CALIBRATION AND VALIDATION OF PARAMICS MICROSCOPIC SIMULATION MODEL FOR LOCAL TRAFFIC CONDITION IN SAUDI ARABIA

BY

IMRAN REZA

A Thesis Presented to the

DEANSHIP OF GRADUATE STUDIES

KING FAHD UNIVERSITY OF PETROLEUM & MINERALS

DHAHRAN, SAUDI ARABIA

In Partial Fulfillment of the

Requirements for the Degree of

MASTER OF SCIENCE

In

CIVIL AND ENVIRONMENTAL ENGINEERING

OCTOBER 2013

KING FAHD UNIVERSITY OF PETROLEUM AND MINERALS

DHAHRAN 31261, SAUDI ARABIA

DEANSHIP OF GRADUATE STUDIES

This thesis written by **IMRAN REZA** under the direction of his thesis advisor and approved by his thesis committee, has been presented to and accepted by the Dean of Graduate Studies, in partial fulfillment of the requirements for the degree of **MASTER OF SCIENCE IN CIVIL AND ENVIRONMENTAL ENGINEERING**.

Thesis Committee

15 DEC 2013

Dr. Nedal T. Ratrout (Advisor)

Dr. Khalaf Al Ofi (Member)

Dr. Syed Masiur Rahman (Member)

Dr. Nedal T. Ratrout 17 DEC 2013
Chairman of Civil and Environmental Engineering Department

Dr. Salam A. Zummo
Dean of Graduate Studies

21/12/13
Date

عمادة الدراسات العليا
DEANSHIP OF GRADUATE STUDIES

IN THE NAME OF ALLAH, THE MOST GRACIOUS, THE MOST MERCIFUL

Dedicated

to

My Beloved Parents

ACKNOWLEDGEMENTS

All praise is due only to ALLAH subhana wa ta’ aala, the sustainer of the worlds, the most merciful for granting me patience, health and knowledge to complete this work. I would like to thank the authority of King Fahd University of Petroleum and Minerals for providing me the opportunity and financial assistance in pursuing MS program in Civil Engineering.

I acknowledge my sincere appreciation and thanks to Professor Nedal T. Ratrout for his supervision and guidance throughout this research. He has been acting not only as a supervisor but a mentor throughout my stay at KFUPM. I am very much grateful to my thesis committee member Dr. Syed Masiur Rahman for his continuous support and untiring effort in completing this research. I would also like to take this opportunity to thank another of my thesis committee member Dr. Khalaf Al Ofi for his co-operation. I am also obligated to the Department Chairman and other faculty members for their support.

I acknowledge the assistance and help of traffic lab technician Mr. Niaji, my colleagues Mr. Uneb Gazder, Mr. Yazan Issa, Mr. Khalid and few undergraduate students in the context of data collection for this research. I should also mention few other names Dr. Rafiul Hassan, Mr. Imtiaj, Mr. Sujel, Mr Himu, Mr. Anshari, Mr. Mahfuz and many others for taking care of me and for being very generous and supportive. Lastly, special thanks are due to my beloved parents and my sister for their sacrifices, efforts, prayers and encouragement during all the stages of my life. A special thank of appreciation goes to Ms Mahiya Mehrin for according me distant desolate soutenir in adversity.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

THESIS ABSTRACT

Name: IMRAN REZA

Title: CALIBRATION AND VALIDATION OF PARAMICS MICROSCOPIC SIMULATION MODEL FOR LOCAL TRAFFIC CONDITION IN SAUDI ARABIA

Degree: MASTER OF SCIENCE

Major Field: CIVIL AND ENVIRONMENTAL ENGINEERING

The unstable growth of mostly car based transportation system of Saudi Arabia has led to noticeable urban traffic congestion which has emerged as a potential problem in all large metropolitan cities in recent years. Therefore, proper understanding of the unique traffic behavior in this region in order to improve traffic signal operation and proper travel management is inevitable. Simulation modeling is an increasingly popular and effective tool for analyzing transportation problems with the least cost. Recent advancements in computer technology have led to the development of high fidelity microscopic simulation models which is safer, less expensive and faster than field implementation and testing. Testing road designs and traffic control systems, analysis of intelligent transportation systems, evaluating traffic management schemes and calibrating adaptive control systems are important applications of microscopic models. Whilst the models are useful to the profession, they must be calibrated and validated before they can be used to provide realistic results.

The main objective of this study is to calibrate and validate the microscopic traffic simulation model PARAMICS to the traffic conditions in some selected urban arterials in the city of Al-Khobar, Saudi Arabia. PARAMICS is one of the few comprehensive microscopic traffic simulators covering a wide range of traffic situations including traffic and transit on urban roads and motorways. To achieve this main objective several default values of the parameters such as driver familiarity, aggressiveness, mean target headway and mean reaction time were modified to mimic the field conditions. An important step of model calibration was to develop an Origin-Destination (OD) matrix that represents the turning volume count at the intersections. The results with modified values of selected parameters showed satisfactory results between the models simulated Measure of Effectiveness (MOE's) and the field observed MOE's. In order to use the calibrated model regionally, the model was validated on a different network chosen in Al Khobar city using a different data set. The result in validating the calibrated model was successful in terms of pre-set target criteria within an acceptable range. Later, using TRANSYT-7F and SYNCHRO signal plans of the new network were optimized and used in PARAMICS for further analysis.

MASTER OF SCIENCE DEGREE

KING FAHD UNIVERSITY OF PETROLEUM AND MINERALS

Dhahran, Saudi Arabia

THESIS ABSTRACT (ملخص الرسالة)

الإسم : رضا عمران

عنوان الرسالة : المعايرة و التحقق من صحة نموذج المحاكاة المجهري (PARAMICS) لظروف حركة المرور في مدينة الخبر بالمملكة العربية السعودية

التخصص : الهندسةالمدنية

الدرجة العلمية : ماجستير بالعلوم الهندسية

تاريخ التخرج : أكتوبر2013

أدى النمو غير المستقر في نظام النقل المعتمد على المركبات في المملكة العربية السعودية الى ازدحام الحركة المرورية في المدن بشكل ملحوظ وقد برزت هذه المشكلة باعتبارها مشكلة محتملة في جميع المدن الحضرية الكبيرة في السنوات الاخيرة. لذلك فإن الفهم الصحيح لسلوك الحركة المرورية في هذه المنطقة من اجل تحسين عمل اشارات المرور و ادارة السفر امر لا مفر منه. ان نماذج المحاكاة المجهرية هي عبارة عن اداة معروفة و فعالة لتحليل مشاكل النقل بتكلفة قليلة. وقد ادت التطورات الاخيرة في تكنولوجيا الكمبيوتر الى زيادة دقة نماذج المحاكاة التي هي أقل تكلفة و أسرع في التنفيذ و الاختبار الميداني. ان اختبار تصاميم الطرق وأنظمة مراقبة حركة المرور، تحليل نظم النقل الذكية، وتقييم خطط إدارة المرور ومعايرة أنظمة التحكم التكيفية هي التطبيقات الهامة من نماذج المحاكاة المجهرية. و على الرغم من ان هذه النماذج مهمة لحل مشكلة الازدحام, الا انه لا بد من معايرتها و التحقق من صحتها قبل استخدامها لتقديم نتائج واقعية.

الهدف الرئيسي من هذه الدراسة هو معايرة و التأكد من صحة نموذج المحاكاة المجهري (PARAMICS) لظروف حركة المرور في بعض الطرق الشريانية المختارة في مدينة الخبر بالمملكة العربية السعودية. ان نموذج (PARAMICS) هو عبارة عن نموذج محاكاة الحركة المرورية و يغطي حركة المرور على الشوارع و الطرق السريعة في المناطق الحضرية.. أظهرت النتائج المعتمدة على القيم المعدلة لبعض المتغيرات المختارة نتائج مرضية عند مقارنتها مع المشاهدات الميدانية. و قد تم التحقق من صحة النموذج على شبكة مختلفة في مدينة الخبر باستخدام مجموعة بيانات مختلفة. وقد كانت نتائج التحقق من صحة النموذج الذي تمت معايرته مرضية .

ماجستير بالعلوم الهندسية
جامعة الملك فهد للبترول والمعادن
الظهران- المملكة العربية السعودية

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The increasing traffic in urban areas has exacerbated congestion and become a serious socio-economic problem that has worsened lately in large metropolitan cities around the world. While congestion cannot be eliminated completely, measures can be adopted to alleviate the traffic condition. To minimize this problem, careful transport planning and efficient transport-infrastructure management are inevitable. The increasing power of computer technologies, the evolution of software engineering and the advent of the intelligent transport systems has prompted traffic simulation to become one of the most appropriate approaches for traffic analysis for the design and evaluation of traffic systems. The ability of traffic simulation to emulate the time variability of traffic phenomena makes it a unique tool for capturing the complexity of any traffic systems.

A model may be defined as the method of simulating real-life situations with mathematical equations to forecast their impending behavior which involves identifying and selecting relevant features of a real-world situation, representing those features symbolically, analyzing and reasoning about the model and the characteristics of the situation. In transportation engineering, mathematical models are used to represent established relationships which evolve from some processes such as the interactions among speed, flow, and density in a specified traffic stream.

In traffic engineering, depending on the simulation objectives, models range from macroscopic models that use traffic descriptors such as flow, density and speed to microscopic models, which detail the movement of individual vehicles (HCM, 2000). Generally, traffic simulation models have been classified as either macroscopic or microscopic. Some models, called mesoscopic models, combine elements of both the macroscopic and microscopic models. A new mode of model called nanoscopic simulation has emerged only recently.

