busy with no obvious pattern; however, it appears that the degree of variation may be related to the LOS of the approaches. Table 2.5 gives the standard deviations obtained for the delay/vehicle of all approaches, sorted by approach LOS.

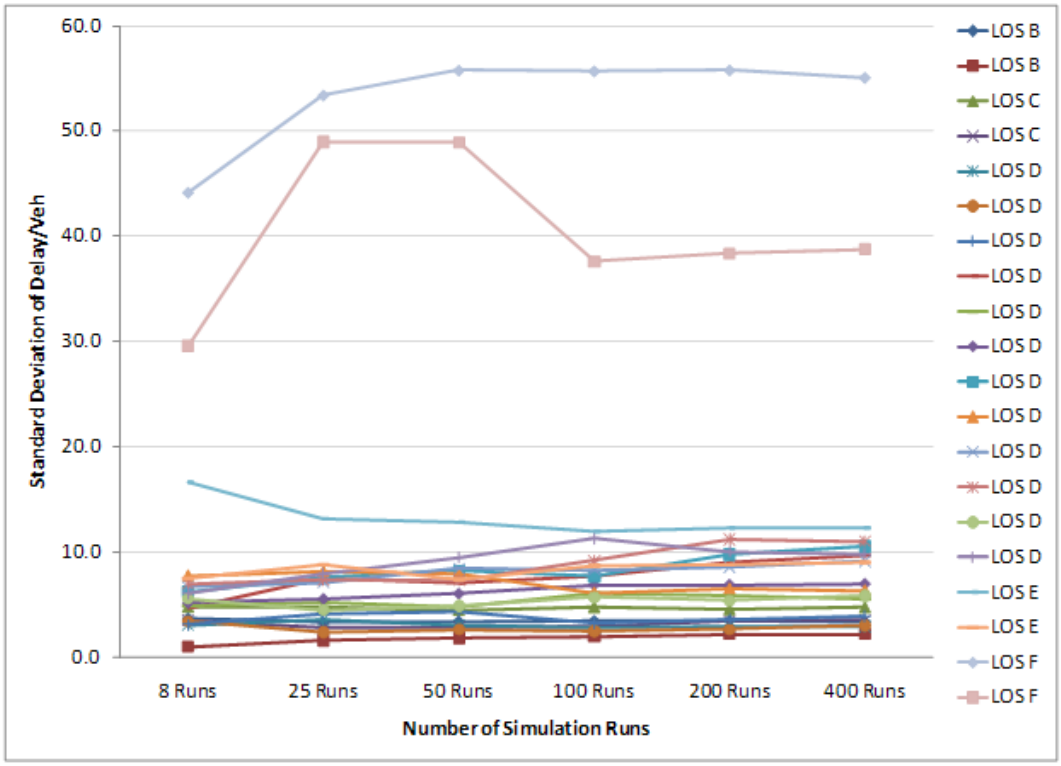

Figure 2.2. Trend of Standard Deviation of Delay/Vehicle for Increased Simulation Runs (All Approaches)

Table 2.5 Standard Deviation of Total Delay/Vehicle for Different Simulation Runs

Intersection	STDEV of Delay/Vehicle (simulation time - 10 minutes)						
	8 Runs	25 Runs	50 Runs	100 Runs	200 Runs	400 Runs	LOS
13th & US-82, NB	3.7	3.3	3.4	3.4	3.5	3.4	B
37th & US-82, NB	1.0	1.6	1.9	2.0	2.2	2.2	B
13th & US-82, SB	4.8	4.8	4.5	4.8	4.6	4.7	C
37th & US-82, SB	3.5	2.8	2.9	2.9	3.4	3.5	C
13th & US-82, WB	6.2	7.6	8.2	7.7	9.8	10.5	D
15th & US-82, WB	4.9	5.2	4.7	6.1	5.9	5.5	D
15th & US-82, NB	7.8	8.2	8.0	6.1	6.5	6.3	D
15th & US-82, SB	5.2	5.6	6.1	6.9	6.9	7.0	D
37th & US-82, EB	6.7	7.1	8.4	8.3	8.6	9.1	D
37th & US-82, WB	7.0	7.4	7.3	9.2	11.2	11.0	D
Hargrove & US-82, NB	3.1	4.1	4.4	3.2	3.6	3.9	D
Hargrove & US-82, SB	4.7	7.6	7.1	7.7	9.0	9.6	D
Skyland & US-82, EB	5.6	4.5	4.9	5.7	5.4	5.9	D
Skyland & US-82, WB	3.4	2.4	2.6	2.5	2.7	3.1	D
Skyland & US-82, NB	6.1	8.0	9.5	11.3	10.1	9.8	D
Skyland & US-82, SB	3.1	3.6	3.1	2.8	2.9	2.9	D
13th & US-82, EB	16.6	13.2	12.9	12.0	12.3	12.2	E
15th & US-82, EB	7.5	8.8	7.4	8.7	8.9	9.0	E
Hargrove & US-82, EB	44.1	53.4	55.8	55.7	55.8	55.1	F
Hargrove & US-82, WB	29.6	48.9	48.9	37.6	38.4	38.8	F

Using the data in Table 2.5, trend charts were prepared for MOE groups to further examine the trend of standard deviation of MOE (delay/vehicle) with increased simulation runs. Figure 2.3 shows the trends for approaches with LOSs B and C. From this figure, it can be observed that the standard deviation of delay/vehicle becomes somewhat stable after 25 simulation runs. In other words, using more than 25 simulation runs did not appreciably change the standard deviation for these approaches. Because of the limited number of approaches with an LOS of C or higher, the best that can be said is that the figure suggests that more than 25 repetitions might not be necessary.

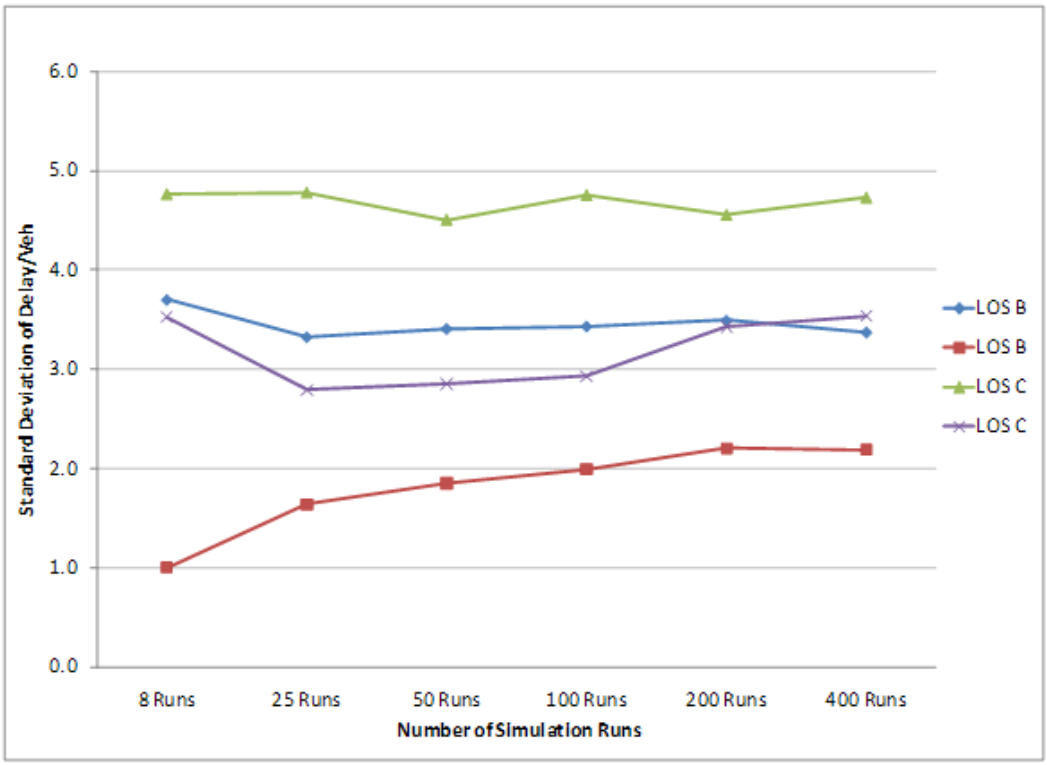

Figure 2.3. Trend of Standard Deviation of Delay/Vehicle for Increased Simulation Runs (Approaches with LOS B and C)

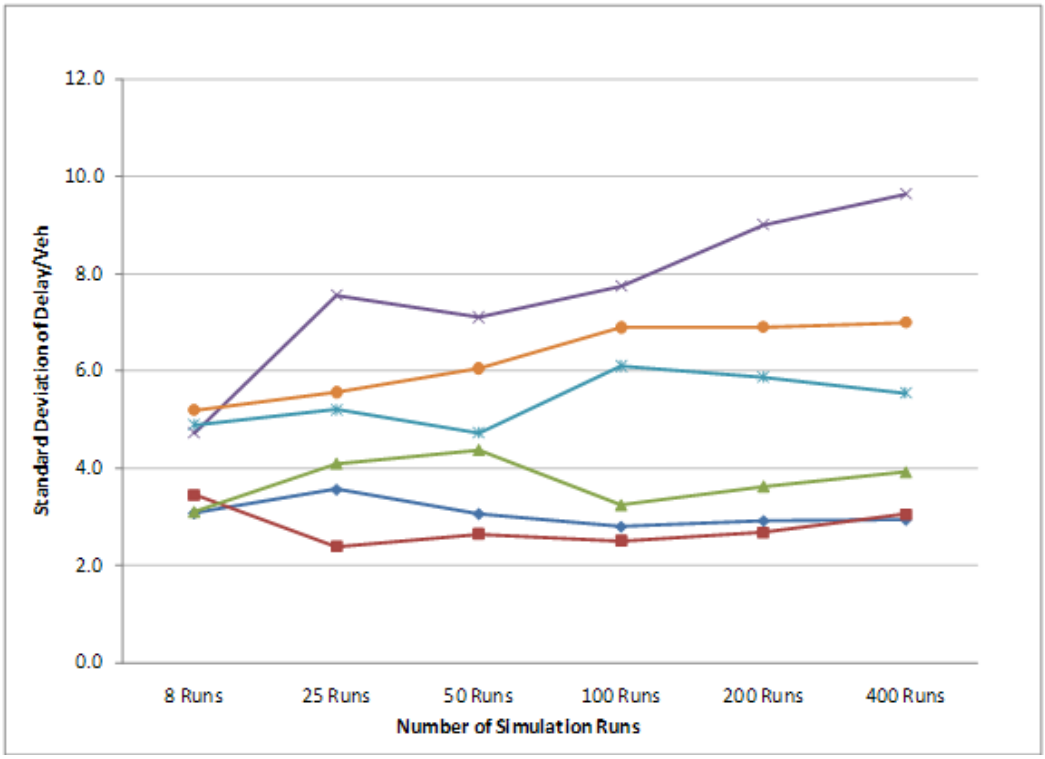

Figure 2.4(a). Trend of Standard Deviation of Delay/Vehicle for Increased Simulation Runs (1st Set of Approaches with LOS D)

Since there were 12 approaches with LOS D, they were separated into two groups for plotting of trends in Figures 2.4(a) and 2.4(b), based upon the magnitude of the standard deviations. In Figure 2.4(a) there are two overall patterns for the 400 runs. Three approaches increased noticeably above the initial standard deviation, and three meandered up and down but stayed relatively near the initial value. A detailed analysis showed that for the six approaches, the standard deviations of two reached stability after about 50 runs and three approaches reached stability after about 100 runs. The sixth approach continued to increase throughout the 400 repetitions and did not reach stability.

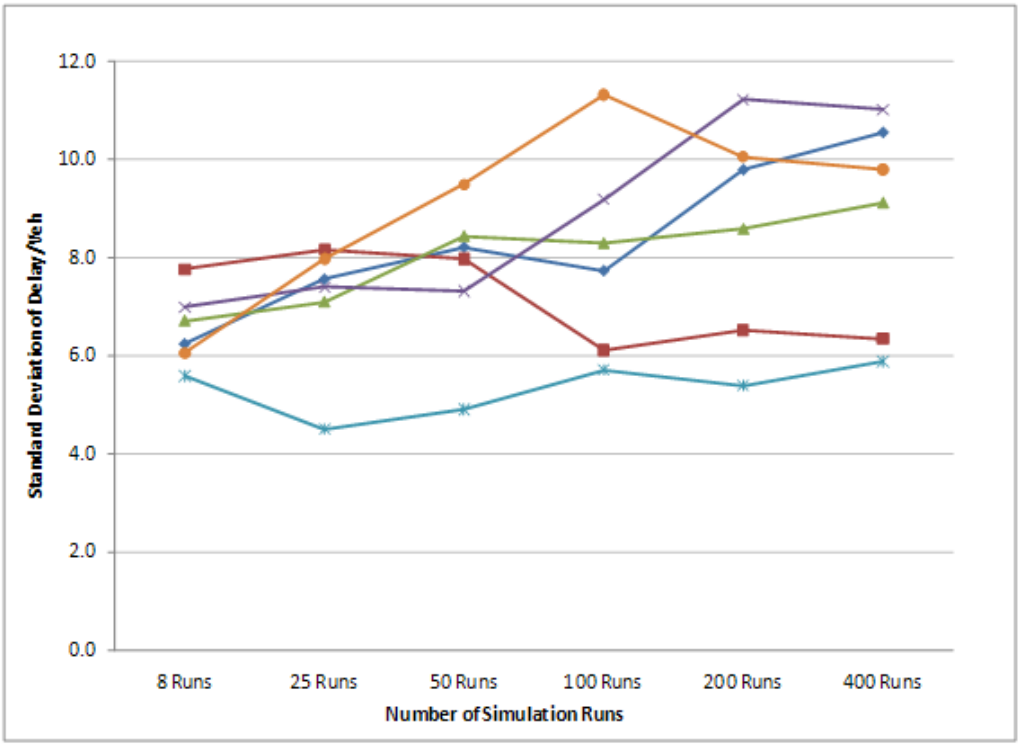

Figure 2.4(b). Trend of Standard Deviation of Delay/Vehicle for Increased Simulation Runs (2nd Set of Approaches with LOS D)

Figure 2.4(b) contains the trend of standard deviation of delay/vehicle for the second set of six approaches with LOS D. This group of approaches had higher initial standard deviations than the previous group of LOS D approaches. Also, the approaches in Figure 2.4(b) display greater variability than the prior group. It appears that the standard deviation of delay/vehicle for one of the six approaches reached stability after 50 runs and two other approaches reached stability after about 100 runs. For the other three approaches the trend was not clear although it is possible that they were approaching stability after 200 runs.

By combining observations from figures 2.4(a) and 2.4(b), the standard deviation of delay/vehicle for 8 out of 12 approaches reached stability at or before about 100 runs. But stability one third of the approaches could not be verified in this study.

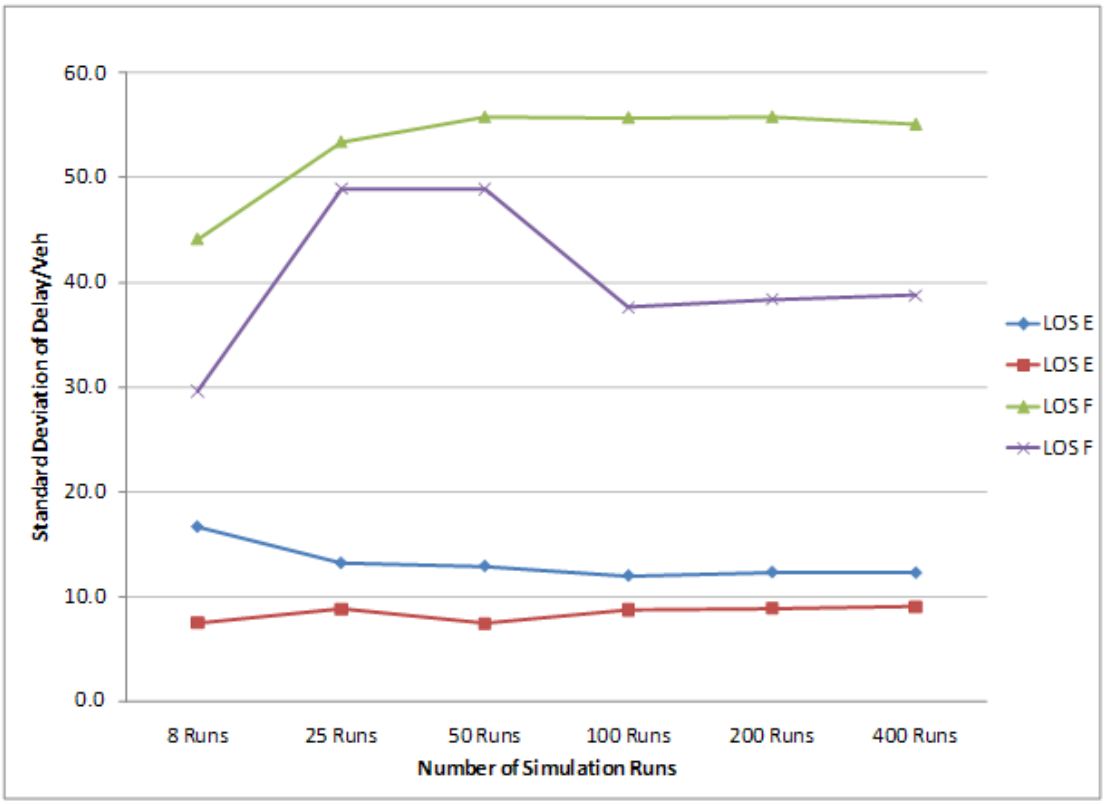

Figure 2.5. Trend of Standard Deviation of Delay/Vehicle for Increased Simulation Runs (Approaches with LOS E and F)

Figure 2.5 shows the trend of standard deviation of delay/vehicle for four approaches with LOSs E and F. From the figure, it appears that standard deviation for two of the four remained steady after 25 runs, one approach stabilized after 50 runs, and the final stabilized after 100 runs.

By combining observations from figures 2.3, 2.4(a), 2.4(b), and 2.5, it appears that, for this study site and the stated simulation period and traffic volumes, the level of congestion certainly has an impact on the number of simulation runs needed to obtain a stable standard deviation for a particular MOE. For approaches with LOS B and C, the standard deviation apparently reached a stable state after 25 runs, but for lower values of LOSs most of the approaches stabilized at a much higher number of repetitions.

This research step produced interesting findings, even though the data sample (number of approaches) was small. Though this investigation step helped in understanding the trend of standard deviation of MOE with respect to different levels of congestion, the value of standard deviation itself was considered high for most of the studied approaches. Furthermore, the standard deviation rarely decreased with an increase in the number of simulation runs, which was contrary to expectations. Only 5 of 20 approaches showed improvement in the standard deviation with an increase in runs. The authors believe that the reason behind this finding could be the simulation time, which was set at 10 minutes. From the available literature [10], it was found that a higher simulation time (60 mins) is expected to reduce the variations in the MOE between each simulation runs. Unfortunately, it was not possible to repeat this study for 400 repetitions at the 60 minute level due to limitations on the amount of time available for this research project.

The small data sample did not allow statistical testing of the results; however, this study points out the need for larger controlled studies of much larger systems. Such studies could include multiple MOEs, longer simulation periods, and up to 1000 repetitions.

2.8.4 Step IV

This step compared the MOEs obtained from 10 minute and 60 minute simulation times, and observed the trend in standard deviation of the MOEs with an increase in the number of simulation runs (60 minute simulation time) and then related it to the LOS of approaches.

The MOEs obtained in the previous step came from a simulation time of only 10 minutes. As mentioned earlier, it was found that a higher simulation time (say 60 minutes) would yield MOEs with lesser variation than that with 10 minutes simulation time. Hence, in this step, fifty simulation runs were performed for each selected approach with a 60 minute simulation time. The standard deviations of MOEs thus obtained for the approaches under consideration were

compared with those obtained for 10 minutes of simulation time. Table 2.6 gives the standard deviation values of MOEs for 10 minute and 60 minute simulation runs. To compare the values obtained, Table 2.7 was prepared showing the average change in standard deviation of MOEs for approaches grouped with respect to their corresponding LOS.

Table 2.6 Standard Deviations of MOEs for 10 minute and 60 minute Simulation Runs

Intersection and Approach	LOS	STDEV (Simulation Time: 10 mins)			Total 1	STDEV (Simulation Time: 60 mins)			Total 2	%Change Total 1vs2
		8 Runs	25 Runs	50 Runs		8 Runs	25 Runs	50 Runs		
13th & US-82, NB	B	3.7	3.3	3.4	10.4	0.9	1.4	1.4	3.7	-65%
37th & US-82, NB	B	1.0	1.6	1.9	4.5	0.6	0.8	0.9	2.2	-51%
13th & US-82, SB	C	4.8	4.8	4.5	14.0	2.5	2.4	2.7	7.6	-46%
37th & US-82, SB	C	3.5	2.8	2.9	9.2	1.7	1.3	1.4	4.3	-53%
13th & US-82, WB	D	6.2	7.6	8.2	22.0	3.8	4.3	4.0	12.1	-45%
15th & US-82, WB	D	4.9	5.2	4.7	14.8	2.9	2.3	2.4	7.6	-49%
15th & US-82, NB	D	7.8	8.2	8.0	23.9	2.4	3.2	3.0	8.6	-64%
15th & US-82, SB	D	5.2	5.6	6.1	16.8	4.0	3.8	3.7	11.5	-32%
37th & US-82, EB	D	6.7	7.1	8.4	22.2	3.8	3.0	3.6	10.3	-54%
37th & US-82, WB	D	7.0	7.4	7.3	21.7	3.6	6.7	5.7	16.0	-26%
Hargrove & US-82, NB	D	3.1	4.1	4.4	11.6	1.9	1.5	1.6	4.9	-58%
Skyland & US-82, EB	D	5.6	4.5	4.9	15.0	1.9	1.7	2.5	6.2	-59%
Skyland & US-82, WB	D	3.4	2.4	2.6	8.5	1.2	1.6	1.3	4.2	-51%
Skyland & US-82, NB	D	6.1	8.0	9.5	23.5	3.3	4.3	5.5	13.1	-44%
Skyland & US-82, SB	D	3.1	3.6	3.1	9.7	1.3	1.2	1.1	3.6	-63%
13th & US-82, EB	E	16.6	13.2	12.9	42.7	4.9	6.6	5.6	17.1	-60%
15th & US-82, EB	E	7.5	8.8	7.4	23.7	2.1	4.1	4.4	10.6	-55%
Hargrove & US-82, SB *	E	4.7	7.6	7.1	19.4	8.7	5.6	7.2	21.4	10%
Hargrove & US-82, EB *	F	44.1	53.4	55.8	153.3	101.6	96.3	82.6	280.5	83%
Hargrove & US-82, WB *	F	29.6	48.9	48.9	127.5	42.4	50.7	62.4	155.5	22%
*Note: Approaches marked with * had positive changes with respect to their Standard Deviations of Delay*										

Table 2.7 Change in Standard Deviation between 10 minute and 60 minute Simulation Time (based on Table 2.6)

Approaches Grouped by LOS	No of Observations	Average Change in STDEV	Direction of Change
B	2	-58%	all negative
C	2	-49%	all negative
D	10	-49%	all negative
E	3	-35%	mixed
F	2	52%	Positive

The following observations could be drawn from Table 2.6 and 2.7;

- For the limited number of approaches tested, as the simulation time becomes longer:
    - The standard deviation of delay increases as LOS decreases (B to C to D). This trend was consistent regardless of the number of runs (8, 25, or 50 runs).
    - But approaching saturation and during saturated conditions, the behavior was different.
- At LOS E, there was a mixture of both negative and positive changes to the standard deviation of delay.
- At LOS F, there were only positive changes to the standard deviation of delay.
- The length of simulation time could lead to a considerable difference in the results.
- To help understand the trend observed in Table 2.6, the researchers decided to conduct a study on the source of variation in MOEs.

2.8.5 Step V

This step focused on identifying the sources of variation of MOE and finding the major contributor among them.

One of the observations of Step IV was that a higher simulation time does decrease the variations in the MOEs. However, an increase in the number of simulation runs did not decrease variations within the MOEs. Hence, it was desired to explore why the variations within MOEs still existed in spite of an increase in the number of simulation runs. Specifically, the goal was to determine the sources causing such variations within the MOEs.

Earlier research noted that vehicle and driver types that populate the network and the rate and proportions at which they do so are the main contributors to variations in MOEs [Traffic Analysis Toolbox, Volume IV, 2007]. These parameters are addressed by increasing the number

of runs and changing the rates and proportions via random seeds. From these changes, variance is introduced by the specification of car-following, gap acceptance, and lane changing parameters (all related to driver and vehicle type but handled in separate algorithms).

In this research step, the UTCA researchers held one of these three parameters constant while the other two were allowed to vary. This was repeated for the other two parameters. In other words, when the specifications for lane changing were held constant, car following and gap acceptance were allowed to vary. A total of 150 simulation runs were performed (50 runs while holding each of the three parameters constant). It was expected that this procedure would help to isolate the source of variability and the effects of congestion. Table 2.8 gives the embedded specifications for the car following, gap acceptance, and lane changing parameters in SimTraffic.

Table 2.8 Embedded Specifications of Lane Changing, Gap Acceptance and Car Following Parameters in SimTraffic

Item	Parameter	Value
Lane Changing Parameters	Positioning Advantage (veh)	15
	Optional Advantage (veh)	2.3
	Mandatory Dist Adj (%)	200
	Positioning Dist Adj (%)	150
Gap Acceptance Parameter	Gap Acceptance Factor	1.15
Car Following Parameters	Yellow Decel (ft/sec^2)	10
	Courtesy Decel (ft/sec^2)	5
	Yellow React (sec)	1
	Green React (sec)	1
	Max Accel Rate (ft/sec^2)	10
	Headway at 0 mph (sec)	0.5
	Headway at 20 mph (sec)	1.5
	Headway at 50 mph (sec)	1.75
	Speed Factor (%)	1
	Maximum Speed (mph)	60

The results of the simulation runs performed with fixed specifications for the lane changing parameter are given in Table 2.9.

Table 2.9 Standard Deviation of MOE with Constant Specifications of Lane Changing Parameter

Approach	STDEV of Delay/Vehicle (sec)			LOS	+/-/..	Remarks
	8 Runs	25 Runs	50 Runs			
37th, NB	0.8	0.8	0.8	B	.. ..	
13th, NB	1.9	1.3	1.4	C	- ..	
37th, SB	3.9	2.9	2.7	C	- -	
13th, SB	3.7	3.2	2.9	C	- -	
Skyland, SB *	0.9	1.1	1.1	D	+ ..	not much distance for lane changing
Skyland, WB	2.2	1.6	1.5	D	- -	
Hargrove, NB	2.8	2.3	2.2	D	- -	
13th, WB	6.1	4.7	4.8	D	- ..	
15th, WB	2.6	2.3	2.3	D	- ..	
37th, EB	3.8	3.2	3.4	D	- ..	
Skyland, NB	4.5	3.9	4.1	D	- +	
37th, WB *	3.6	5.4	6.6	D	+ +	single lane no room for lane change
Skyland, EB	3.6	2.6	2.4	D	- -	
15th, SB	4.8	4.3	4.2	D	- ..	
15th, NB *	3.8	3.2	4.8	E	- +	Congested
13th, EB *	5.6	7.0	6.2	E	+ -	congested, single lane
Hargrove, SB *	4.1	5.3	5.0	E	+ -	Congested
15th, EB *	7.9	9.0	8.3	E	+ -	Congested
Hargrove, WB *	79.9	81.7	81.7	F	.. ..	Congested
Hargrove, EB *	83.9	76.2	86.2	F	- +	Congested

Note: Pattern is + gain, - loss, … continue
Note: Items with does not follow declining trend in their standard deviation values, refer the remarks column for explanation*

From Table 2.9, it was observed that lane change parameters were playing a major role in contributing variation to the MOE (delay/vehicle). From Table 2.9, it can be observed that many approaches, except those that are highly congested (LOS E and F), showed a decrease in the standard deviation of delay with an increase in the number of simulation runs. Some approaches with better LOSs did not follow this trend because they were either approaches with a single lane or those with not enough approach length, and hence the lane change parameters had no effect or limited effect on them.

The results of the simulation runs for constant specifications for the gap acceptance and car following parameters are shown in tables 2.10 and 2.11, respectively. No clear trends for MOE standard deviations were found in either table. This is opposite to the trend observed while lane changing parameters were held constant. Several reasons may account for this situation:

- When gap acceptance and car following parameters were kept constant, the lane changing parameter was allowed to vary. It is worth noting that Table 2.9 already confirmed that the lane changing parameter is indeed a major source of variation in MOEs, which explains the variation in MOEs observed in tables 2.10 and 2.11.
- The car following algorithm is very complex, and by merely holding the specifications of the parameter constant, the variation caused by the parameter cannot be completely controlled.
- The values selected for the parameters in Table 2.8 also play a major role in the outcome of the results. The chosen values can be conservative or aggressive.