Macroscopic models are generally applied over large geographical areas and are more useful for transportation planning and corridor operations analysis rather than detailed traffic engineering in areas with complicated geometry and limited right-of-way for the traffic operations. In macroscopic models, vehicle movement is governed by the flow-density relationship without tracking individual vehicles (Owen et al., 2000). The simulation takes place on a section-by-section basis and is based on deterministic relationships of flow, speed, and density in the traffic stream (Alexiadis et al., 2004). Microscopic modeling is used to track individual vehicles right from entry into the network until departure from it and each vehicle type specific performance capabilities such as maximum speeds and acceleration and deceleration rates are considered. In addition, individual vehicle movements can be described by the use of appropriate models that can be drawn for both lateral and longitudinal movements along the road (Oketch et al.,2005). Mesoscopic models were developed as a compromise between computationally intensive microscopic models and more efficient macroscopic models so that traffic stream can be analyzed both in platoons and individually. With the increased popularity of using microscopic model, mesoscopic models are becoming less popular as the computing power necessary for it becomes more available. In mesoscopic models, the unit of traffic flow is the individual vehicle, but the movement is governed by the average speed on the link (Alexiadis et al., 2004). Nano-simulation or traffic safety modeling is a relatively new area of simulation which attempts to model drivers' steering behavior and more detailed components of perception-reaction time in order to include traffic safety in the model.

Microscopic simulation models can reproduce queues, shock waves, weaving areas, merging zones, gap acceptance, fixed and actuated signals and may other traffic characteristics observed in real life. The main advantage of micro-simulation models lies in their ability to model relatively large networks in sufficient details to enable operational outputs at the link or intersection level while correctly accounting for wide area impacts of localized activities (Oketch et al., 2005). The majority come with dynamic assignment tools that facilitate realistic modeling of route choice decisions and hence better network performance. Moreover, their powerful animation and graphical user interface endear microscopic models to users, especially when the results of the analysis are to be communicated to non-technical persons. Microscopic models have been successively used in testing alternative road designs, alternative traffic control systems, intelligent transportation systems, and toll and pricing schemes. Other applications include incident management analysis, public transit impacts, bus priority, high occupancy vehicle lanes, the impact of heavy vehicles, route guidance systems and the calibration of adaptive traffic control systems.

PARAMICS is one of the most powerful microscopic urban and freeway traffic simulation software among all commercially available software that is used to model the movement and behavior of individual vehicles on road networks. PARAMICS is developed on a sophisticated microscopic car following and lane changing model, dynamic and intelligent routing, inclusion of intelligent transportation systems and the ability to interface with the real time traffic input data sources. It takes full account of public transport and its interaction with other modes at bus stops. The animation generated in PARAMICS allows the user to observe the traffic flow on-screen and inappropriateness or inefficiencies can be noticed in signal timings and offsets, queue spillback, insufficient storage and weaving problems. Traffic data like route travel time, delay, queue length, and link volumes can be collected during each simulation run and stored in data files for off-line analysis. The most important feature of PARAMICS is its ability of overriding or extending the default models such as car following, lane changing, route choice, etc., using its Application Programming Interface (API) (Ozbay et. al, 2005). This feature helps the modelers to incorporate customized functionalities and test their own models. Another important feature of PARAMICS over the other available software is that it has an integrated ITS (Intelligent Transportation System) functionality. Special ITS features in the form of High Occupancy Tolling (HOT), Variable Speed Limits (VSL), Vehicle Actuated Signals (VA) makes it popular among the researchers and transportation engineering professionals.

This research focuses on the calibration and validation of PARAMICS model for the local traffic condition in Saudi Arabia. A comprehensive literature review and Justification of choosing PARAMICS will be discussed in the following chapter.

1.2 PROBLEM STATEMENT

Traffic-simulation modeling is a powerful tool to analyze a wide variety of dynamic problems that are otherwise difficult to assess in real field. Such models can simulate real network conditions and perform analysis and forecasting by replacing physical experiments with computer representations.

However, simulation models have limitations. A simulation is not always the best way to solve a problem. The modeler must always consider alternative resources. For a model to reflect reality, calibration and validation must be performed after checking and evaluating the codified network. If the required calibration steps are poorly implemented, the model will not be reliable. Model calibration is the process by which network elements, model parameters and trip patterns are adjusted in order to obtain a model capable of reproducing observed traffic characteristics such as queuing, travel time, traffic volumes, routing, turn proportions, driving behavior and vehicle characteristics. Model calibration is one of the essential tasks in transportation modeling and analysis because its accuracy directly determines the usefulness of the model used. Unfortunately the number of simulated events and the parameters associated with them make the calibration process a complex and time consuming and tedious job that sometimes impede the benefits of microscopic traffic simulation.

It is elicited from an extensive literature survey that only a few microscopic simulation models such as NETSIM, SimTraffic, AIMSUN and VISSIM models are calibrated and validated using local traffic conditions and driving behavior of Saudi Arabia. However, few case studies clearly demonstrated the unique traffic behavior prevalent in the Kingdom justifies the model calibration and validation using local traffic data. Only a few of the mentioned calibrated models are suitable for evaluating ITS applications and provide Application Programming Interface (API) to interact deeply with the basic models. However, PARAMICS model can be used to investigate different ITS applications and provide seamless model of surface streets and freeway road network along with API functionality.

Based on the investigation of available literature it seems that probably the microscopic model PARAMICS has not been used in the Kingdom for traffic analysis, policy making and travel demand management in the whole transportation system. Therefore, it is expected that the appropriate calibration and validation of the PARAMICS model will help in identifying and addressing a number of traffic related problems that the Kingdom of Saudi Arabia is encountering over the years.

1.3 OBJECTIVES

The main objective of this study is to investigate a few traffic engineering applications of Quadstone PARAMICS, at a particular arterial with few intersections at the city of Al Khobar in Saudi Arabia. To address this need, this research seeks to provide a comprehensive introduction to the concepts, experiences with, and performance of early-generation traffic simulation models. The specific purposes of this study are as follows

(1) To review and study available microscopic simulation models along with their specific pros and cons.

(2) To review the past and present research activities in the Kingdom related to different microscopic simulation models.

(3) To review and study different methodologies for calibrating and validating microscopic simulation models.

(4) To study different methodologies available in the literature for calibrating PARAMICS model.

(5) To identify appropriate parameters for calibrating PARAMICS model.

(6) To calibrate PARAMICS model for the local traffic conditions in Al-Khobar city, Kingdom of Saudi Arabia.

(7) To validate the calibrated PARAMICS model.

(8) To compare the simulated output of PARAMICS to TRANSYT-7F and SimTraffic that is commonly used in this region.

(9) To utilize optimized signal plan from the above mentioned software in PARAMICS and to compare their results.

1.4 THESIS ORGANIZATION

This thesis is organized in a total of 5 chapters. The content of each of these chapters is explained below.

Chapter 1: This chapter consists of the background of the thesis work, and a brief description for the need of this research is explained. Then the thesis objectives are stated.

Chapter 2: In this chapter, a detailed literature review is presented and focus was given mainly to microscopic traffic simulation in the context of the larger range of traffic analysis tools. Several categories of traffic analysis tools are discussed, with emphasis on commercially available microscopic traffic simulation software. Finally, recent research conducted in Saudi Arabia and other countries on the calibration and validation of microscopic traffic simulation models is summarized and discussed.

Chapter 3: Chapter three presents in detail description of the selected PARAMICS software. The lane changing and car following logic used in PARAMICS is briefly discussed along with other regular features of PARAMICS.

Chapter 4: The detailed research methodology has been discussed in this chapter. The selection of study area and data collection process has been illustrated in brief.

Chapter 5: Data analysis includes the description of preparation of network model and calibration and validation process. This chapter also includes a sensitivity analysis of few of the parameters and their impact on the overall result output.

Chapter 6: This chapter has been dedicated to the conclusions and recommendations based on the discussion from the previous chapters.

CHAPTER 2

LITERATURE REVEIW

2.1 TRAFFIC SIMULATION MODEL

The increasing levels of traffic in cities and towns continue to create significant problems for city planners. Limited funding for infrastructure and environmental issues has resulted in the need to find solutions that increase road capacity without the requirement of new road construction. Increasing capacity without further road construction requires a good understanding of the factors and variables involved in traffic operations. Traffic flow is a complex human-machine dynamic system that varies by the hour, day, week and year. Traffic in general displays a considerable amount of randomness mainly produced by different driver behaviours, a changing network capacity and demand-adaptive traffic control systems.

Traffic simulation modeling has become a widely used tool in transportation engineering that is able to reproduce some of the complex patterns observed in traffic flows. Traffic simulation is achieved by developing a computer traffic model that relates the main variables of the traffic stream and the main components of the transportation system in real time. Through simulation, transportation specialists can study the formation and dissipation of congestion on roadways, assess the impacts of control strategies and compare alternative geometric configurations. May, A.D. (1990) defined simulation as follows:

> "Simulation is a numerical technique for conducting experiments on a digital computer, which may include stochastic characteristics, be microscopic or macroscopic in nature and involve mathematical models that describe the behavior of a transportation system over extended periods of real time".

Simulation is increasingly being used in the transportation and traffic engineering field, not only because of its strength in analyzing complex systems requiring a large number of calculations, but also because of its capabilities in providing users statistical measures of effectiveness. Mathematical modeling of traffic flow behaviour is a prerequisite for a number of important analytical tasks such as transportation planning, traffic surveillance and monitoring, incident detection, control design, forecasting, energy consumption, environmental impact and vehicle guidance systems.

There is a wide range of uses of traffic simulation models:

1. Evaluation of alternative treatments by controlling the experimental environment and the range of conditions to be explored.

2. Testing new designs by studying the effect of different geometric designs before the construction takes place.

3. Being embedded in other models, simulation sub-models can be integrated within software tools designed to perform other functions. For example, the flow model within the TRANSYT-7F signals optimization.

4. Simulation can be used in the context of a real-time laboratory to train operators of Traffic Management Centers.

5. Simulation can be effectively used for road safety analysis and to build safer vehicles and roadways.

6. Evaluation of transit priority scheme and transit impact on delay.

7. Impact of route guidance system.

8. Long term and short term forecasting.

9. Effect of traffic calming and incident impact

10. Traffic Impact Assessment Study.

11. Emission modeling and quantifying energy savings.

However, the use of simulation model is considered when:

- Other analytical approaches may not be appropriate.
- The assumptions underlying a mathematical formulation (e.g., a linear program) or a heuristic procedure (e.g., those in the Highway Capacity Manual) generate doubts on the accuracy or applicability of the results.
- The mathematical formulation represents the dynamic traffic/control environment as a simpler quasi steady-state system.
- There is a need to view vehicle animation displays to gain an understanding of how the system is behaving in order to explain why the resulting statistics were produced.
- Congested conditions persist over a significant time.