Table 2.10 Standard Deviation of MOE with Constant Specifications of Gap Acceptance Parameter

Approach	Standard Deviation of Delay/Vehicle (sec)			LOS	+/-/..
	8 Runs	25 Runs	50 Runs		
37th, NB	0.9	0.7	0.8	B	- +
13th, NB	1.1	1.2	1.3	B	+ +
37th, SB	1.5	1.5	1.5	C	.. ..
13th, SB	1.0	1.5	1.9	C	+ +
Skyland, SB	0.8	0.9	1.1	D	+ +
Skyland, WB	0.9	1.1	1.2	D	+ +
Hargrove, NB	2.0	1.6	1.9	D	- +
13th, WB	5.4	4.5	4.8	D	- +
15th, WB	2.2	1.5	2.8	D	- +
37th, WB	4.9	5.5	5.0	D	+ -
Skyland, NB	5.7	5.0	4.6	D	- -
37th, EB	4.6	3.5	3.7	D	- +
Skyland, EB	2.3	1.9	2.0	D	- +
15th, SB	6.0	4.1	4.0	D	- ..
15th, NB	3.2	3.3	3.2	D	.. ..
Hargrove, SB	6.4	5.5	5.5	E	- ..
13th, EB	4.5	4.5	4.9	E	.. +
15th, EB	4.5	4.3	4.4	E	.. ..
Hargrove, WB	34.1	42.7	47.2	F	+ +
Hargrove, EB	129.7	102.0	91.8	F	- -
Note: Pattern is + gain, - loss, … continue					

Table 2.11 Standard Deviation of MOE with Constant Specifications of Car Following Parameter

Approach	Standard Deviation of Delay/Vehicle (sec)			LOS	+/-/..
	8 Runs	25 Runs	50 Runs		
37th NB	0.9	1.0	0.9	B	+ -
13th NB	2.1	1.6	1.5	B	- -
37th SB	1.8	1.8	2.1	C	.. +
13th SB	2.4	2.3	2.3	C	.. ..
Skyland SB	0.9	1.0	1.1	D	+ +
13th WB	4.5	5.9	7.2	D	+ +
Skyland WB	0.8	1.2	1.2	D	+ ..
Hargrove NB	3.1	2.8	2.9	D	- ..
15th WB	1.6	2.4	2.4	D	+ ..
37th WB	3.8	5.4	5.2	D	+ ..
37th EB	4.8	6.7	5.6	D	+ -
Skyland NB	9.2	7.3	6.4	D	- -
Skyland EB	3.7	3.2	2.9	D	- -
15th EB	11.3	9.2	11.8	E	- +
13th EB	13.2	8.6	7.1	E	- -
15th SB	4.6	4.9	5.0	E	+ ..
15th NB	5.6	4.8	5.2	E	- +
Hargrove SB	13.0	20.4	18.2	E	+ -
Hargrove WB	69.0	75.3	67.8	F	+ -
Hargrove EB	85.5	87.9	100.6	F	.. +
Note: Pattern is + gain, - loss, … continue					

2.9 Conclusions

This portion of the study explored three major topics:

- The sensitivity of traffic parameters to simulation initialization times.
- The relationship between levels of congestion and the number of simulation runs needed for statistically optimum MOE metrics.
- The source of variability of MOEs between simulation runs, and its relation to different levels of congestion.

The major findings from this study are listed below:

- The MOEs of an intersection are indeed sensitive to initialization times of simulation.

- The larger the difference in initialization times (say 7 and 13 minutes instead of 7 and 10 minutes), the larger the difference between their means MOEs.
- While choosing the initialization time, practitioners should check with the guidelines given in the Toolbox [Traffic Analysis Toolbox Volume III, 2004] and the specifications given in the software manual.
- For this kind of sensitivity analysis of an intersection, it appears better to analyze each approach of an intersection separately. There are large variations in any MOE between approaches within the same intersection.
- The variation within MOEs for different numbers of runs definitely increases with increases in congestion. In other words, the deviation of MOEs for a saturated traffic condition is much larger than that obtained for a free flow traffic condition.
- An increased number of simulation runs certainly helps in stabilizing the variability of the MOE. In addition to that, it appears that the level of congestion also has an impact on the number of simulation runs needed to obtain a stable standard deviation for simulation results. For approaches with better LOSs such as B and C, the standard deviation reached a stable state after 25 runs. For approaches with LOSs of E and F, most of the approaches stabilized at a much higher number of repetitions. It was difficult to understand the trend of MOE variability for approaches with LOS D. The reason could be that LOS D represents the transition phase between congested and free flow condition.
- The length of simulation time makes a considerable difference in the simulation results. A larger simulation time decreases the variance in MOEs for undersaturated conditions. For near-saturation and saturated conditions (LOS E and F), an increase in simulation

time does not necessarily decrease the variance in MOEs (this observation was based on only a few data points on a single arterial).

- Lane change parameters play a major role as a source of variation of MOEs (Delay/Vehicle) obtained for different random seeds. But this is not conclusive for congested conditions (LOS E and F).
- The extent of variation on MOEs caused by car following and gap acceptance parameters could not be identified. It was understood that the car following algorithm is actually very complex. By merely holding the specifications of the parameter constant, the variation caused by the parameters cannot be completely controlled.

2.10 Ideas for Future Research

Future research should focus on the following areas:

- For the sources of variation of MOEs, namely lane changing, gap acceptance and car following parameters, further work should be done by choosing different embedded specifications of these parameters in the simulation software.
- A movement-wise (through, left, right) analysis of the approaches of an intersection is also recommended to further this work.
- Since it was difficult to understand the behavior of MOEs under near-saturated and saturated conditions (LOS E and F), further research should particularly focus along these lines.
- An even larger number of simulation runs (e.g., 1000 runs) could be performed to study the MOEs, specifically for those approaches and intersections having poor LOS values. Accordingly, the simulation models should be modified to enable running larger numbers of repetitions.

2.11 Acknowledgement

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CHAPTER 3

OBSERVED VARIATIONS IN GAP ACCEPTANCE AND LANE CHANGING IN RELATION TO TRAFFIC FLOW RATES IN URBAN STREETS

3.1 Abstract

This paper investigates the variations in gap acceptance and lane changing with respect to traffic flow rates on urban streets. The road network chosen for this study is a six lane two way main traffic artery located in Tuscaloosa, Alabama, USA. Data was collected for the number of lane changes and gap acceptances per cycle of green time for southbound traffic during the AM peak, midday and PM peak on a normal working day. For each lane change and gap acceptance, the duration of the change maneuver (in seconds) and the size of the accepted gap (in seconds) were collected. The collected data were grouped under four different flow rate bins: 10 – 30, 30 – 50, 50 – 70, and 70 – 90 vehicles per minute. Descriptive statistics (mean, median, standard deviation, and skewness) and best fit distributions were obtained for the collected data to observe the variations in size of accepted gaps and duration of lane changes corresponding to different flow rate bins. From the analysis the authors found that the size of accepted gaps and duration of lane changes were the least for the highest flow rate. In other words, it was observed that when traffic flow approached saturation, a considerable number of drivers accepted smaller gaps and changed lanes more rapidly. The authors also observe that the findings from this study may have direct implications upon the gap acceptance and lane changing parameters used in microscopic traffic simulation, particularly during model calibration.

3.2 Introduction

Traffic congestion is a common concern on urban streets. During peak hours, traffic demand at a signalized intersection increases and often exceeds the capacity of the intersection when the length of the green signal indication is insufficient to clear the vehicles waiting to pass through the intersection. In this event, the intersection is said to be saturated. Driver behavior varies during the transition from undersaturated to saturated. Understanding the changes in driver behavior may help us understand the underlying contributors to congestion and thereby help devise solutions for it. Among the many operational parameters related to driver behavior, lane changing and gap acceptance appear to be the most strongly related to the increase in congestion.

Lane changing refers to drivers changing roadway lanes without interfering with vehicles in the destination lane. Gap acceptance is the minimum size of gap (seconds) in traffic flow that drivers are willing to accept when entering or crossing a traffic stream, or while changing lanes. Over the years, these two driver behavior parameters have attracted attention from researchers. However much of the existing research has been done on lane changes on freeways, not on urban streets. This study is an attempt to enhance traffic operations by improving our understanding of lane changing and gap acceptance parameters on urban streets under different levels of traffic flow.

3.3 Objective

This study is a part of on-going research project 07112 by the University Transportation Center of Alabama (UTCA), “Characterization of Arterial Traffic Congestion”. The broad objective of this research is to investigate the changes between undersaturated and oversaturated traffic conditions in terms of performance measures (delay) and operational parameters (gap acceptance and lane changing). This paper contributes to that project by exploring the changes in

lane changing and gap acceptance parameters in urban streets under different levels of traffic flow.

3.4 Background

Historically, there has been great interest in parameters related to driver behavior. Past research on these parameters has mainly focused on freeways rather than urban streets. The reason could be that traffic operations on urban streets are more complex than on freeways due to the presence of traffic signals and the lack of sufficient access management on urban streets.

Previous research has shown that gap acceptance and lane changing on freeways depend on driver parameters like aggressiveness, urgency, and impatience [Goswami and Bham, 2007]. It is known that drivers behave differently under diverse traffic, geometric, and environmental conditions. Similarly, the same driver can behave differently under varying traffic and control conditions [Goswami and Bham, 2007]. The authors studied driver behavior in terms of gap acceptance and critical gaps (the minimum value of an accepted gap) for mandatory lane changes in congested and uncongested traffic conditions. Data collected by the Next Generation Simulation (NGSIM) project on I-80 was used for this study. The authors proposed a gamma distribution for accepted time gaps for mandatory lane change maneuvers under both uncongested and congested traffic flow conditions. They also found that drivers were more sensitive to trailing critical gaps (between the subject vehicle and the following vehicle) than to leading critical gaps (between the subject vehicle and the following vehicle).

Modeling the lane changing process has attracted interest among researchers. One such study done by Toledo and Zohar, 2007, focused on modeling the duration of lane changes. The authors used vehicle trajectory data at a high time resolution collected from cameras mounted on

I-80. They found that lane changes were not instantaneous events as modeled by most microscopic traffic simulation software models. They have durations in the range of 1.0 to 13.3 seconds, with a mean of 4.6 seconds. They also indicated that the lane change durations for passenger cars differed significantly from that of heavy vehicles. However, with both vehicle types, lane change durations were longer when the maneuver was riskier or when the task was complicated by the relationship of the subject vehicle to other vehicles.

Earlier studies also focused on risk taking behavior of drivers with respect to congested traffic conditions. One of the studies conducted by Mahlavat and Zhang, 2008, on I-35 determined the safe headway adopted by drivers during different levels of traffic flow. The authors attempted to identify the relationship between increasing congestion levels and drivers' willingness to adopt unsafe headways. They found that at all traffic flow levels there were vehicles traveling at unsafe headways, but more drivers seemed to adopt unsafe headways as the flow level increased.

A pilot study was conducted by Coifman et al, 2006, on the effects of lane change maneuvers on freeway delays. The authors proposed a method to estimate delays caused by a lane change maneuver within a given lane, relative to the situation in which no lane change occurred. The authors used vehicle trajectory data from I-405. Their methodology was to determine the difference between the measured and estimated travel times. The measured travel time was obtained from trajectory data and the estimated travel time was found by an algorithm which reflected conditions in the lane if there had been no lane changes. The authors showed that the method was feasible, but further research was needed to evaluate the impact of some of the assumptions made in the study.

3.5 Data Description

The UTCA Intelligent Transportation Systems/Traffic Management Center (ITS/TMC) lab was used extensively for collecting the required data for this study. The ITS/TMC lab receives live feed of real-time traffic by closed circuit television (CCTV) cameras placed along the study road (McFarland Boulevard, Tuscaloosa, AL). One of the cameras was adjusted to view the intersection of McFarland Boulevard and 37th Street, including about 1000 feet of southbound approach of McFarland Boulevard to the signal. As per 2007 traffic counts by Alabama Department of Transportation (ALDOT), the Average Daily Traffic (ADT) of McFarland Boulevard is 58,000 and the speed limit is 55 mph. As lane changing and gap acceptance took place, data were collected as per the methodology and timing plans described later in this article. A sketch of the study road is given in Figure 3.1, and a snap shot of the camera view is given in Figure 3.2.

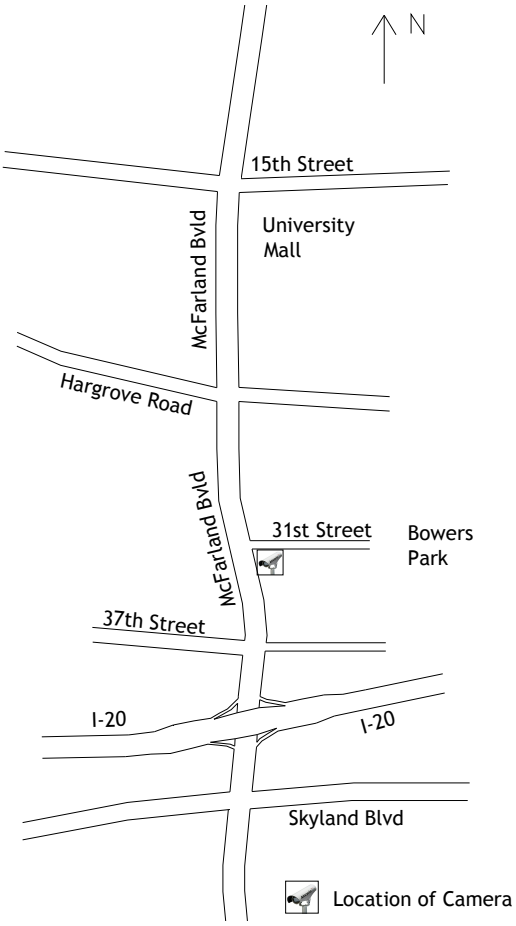

Figure 3.1 Study Road (McFarland Boulevard)

Figure 3.2 Snap Shot of the Approach in the Study Road

3.6 Methodology

Real time traffic flow was recorded using WinTV® software available in the ITS/TMC lab. The data collection protocol captured data from the AM peak (7 AM to 9 AM), midday (11 AM to 1 PM), and the PM peak (4 PM to 6 PM) on a normal working day (February 2008), thus covering both undersaturated and saturated traffic conditions.

Field measurements were collected in time intervals corresponding to the green phase for the southbound traffic at the signalized intersection. Data was collected for the number of lane changes and gap acceptances per cycle of green time. For each lane change and gap acceptance, the duration of the change maneuver (in seconds) and the size of the accepted gap (in seconds) were collected.

3.6.1 Data Collection Procedure

The recorded videos were viewed in Windows Movie Maker® due to its editing capabilities. For instance, the video can be observed as slowly as one frame for every one-fifteenth of a second and can move forward and backwards as necessary. This improves accuracy of data collection and reduction.

In this study, gap acceptance refers to accepting a gap during a lane change maneuver. Hence the gap acceptance and lane change duration parameters are related to each other. Figure 3.3(a) shows all the lane change maneuvers possible within a three lane road and Figure 3.3(b) shows the process of vehicles accepting gaps while changing lanes. In the figure, gap refers to the time gap available to the subject vehicle in the target lane and accepted gaps are those gaps accepted by the subject vehicle. The accepted gaps were collected as follows:

- The time (T1) and location of the leading vehicle in the target lane were noted at the instant the subject vehicle first began changing its alignment in the present lane.
- After the subject vehicle changed lanes, the time (T2) was noted when the following vehicle crossed the same spot as that noted in the first step.
- The difference between times T1 and T2 gives the accepted gap size (in seconds).

Similarly, the duration of the lane change maneuver was calculated as the time taken by the subject vehicle to completely change from one lane to another.

Past studies have indicated that a leading gap or a trailing gap of 200 feet or more will result in vehicles not interacting with each other [Goswami and Bham, 2007]. In this study, gaps are defined as the time between the leading and trailing vehicles (see Figure 3.3(b)). So a gap size of 400 feet was taken as the limit for vehicles to be in interaction. Assuming the speed of vehicles as 35 mph, about 8 seconds is needed to clear 400 feet. In other words, when a vehicle changed lanes, there should be 200 feet between it and the leading vehicle, and between it and the following vehicle. Thus, gap sizes equal to or more than 8 seconds were not counted in the data collection process.

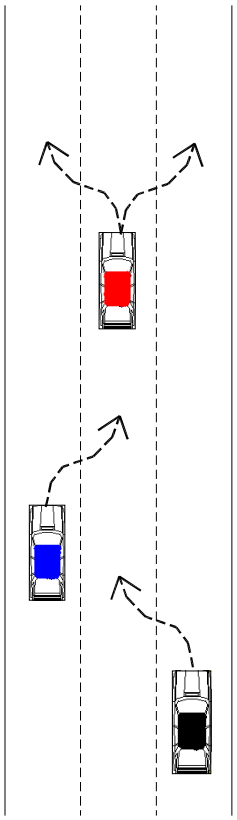

Figure 3.3(a) Possible Lane Changes on a Three Lane Road

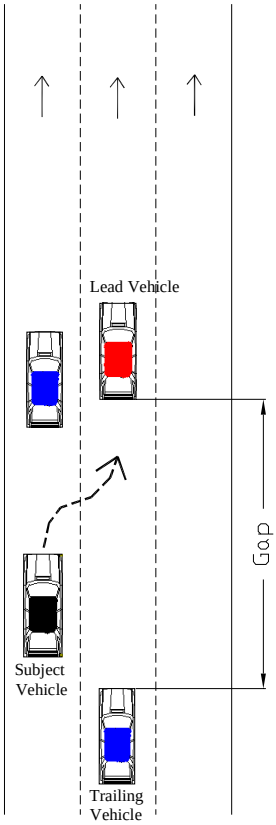

Figure 3.3(b) Gap Acceptance while Changing Lanes

The approach under consideration has three through lanes. Lane changes and gap acceptances occurring only within these three lanes were collected for the study. The data form used to collect duration of lane changes and accepted gaps is shown in Table 3.1. Sample data is shown in the form. As mentioned earlier if the gap sizes were equal to or more than 8 seconds they were not taken into account. This explains the blank spaces corresponding to accepted gaps in Table 3.1

Table 3.1 Data Collection Form and Sample Data

Green Time					Accepted Gap						Duration of Lane Change						Volume
Start	hr	min	sec	ms	time	min	sec	ms	sec	diff	time	min	sec	ms	sec	Diff	# of veh
	4	6	40	40	T1	6	40	47	40.47	4.86	T1	6	40	47	40.47	4.13	78
					T2	6	45	33	45.33		T2	6	44	60	44.60		
					T1	6	43	07	43.07	5.00	T1	6	43	07	43.07	3.93	
					T2	6	48	07	48.07		T2	6	47	0	47.00		
											T1	6	52	13	52.13	3.60	
											T2	6	55	73	55.73		
					T1	7	2	40	2.40	3.47	T1	7	2	40	2.40	5.33	
					T2	7	5	87	5.87		T2	7	7	73	7.73		
					T1	7	9	53	9.53	3.00	T1	7	9	53	9.53	3.47	
					T2	7	12	53	12.53		T2	7	13	0	13.00		
					T1	7	11	20	11.20	8.47	T1	7	11	20	11.20	2.93	
					T2	7	19	67	19.67		T2	7	14	13	14.13		
					T1	7	27	27	27.27	4.00	T1	7	27	27	27.27	4.93	
					T2	7	31	27	31.27		T2	7	32	20	32.20		
											T1	7	44	73	44.73	3.14	
End	4	7	51	87							T2	7	47	87	47.87		

Note: T1 and T2 for accepted gaps and lane changing are defined separately in methodology.

3.6.2 Creation of Flow Rate Bins and Categorizing Data

The flow rate of each cycle of green time was obtained by dividing the volume of traffic clearing the intersection during green with the green time (in minutes). The flow rate is expressed as the number of vehicles per minute of green time. Hence, each cycle has a flow rate associated with it. The lane changes and gap acceptances occurring in a particular cycle correspond to the flow rate of that cycle. For analysis purposes, four different flow rate bins were created to cover the range of traffic flow:

- 10 – 30 vehicles per minute of green
- 30 – 50 vehicles per minute of green
- 50 – 70 vehicles per minute of green
- 70 – 90 vehicles per minute of green

After creating the flow rate bins, each lane change and accepted gap was recorded in the appropriate bin. As a result, each flow rate bin contained a considerable number of lane changes and accepted gaps. To ensure a valid statistical analysis of the grouped data, the minimum sample requirement of each bin was calculated.

3.7 Sample Requirements

The minimum sample size was calculated using the standard procedure given in reference [Currin, 2001]. A confidence level of 95%, a sample standard deviation(s) of 0.5 for 6 samples and a permitted error of the estimate of 5% were assumed. The number of required samples was obtained as 64. This number was taken as the number of lane changing or gap acceptance maneuver required for statistical validity.

The number of samples available in each bin is shown in Table 3.2. It should be noted that for an accepted gap both the lead and following vehicle must be present in the target lane. This explains the difference in the number of lane changes and accepted gaps in Table 3.2 for each flow rate bin.

Table 3.2 Available Number of Samples

Flow Rate bins	Number of Samples Available	
	Accepted Gaps	Lane Changes
10 – 30 veh/min of green	63	156
30 – 50 veh/min of green	171	322
50 – 70 veh/min of green	139	230
70 – 90 veh/min of green	65	106

3.8 Analysis of Data

3.8.1 Descriptive Statistics for Accepted Gaps

The accepted gaps corresponding to each flow rate bin were analyzed to obtain basic statistics such as mean, median, standard deviation, and skewness. Table 3.3 shows the basic

statistics obtained using Minitab. Examples of two such analyses are shown in figures 3.4 and 3.5.

Table 3.3 Basic Statistics of Accepted Gaps for Different Flow Rates

Flow Rate	Mean	Standard Deviation	Median	Skewness
10 to 30 veh/min	4.33	1.65	4.33	0.104
30 to 50 veh/min	4.35	1.79	4.33	0.169
50 to 70 veh/min	4.36	1.69	4.19	0.342
70 to 90 veh/min	4.04	1.61	3.74	0.635

Before drawing inferences from Table 3.3, the differences in the mean values of the accepted gaps for different flow rates were checked by conducting a hypothesis test. The methodology adopted to conduct the test is given in the following text.

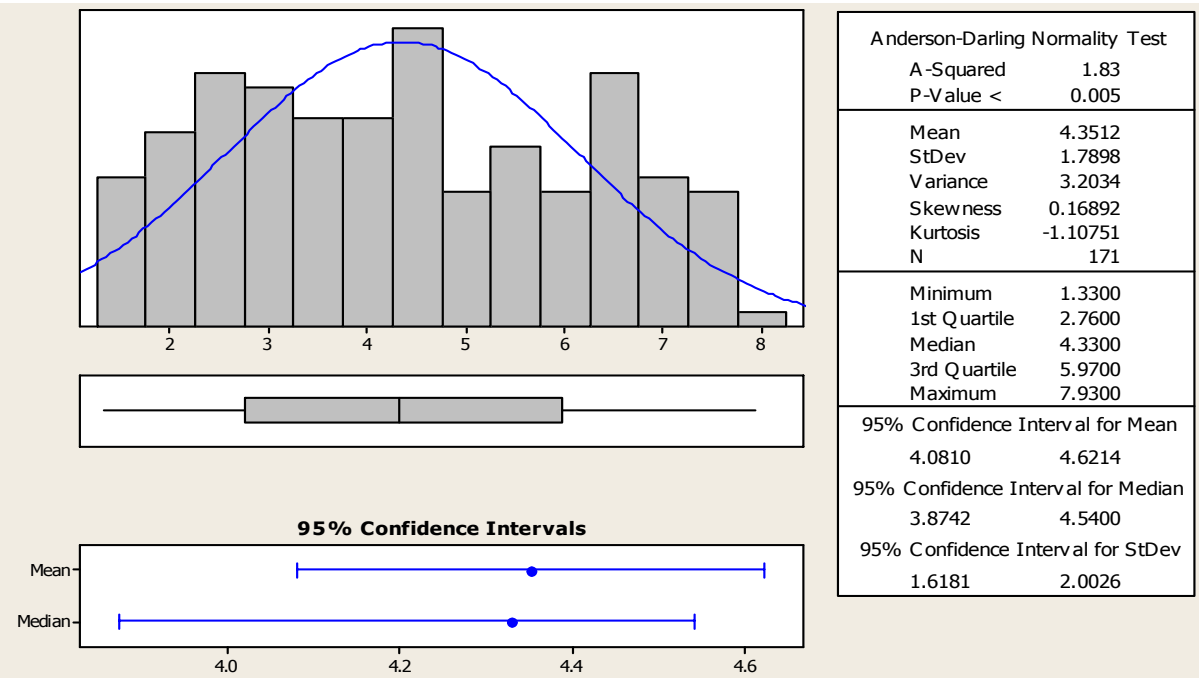

Figure 3.4 Statistics of Accepted Gaps for Flow Rate 30–50 veh/min

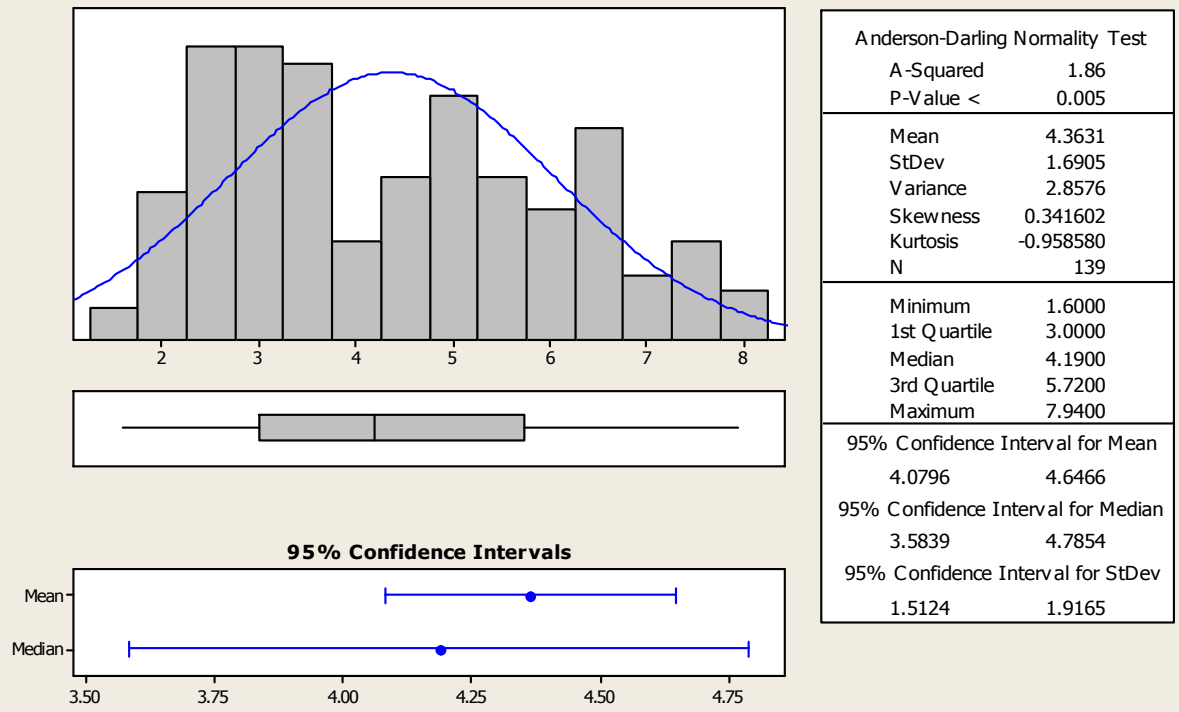

Figure 3.5 Statistics of Accepted Gaps for Flow Rate 50–70 veh/min

3.8.2 Hypothesis Test for Difference between Means of Accepted Gaps

The hypothesis test was performed at a 90% confidence interval to check whether the mean values of accepted gaps were different for different flow rates. Minitab was used to conduct the hypothesis test. The results are given in Table 3.4. The null and alternative hypothesis are stated below,

H_0: The mean values of accepted gaps obtained from two different flow rates are the same.

H_1: The mean values of accepted gaps obtained from two different flow rates are different.

Table 3.4 Hypothesis Test for Accepted Gaps (Mean)

Flow Rate Bins	Z-Statistic	Z-Critical	Hypothesis (H_0/H_1)
10-30 & 30-50 veh/m	0.11	1.28	Do not Reject H_0
10-30 & 50-70 veh/m	0.17	1.28	Do not Reject H_0
10-30 & 70-90 veh/m	1.39	1.28	Reject H_0
30-50 & 50-70veh/m	0.09	1.28	Do not Reject H_0
30-50 & 70-90 veh/m	2.49	1.28	Reject H_0
50-70 & 70-90 veh/m	2.33	1.28	Reject H_0

The results of the hypothesis test indicate that the mean value of the accepted gap for the highest flow rate (70 to 90 veh/m) is smaller than that obtained for all the other flow rates: 10 to 30 veh/m, 30 to 50 veh/m, and 50 to 70 veh/m.

3.8.3. Best Fitted Distributions for Accepted Gaps under Different Flow Rates

The samples collected for accepted gaps for different flow rates were used to find the corresponding fitting distribution. Arena®, a software used for statistical design and analysis, was used to fit the distributions. In order to overlay the fitted distributions of accepted gaps of four different flow rates, the shape, scale and location parameters obtained from Arena were used to recreate the distributions. The parameters of the best fitted distributions and their square errors are given in Table 3.5, and the fitted distributions are shown in Figure 3.6.

Table 3.5 Parameters and Square Errors of Best Fitted Distributions for Gap Acceptance under Different Flow Rates

Flow Rate (veh/min)	Best Fitted Distribution	Parameters		Square Error
		Scale	Shape	
10 to 30	Weibull	2.140	3.75	0.0125
30 to 50	Weibull	1.950	3.78	0.0066
50 to 70	Gamma	0.948	3.55	0.0076
70 to 90	Gamma	0.862	3.53	0.0055

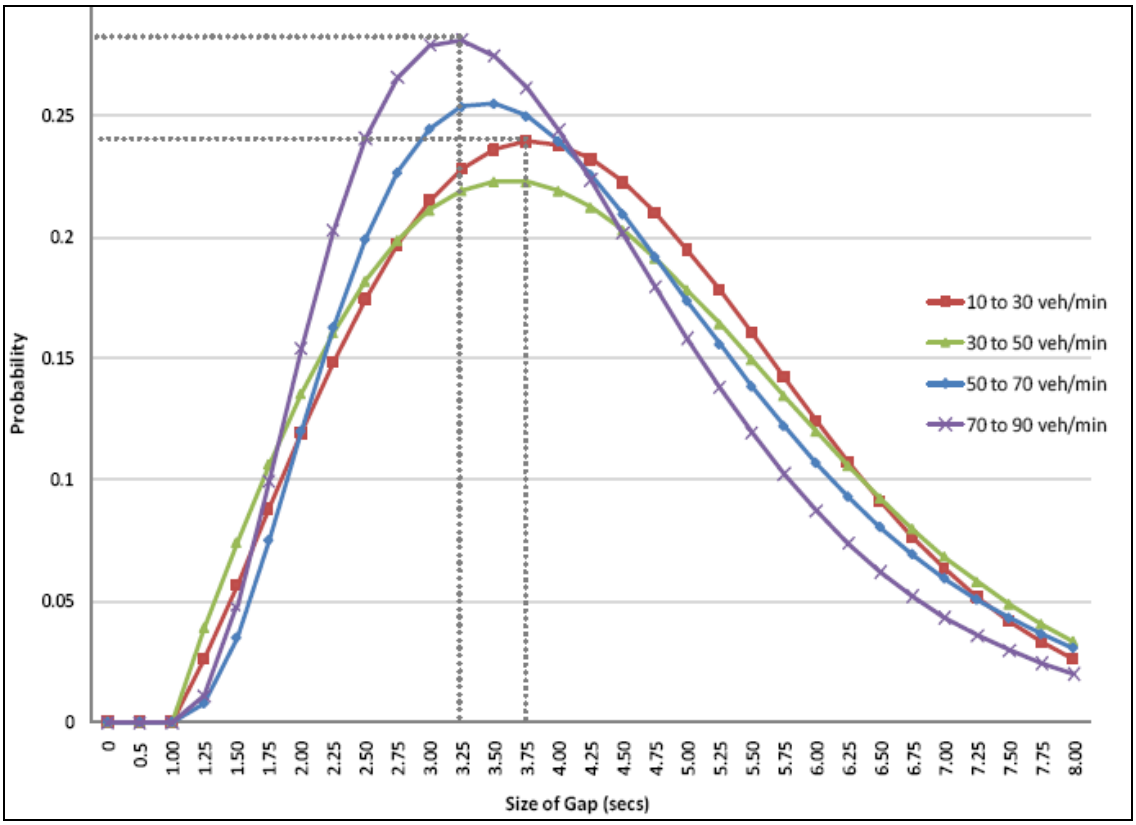

Figure 3.6 Best Fitted Distributions for Gap Acceptance under Different Flow Rates

From Figure 3.6, it can be observed that as the flow rate increases the accepted gap size corresponding to the highest probability decreases (the expected outcome). For instance, consider the flow rates of 10 to 30 veh/min and 70 to 90 veh/min. The accepted gap size corresponding to the highest probability (24%) in the former is 3.75 seconds, while that of the latter is 3.25 seconds (highest probability 28%). Similarly, the accepted gap size corresponding to the highest probability of the other two flow rates (30 to 50 and 50 to 70 veh/min) falls in between the two extremes. It can also be observed that the distributions are increasingly skewed to the right with increasing flow rate.