Simulation models also have some shortcomings. Few of these are listed below:

- There may be easier ways to solve the problem
- Simulation models may require verification, calibration, and validation, which, if overlooked, make such models useless or not dependable
- Development of simulation models requires knowledge in a variety of disciplines, including traffic flow theory, computer programming and operation, probability, decision making, and statistical analysis
- The simulation model may be difficult for analysts to use because of lack of documentation or need for unique computer facilities
- Some users may apply simulation models and not understand what they represent
- Some users may apply simulation models and not know or appreciate model limitations and assumptions
- Simulation models require considerable input characteristics and data, which may be difficult or impossible to obtain
- Results may vary slightly each time a model is run

2.2 CLASSIFICATION OF TRAFFIC SIMULATION MODEL

Traffic simulation models/software can be classified according to different basis. They can be classified according to their typical applications, the level of aggregation, the uncertainty content, or the manner their systems are updated (Prevedouros, 2000).

2.2.1 Application Oriented

Based on this classification simulation models/software are classified as transportation planning, transportation design, transportation safety, or traffic operation. Transportation planning models enable planners to evaluate alternative urban development patterns, and to produce information on population, employment, and land use for use in estimating travel and transportation demand. The primary concern of transportation planning is demand estimation. Examples of these models are TRANSCAD, TRANPLAN and TRANSIMS.

Traffic operation models have different scales of applications. Examples of these applications and sample of the software used with each application are as follows:

- Isolated intersections: SIDRA, SIGNAL, SOAP, etc.
- Arterial and highways: PASSER II, PASSER III, etc.
- Urban Street Networks: TRANSYT-7F, SYNCHRO, PASSER IV, etc.
- Freeways and Freeways Corridors: FREQ, INTEGRATION, KWaves, etc.
- Integrated Networks: VISSIM, DYNEMO, CORSIM, etc.

2.2.2 Uncertainty Content

This is the common classification method for simulation models. It represents the deterministic or stochastic nature of simulation and the time horizon that represents the static or dynamic properties of simulation. If no element of a model is subject to randomness, the model is considered deterministic and if random seeds are embedded in a model, the model is considered stochastic.

2.2.3 System update

If the status of the traffic system keeps updated with the time intervals, the model is said to be continuous. But if the traffic system updating is not at fixed time intervals, the model will be discrete. There are two types of discrete models, discrete time and discrete event. When discrete time models are used, the state of the traffic system is examined and the elements of the system are recomputed based on fixed time intervals. In the discrete event models, the traffic situation is updated when events of importance to traffic operations occur. For example, at a signalized intersection, the traffic situation will be updated whenever signal changes its phase.

2.2.4 Level of Aggregation

According to the level of aggregation, traffic simulation models can be classified as Microscopic (low fidelity), Mesoscopic (mixed fidelity) and Macroscopic (high fidelity). Macroscopic models model traffic as an aggregate fluid flow by using continuity equation representing the relationship among the speed, density and flow-generation rate. In these models traffic flow represented by aggregate measures such as flow rate, speed and density. Microscopic models are based on car-following and lane-changing theories that can represent the traffic operations and vehicle/driver behaviors in detail. These models incorporate queuing analysis, shock-wave analysis and other analytical techniques. Mesoscopic models represent traffic flow at a high level of detail but describe their activities and interactions at a much lower level of detail than would the microscopic models. A limited number of simulation models fall into category of mesoscopic models.

From the perspective of traffic demand input data, traffic simulation models can be classified into flow-based simulation models (for example, CORSIM, SimTraffic), or path-based simulation models (for example, VISSIM, PARAMICS).

Flow-based traffic simulation models are designed mainly to reproduce link performance. Such models use entry volumes and turn percentages as the traffic input demand. Once inside the network, vehicles are assigned to downstream links according to prescribed turning probabilities.

By contrast, path-based simulation models concentrate on reproducing network trip making behavior. Therefore, Origin Destination (OD) matrices represent the input traffic demand. In this kind of models, traffic assignment is performed using specified routing algorithms based on minimizing total travel costs, or some variation thereof.

2.3 SIMULATION SOFTWARE PACKAGES

2.3.1 Macroscopic Model

In macroscopic models, vehicle movement is governed by the flow-density relationship without tracking individual vehicles (Owen et al., 2000). The simulation takes place on a section-by-section basis and is based on deterministic relationships of flow, speed, and density in the traffic stream (Alexiadis, 2004). While this can adequately represent reality at a large scale, macroscopic models make some counterintuitive assumptions. For example, a car exists simultaneously at every point along its route during the entire period (morning peak, mid-day, evening peak, and off-peak) when its trip takes place (Druitt, 1998). Some of the existing macroscopic traffic simulation models include: TRANSYT-7F, TRAF-CORFLO (CORridor FLOw Model) (CORFLO, 2007), KRONOS (Kwon, 2007), and PASSER (Series).

TRANSYT-7F (TRAffic Network StudY Tool) (TRANSYT-7F Users Guide, 1998) TRANSYT-7F, a macroscopic simulation model, was developed by the FHWA. It is used to analyze existent traffic signal timing and optimize it to reduce delays, stops, and fuel consumption for a two-dimensional network.

PASSER (Progression Analysis and Signal System Evaluation Routine), a macroscopic simulation model, was developed by researchers at the Texas Transportation Institute (Boxill et al., 2000). The PASSER model includes traffic signal timing optimization software programs. PASSER-Ⅱ is used to optimize a single signalized roadway, while PASSER-Ш is used for diamond interchanges and PASSER-IV for single, multiple roadway and diamond interchanges.

2.3.2 Mesoscopic Model

Mesoscopic models were developed as a compromise between computationally intensive microscopic models and more efficient macroscopic models. Mesoscopic models are becoming less common as the computing power necessary for microscopic modeling becomes more available. In mesoscopic models, the unit of traffic flow is the individual vehicle, but movement is governed by the average speed on the link (Alexiadis, 2004). Mesoscopic models assume that packets or platoons of vehicles are moved together or that some patterns of decisions are modeled instead of individual decisions. A packet is a group of vehicles that is treated as a single group of individual decisions (Yuhao, 1996). These models incorporate equations that indicate how these clusters of vehicles interact.

Another way of representing flow is obtained by moving vehicles on a road from an intersection to another based on calculating the travel time in the link. The travel time depends on parameters like the length, the number of lanes, and the speed limit of the road as well as on dynamic variables such as density of vehicles currently on the road.

Some of the existing mesoscopic models include CONTRAM (CONtinuous TRaffic Assignment Model) (Contam, 2007), DYNAMIT-P (DYNAmic traffic assignment

Massachusetts Institute of Technology) (Sundaram, 2002), and DYNASMART-P (DYnamic Network Assignment-Simulation Model for Advanced Roadway Telematics) (DYNASMART-P) and SATURN.

DYNAMIT (Boxill et al., 2000), a mesoscopic traffic simulation tool, was developed by Ben-Akiva et al. (www.ivhs.mit.edu/products/simlab) It is a Dynamic Traffic Assignment (DTA) system developed for route guidance and traffic prediction and estimation. This tool can control real-time operations and accept real-time surveillance data. In addition, time-dependent O-D flows are estimated and predicted based on DynaMIT. This system also has self-calibration and route-guidance generation capabilities.

SATURN (Simulation and Assignment of Traffic in Urban Road Networks) is a combined traffic simulation model suitable for the analysis of relatively small networks, which may include changes, such as, the introduction of one-way streets, changes to junction controls, bus only streets, etc. Being a combined simulation and assignment model SATURN can function as a conventional traffic assignment model and as a pure junction simulation model (Drick, 2000)

2.3.3 Microscopic Simulation Models

Microscopic computer simulation of traffic was first introduced in 1955, when D. L. Gerlough published his dissertation, “Simulation of Freeway Traffic on a General purpose Discrete Variable Computer” at the University of California, Los Angeles (Figueiredo et al., 2004). Microscopic models track individual vehicles, each with its own set of driver and vehicle characteristics. Whereas macro- and mesoscopic models track only the lateral movement of vehicles, microscopic models also examine behavior between lanes of traffic, creating a two-dimensional model (referring to the analysis, not to the animations created in postprocessing). Driver and vehicle characteristics, interactions with the network geometry, and interactions between vehicles are all factors that determine movements (Owen et al., 2000). These models are driven by car-following, lane-changing, and gap acceptance models (which can be thought of as sub-models). Most microscopic traffic simulation models utilize variations on the General Motors (GM) model (Figueiredo et al., 2004), which remains the industry standard today.

2.3.3.1 Common Microscopic Traffic Simulation Models

Microscopic simulation models, in which the dynamic behaviour of individual agents is explicitly simulated over both time and space to generate aggregate system behaviour, have been applied with increasing frequency over the past decade or more in the field of transportation systems analysis. Perhaps the best developed application is in the area of transportation network simulation models, in which a number of operational (and often commercially supplied) software packages exist, which model second-by-second operations of individual road and/or transit vehicles over very high fidelity representations of urban transportation networks (Miller et al., 2004). Over the last two decades, research groups and software companies have developed a number of microscopic traffic simulation software packages. Many of these packages have been produced for research purposes but others have been developed to solve day-to-day traffic engineering problems. Micro-simulators are specifically developed to solve particular problems although some of them are more generic in that they are intended for variety of transportation application. Information about Microscopic traffic simulation models were very scanty until the report on a research project “The SMARTEST Project” funded by the European Union was published whose objective was to review existing micro-simulation models and to identify their pros and cons in order to enhance the capability of state of the art packages. Another source of information is the website of the commercially available software which they use for promoting their package. Among the microscopic models few models are only used for research purpose and the rest are available for commercial use.

Research models have been present in the academic world for many years but their evolution has been limited compared to commercial models. As this models are developed for some specific purpose, their development to encompass other aspects of traffic application remains very slow. On the other hand commercial software packages are more dynamic when it comes to the development of the product, showing responsiveness to the market need. This fast evolution has transformed these models into powerful tools that are capable of solving a significant variety of transportation problems. A list of some of the existing microscopic traffic simulation models and commercial software is appended in Table 2.1 and few of those widely used software is described briefly.

Table 2-1 List of mostly available microscopic simulation model

Sl No	Model	Organization	Country
1	AIMSUN 2	Universitat Politècnica de Catalunya, Barcelona	Spain
2	ANATOLL	ISIS and Centre d'Etudes Techniques de l'Equipement	France
3	ARTEMIS	University of New Wales, School of Civil Engineering	Australia
4	ARTIST	Bosch	Germany
5	CASIMIR	Institut National de Recherche sur les Transports et la Sécurité	France
6	CORSIM	Federal Highway Administration	USA
7	DRACULA	Institute for Transport Studies, University of	UK

Sl No	Model	Organization	Country
		Leeds	
8	FLEXSYT II	Ministry of Transport	Netherlands
9	FREEVU	University of Waterloo, Department of Civil Engineering	Canada
10	FRESIM	Federal Highway Administration	USA
11	HUTSIM	Helsinki University of Technology	Finland
12	INTEGRATION	Queen's University, Transportation Research Group	Canada
13	MELROSE	Mitsubishi Electric Corporation	Japan
14	MICROSIM	Centre of parallel computing (ZPR), University of Cologne	Germany
15	MICSTRAN	National Research Institute of Police Science	Japan
16	MITSIM	Massachusetts Institute of Technology	USA
17	NEMIS	Mizar Automazione, Turin	Italy
18	PADSIM	Nottingham Trent University - NTU	UK
19	PARAMICS	The Edinburgh Parallel Computing Centre and Quadstone Ltd	UK
20	PHAROS	Institute for simulation and training	USA
21	PLANSIM-T	Centre of parallel computing (ZPR), University of Cologne	Germany
22	SHIVA	Robotics Institute - CMU	USA
23	SIGSIM	University of Newcastle	UK
24	SIMDAC	ONERA - Centre d'Etudes et de Recherche de Toulouse	France
25	SIMNET	Technical University Berlin	Germany
26	SISTM	Transport Research Laboratory, Crowthorne	UK
27	SITRA-B+	ONERA - Centre d'Etudes et de Recherche de Toulouse	France
28	SITRAS	University of New South Wales, School of Civil Engineering	Australia
29	THOREAU	The MITRE Corporation	USA
30	TRANSIMS	Los Alamos National Laboratory	USA
31	TRAF-NETSIM	Federal Highway Administration	USA
32	VISSIM	PTV System Software and Consulting GMBH	Germany

Source: Smartest, (1997)

Few of the popular research models and commercial packages are briefly discussed in the following section.

PARAMICS (PARAllel MICroscopic Simulation), a micro stochastic simulation model, is developed by Quadstone Limited and includes five software modules: Modeller, Processor, Analyzer, Programmer, and Monitor. PARAMICS can simulate individual vehicle movements based on a microscopic car-following and lane-changing model on freeways, arterial networks, advanced signal controls, roundabouts, incidents, high occupancy vehicle (HOV) lanes, etc. A Graphical User Interface (GUI) with graphical windows provides a three-dimensional animation of car movements through a simulated network. An Application Programming Interface (API) can customize car-following, gap acceptance, lane-changing, and route choice simulations, and the simulation results can be matched with real-world conditions. The API also uses signal optimization, adaptive ramp-metering, and incident detection as control strategies. Input parameters can be categorized into four different types: network characteristics, demand data, assignment, and general configuration. The output parameters are travel time, flows, queue length, delay, speed, and density.

CORSIM (CORridor SIMulation) (Boxill et al., 2000), a microscopic stochastic simulation model, was developed by the U.S. Federal Highway Administration (FHWA), and it consists of the NETSIM and FRESIM models. The NETSIM model is used for surface street design, while the FRESIM model is used for freeway design. In the case of a multiple-model network, an urban sub-network is built using NETSIM and freeway sections are modeled using FRESIM, both at the same time. Each vehicle in NETSIM can be classified into one of nine different types, and driver behavioral characteristics are assigned. Speed, acceleration, and status of vehicle can also be specified. Each vehicle's movement and position on the link responds to control devices and demands, and calculations are based on car-following logic. Traffic operations are affected by fleet components, load factor, turn movement bus operations, HOV lanes, and queue discharge distribution, among others. The FRESIM model, a microscopic freeway simulation model, is capable of simulating more complex geometric calculations. This model represents more detailed freeway situations, with such operational features as a lane-changing model, clock-time and traffic-responsive ramp-metering, representations of nine different vehicle types, heavy-vehicle movements, 10 different driver habits, and driver reactions to upcoming geometric changes.

MITSIM (MIcroscopic Traffic SIMulator) (Boxill et al., 2000) was developed by Ben-Akiva at the MIT ITS program and evaluates advanced traffic management systems (ATMS) and route guidance systems. MITSIMLab consists of three modules: a Microscopic Traffic Simulator (MITSIM), a Traffic Management System (TMS), and a GUI. By modifying driver behavior factors such as desired speed, aggressiveness, etc. MITSIM can specify each vehicle’s characteristics. Individual vehicle movements are simulated based on a car-following model and a lane-changing model. Real-time information is provided for drivers by route guidance systems, so they can make route-choice decisions. Control and routing strategies-such as ramp control, freeway mainline control, intersection control, variable message sign, and in-vehicle route guidance- are evaluated through the traffic management simulator. A visualization of vehicle movements is available through the GUI, to monitor traffic impact.

AIMSUN, which is short of Advanced Interactive Microscopic Simulator for Urban and Non- Urban Networks, was developed by the Department of Statistics and Operational Research, Universitat Poletecnica de Catalunya, Barcelona, Spain.(Xiao et al., 2005). This microscopic traffic simulation software is capable of reproducing various real traffic networks and conditions on a computer platform. The driver behavior models inside

AIMSUN such as car-following model, lane changing model and gap-acceptance model provide the behavior of each single vehicle of the entire simulation period. (TSS, 2006) As developed in the GETRAM simulation environment, AIMSUN has the Application Programming Interface (API), which enables it to communicate with some user-defined applications. The advantage of AIMSUN also includes the capability of modeling a traffic network in detail and producing a number of measures of effectives. The latest version of AIMSUN at the time of the study was Version 7.0, released on 14 September, 2011.

VISSIM is a time step and behavior based microscopic traffic simulation model developed at the University of Karlsruhe, Karlsruhe, Germany, in the early 1970s. PTV Transworld AG, a German company, began the commercial distribution of VISSIM from 1993 and continues to maintain the software up to this date. This traffic simulation software is developed to model urban traffic and public transit operations and it is composed of two main components: a traffic simulator and signal state generator. The traffic simulator is in charge of the movement of vehicles, while the signal state generator models the signal status decision from detector information of the traffic simulator and then passes the signal status back to the traffic simulator. (Bloomberg et al., 2000) The VISSIM model can produce almost all the commonly used measurements of effectiveness in the traffic engineering area. Also, it is capable of modeling different vehicle types for both freeways and arterials under different complex traffic control situations. (Moen et al., 2000). The latest version of VISSIM is Version 5.40 at the time of this study.

The INTEGRATION, developed by the late Michel Van Aerde in 1983, is a trip-based microscopic traffic simulation model. Professor Hesham Rakha continues with the development of this model since 1999. The two most important features of the INTGERATION software are first, it is the first model to attempt to integrate both freeways and arterials; second, it integrates traffic assignment and microscopic simulation within the same model. The name INTEGRATION stems from this fact. The INTEGRATION model is capable of providing sufficient detailed driver behavior data by tracing individual vehicle movements from its origin to its destination at a level of resolution of one deci-second. Also, the model is capable of computing a number of measurements of effectiveness including vehicle delay, vehicle stops, emissions and fuel consumption as well as the crash risk for 14 crash types. (Van Aerde and Rakha, 2007).

SimTraffic, was developed to work hand in hand with the signal optimization program Synchro and to provide a user-friendly modeling and visualization alternative to CORSIM. While the primary strength of SimTraffic lies in its ability to model signalized intersections, SimTraffic developers claim that it can be applied to freeways and larger networks as well. SimTraffic was developed by Trafficware and bases its vehicle and driver performance characterstics on the vehicle and driver performance characteristics developed by the FHWA. As of Version 6, SimTraffic does not simulate transit, ramp metering, on-street parking, or high-occupancy vehicle (HOV) lanes. It can model most network geometries, including limited applications of roundabouts.

HUTSIM is a software package created in Finland by the Helsinki University of Technology. It is a tool developed especially for traffic signal simulation and can be connected to real signal controllers. This makes it possible to evaluate control strategies, intelligent transportation system and new control systems. This model allows a detailed representation of intersections and their approaches.

FLETSYT II was created for the Ministry of Transport of the Netherlands. The aim of this software was to enable the analysis of dynamic traffic management strategies involving signals, ramp meter, toll plazas, special lanes etc. This model is fully event-based and moves the vehicles through the network on a stochastic basis. This model in not included with assignment algorithm and can only reproduce small networks.

THOREAU, a research based software was developed by the MITRE corporation in the United States of America. THOREAU as developed to quantify the benefits of intelligent transportation systems, primarily Advanced Traveler Information Systems (AITS) and Advanced Traffic Management Systems (ATMS). It has been used for evaluating various adaptive traffic signal algorithms. This model uses both macroscopic and microscopic approach to achieve the desired performance, simulation speed and granularity.

SITRAS is an Australian software developed at the University of New South Wales. This software emphasizes the simulation of urban road networks under congestion conditions for the purpose of analyzing and evaluating intelligent transportation systems. SITRA is a time-interval update simulator based on car following and lane changing theory, and route selection based on individual driver characteristics. Fixed time, coordination and adaptive traffic signal control strategies can be programmed into the model. Incidents may be programmed at any point and time and it is possible to model route guidance systems.

SIMNET was created by the Technical University Berlin, Germany. This is a research tool whose main purpose is the evaluation of traffic control strategies. SIMNET uses a combination of discrete event simulation and quasi-continuous simulation. It simulates individual vehicles whose positions are defined as queue-positions on a lane in the queuing model and as real positions on a lane in the quasi continuous mode.