3.8.4 Findings

From the analysis of accepted gaps including basic statistics, hypothesis testing and best fitted distributions, the following observations can be drawn:

- Since the mean value of the accepted gaps during the highest flow rate (70 – 90 vehicles per minute of green time) was less than the other mean values of accepted gaps, it can be understood that average drivers accept smaller gaps when traffic flow is heavy.
- The steady decline in the median value of accepted gaps with increasing flow rates (4.33, 4.33, 4.19, and 3.74 seconds for 10-30, 30-50, 50-70, and 70-90 veh/m flow rates respectively) supports the first finding.
- The shape and skewness of the best fitted distributions showed that the probability of drivers accepting smaller gaps is higher when the traffic flow is heavy.
- From the above findings, it appears that drivers take higher risks to change lanes when congestion reaches higher conditions. On the same lines, it can be noted that drivers reduce their speed during congested traffic conditions and hence are willing to accept shorter gaps to change lanes.

3.8.5 Descriptive Statistics for Lane Change Durations

The mean, median, standard deviation, and skewness were obtained for lane change durations of each flow rate bin. These statistics were obtained using Minitab and are given in Table 3.6.

Table 3.6 Basic Statistics of Lane Change Durations under Different Flow Rates

Flow Rate	Mean	Standard Deviation	Median	Skewness
10 to 30 veh/min	4.34	0.94	4.25	0.290
30 to 50 veh/min	4.33	0.93	4.31	0.149
50 to 70 veh/min	4.25	0.90	4.20	0.108
70 to 90 veh/min	4.19	0.81	4.20	0.016

Before drawing inferences from Table 3.6, the differences in the mean values of the change durations for different flow rates were checked by conducting a hypothesis test.

3.8.6 Hypothesis Test for Difference between Means of Lane Change Durations

The hypothesis test was performed at a 90% confidence interval to check whether the mean values of lane change durations were different during higher flow rates. The results of the hypothesis test obtained using Minitab is given in Table 3.7.

Table 3.7 Hypothesis Test for Lane Change Duration (Mean)

Flow Rate Bins	Z-Statistic	Z-Critical	Hypothesis (H_0/H_1)
10-30 & 30-50 veh/m	0.06	1.28	Do not Reject H_0
10-30 & 50-70 veh/m	1.10	1.28	Do not Reject H_0
10-30 & 70-90 veh/m	2.13	1.28	Reject H_0
30-50 & 50-70veh/m	1.51	1.28	Reject H_0
30-50 & 70-90 veh/m	2.98	1.28	Reject H_0
50-70 & 70-90 veh/m	1.12	1.28	Do not Reject H_0

The rejections of null hypothesis for the flow rates shown in Table 3.7 imply that the mean values of lane change duration of higher flow rates (i.e., 70 to 90 veh/min and 50 to 70 veh/min) are smaller than those obtained for lower flow rates (i.e., 10 to 30 and 30 to 50 veh/min). In other words, with increasing flow rates, time taken to change lanes decreases. However the decrease is relative less when compared with that obtained for accepted gaps.

3.8.7 Best Fitted Distributions for Lane Change Durations under Different Flow Rates

The procedure adopted to create the best fitted distributions for lane change durations is the same as that adopted for accepted gaps. The parameters of the fitted distributions and their square errors are given in Table 3.8, and the fitted distributions are shown in Figure 3.7.

Table 3.8 Parameters and Square Errors of Best Fitted Distributions for Lane Change Durations under Different Flow Rates

Flow Rate (veh/min)	Best Fitted Distribution	Parameters		Square Error
		Scale	Shape	
10 to 30	Weibull	2.63	2.69	0.0082
30 to 50	Weibull	2.62	2.74	0.0043
50 to 70	Weibull	2.53	2.72	0.0040
70 to 90	Weibull	2.18	2.61	0.0073

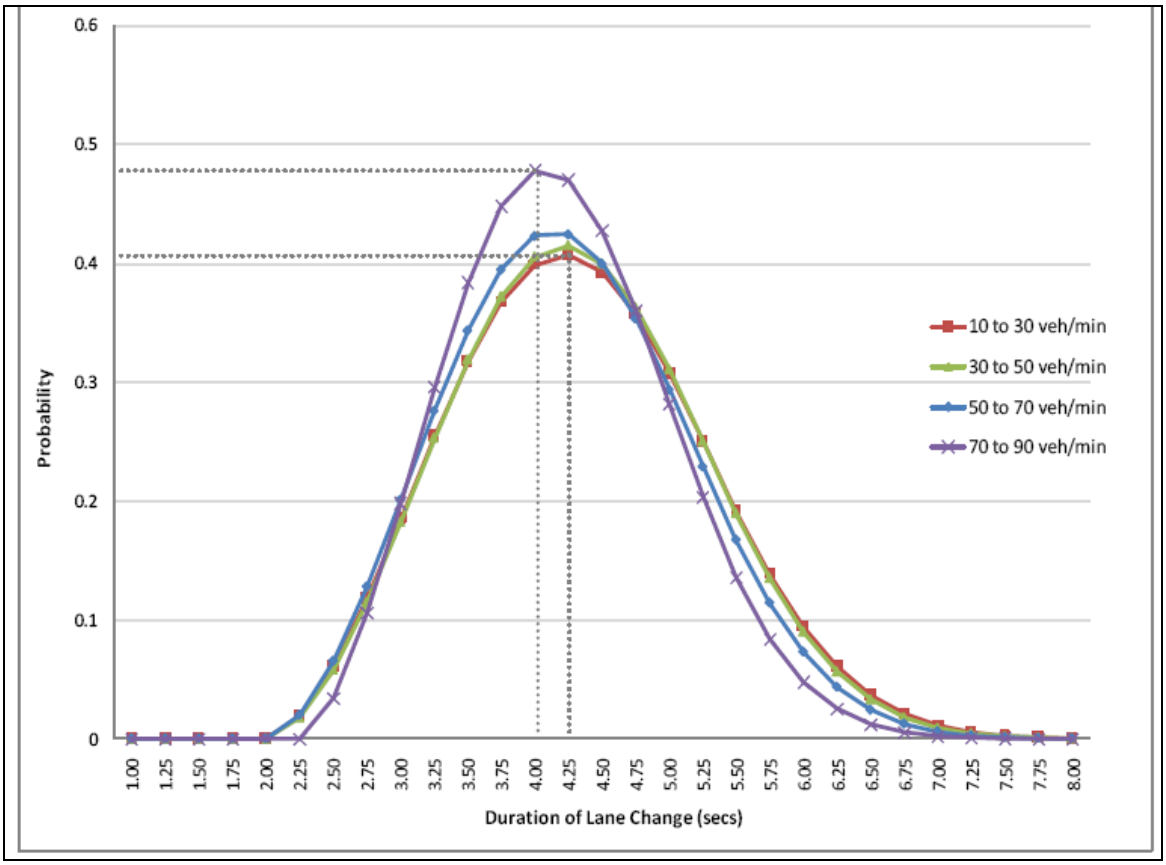

Figure 3.7 Best Fitted Distributions for Lane Change Durations under Different Flow Rates

Figure 3.7 shows that with increasing flow rate the lane change duration corresponding to the highest probability decreases. For instance, for the flow rate of 10 to 30 veh/min, the lane change duration of 4.25 seconds corresponds to the highest probability of 41%, while for the flow rate of 70 to 90 veh/min a lane change duration of 4 seconds corresponds to its highest probability of 48%.

3.8.8 Findings

From the analysis of lane change durations including basic statistics, hypothesis testing and best fitted distributions, the following observations can be drawn:

- The mean values of the lane change duration show a decreasing trend with increasing flow rates (similar to the observation obtained for accepted gaps), but the change is relatively less when compared with that obtained for accepted gaps.

- The standard deviations of lane change durations from lower to higher flow rates were found to be 0.94, 0.93, 0.90, and 0.81 seconds. These values are much less than those obtained for accepted gaps, indicating that the variation of lane change duration values is less than that of accepted gaps.
- The shape and skewness of the best fitted distributions showed a higher probability that drivers change lanes more rapidly when traffic flow is heavy.
- By combining the above observations with that obtained for accepted gaps, it can be understood that during heavy traffic flow conditions considerably more drivers accept smaller gaps and change lanes a bit faster.

3.9 Rate of Lane Changes

In order to determine the trend in the rate of lane changes with increasing flow rates, the rate of lane changes was obtained by dividing the number of changes for each flow rate bin with the average flow rate. For instance, the average flow rate of the 10-30 flow rate bin was found to be 20 vehicles per minute. The rates of lane changes thus obtained for different flow rate bins are given in Table 3.9, and they are plotted on Figure 3.8.

Table 3.9 Rate of Lane Changes

Flow Rate Bins	No of Lane Changes	Rate of Lane Changes/veh/m
10-30 veh/m	144	7.2
30-50 veh/m	302	7.6
50-70 veh/m	223	3.7
70-90veh/m	103	1.3

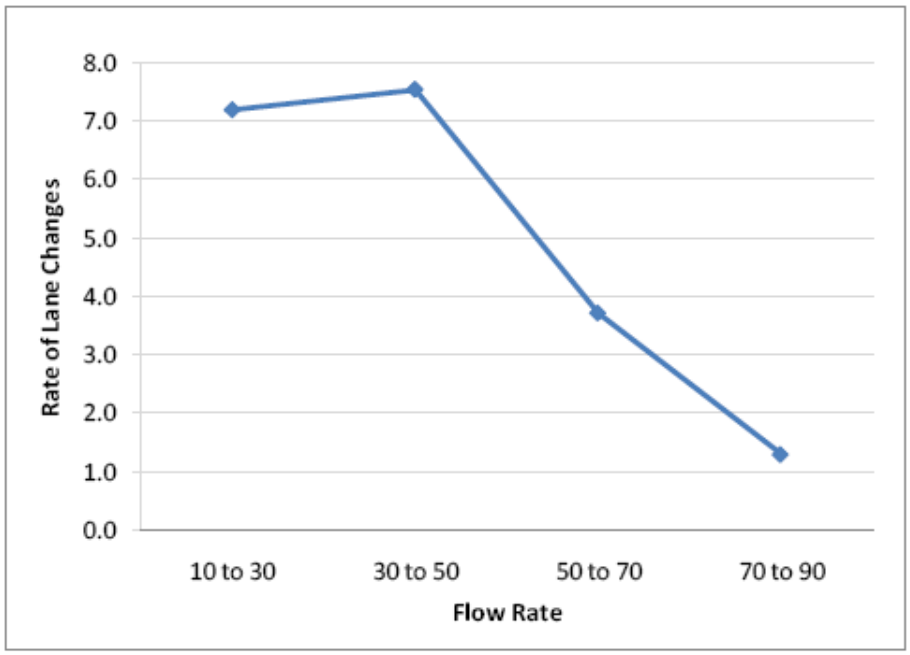

Figure 3.8 Trend of Rate of Lane Changes

The table and figure show that the rate of lane changes decreases with increase in traffic flow. This finding is reasonable because the available gaps decrease with increasing flow rates. It can be concluded that since there are fewer gaps during higher traffic flow or saturated flow, drivers accept smaller gaps.

3.10 Conclusions

There is a keen interest in changes in driver behavior parameters such as lane changing and gap acceptance under varying traffic flow conditions. This study explored the changes in gap acceptance and lane changing parameters on one urban street under different levels of traffic flow. The major finding of this research is that when traffic flow on a typical arterial approaches saturation, a considerable number of drivers accept smaller gaps and change lanes more rapidly. In other words, drivers take higher risks to change lanes during congested traffic flow. This finding is validated by the following observations,

- The mean values of accepted gaps ranged from 4.33 to 4.04 seconds for average flow rates of 20 to 80 vehicles per minute, and standard deviations were around 1.6 seconds. A hypothesis test proved that the extreme values (4.33 and 4.04) are indeed different.
- There was a steady decline in the median value of accepted gaps with increasing flow rates. The highest was 4.33 seconds and the lowest was 3.74 seconds, corresponding to flow rates of 10-30 and 70-90 veh/min, respectively.
- The shape and skewness of the best fitted distributions showed that the probability of drivers accepting smaller gaps is higher when the traffic flow is heavy.
- The mean values of lane change durations ranged from 4.34 to 4.19 seconds for average flow rates of 20 to 80 vehicles/minute, and the standard deviations were around 0.9 seconds. A hypothesis test proved that the extreme values (4.34 and 4.19) are indeed different.
- The variation of lane change durations is less than the variation of accepted gaps, which indicates that gap acceptance is more sensitive to changes in traffic volume than lane change duration.
- The best fitted distributions of lane change durations showed that the probability of drivers changing lanes more rapidly is higher when traffic flow is heavy.
- The rate of lane changes decreases with increasing traffic flow rates, probably because the number of available gaps decreases with increasing flow. It can be concluded that since there are less available gaps during higher traffic flow or saturated flow conditions, drivers accept smaller gaps.

All of these findings have direct implications upon the gap acceptance and lane changing parameters used in microscopic traffic simulation, particularly during model calibration. The

default values of lane changing and gap acceptance parameters used in some of the common traffic simulation tools are indeed different from those obtained in this study. For instance, CORSIM has a default lane change duration value of 3 seconds, while the results obtained in this study show that the mean value of lane change duration varied from 4.19 to 4.34 seconds depending on the traffic flow rate. Also, the gap acceptance (while changing lanes) distributions obtained in this study may be used in lane change models in microscopic traffic simulation models.