SISTM was developed in the United Kingdom by the Transport Research Laboratory, Crowthrone. SISTM has been designed to study motorway traffic in congested conditions with the aim of developing and evaluating different strategies for reducing congestion. It simulates traffic based on a car following algorithm and two driver behaviour parameters (aggressiveness and awareness) that produce a distribution of desired speed and desired headway. By controlling lane changing stimulus the lane changing can be accomplished here. It does not include route assignment.

SHIVA was developed at the Robotics institute of the Carnegie Mellon University, USA. This product is designed to support the design and testing of intelligent vehicle algorithms that operate at the tactical level of driving. SHIVA supports heterogeneous vehicle control algorithms where different cars are equipped with different sensors and may use different algorithms for driving.

DRACULA was created by the institute of Transport Studies at the University of Leeds, UK. The main objective was to test the fundamental issues in network modeling and assessment of future transport strategies and policies related to public transport, Urban Traffic Control (UTC), pricing strategies, fuel consumption and exhaust emissions. DRACULA is a time-based simulator that changes the vehicle state at discrete intervals of 1 second. Vehicles are individually represented and their movement in the network is controlled by a car following model, lane changing model and traffic regulations on the road. Traffic signals may be fixed time, adaptive or may include bus priority conditions.

FREEVU was developed in University of Waterloo, Canada. This is a research tool that estimates the impacts of trucks on freeway traffic streams. It is based on the following models originally incorporated into the FHWA model INTRAS. FREEVU is based on a car following logic that incorporates collision avoidance rules and a mandatory and discretionary lane changing model. Detailed traffic composition is also available in FREEVU.

2.3.3.2 Car Following Theory in Microscopic Simulation Model

"The accuracy of a traffic-simulation system depends highly on the quality of the traffic-flow model at its core, with the two main critical components being the car-following and lane changing models (Panwai et al., 2005)." Car-following models form the basis of microscopic simulation models, and they explain the behavior of drivers in a platoon of vehicles (Aycin et al., 1999). Each traffic simulation model has its unique underlying logic. This logic includes a car-following logic, a lane-changing logic, and gap acceptance logic. Car following theory has evolved over the past forty years from conceptual ideas to mathematical model descriptions, analysis and model refinements resulting from empirical testing and evaluation. Car following model focuses on the task of one car following another in a single lane of a roadway. It forms a tie between individual car following behavior and the macroscopic world of a line of vehicles and their corresponding flow properties. The task of one vehicle following another can be categorized as three specific subtasks: perception, decision making and control. Perception involves information related to speed, acceleration, vehicle spacing, relative speed, collision time etc.

Decision making refers to the interpretation of the perceived information and the definition of driving strategies to control and maneuver the vehicle. The more a person drives a car, the more these activities become automatic and define the driving skills of the driver. Skilled drivers can control the vehicle with dexterity, smoothness and coordination. The approach used assumes that a stimulus-response relationship can accurately describe the driver car-following task.

$$\text{RESPONSE} = \lambda * \text{Stimulus} \; [\lambda \text{ is a proportional constant}]$$

This stimulus-response relationship states that a driver will execute a control task in "response" to a stimulus generated by a perceived change in relative, inter vehicle spacing, vehicle performance etc. The response that is commonly accepted is the acceleration and deceleration of the following vehicle. Acceleration is well accepted because the driver has direct control of this quantity through the "accelerator" and "brake" pedals and also because the driver obtains direct feedback of its effects through the inertial forces. The most common factor used to represent the stimulus is the relative speed between vehicles. The proportional constant λ is the equation component that most of the research has emphasized on and involves leading-vehicle speed and the inter-vehicle spacing factors. Figure 2.1 shows the form of a general equation of car following models.

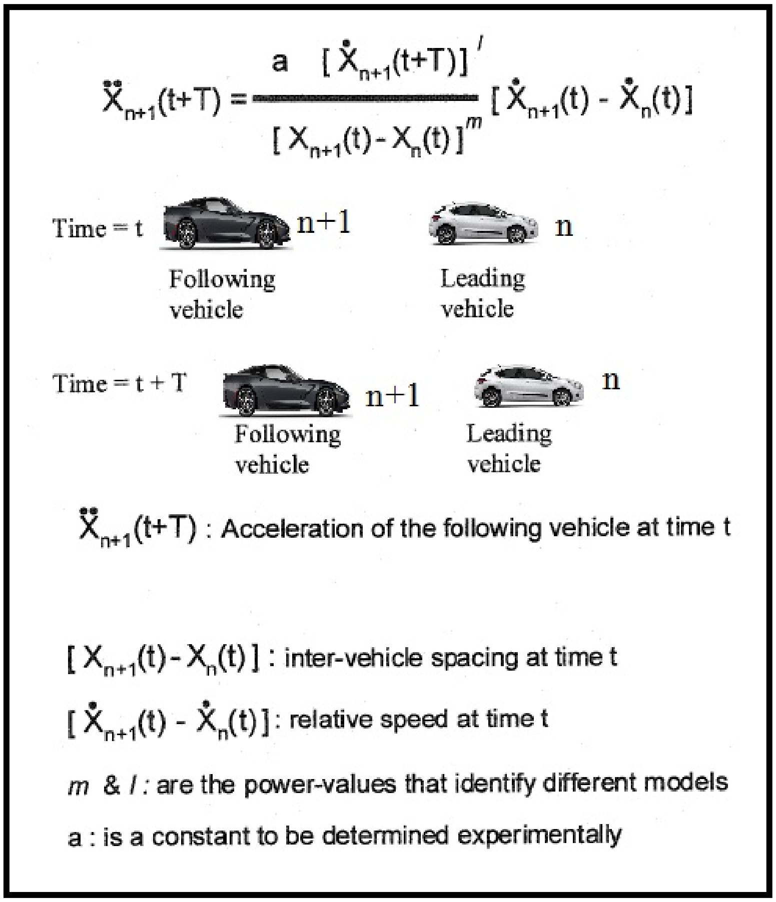


Figure 2-1 Car following general equation (Source: Aldazaba, 2004)

In 1994, Hans Thomas Fritzsche proposed a single lane car following model based on thresholds. This model assumes constant acceleration of the following vehicle until it reaches a new threshold and then a new response (acceleration or deceleration) is defined. The thresholds included in the model are:

1. Positive perception threshold (PTP), This threshold tries to capture the fact that the movement of an object can only be perceived when the reflection of the retina has to exceed a certain minimum speed threshold.
2. Negative perception threshold (PTN), It is similar to PTP but associated to an increasing distance state between following and leading vehicle.

3. Desired distance (AD), This threshold reflects the rule that a following car should maintain a distance (meters) with respect to the leading vehicle of half of the speed shown in the speedometer.
4. Risky distance (AR), This threshold avoids a short risky distance between leading and following cars.
5. Safe distance (AS), This threshold and the "braking distance" threshold keep the following car at a safe distance from the leading vehicle.
6. Braking distance (AB)

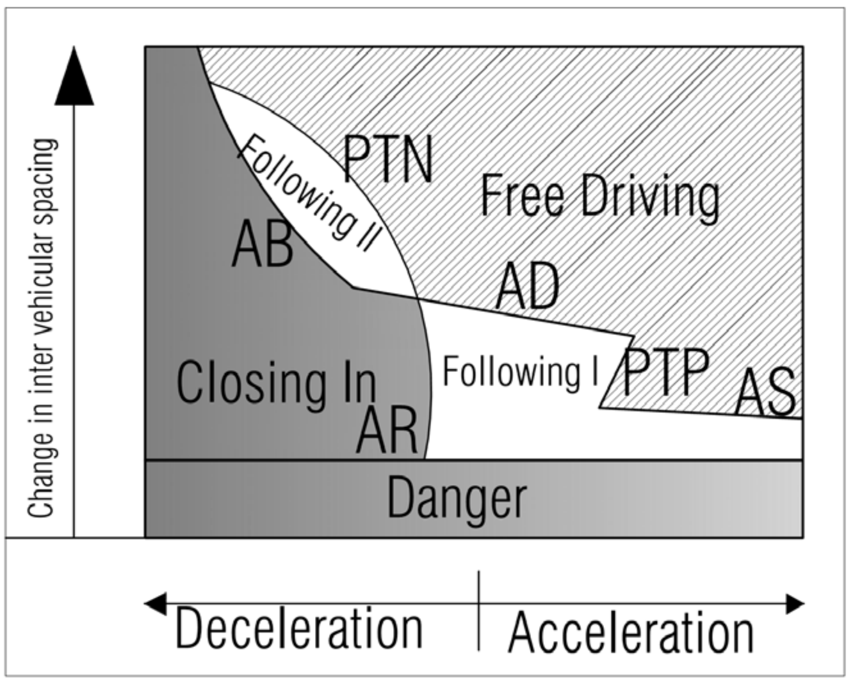


Figure 2-2 Shows a schematic diagram of these thresholds.

When a following vehicle is moving, its inter-vehicular spacing and relative speed with respect to the leading vehicle is changing and this can be represented by a new location in the above diagram. The following car keeps changing its condition with respect to the leading vehicle until it crosses a new threshold.

When the following vehicle crosses a threshold, it has to accelerate, decelerate, or do nothing to the following rules.

1. If the following vehicle enters either the “danger” or “closing in” regions, it has to accelerate to avoid a collision.
2. If the following vehicle enters either the “following I” or “following II” regions, it has to keep its current acceleration which can be positive or negative.
3. If the following vehicle enters the “free driving” region it has to keep its acceleration only until it reaches its desired travel speed.

This model has performed well and has been the base and inspiration of some of the car following models implemented in recent micro-simulation software such as VISSIM and PARAMICS.

2.3.3.2.1 Car-following logic of AIMSUN

The car following model used in AIMSUN is based on the model developed by Gipps (1981), which considers the speed of the following vehicle to be either free or constrained by the leading vehicle. Below is the detailed description of the model. The speed of the following vehicle during the time interval [t, t+T] is calculated using equation (2.1)

$$v_n\ (\mathrm{t+T}) = \min\{v_n^a\ (t+T), v_n^b(t+T)\} \qquad (2.1)$$

Where, $v_n^a\ (t+T)$ is the maximum speed the following vehicle can accelerate and $v_n^b(t+T)$ is the maximum safe speed for the following vehicle with respect to the vehicle in front at time t.

Equation (2.2) is used when the traffic flows freely which means no leading vehicle’s impact on its behavior. Equation (2.3) is used in congested flow conditions, which means the behavior of the following vehicle is constrained by the vehicle ahead of it.

$$v_n^a(t+T) = v_n(t) + 2.5 \cdot a_n^{\max} \cdot T \cdot (1 - \frac{v_n(t)}{v_n^{desired}}) \cdot \sqrt{0.025 + \frac{v_n(t)}{v_n^{desired}}} \quad (2.2)$$

$$v_n^b(t+T) = d_n^{\max} \cdot T + \sqrt{(d_n^{\max} \cdot T)^2 - d_n^{\max} \cdot [2\{x_{n-1}(t) - s_{n-1} - x_n(t)\} - v_n(t) \cdot T - \frac{v_{n-1}(t)^2}{\hat{d}_{n-1}}]} \quad (2.3)$$

Where,

a_n^{max}	Maximum desired acceleration, vehicle n, [m/s2]
d_n^{max}	Maximum desired deceleration, vehicle n, [m/s2]
$\hat{d}_{n-1}$	Estimation of maximum deceleration desired by vehicle n-1, [m/s2]
T	The apparent reaction time, a constant for all vehicles
Sn-1	The effective length of a vehicle, which consists of vehicles length and the user specified parameter- min distance between vehicles.

The leader's desired deceleration $\hat{d}_{n-1}$ can be estimated in the following two ways as demonstrated in equation (4) and (5) (TSS, 2002)

$$\hat{d}_{n-1} = d_{n-1} \quad (2.4)$$

$$\hat{d}_{n-1} = \frac{d_{n+} d_{n-1}}{2} \quad (2.5)$$

Where the first desired deceleration is calculated to be the estimation as the leaders desired deceleration, dn-1and the second desired deceleration is estimated as average of the leader's and the follower's desired decelerations.

2.3.3.2.2 Car-following logic of VISSIM

VISSIM uses a psycho-physical car-following model based on the model developed by Wiedemann (1974), which defines the driver perception thresholds and the regimes formed by these thresholds. There is another car-following model called Wiedemann 99 car-following in VISSIM, the Wiedemann 99 car-following model is in many ways similar to Wiedemann 74 carfollowing model , except that some of the thresholds in the 99 model are defined in a different (sometimes, simpler) way to model freeway traffic better. In addition, many more of the thresholds are user adjustable in the Wiedemann 99 model.

2.3.3.2.3 Car following logic in PARAMICS

The car following model in PARAMICS, similar with Wiedemann's car-following model, is based on a psycho-physical model developed by Fritzche (1994). In Fritzche's model, the perception thresholds and different regimes are defined as demonstrated earlier in figure 2.2. For different regimes the model has its corresponding driver behavior.

In danger regime, the following vehicle uses its max deceleration to extend the headway; in closing in regime, the following need deceleration to keep a distance from the leading vehicle; in following regime, there is no need for action and as the driver doesn't have the ability to maintain the constant speed, a parameter is assigned to model this; in following II regime, no action is necessary because although the following vehicle realizes he/she is closing in the front vehicle but the distance headway is too large to make any adjustment; in free driving regime, the vehicle accelerates to its desired speed first and then drives around this speed as the driver is unable to maintain the constant speed (Olstam and Tapani, 2004).

2.3.3.2.4 Car-following logic of CORSIM

The CORSIM car following model developed by FHWA evolved from two parts: NETSIM and FRESIM models. In which NETSIM models arterials with at grade intersection and FRESIM models uninterrupted facilities.

FRESIM was developed based on INTRAS, a microscopic freeway simulation application introduced in 1980s. The car-following logic in FRESIM is kept the same as in INTRAS which is Pitt car-following model developed by the University of Pittsburgh (Halati et al., 1996). The basic model of CORSIM takes the distance headway and speed differential between the leading and following vehicle as two independent variables, as shown in Equation(2.6) (Rakha and Crowther, 2003)

$$h=hj+c_3u+bc_3\Delta u^2 \tag{2.6}$$

Where h and j h are respectively the distance headway and the jam distance headway (km); u and Δu are respectively the speed of the following vehicle and speed difference between the leading and following vehicles; c3 is the driver sensitivity factor and b is calibration constant.

In NETSIM the basic logic of car-following model is that the following vehicle will move to a certain location where even the leading vehicle decelerates at its maximum deceleration rate, the following vehicle still has enough reaction time and braking ability to stop without resulting in a collision. The basic car-following model is demonstrated in Equation (2.7) (Rakha and Crowther, 2003). NETSIM utilizes a time step of 1 second in simulation.

$$h=h_j+\Delta s+\Delta r+S_F-S_L \tag{2.7}$$

Where,

Δs = distance traveled by following vehicle over the time interval (km)

Δr = distance traveled by following vehicle during its reaction time (km)

S_F = distance required by following vehicle to come to a complete stop (km)

S_L = distance required by lead vehicle to come to a complete stop (km)

2.3.3.2.5 Car-following logic of INTEGRATION

The INTEGRATION software uses the car-following model proposed by Van Aerde (1995) and Van Aerde and Rakha (1995). The Van Aerde's model combines the Greenshields car-following model and the Pipes car-following model into a single-regime model which overcomes the shortcomings of them. "Specifically, the model overcomes the shortcoming of the Pipes model in which it assumes that vehicle speeds are insensitive to traffic density in the uncongested regime." "Alternatively, the model overcomes the main shortcoming of the Greenshields model, which assumes that the speed-flow relationship is parabolic". (Rakha and Crowther, 2002).

2.3.3.3 Lane Changing Theory in Microscopic Simulation Model

After the car following models, lane changing models are the next most important element in microscopic modeling and simulation. Lane changing is a complex and common phenomena in real traffic. Lane changing replicates the phenomena of one vehicle moving from one lane to another. This phenomenon usually takes place in a short space of time and in most situations involves more than one vehicle.

In lane changing maneuver the following issues are involved:

- A vehicle wishing or needing to make a lane change moves from its current lane so the driver quickly checks the road and chooses a target lane.

- To be able to move, the subject vehicle (Figure: 2. 3) verifies the gaps between vehicles travelling in the target lane and selects one of the gaps as a target.
- After choosing the lane and the gap in this lane, the subject vehicle examines the front gap to the vehicle right in front and the lead and lag gaps with respect to the vehicles in the target lane.
- The front gap has to have a minimum desired distance to the front vehicle. This desired distance assumes that the driver will be able to safely stop in case of sudden braking by the lead vehicle.
- The lead gap has to be large enough to avoid a collision with the front vehicle. So it has to include a safety distance and some additional space to undertake the maneuver in a comfortable way.
- The lag gap has to be large enough to allow the vehicle to carry out the lane change without forcing the lag vehicle to brake suddenly and to keep a safe distance.
- If the gaps (front, lead and lag) are acceptable, the lane change is executed instantaneously.
- There are circumstances in which drivers are not able to find a desired gap in the required lane so they have to either continue in the same lane until a gap is available or stop and wait until somebody voluntarily brakes to create a gap for the candidate vehicle.

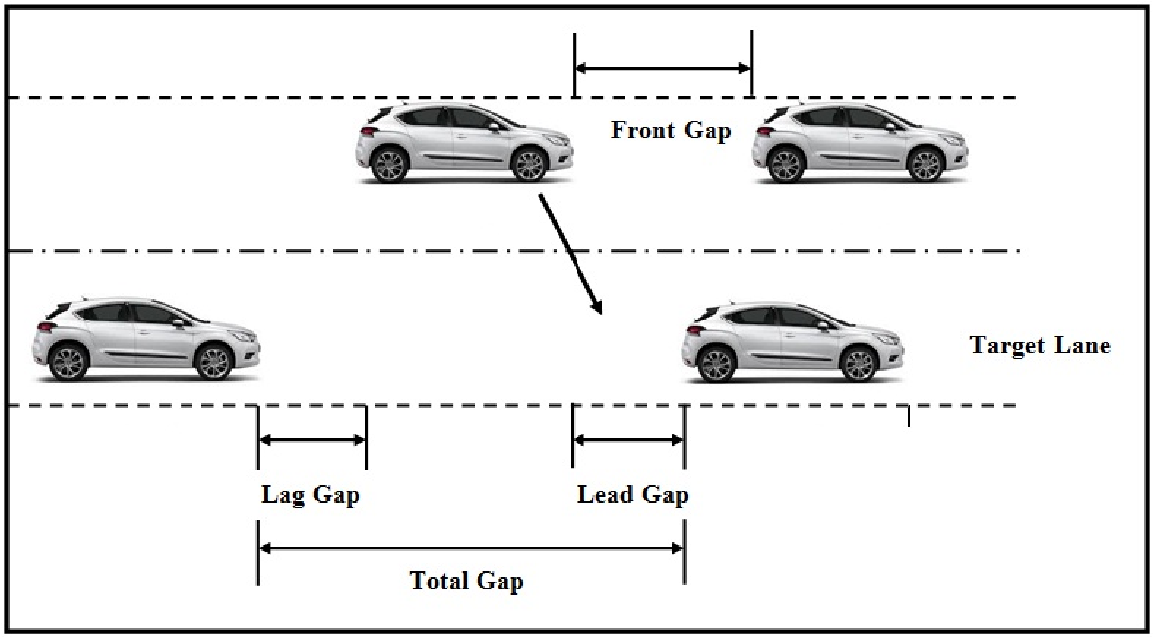


Figure 2-3 Lane Changing Diagram

The main efforts in lane changing models have focused on gap acceptance behavior. In 1986, Gipps formulated an urban model that established the following three driving situations:

- The driver is far from his next turn and the only motivation to change lane is to reach a desired speed.
- The driver is somewhat close to his next turn and needs to change lane in order to be in position for doing such turn.
- The driver is close to the next turn. No lane change is performed in spite of slower speeds.

This model implies that there is no interference between vehicles in the destination lane when undertaking the maneuver. A vehicle changes lane without forcing other vehicles in the destination lane to slow down or stop. Currently some simulation models are including models that consider forced and cooperative lane changing situations.