3.11 Future Research

To further the current research, the following studies are suggested:

- Increase the sample size of data collection to reduce data-related variation.
- Instead of flow rates, driver behavior parameters should be compared with more representative parameters such as delay, speed or LOS of the arterial approach under consideration.
- Focus on how the driver behavior parameters, as currently used in microscopic traffic simulation models, relate to the field data obtained in this research. Both logical and statistical relationships can be examined.
- Perform similar research in other streets in other cities.

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CHAPTER 4

SENSITIVITY ANALYSIS OF LANE CHANGE PARAMETERS TO SATURATED AND FREE FLOW TRAFFIC CONDITIONS IN MICROSCOPIC TRAFFIC SIMULATION MODELS

4.1 Abstract

Lane change models form an integral part of microscopic traffic simulation. Among many parameters that govern lane change models, the duration of a lane change maneuver and the mean longitudinal distance to perform a lane change are studied in this paper. A sensitivity analysis is performed using default values (embedded in the simulation software) and values obtained from field observations and the available literature for these two parameters. CORSIM is the microscopic traffic simulation model used for this study, and the road network chosen to simulate is McFarland Boulevard (US-82), a six-lane main traffic artery located in Tuscaloosa, Alabama. The sensitivity analysis is performed for the study network under both saturated and free flow traffic conditions. The lane change duration parameter values selected for the analysis are 3, and 7 seconds, and the parameter values for mean longitudinal distance to make a lane change are 300 feet and 2,500 feet. For each value of the parameter under consideration, 10 simulation runs were performed and each run was simulated for 60 minutes. Measures of effectiveness (MOE) such as delay and the number of lane changes were collected for each run. Then a statistical analysis including hypothesis testing using the Student's t-Test was performed to look for trends and statistical differences between MOEs obtained with different parameter values. From the results, it was found that the MOEs are indeed sensitive to lane change duration

and the longitudinal distance for making lane changes. Slower lane changes or larger look ahead distances resulted in a decrease in the number of lane changes or an increase in the delays. Hence, the authors recommend that the current lane changing models be calibrated with the specific lane change duration of the studied location. Also, the value of the parameter for the mean longitudinal distance for making lane changes should be at least larger than the length of storage bays for turning movements, particularly, during saturated conditions. Practitioners must take sufficient care while calibrating this parameter so that it reflects the field conditions of the studied location.

4.2 Introduction

Traffic simulation has been widely used as a traffic analysis tool in transportation analyses, particularly to analyze the operation of complex and congested transportation systems. However, despite the widespread use of traffic simulation software, there is still ongoing research to improve accuracy of simulation and model calibration. This paper focuses on the parameters used in lane change models (an integral part of microscopic traffic simulation models), namely, the duration of the lane change maneuver and the mean longitudinal distance to perform a lane change (look ahead distance).

This study is a part of the on-going Research Project 07112 by the University Transportation Center of Alabama (UTCA), “Characterization of Arterial Traffic Congestion”. The broad objective of this research is to investigate the changes between undersaturated and saturated traffic conditions in terms of performance measures (delay) and operational parameters (gap acceptance and lane changing). This paper contributes to the research by exploring the sensitivity of lane changing parameters during undersaturated and saturated traffic conditions.

4.3 Background and Motivation

Driving is a highly complex task that requires continual integration of perception, cognition, and motor response; and lane changing is one subtask that incorporates many of these critical aspects of driving (1). Lane changes have a significant impact on the characteristics of traffic flow (2). It is suspected that lane change maneuvers can be a source of traffic flow disruption and thereby contributing to congestion and delays (3). Hence, sufficient care must be taken while modeling lane changes.

Lane change models form an integral and important part of microscopic traffic simulation. Hence, a great number of studies were conducted on the characteristics and modeling of a lane change process (2, 4, 5, 6, and 7). The functioning of lane change models is usually governed by driver behavior parameters such as duration of lane changes and longitudinal distance to perform a lane change. With increase in the usage and popularity of microscopic traffic simulation, the quest for accurate and reliable lane change models emerged to describe this aspect of driving activity.

Existing models of lane changing behavior emphasize the decision-making aspects of the task, but generally neglect the detailed modeling of the lane changing action itself and only model it as an instantaneous event (2). Some of the commonly used simulation software packages, such as SimTraffic and AIMSUN, do not explicitly state the duration of lane changes (8 and 9). CORSIM has a three second default value as the duration of lane change (10). However, research (2, 4, and 6) has shown that lane change duration is not an instantaneous process or a constant value, instead, it follows a range of values and a specific distribution based on locally specific conditions (geometry, traffic levels, speeds, etc.). It has been documented based on previous field studies that the mean lane change duration is 4 to 6 seconds with ranges

from 3 to 8.5 seconds (2, 4, and 6). Hence, it was expected that these values of lane change duration, when incorporated into the existing lane change models, will result in drastic variation in the results or MOEs (Measures of Effectiveness) of the simulation from those currently incorporated into the models.

The other parameter considered in this study, the longitudinal distance to perform a lane change, refers to the distance required for a vehicle to contemplate and perform one lane change (10). In CORSIM, the default value for this parameter is between 225 and 375 feet (69 and 114 meters) based upon the driver type (10). Similarly, SimTraffic addresses this condition with two parameters; mandatory distance (the distance back from the stop bar where a lane change must commence) and positioning distance (the distance back from the mandatory point where a vehicle first attempts to change lanes). These two parameters in SimTraffic also have default values involving speed, cycle length and driver type. In AIMSUN, the longitudinal distance to perform a lane change is governed by defining zones. Zone 1 is the farthest distance from the next turning point. The lane changing decisions are mainly governed by the traffic conditions of the lanes involved. Zone 2 is the intermediate zone. It is mainly the desired turning lane that affects the lane-changing decision. Zone 3 is the shortest distance to the next turning point. Vehicles are forced to reach their desired turning lanes, reducing speed if necessary, and even coming to a complete stop in order to make the lane change.

Previous studies suggest that lane change duration is a range and not a constant value. However, no studies were identified which explored the changes in the simulated results by using a range of values for lane change duration rather than keeping it constant. Studies dealing with the longitudinal distance to make a lane change were not identified during the preparation of this paper. Hence, in this study, a sensitivity analysis of the above mentioned parameters in

microscopic traffic simulation is performed. The authors anticipated that the findings from this study will include recommendations for the improvement of the existing lane change models in microscopic traffic simulation in terms of model calibration and accuracy of results i.e., MOEs.

4.4 Simulation Model

CORSIM 5.1 was selected for this study. It was developed for the Federal Highway Administration (FHWA) and distributed by McTrans. All the simulation models addressed in the literature (CORSIM, SimTraffic, and AIMSUN) have their own lane changing algorithms for performing lane changes within the traffic stream. However, CORSIM had unique characteristics that led to its selection for this study,

- CORSIM is reported to be the most widely used micro-simulation program in the U.S. (11).
- CORSIM allows the user to calibrate a comprehensive set of parameters (acceleration/deceleration, headway, look-ahead distance etc.) affecting the lane changing process.
- CORSIM allows the user to modify the default values of lane change maneuver duration and the mean longitudinal distance for making a lane change.

4.5 Study Road and Data Set

The road network chosen for the study is McFarland Boulevard (US-82), a six-lane, congested, main traffic artery located in Tuscaloosa, Alabama. The study stretch as shown in Figure 4.1 extends 2.3 miles from Skyland Boulevard to 13th Street. There were nine intersections, three major and six minor, all of which are controlled by actuated and coordinated signals. Most of the segments between the intersections are long enough, about 1,000 to 3,000 feet (305 to 915 meters) to facilitate lane changes.

The volume data (turning movements), signal timing, phasing data, and geometrics of the study road were available as a Synchro file from a previous study done at the UTCA. This Synchro file was exported to CORSIM. In order to represent a saturated traffic condition, the PM peak hour volume was used. To represent a free flow condition the PM peak volume was reduced by 50%.

Figure 4.1 Study Road

4.6 Simulation Runs

The CORSIM file was simulated with 10 repetitions and a mean value of the MOEs under consideration were obtained. A simulation time of 60 minutes for each run was chosen.

For the lane change duration parameter, two values were used for the sensitivity analysis, namely 3 and 7 seconds. These values were selected based on the default value in CORSIM, as well as from the available literature (2, 4, and 6). Similarly, for the other parameter, the mean longitudinal distance to make a lane change, the two values selected were 300 and 2,500 feet (92 and 763 meters). The test values lane change duration and look ahead distance parameter were widely separated to make it easier to isolate the effects of the change in the parameters, as compared to the default values embedded in CORSIM.

4.7 Investigation Steps

4.7.1 Step I

This step focused on the sensitivity of lane change durations. It involved simulating the study road network with two different lane change durations mentioned earlier (i.e., 3 and 7 seconds). All other parameters were unchanged during the simulation. For this step, the volume data corresponding to the PM peak hour was used. The MOEs such as vehicle miles, total time, delay, and average speed of the entire network were obtained for 10 simulation runs. Network MOEs were considered because it was thought that changes in lane change durations may affect the changes in MOEs for the entire network. Table 4.1 shows the mean, standard deviation, and difference between the means of the MOEs taken into consideration.

Table 4.1 Simulation Results with Two Different Lane Change Durations

MOE	LC Duration – 3 sec		LC Duration – 7sec		Difference in Mean (%)
	Mean	STDEV	Mean	STDEV	
Vehicle Miles	38165.88	213.87	38282.27	225.47	0.31
Delay (veh-hrs)	701.66	8.12	705.09	8.61	0.49
Total Time (veh-hrs)	1670.16	12.01	1675.91	12.68	0.34
Avg. Speed (mph)	22.85	0.07	22.84	0.08	-0.03

From Table 4.1, it can be observed that with an increase in the lane change duration from 3 to 7 seconds, there is a marginal increase in the vehicle miles, delay and total time, and a

marginal decrease in the average speed. The observed trend is as expected. However, there is a no statistically significant difference between the two results. A t-test for determining the difference in means proved that the two sets of results are not different at a 90% confidence interval. This step involved comparing network level MOEs. Hence, in the next attempt MOEs of each link were taken into consideration. It was expected that link level MOEs obtained from sensitivity analysis of test parameters may yield statistically different results.

4.7.2 Step II

This step involves link level analysis, the MOEs considered were number of lane changes, through movement delay, and left movement delay. Table 4.2 shows the mean number of lane changes, mean delay of through traffic, and mean delay of left turning traffic traveling south for each segment in the study road. The table also includes the difference (%) between the mean values obtained from the two lane change durations, and the result of hypothesis testing for the null or alternate hypothesis. Hypothesis testing for the difference between the means was conducted with a t-test at a 90% confidence interval; the null and alternate hypothesis are defined as follows,

H_0: There is no difference between the MOEs obtained for different lane change duration or look ahead distance.

H_A: There is difference between the MOEs obtained for different lane change duration or look ahead distance.

Table 4.2 MOEs of All the Links in the Study Road

Link/Stretch in US-82	Mean Number of LCs			H_0 or H_A	Mean Thru Delay (vehicle minuets)			H_0 or H_A	Mean Left Delay (vehicle minuets)			H_0 or H_A
	LCD = 3 sec	LCD = 7 sec	Diff (%)		LCD = 3 sec	LCD = 7 sec	Diff (%)		LCD = 3 sec	LCD = 7 sec	Diff (%)	
North of 13th St	1370	1364	-0.4	H_0	561	562	0.2	H_0	134	131	-1.6	H_0
13th and 15th St	1298	1305	0.5	H_0	701	676	-3.6	H_0	719	711	-1.1	H_0
U-Mall and 15th St	1014	1003	-1.1	H_0	533	542	1.8	H_0	174	172	-1.2	H_0
Hargrove and Mall	1500	1480	-1.3	H_0	975	1005	3.0	H_0	383	378	-1.3	H_0
31st and Hargrove	1171	1126	-3.9	H_A	274	277	1.3	H_0	7	7	0.0	H_0
31st and 37th St	903	897	-0.6	H_0	507	512	1.1	H_0	103	103	0.0	H_0
37th and Ramp	514	463	-9.9	H_A	92	91	-0.1	H_0	0	0	0.0	H_0
Ramp and 37th St	1078	999	-7.3	H_A	196	197	0.7	H_0	0	0	0.0	H_0
Between I-20 Ramps	1338	1302	-2.7	H_0	194	216	11.6	H_A	265	264	-0.2	H_0
Mall and I-20 Ramp	1367	1350	-1.2	H_0	157	186	18.8	H_A	399	394	-1.3	H_0
Skyland and Mall	1176	1314	11.7	H_A	361	422	17.0	H_A	873	886	1.4	H_0

The results in Table 4.2 indicate that, in general, the mean number of lane changes decreased with an increase in lane change duration, and the mean values of through movement delay increased with an increase in lane change duration. The hypothesis testing proved that the mean numbers of lane changes for four stretches in the study road with a 7 second lane change duration are statistically different, and less than those obtained with a 3 second lane change duration. These observations were as expected. However, there were a few stretches that did not adhere to the general trend (refer to the shaded values on Table 4.2). These stretches either showed an increase in the number of lane changes with increased lane change duration, or a decrease in the mean through delay with increased lane change duration. Also, the trend was mixed for mean values of left-turn movement delay, with increased lane change duration. To understand this, the percentage of southbound turning movements was evaluated (Table 4.3). From this table, It was found that the proportion of turning movements (left and right turns) at those stretches which deviated from the general trend were considerably higher than those of other stretches.

Table 4.3 Percentage of Turning Movements and Volumes for All Links in the Study Road

Link/Stretch in US-82	Turning Movements (%)			Volume (vehicles per hour)
	Left	Thru	Right	
North of 13th St	9	90	1	1956
Between 13th and 15th St	25	63	12	1830
Between Mall and 15th St	8	92	0	1893
Between Hargrove Rd and Mall	12	82	6	2045
Between 31st St and Hargrove Rd	1	91	8	1970
Between 31st and 37th St	4	91	5	1905
Between 37th St and Ramp	0	100	0	1964
Between Ramp and 37th St	0	83	17	1964
Between I-20 Ramps	14	86	0	1776
Between Mall and I-20 Ramp	13	81	6	2059
Between Skyland Blvd and Mall	38	47	15	1503

From the road network, it was also observed that the length of two arterial sections, between I-20 and the Mall and the one between the Mall and Skyland Boulevard, were relatively small but had a high number of lane changes. To give additional insight into traffic behavior of these locations a visual observation was conducted as the simulation in CORSIM was underway. The simulation with each lane change duration was observed separately. The visual observation led to an interesting finding which is described in the following paragraph.

When some of the vehicles turning left or right received instruction from the simulation model (as indicated by a change in the color of vehicles on the display screen) to change lanes for the storage bay, the bay was already queued. These vehicles stopped in the middle of a through lane, causing the following vehicles to change lanes to pass them, and then, to move back to their original lanes. This is a clear case where the simulation model provided insufficient longitudinal distance to make a lane change. The default value for the mean longitudinal distance to make a lane change in CORSIM is 300 feet and this value can go as low as 225 feet because of the decile distribution used for this parameter. This parameter coupled with an increased lane change duration value of 7 seconds could have resulted in the increased number of lane changes

for the stretches which deviated from the general trend. Visual observation of the simulation also showed that this condition was more prevalent with increased lane change duration. However, visual observation failed to shed light on the anomaly with respect to delay in one of the stretches where increased lane change duration resulted in decreased delay.