Lane changing models are implemented by cycling algorithms that may include many of the following subroutines

1. **Defining a need for lane changing**

It defines when a vehicle must be aware of the necessity or wish to change lane. A lane changing is carried out to prepare for a turn movement, to avoid slower vehicles ahead in the same lane, to avoid lane closures or incidents or to move into a faster lane to achieve the desired travel speed.

2. **Identifying possible lanes to achieve the objective**

The vehicle identifies a set of admissible lanes, based on lane changing regulations, lane use signs, prevailing traffic conditions, desired route etc.

3. **Choosing a target lane**

Lane selection is based on a combination of factors such as: intended turning movement, lane blockage, speed, queuing advantages, special turning lanes, sharing straight-turning lanes, heavy vehicle presence, and transit presence.

4. **Evaluating the gap in the target lane and defining the rules or type of lane changing**

One of the roles of the lane-changing model is to determine the type of lane changing situation based on traffic conditions in the target lane. The process takes into account the spacing and speed of its potential leader and follower vehicles in the target lane. The different possible situations are appended below:

Mandatory or forced Change

In a mandatory or forced lane changing, a vehicle is forced to change lane in order to reach its destination. The reasons for such changes are:

- Connecting to the next link on the path
- Being prepared for the next turn
- The destination requires a change to other lane
- Avoiding a restricted use lane
- Bypassing a lane blockage downstream
- Responding to a variable message sign (VMS)
- An incident in the same lane
- The current lane is blocked
- The current lane is merging to another lane.

When a vehicle is aware of the necessity of undertaking a change of lane, it still has a distance to plan the movement and wait for a gap but the vehicle must merge into the target lane by a certain position on the current link.

Discretionary or Voluntary Change

A discretionary or voluntary lane change will be required when a car is in one of the following situations:

- A vehicle wants to overtake a slower or heavy vehicle;
- A vehicle wants to choose the shorter queue at a junction entrance;
- A vehicle wants to increase its travel speed.

Normally a gap for a voluntary lane change is acceptable when it is greater than a safety distance, which the vehicle wants to keep Incase of sudden braking by the vehicle ahead.

Free Lane Change

A free lane change occurs when the gap between the leading and the following vehicle is large enough so that the maneuver does not disturb the following vehicle.

Forced Lane Change

This type of lane change occurs when a vehicle is losing to its target point but it is not able to find a gap even when its gap-size expectation decreases as the car get closer. Under this situation, the vehicle will slow down and eventually stop to wait for a an opportunity to make the maneuver. After waiting for a few seconds the vehicle may nose into the target lane to “force” the following vehicle to yield. Lead and gaps start to widen after the subject vehicle enters the lane. This has an impact on the car following behavior and the models have to take account of this situation.

Cooperation Lane Change

This lane change refers to a situation when the following car perceived the need of a vehicle to make a lane change and decides to voluntarily slow to create the required gap. This kind of situation is normally associated with congested conditions when the drivers are more willing to understand the difficult situation of other vehicles wishing to change lanes as there is very little opportunity to find a natural gap in the flow.

Performing the Lane Change

Once a vehicle defines the kind of lane change to be executed, it just has to follow a predefined trajectory to move from one lane to the other. Common trajectories are circular arcs and polynomial arcs. A trajectory has to meet the following basic criteria:

- First , the trajectory curve should be continuous, and even the derivative should be continuous as well;
- Second the trajectory should be easy to generate and suitable for different situations, like different velocities;
- Finally, the trajectory should be reasonable and not unrealizable for cars.

2.3.3.3.1 *Lane-changing logic of AIMSUN*

The lane-changing model applied in AIMSUN is also developed based on the Gipps's lane changing model (Gipps, 1986). Similar with the other lane-changing models, the lane-changing model in AIMSUN is also a decision based model which addresses three questions: The necessity, desirability and feasibility of the lane change.

In AIMSUN, three different zones corresponding to different lane changing motivations are considered to generate a more accurate decision, as demonstrated in Figure (2-4). These three zones are defined by the distance to zone 1 and distance to zone 2 in seconds.

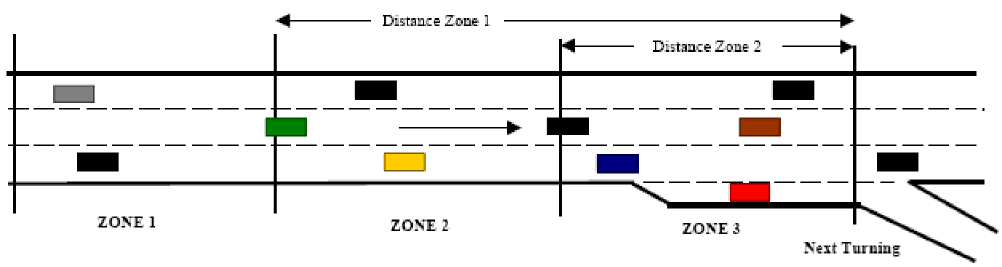


Figure 2-4 Lane Changing zones of AIMSUN lane changing model (Source: Gao, 2008)

For zone 1, the main concern about lane change is the traffic condition of these lanes; for zone 2, the desired turning lane is the main concern; for zone 3, the decision of lane changing mainly depends on the feasibility, which means whether the lane change is possible. (Barcelo et al., 2004)

2.3.3.3.2 *Lane changing logic in VISSIM*

The lane-changing model in VISSIM was originally developed by Willmann and Sparmann (1978). In Sparmann's model, the lane-changing behavior is divided into two types: Lane change to a faster lane and lane change to a slower lane. To make the decision of lane change, three questions need to be evaluated: Whether there is a desire to change the lane, whether the present driving situation in the neighboring lane is favorable, whether the movement to a neighboring lane is possible (Kan and Bhan, 2007). Similar with INTEGRATION, there are also two kinds of lane changes in VISSIM: Necessary lane change and free lane change. The necessary lane change is applied when the vehicle needs to reach the connector of next routine. The free lane change happens when the vehicle is seeking more space or higher speed. No matter which type of lane change it is, the first step for the vehicles in VISSIM is to find "a suitable gap (time headway)" (PTV, 2007).

2.3.3.3.3 Lane-changing logic of PARAMICS

Two zones are defined in the PARAMICS lane changing model. For the lane changing zone one, the vehicle has a distance to the junction and the only reason for its lane changes is to overtake a slower vehicle. For the lane changing zone two, the vehicle is approaching the junction and it may choose not to overtake anymore. The lane changes are only for reaching the appropriate lane to make the turn for this zone. (Jiménez et al, 2004). Duncan (2000) stated that the lane changing logic in PARAMICS is applied using "a gap acceptance policy". It means that when the vehicle is trying to change to another lane, the following two conditions have to be satisfied: The subject vehicle will not result in a collision with the front vehicle in the target lane; the subject vehicle will not result in a collision with the vehicle behind it in the target lane.

2.3.3.3.4 Lane-changing logic of CORSIM

Lane changing logic in CORSIM is based on Gipps's decision model (1981) which is described earlier. The logic considers mandatory and discretionary lane changes. A mandatory lane change is defined as when the driver must leave the current lane for the next exit. Discretionary lane change is defined as when the driver is seeking better traffic condition in the target lane. (Rakha and Zhang, 2004)

2.3.3.3.5 Lane-changing logic of INTEGRATION

Both mandatory and discretionary lane changes are considered in INTEGRATION's lane changing logic. Mandatory lane change is applied when there is "a need for vehicles to maintain lane connectivity at the end of each link". For discretionary lane changes, first the potential speed at which vehicle could continue to drive in its current lane and the potential speed at which the vehicle could drive after changing to the adjacent left or right lane are computed and compared every deci-second based on the available headway in each lane. The model also scans all lanes on a roadway every 0.5 s. The precondition of the discretionary lane change is that there must be an adequate gap in the new lane. After the discretionary lane changes are made, the mandatory lane changes become primary in respect of the lane connectivity at the end of the link. The lane changing model in INTEGATION internally computes the lane connectivity at any diverge or merge, which saves a lot of time for model users of coding link connectivity. (Van Aerde and Rakha, 2007).

2.4 TRAFFIC CHARACTERISTICS IN THE KINGDOM OF SAUDI ARABIA (KSA)

Saudi Arabia has one of the highest fatality risk levels in the world in terms of traffic accident fatalities with around 29 deaths per 100,000 people. In numbers, more than 6450 people get killed and more than 36400 get injured due to traffic accidents in Saudi Arabia annually (WHO, 2009). This is considered a very high rate when compared with other countries. Many researchers already investigated on that issue and found improper driver behavior is the primary cause of accident at signalized urban intersection; running a red light and failing to yield are the primary contributing causes (Al-Ghamdi, 2003). Literature shows that erratic driving in some cases is highly associated with failing to judge an appropriate time and space gap to complete a safe maneuver in the complex intersections. This phenomenon is related to the time headway and the reaction time of the driver. A proper match of the two parameters would reflect the driver behavior in real field condition.

Many Studies have assessed the state of traffic safety in Saudi Arabia. All of these authors agreed that road safety is a very serious problem in Saudi Arabia despite the existing wide and well-maintained roads network. They also agreed that this problem is partially due to the wrong behaviour of drivers and other road users. The most frequently cited and observed violations on the roads are over speeding, red-light crossing, excessive lane changing, tailgating, not wearing seat belts and turning from the wrong lane (Bendak, 2011). There are a number of reasons that seem to be contributing to this behaviour of ignoring red lights.

In several studies of time headway analysis it was reported that, the sample coefficient of variation CV (The proportion of sample standard deviation and sample mean) values fall in the range of 0.5 to less than 1.5 over a range of flow rates from less than 500 to greater than 2,000 vph (Breiman et al., 1977). Over the same range of flow rates (500 to above 2,200 vph), this study shows that the CV is less than 1 in all samples (the range is from 0.32 to 0.82) for time headway. Therefore, the CV from this study is generally shorter than corresponding values from international research (countries outside Saudi Arabia), indicating that a motor car leaves a shorter headway from the car ahead than corresponding drivers in the developed world. This finding may reflect the difference in traffic conditions, particularly driving behavior, in Saudi Arabia (a developing country) and those in developed countries. Such differences may be attributable to the fact that driving behavior in Saudi Arabia tends to be more aggressive (Al-Saif et al., 1990). In studying driving behavior at signalized intersections, Al-Ghamdi (1999a) found that the mean of discharge headways is shorter in Riyadh (Capital of Saudi Arabia) than that in other cities and, accordingly, the saturation flow rate levels are higher (Ali Al Ghamdi, 2001). In addition, the occurrence of traffic accidents due to cars following each other too closely is a typical problem in this country.