Because the road network was simulated with the PM peak volume data, it was suspected that the anomalous results may be due to saturation of some of the traffic signals in the study road. Hence the authors thought that a comparison of results between saturated and free flow conditions could help explain the anomalous results obtained in this step. As mentioned earlier, the volume of the free flow network was 50% of that of a saturated condition. Hence, in the next step, a comparison of simulation results between saturated and free flow conditions was performed with a much smaller network to simplify the analysis.

4.7.3 Step III

The reduced network consisted of two intersections, the link between them and one link beyond both of them. Lane change durations of 3 and 7 seconds, and the look ahead distances of 300 and 2500 feet were analyzed. Simulation runs were performed for a saturated network (with PM peak hour volume data) and a network with free flow conditions (half of the PM peak hour volume). The experimental design for performing this sensitivity analysis is shown in Figure 4.2.

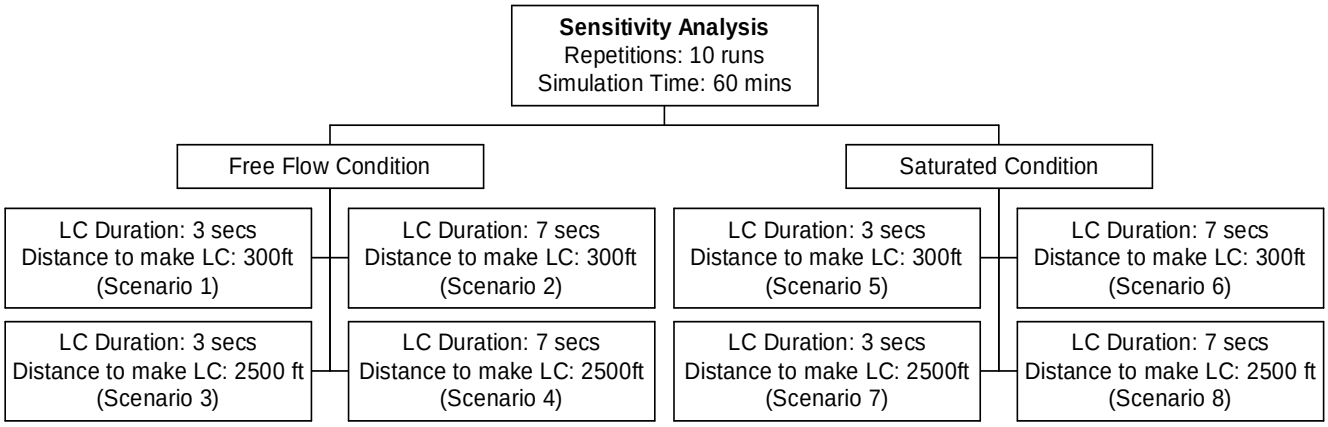

Figure 4.2 Experimental Design Setup for Sensitivity Analysis of Selected Parameters to a Saturated and a Free Flow Network

Eight scenarios were performed as shown in Figure 4.2. For each scenario the number of lane changes and through movement delay on the links of the reduced road network were obtained. Tables 4.4 and 4.5 show the mean number of lane changes for scenarios 1 to 4, and scenarios 5 to 8, respectively.

Table 4.4 Mean Number of Lane Changes from Sensitivity Analysis of Lane Change Duration and Look Ahead Distance in a Free Flow Condition

Link/Stretch in US-82	Mean Number of LCs											
	Testing for Lane Change Duration						Testing for Look Ahead Distance					
	Scenario1 Free flow 3 s, 300'	Scenario2 Free flow 7 s, 300'	Diff (%)	Scenario3 Free flow 3 s, 2500'	Scenario4 Free flow 7 s, 2500'	Diff (%)	Scenario1 Free flow 3 s, 300'	Scenario3 Free flow 3 s, 2500'	Diff (%)	Scenario2 Free flow 7 s, 300'	Scenario4 Free flow 7 s, 2500'	Diff (%)
South of UMall, NB	306	298	-2.5	316	305	-3.6	306	316	3.3	298	305	2.3
UMall - 15th, NB	418	417	-0.2	371	371	0.0	418	371	-11.0	417	371	-11.0
North of 15th Street, NB	123	113	-8.2	123	117	-4.7	123	123	0.0	113	117	3.0
North of 15th Street, SB	512	512	0.0	514	502	-2.4	512	514	0.4	512	502	-2.0
UMall - 15th, SB	327	331	1.3	325	313	-3.6	327	325	-0.6	331	313	-5.4
South of UMall, SB	198	195	-1.7	196	190	-2.8	198	196	-1.0	195	190	-2.6

In Table 4.4, the sensitivity testing of lane change duration for a free flow condition include the comparison of results from scenarios 1 and 2 and scenarios 3 and 4. As shown in Figure 4.2, in scenarios 1 and 2, the look ahead parameter was kept constant at 300 feet but lane change durations varied (3 seconds for scenario 1 representing faster lane changes and 7 seconds for scenario 2 representing slower lane changes). Scenarios 3 and 4 are similar to scenarios 1 and 2 except that the look ahead parameter is held constant at 2,500 feet. From this testing, it can be observed that irrespective of the look ahead distance, increased lane change duration or slower lane changes resulted in a decrease in the number of lane changes. This is the expected result. A comparison of results from scenarios 1 and 2 shows that in four out of six links the number of lane changes decreased as lane change duration increased. Similarly, considering scenarios 3 and 4, the number of lane changes for five out of six links decreased with an increased lane change duration.

The sensitivity testing of look ahead distance involved holding the lane change duration constant, and varying the look ahead parameter. The results of this test include comparison of scenarios 1 and 3 and scenarios 2 and 4. They indicate that an increased look

ahead distance resulted in mixed results, i.e., with an increased look ahead distance, some links showed a decrease in the number of lane changes, and for other links, the number of lane changes increased. Between scenarios 1 and 3, only three out of six links showed a decrease in the number of lane changes with increased look ahead distance. Similarly, between scenarios 2 and 4, four out of six links showed a decrease in the number of lane changes with increased look ahead distance.

Table 4.5 Mean Number of Lane Changes from Sensitivity Analysis of Lane Change Duration and Look Ahead Distance in a Saturated Condition

Link/Stretch in US-82	Mean Number of LCs											
	Testing for Lane Change Duration						Testing for Look Ahead Distance					
	Scenario5 Saturated 3 s, 300’	Scenario6 Saturated 7 s, 300’	Diff (%)	Scenario7 Saturated 3 s, 2500’	Scenario8 Saturated 7 s, 2500’	Diff (%)	Scenario5 Saturated 3 s, 300’	Scenario7 Saturated 3 s, 2500’	Diff (%)	Scenario6 Saturated 7 s, 300’	Scenario8 Saturated 7 s, 2500’	Diff (%)
South of UMall, NB	688	684	-0.6	676	667	-1.4	688	676	-1.7	684	667	-2.5
UMall - 15th, NB	1163	1162	-0.1	1031	1044	1.3	1163	1031	-11.3	1162	1044	-10.2
North of 15th Street, NB	306	282	-7.8	298	287	-3.6	306	298	-2.6	282	287	1.8
North of 15th Street, SB	1209	1243	2.8	1204	1231	2.3	1209	1204	-0.4	1243	1231	-1.0
UMall - 15th, SB	927	924	-0.3	902	893	-0.9	927	902	-2.7	924	893	-3.4
South of UMall, SB	485	456	-6.0	473	452	-4.5	485	473	-2.5	456	452	-0.9

The results in Table 4.5 obtained from saturated traffic flow are similar to those obtained from Table 4.4 for free flow conditions. The sensitivity testing for lane change duration indicated that increased lane change duration resulted in a decrease in the number of lane changes. A comparison of results from scenarios 5 and 6 shows that a higher lane changing time resulted in a decreased number of lane changes. The mean number of lane changes decreased for five out of six links when lane change duration increased. The same trend was observed in four out of six links while comparing results from scenarios 7 and 8. Similarly, the sensitivity testing for the look ahead parameter indicated that an increased look ahead parameter resulted in a decrease in the number

of lane changes. In other words, when drivers in CORSIM had longer distances to decide to change lanes, they changed lanes fewer times. By comparing scenarios 5 and 7, it can be observed that all the links showed a decrease in the number of lane changes with increased look ahead distance. Between scenarios 6 and 8, the number of lane changes decreased for five out of six links, when look ahead distance increased. The southward traffic movement on the US-82 link, north of 15th street, deviated from the general trend. In this link, the number of lane changes actually increased with increase in the duration of lane changes. As explained earlier in step II, this counter intuitive result in this link could be attributed to two reasons. The first one is insufficient look ahead distance; 300 feet is less than the storage bay length for left turns. The other reason is that the queue on the left turn bay spills back to the through lane, causing drastic variations in driver behavior in the simulation model.

The observations from tables 4.4 and 4.5 generally matched what was intuitively expected,

- Slower lane changes need relatively larger gaps when compared to gaps needed for faster lane changes. Hence, an increased lane change duration (slower lane changes) resulted in a reduced number of lane changes.
- The look ahead distance governs lane changing to reach a specific goal lane. Perhaps, an increased look ahead distance directs drivers to change lanes at a more upstream location thus reducing the opportunity for any desired lane changes further downstream till the turn movement. Hence, an increased look ahead distance results in a reduced number of lane changes.

Table 4.6 shows the mean through delay for scenarios 1 to 4. The sensitivity testing for lane change duration indicated that in a free flow traffic condition, faster or slower lane changes did not increase or decrease the through movements delay significantly. A comparison of results between scenarios 1 and 2 and scenarios 3 and 4 of Table 4.6 shows that the through delay at all the six links did not increase or decrease significantly (the change in mean through delay was less than 1%). Similarly, the look ahead distance did not have any significant impact on the through delay for a free flow traffic condition. The testing for look ahead distance in Table 4.6 shows that five out of six links had changes in mean through delay of less that 1%. It can be noted that the changes observed in the number of lane changes in Table 4.4 had a minimal effect on the through movement delays.

Table 4.6 Mean Through Delay Values from Sensitivity Analysis of Lane Change Duration and Look Ahead Distance in a Free Flow Condition

Link/Stretch in US-82	Mean Through Delay (vehicle minutes)											
	Testing for Lane Change Duration						Testing for Look Ahead Distance					
	Scenario1 Free flow 3 s, 300'	Scenario2 Free flow 7 s, 300'	Diff (%)	Scenario3 Free flow 3 s, 2500'	Scenario4 Free flow 7 s, 2500'	Diff (%)	Scenario1 Free flow 3 s, 300'	Scenario3 Free flow 3 s, 2500'	Diff (%)	Scenario2 Free flow 7 s, 300'	Scenario4 Free flow 7 s, 2500'	Diff (%)
South of UMall, NB	68.5	68.6	0.1	69.6	69.4	-0.3	68.5	69.6	1.6	68.6	69.4	1.2
UMall - 15th, NB	171.8	171.7	-0.1	171.8	171.1	-0.4	171.8	171.8	0.0	171.7	171.1	-0.3
North of 15th Street, NB	38.2	38.3	0.3	38.2	38.4	0.5	38.2	38.2	0.0	38.3	38.4	0.3
North of 15th Street, SB	176.9	176.8	-0.1	178	176.5	-0.8	176.9	178	0.6	176.8	176.5	-0.2
UMall - 15th, SB	86.1	86	-0.1	85.8	85.5	-0.3	86.1	85.8	-0.3	86	85.5	-0.6
South of UMall, SB	41.3	41.5	0.5	41.2	41.5	0.7	41.3	41.2	-0.2	41.5	41.5	0.0

Similarly, from Table 4.7, the sensitivity testing of lane change duration for a saturated traffic condition shows that irrespective of the look ahead distance, slower lane changes resulted in increased through delay. This is intuitive (slower lane changes increase travel time and hence increase delay). In Table 4.7, a comparison between the results of scenario 5 and 6 shows that the mean through

delay increased for four out of six links. The same comparison between scenarios 7 and 8 showed that the mean through delay increased for five out of six links. The sensitivity testing for look ahead distance yielded mixed results.

Table 4.7 Mean Through Delay Values from Sensitivity Analysis of Lane Change Duration and Look Ahead Distance in a Saturated Condition

Link/Stretch in US-82	Mean Through Delay (vehicle minutes)											
	Testing for Lane Change Duration						Testing for Look Ahead Distance					
	Scenario5 Saturated 3 s, 300'	Scenario6 Saturated 7 s, 300'	Diff (%)	Scenario7 Saturated 3 s, 2500'	Scenario8 Saturated 7 s, 2500'	Diff (%)	Scenario5 Saturated 3 s, 300'	Scenario7 Saturated 3 s, 2500'	Diff (%)	Scenario6 Saturated 7 s, 300'	Scenario8 Saturated 7 s, 2500'	Diff (%)
South of UMall, NB	294.4	295.0	0.2	299.6	301.8	0.7	294.4	299.6	1.8	295.0	301.8	2.3
UMall - 15th, NB	719.5	707.6	-1.7	704.5	702.7	-0.3	719.5	704.5	-2.1	707.6	702.7	-0.7
North of 15th Street, NB	92.4	93.1	0.8	92.7	92.9	0.2	92.4	92.7	0.3	93.1	92.9	-0.2
North of 15th Street, SB	795.4	814.8	2.4	798.7	818.5	2.5	795.4	798.7	0.4	814.8	818.5	0.5
UMall and 15th, SB	497.3	503.5	1.2	495.7	497.7	0.4	497.3	495.7	-0.3	503.5	497.7	-1.2
South of UMall, SB	126.1	125.4	-0.6	125	125.7	0.6	126.1	125	-0.9	125.4	125.7	0.2

From the sensitivity analysis of lane change duration and look ahead distance in both free flow and saturated traffic condition, the following broad observations were drawn,

- Lane changing parameters are less sensitive to free flow traffic conditions than congested conditions. A change in the lane change duration or the look ahead distance does not significantly affect the results of microscopic traffic simulation.
- Saturated traffic conditions are complex. Sensitivity analyses of lane changing parameters in saturated conditions yielded mixed results, thus clear patterns could not be drawn.
- Speed of lane changing is easier to interpret than look ahead distance. Irrespective of the look ahead distance, increased lane change durations resulted in decreases in the number of lane changes and increases in delay.

Apart from these sensitivity analyses, an attempt was made to study the combined effect of speed of lane changes and look ahead distance; for instance, the simulation results obtained using 3 seconds lane change duration and 300 feet look ahead distance were compared with that obtained using 7 seconds lane change duration and 2500 feet look ahead distance. However, the comparison did now show any clear pattern. The investigators concluded that it would take a more rigorous analysis to identify the effects of changes to multiple parameters.

4.8 Extension of the Understanding of Lane Changing Parameters in CORSIM to Other Simulation Tools

CORSIM was used for this study, but since the lane changing algorithms in other simulation tools are similar in principle, an attempt was made to extend the findings of this study to these simulation tools. Due to time and resource constraint only SimTraffic and AIMSUN were considered. The findings are listed below,

- In SimTraffic, ‘positioning distance’ is similar to the mean longitudinal distance to make a lane change in CORSIM. This positioning distance is determined using the formula Max(300’, v * 30s) where ‘v’ is the speed of the vehicle. For instance, assuming the range of speed for vehicles to be within 30 to 50 mph for an urban street, the range of the parameter is obtained as 1,320 to 2,200 feet. This range appears to be much larger than the length of most of the storage bays for turning movements on urban arterials. Hence, it can be expected that the lane changing issues relating to insufficient length of storage bays during queuing conditions will not arise. The other parameter, the duration of lane change, could not be found in the SimTraffic user manual.

- Similarly, in AIMSUN, ‘Zone 2’ is the parameter which is equivalent to the positioning distance in SimTraffic and mean longitudinal distance to make a lane change in CORSIM. Zone 2 is calculated using the following formula,

$$D_m = D_t . S_{\lim it}(s) . \left[\frac{S_{\lim it}(s)}{v_{\max}(i,s)} \right] \quad \text{………..……………………………..…………………(4.1)}$$

where,

D_m : Distance in meters

D_t : Distance in seconds (default value 23 seconds)

S_{limit} : Speed limit of the section ‘s’

v_{max} (i, s) : Maximum desired speed of vehicle ‘i’ on a section or turning ‘s’

In this function, faster vehicles (with respect to the speed limit) require shorter zones than slower vehicles. Assuming an urban street with a speed limit of 50 mph and the speed of a vehicle ranging between 40 and 60 mph, the value of the parameter ‘Zone 2’ will be between 2,110 and 1,410 feet, respectively. This range of the parameter appears to be much larger than most of the storage bay lengths for turning movements on urban arterials. In this case, it appears that the lane changing issues relating to insufficient length of storage bays during queuing conditions will not arise. The other parameter, namely the duration of lane change or any other equivalent parameter, could not be found in the AIMSUN user manual.

4.9 Conclusions

This study explored the sensitivity of two lane changing parameters, the lane change duration, and the mean longitudinal distance for making lane changes, in saturated and free flow traffic conditions. The following conclusions were drawn from the study of one location,

- The MOEs are sensitive to lane change duration. An increase in lane change duration resulted in a decrease in the number of lane changes on the links of the study road. Since the second research step and earlier studies have shown that lane change duration follows a specific distribution, it seems reasonable that the current simulation lane changing models be calibrated with the specific distribution of lane change duration of the location that is being studied.
- The MOEs are also sensitive to the mean longitudinal distance needed to make lane changes. Since this parameter affects vehicles changing lanes in preparation for a turn (left or right) at the approaching intersection, the mean value of this parameter should be larger than the length of storage bays for turning movements so that vehicles can change lanes well ahead of approaching the turning bay, thus reflecting field conditions, particularly during saturated traffic flow. For instance, the default value of the mean longitudinal distance to make a lane change in CORSIM is only 300 feet, and this value can be as small as 225 feet because of the decile distribution used for this parameter. Some of the storage bays for turning movements may be longer than 225 feet; so some vehicles may not even receive instructions to change lanes even while nearing the turn bay.
- For saturated links in general, and particularly those with a high percentage of turning movements, it was difficult to study the trend in MOEs with changes to lane change parameters. This could be because of high interaction between vehicles in such traffic conditions, but the researchers were not able to establish such a finding.

- Lane changing parameters are relatively less sensitive during free flow traffic conditions. A change in the lane change duration or the look ahead distance does not significantly affect the results of microscopic traffic simulation.
- Speed of lane changing is easier to interpret than look ahead distance. Irrespective of the look ahead distance, an increased lane change duration resulted in a decrease in the number of lane changes or an increase in delay.
- Observations from sensitivity analysis of lane change parameters showed what was thought intuitively i.e., slower lane changes or higher look ahead distances resulted in lesser number of lane changes.
- For a saturated network, the two parameters considered in this study alone cannot explain the results. The anomalies and mixed results observed in the sensitivity analysis may be explained with more rigorous analysis with different parameters governing the lane changing process.

4.10 Future Research

Though this study showed that the number of lane changing and delay are sensitive to changes in lane change parameters, due to time constraints, the model was not validated with field results. Hence, further work should be done to calibrate the model with lane changing duration and longitudinal distance parameter for making lane changes, reflecting field conditions. They should also be validated for the same model with field results. Other lane changing parameters such as minimum deceleration for lane changing, mean time for a driver to react to a sudden deceleration of the lead vehicle, headway below which all drivers will attempt to change lanes, etc., should be explored particularly in saturated traffic conditions.

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CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 Summary

This dissertation research is a part of on-going UTCA project 07112 “Characterizing Arterial Traffic Congestion”. The objective of the UTCA project is to investigate the changes between undersaturated and saturated traffic conditions at arterial signals in terms of performance measures (delay) and operational parameters (gap acceptance and lane changing).

This dissertation research utilized microscopic traffic simulation and field data to investigate free flow and saturated traffic conditions of traffic signals on arterial highways. Despite the robustness and wide spread use of traffic microsimulation models for this type of analysis, some gaps and limitations still exist that can affect the accuracy of the results produced by the models. Also, changes in driver behavior parameters such as lane changing and gap acceptance under different traffic conditions are not completely understood. Hence, the purpose of this study was to enhance the understanding of traffic characteristics and driver behavior parameters such as lane changing and gap acceptance under different traffic conditions to improve the accuracy of microscopic traffic simulation, particularly, while simulating saturated traffic conditions.

5.2 Conclusions

This dissertation research was conducted on a single test site. It is likely that driver behavior, signal system design, and roadway geometry influenced the results of this research. Hence these results only represent the site considered in this study and are not generic. The entire dissertation research involved three related research efforts, each conducted along the topic of this dissertation. The specific findings of each research effort are given in the next section. The following are the broad findings of the entire dissertation,

- Traffic flows at signals that are approaching saturation are still complex to analyze, and the interactions between traffic parameters are not well understood.
- When traffic flow on a typical arterial approaches saturation, drivers take higher risks. For instance, drivers accept smaller gaps.
- A statistical analysis of gap acceptance and lane changing behavior confirmed what is suspected intuitively, i.e., the distributions of gap acceptance and lane change durations.
- Existing microscopic traffic simulation tools simplify some of the traffic parameters in simulation models. These parameters may be recoded or recalibrated for better accuracy of simulation results.
- An increased number of repetitive simulation runs helps in achieving stable standard deviation of MOEs. Further, the level of congestion affects the required number of simulation runs needed for the MOE variability to reach a steady state.
- In microscopic traffic simulation, it is advisable to use a longer simulation times (eg. 60 minutes) to reduce the variation of MOEs when repetitive runs are used.

5.2.1 Findings from First Research Emphasis Area

The first research effort of this dissertation investigated the sensitivity of microscopic traffic simulation MOEs to initialization time and number of repetitions. It also explored the parameters responsible for variability in MOEs. The major findings of this part of the research drawn from study of a single arterial are given below,

- MOEs are indeed sensitive to initialization times of simulation. A change in simulation initialization time produces changes in the MOEs, and the larger the difference in initialization times (say 7 and 13 minutes instead of 7 and 10 minutes), the larger the difference between their resulting MOEs.
- There are large variations in any MOE between approaches at the same intersection (a typical cross intersection has four approaches). Hence, a separate sensitivity analysis of each approach of an intersection was considered a better way of analyzing an intersection.
- The variation of MOEs for repetitive runs rises as congestion increases. In other words, the deviation of MOEs for repetitive runs with a saturated traffic condition is much larger than that obtained for a free flow traffic condition.
- An increased number of simulation runs certainly helps in stabilizing the variability of the MOE. In addition to that, it appears that the level of congestion also has an impact on the number of simulation runs needed to obtain a stable standard deviation for simulation results. For approaches with better LOS values such as LOS B and C, the standard deviation reached a stable state after about 25 runs, but for lower values of LOSs most of the approaches stabilized at a much higher number of repetitions. For approaches with LOS D, the number of simulation runs needed to stabilize the MOE variability could not

be clearly identified. The reason could be that LOS D represents the transition phase between congested and free flow traffic conditions.

- The length of the simulation time makes a considerable difference in the simulation results. A larger simulation time decreases the variance in MOEs for undersaturated conditions. For near saturation and saturated conditions (LOS E and F), an increase in simulation time does not necessarily decrease the variance in MOEs (this observation was based on only a few data points on a single arterial). These observations should be considered before choosing a simulation time length.
- Lane change parameters play a major role as a source of variation of MOEs (delay/vehicle) obtained for different random seeds. But this is not conclusive for congested conditions (LOS E and F). Hence, it is important to check these parameters before the simulation to ensure that they reflect field conditions.

5.2.2 Findings from Second Research Emphasis Area

The second research step was a field investigation of the variations in gap acceptance and lane changing in relation to different traffic flow rates. It compared the field results with corresponding values in some of the common microscopic traffic simulation models. The key findings of this research effort are given below,

- The sizes of accepted gaps (seconds) are statistically different for free flow and saturated traffic conditions. Specifically, the mean values of accepted gaps ranged from 4.33 to 4.04 seconds for average flow rates of 20 and 80 vehicles per minute, and standard deviations were around 1.6 seconds. A hypothesis test proved that the extreme values (4.33 and 4.04) are statistically different.

- With increased flow rate, there was an increase in the number of drivers accepting smaller gaps. In other words, the median value of accepted gaps for the lowest flow rate (20 vehicles per minute) was 4.33 seconds, and that for the highest flow rate (80 vehicles per minute) was 3.74 seconds, respectively.
- The shape and skewness of the best fitted distributions for accepted gaps also showed a higher probability of drivers accepting smaller gaps when the traffic flow is heavy.
- The observations of lane change durations were similar to observations of accepted gaps. The lane change durations corresponding to free flow and saturated traffic conditions were statistically different. Specifically, the mean value of the lane change durations ranged from 4.34 to 4.19 seconds for average flow rates of 20 and 80 vehicles/minute, and the standard deviations were around 0.9 seconds. A hypothesis test proved that the extreme values (4.34 and 4.19) are indeed different.
- The variation of lane change durations is less than the variation of accepted gaps, which indicates that gap acceptance is more sensitive than lane change duration.
- The median values of lane change duration irrespective of the flow rate were around 4.25 seconds, which shows that 50% of the drivers took 4.25 seconds to change lanes.
- The shape and skewness of the best fitted distributions of lane change durations showed that the probability of drivers changing lanes more rapidly is marginally higher when traffic flow is heavy.
- The rate of lane changes decreased with increased traffic flow, probably because the number of available gaps decreased with increased traffic flow. Also, since there is less available gaps during higher traffic flow or saturated flow conditions, drivers accept smaller gaps.

- To summarize the findings of this part of the research, it can be observed that when traffic flow on a typical arterial approaches saturation, a considerable number of drivers accept smaller gaps and change lanes more rapidly. In other words, drivers take higher risks to change lanes during congested traffic flow.
- All the above findings in this research have direct implications upon the gap acceptance and lane changing parameters used in microscopic traffic simulation, particularly during model calibration. The default values of lane changing and gap acceptance parameters used in some of the common traffic simulation tools are different from those obtained in this study. For instance, CORSIM has a default lane change duration value of 3 seconds while the results obtained in this study show that the mean value of lane change duration varied from 4.19 to 4.34 seconds depending on the traffic flow rate. Moreover, the gap acceptance (while changing lanes) distributions obtained in this study may be used in lane change models in microscopic traffic simulation models.

5.2.3 Findings from Third Research Emphasis Area

The final research effort investigated the sensitivity of lane change parameters, including lane change duration and look ahead distance, to free flow and saturated traffic flow in microscopic traffic simulation models. The major findings of this part of the research, drawn from a simulation of a single arterial, are given below,

- The MOEs are sensitive to lane change duration. An increase in lane change duration resulted in a decrease in the number of lane changes on the links of the study road. Since the second research step and earlier studies have shown that lane change duration follows a specific distribution, it seems reasonable that the current simulation lane changing

models be calibrated with the specific distribution of lane change duration of the location that is being studied.

- The MOEs are also sensitive to mean longitudinal distance needed to make lane changes. Since this parameter affects vehicles changing lanes in preparation for a turn (left or right) at the approaching intersection, the mean value of this parameter should be larger than the length of storage bays for turning movements, so that vehicles can change lanes well ahead of approaching the turning bay, thus reflecting field conditions, particularly during saturated traffic flow. For instance, the default value of mean longitudinal distance to make a lane change in CORSIM is only 300 feet and this value can be as small as 225 feet because of the decile distribution used for this parameter. Some of the storage bays for turning movements may be longer than 225 feet; so some vehicles may not even receive instructions to change lanes even while nearing the turn bay.
- For saturated links in general, and particularly those with a high percentage of turning movements, it was very difficult to study the trend in MOEs caused by changes to lane change parameters. This could be because of high interaction between vehicles in such traffic conditions, but the researchers were not able to establish such a finding.
- Lane changing parameters are relatively less sensitive during free flow traffic conditions. A change in the lane change duration or the look ahead distance does not significantly affect the results of microscopic traffic simulation.
- Speed of lane changing is easier to interpret than look ahead distance. Irrespective of the look ahead distance, an increase in lane change duration resulted in a decrease in the number of lane changes or an increase in delay.

- Observations from a sensitivity analysis of lane change parameters showed what was thought intuitively, i.e., slower lane changes or higher look ahead distances resulted in less lane changes.
- For a saturated network, the two parameters considered in this study alone cannot explain the results. The anomalies and mixed results observed in the sensitivity analysis may be explained with more rigorous analysis with different parameters governing the lane changing process.
- In SimTraffic and AIMSUN, the range of default look ahead distances appears to be much larger than the length of most of the storage bays for turning movements on urban arterials. Hence, it can be expected that the lane changing issues relating to insufficient length of storage bays during queuing conditions will not arise. However, the other parameter, the duration of lane change, could not be found in the user manual of either of these simulation models.

5.3 Recommendations for Future Research

This dissertation research has advanced knowledge of traffic flow characteristics at busy traffic signals, and has contributed to enhanced accuracy of microscopic traffic simulation under saturated conditions. The research also has led to additional ideas for future research to increase knowledge in this area. These ideas are listed below:

- In microscopic traffic simulation, this dissertation research conducted an approach-wise analysis of an intersection. To further this research, a movement-wise (through, left, right) analysis of the approaches of an intersection is recommended. It is probable that within each approach, the MOEs of through, right, and left movements may be entirely different.

- Since it was difficult to understand the behavior of MOEs under near-saturated and saturated conditions (LOS E & F), further research should particularly focus along these lines.
- Effect of MOE variability with a very high numbers of simulation runs (1000 runs) should be performed to further understand the trend of MOE variability with increased simulation runs and levels of congestion. To perform this, the microscopic models should be enabled to allow extended simulation such as 1000 runs.
- The sample size of collected data for accepted gaps and lane changes should be increased to reduce data-related variation.
- Instead of flow rates, driver behavior parameters including accepted gaps and lane change durations should be compared with more representative parameters such as delay, speed or LOS of the arterial approach under consideration.
- Though this study showed that MOEs are sensitive to changes in lane change parameters, due to time constraints, the model was not validated with field results. Hence, further work should be done to calibrate the model with the lane change parameters (lane changing duration and the longitudinal distance for making lane changes) reflecting field conditions. The model should also be validated with field results obtained from different locations.
- Other lane changing parameters such as minimum deceleration for lane changing, driver reaction times, and headway below which drivers attempt to change lanes, etc., should be explored, particularly in saturated traffic conditions.
- Enhanced statistical methods should be used for analyzing results of microscopic simulation models.

- Since this study was conducted at one single site, the same study could be repeated on other sites and other roadways to verify or extend the findings of this dissertation research.

CHAPTER 6

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