2.5 CALIBRATION AND VALIDATION OF MICROSCOPIC AND MACROSCOPIC SIMULATION MODELS USED IN THE KINGDOM

A Number of studies have been conducted in the Kingdom of Saudi Arabia using different simulation models to adopt it and calibrating in order to utilize it for traffic application. Some of those are summarized below.

Ratrout et al. (2009) evaluated the adequacy of the state-of-the art TRANSYT-7F and Synchro to the local traffic conditions of Eastern Province, Saudi Arabia. Queue length data were compared to find accuracy of TRANSYT-7F and Synchro. Also, optimal signal timing plans were developed using TRANSYT-7F and Synchro. Each optimal signal timing plan was simulated using TRANSYT-7F and SimTraffic. The main results of this study indicated that queue length calibration process was carried successfully in

TRANSYT-7F but queue length in Synchro could not be calibrated successfully to the field conditions. Signal timing plan resulted by Synchro improves the system performance more than signal timing plan resulted by TRANSYT-7F.

Al-Jaman (2007) calibrate Synchro/SimTraffic model, focusing on local road traffic conditions by using empirical data from several pre-timed intersections in Riyadh. Four parameters: travel speed, turning speed, headway factor and driver type, were modified to calibrate the model in this study. The results with the calibration showed that there is no discrepancy between the field observed MOE's and simulated MOE's. The calibrated model was then successfully validated with a different set of data in another intersection in the city of Riyadh. The percent error between the observed and simulated value was only 7%.

Ahmed (2005), calibrated and validated the microscopic traffic simulation model VISSIM to the traffic conditions of Khobar and Dammam, Saudi Arabia. The default values for the parameters such as number of observed vehicles, additive and multiplicative part of desired safety distance, amber signal decision and distance required in changing lane were modified to emulate the field conditions. The results with these modified values showed no discrepancy between the model simulation MOE's and the field observed MOE's. In order to validate the calibrated model, another network chosen in Dammam city has been used by using a different set of data. The results of the validation showed that the difference between the field observed MOE's and the VISSIM simulation results are within the acceptable range.

Algadhi (1999) conducted this study to aim at alleviating various traffic system's design and management problems during the Hajj by using a computer-based simulation model AIMSUN. The existing Arafat land-uses and roadway network, and the nine highways connecting it with Muzdalifa were represented by utilizing the AIMSUN2 microsimulation package. Enhancements to AIMSUN2 were introduced to satisfy the specific requirements of Ifadha. The model parameters were calibrated such that predicted and observed vehicle volumes on the highways linking Arafat and Muzdalifa are approximately identical. The calibrated microscopic model is then used to simulate and assess the impact of dedicating some of these highways to the shuttle bus operational strategy.

Ratrout (1996) stated in his study that TRANSYT-7F model which was developed on the theory that a platoon of vehicles starting from an upstream intersection will continuously disperse as it travels downstream along the link. He mentioned that the amount of dispersion in the traffic flow pattern, as predicted by this TRANSYT-7F algorithm, depends on the proper value of an empirical constant referred to as the ""Platoon Dispersion Factor"" (PDF). The objective of this study was to determine the value of PDF which best simulates the traffic conditions in the study area along two major arterials in areas of mixed residential and commercial activities. Each arterial consisted of four signalized intersections and four approaches to each of them. The signals at these intersections were pre-timed with four protected phases. It was concluded that the average best fit (calibrated) PDF values in the study area were 28 and 40 for low and moderate friction links, respectively. On the other hand, the TRANSYT-7F manual suggests a value of 25 for low friction links and 35 for moderate friction links. Nevertheless, the result obtained was within the accuracy limit of TRANSYT-7F model.

Al-Ofi (1994) conducted a study on urban intersections in Dammam and Khobar cities to investigate the effect of signal coordination on intersection safety. In his study he considered TRANSYT, SIGOP, PASSER, and MAXBAND models and found TRANSYT model as the suitable model for this study based on its attractive features over other models and it was already subjected to calibration and validation studies in several countries including Saudi Arabia (Ratrout, 1989). It was concluded that the signal coordination reduces intersection accidents and he suggested a methodology to incorporate safety into an inbuilt optimization algorithm of TRANSYT-7F model.

Al-Ahmadi (1985) performed a study on Khobar downtown area, Saudi Arabia in his thesis dissertation entitled “evaluating policy changes using a network simulation model”. In his study he compared several available network simulation models such as SIGOP III, TRANSYT, and NETSIM and came out with a conclusion that NETSIM is a potential simulation model that can effectively be used to evaluate traffic policy changes for road networks in downtown areas.

2.6 MODEL SELECTION AND COMPARISON

Simulation model selection will affect not only the network modeling process and the required labor, but also the simulation results and, therefore, any user conclusions or recommendations. The selection of a simulation model should be based on its capability of producing accurate results as well as the feasibility of its use for specific applications.

Model comparison can assist users in making correct choices with regards to model selection. Performed at different levels, simulation model comparison entails both conceptual model comparison and empirical model comparison. Besides assessing some general considerations, including modeling cost, speed, system needs, etc, a conceptual comparison evaluates the capabilities of each model. Material for this kind of comparison is mostly found in the user guides of the subject simulation models. The conceptual comparison is an efficient way to understand the modeling features and functionalities of different simulation models in a short time.

The information in this section is intended to complement the description of microscopic models provided previously. The information is presented in a set of Tables from which important conclusion have been made. Tables 2.2, 2.3, 2.4, and 2.5 summarize and compare features and capabilities of microscopic models. Tables 2.2 and 2.3 summarize the information related to research microscopic models while Tables 2.4 and 2.5 contains the information related to commercial microscopic models. In these tables; "Network elements" refers to the infrastructure that form the transportation network as well as the users of this network, "Functions" refers to road operations (or phenomena) occurring in a transportation network and "Output" refers to the kind of information and statistics produced by these models.

From Table 2.2 (functions and network elements of research models), the following points can be observed:

- The most common functions represented in research models are actuated traffic signals and route guidance.

- Most research models can deal with commercial vehicles, traffic incidents and vehicle detectors.
- Research models are weak in representing pedestrians and bicycles.
- Research models deal mainly with urban streets.

Table 2-2 Comparison of Research Microscopic Model

Model	**Functions**						**Network Elements**											
	Actuated traffic signal	Transit Priority	Ramp Metering	Variable Message signs	Regional traffic information	Static/ Dynamic route guidance	Vehicle detectors	Weather condition	Parked vehicles	Commercial vehicles	Bicycles/motorbikes	Pedestrians	Incidents	Public transport	Traffic Calming measures	Roundabouts	Urban Streets	Highway
ANATOLL										√								
AUTOBAHN	√		√	√	√	√	√	√		√			√		√	√		√
CASIMIR	√						√										√	
DRACULA	√	√					√	√		√			√	√		√	√	
FREEVU							√											√
MELROSE	√		√			√	√		√	√		√					√	√
MICSTRAN	√	√	√			√	√	√	√	√		√		√			√	
MITSIM	√		√	√		√	√	√	√	√			√		√	√	√	√
MIXIC								√							√			√
NEMIS	√	√		√		√	√			√			√	√	√	√	√	
PADSIM	√					√	√		√							√	√	
PHAROS																√		
PLANSIM-T	√	√	√	√	√	√				√				√		√	√	√
SHIVA							√											
SIGSIM	√	√	√				√		√	√	√		√	√			√	
SIMDAC													√		√			
SIMNET	√	√	√	√		√	√		√				√	√	√	√	√	
SISTM			√	√		√	√	√		√			√					√
SITRA-B+	√	√				√	√		√	√			√	√		√	√	
SITRAS	√					√	√			√			√				√	
THOREAU	√		√	√		√	√	√	√			√	√		√	√	√	

Source: SMARTEST, 1997

Table 2-3 Comparison of Research Microscopic models

Model	**Outputs**									**Others**			
	Travel time	Speed	Congestion	Queue Length	Emission/ Pollution	Noise Level	Number of Accidents	Interaction with pedestrians	Fuel Consumption	Default value of key parameter	User can modify key parameter	Graphical Network Builder	Graphical Animation of Results
ANATOLL										√	√		
AUTOBAHN	√	√	√		√	√					√		
CASIMIR	√			√					√	√	√	√	
DRACULA	√						√		√	√	√		√
FREEVU	√	√								√	√		√
MELROSE	√	√	√	√						√	√	√	√
MICSTRAN	√	√	√	√	√			√		√	√		
MITSIM	√	√	√	√	√					√	√		√
MIXIC	√	√	√		√	√	√	√	√	√	√		√
NEMIS	√	√	√	√	√	√	√			√	√		√
PADSIM		√	√	√						√			√
PHAROS										√	√		√
PLANSIM-T	√	√	√	√			√		√	√	√		√
SHIVA	√	√	√		√	√				√	√		√
SIGSIM	√	√	√	√						√	√		√
SIMDAC		√			√	√					√		√
SIMNET	√	√	√	√			√		√	√	√		
SISTM	√	√	√	√	√					√	√		√
SITRA-B+	√	√	√	√							√		√
SITRAS	√	√	√	√						√	√		√
THOREAU	√	√	√	√	√					√	√		√

Source: SMARTEST, 1997

From Table 2.3 the following observations are important

- Research microscopic models are weak in producing a variety of output data. Most of these models only produce most common statistics (travel time and speed).

- Data input is generally done without the support of a graphical user interface.
- Graphical animation of results is a common feature in research models. This feature allows the user to observe the interactions between vehicles, shock waves, weaving zones and queues.
- A good feature of research models is the fact that users can adjust key parameters, which are in most of the cases calibrated for general conditions.

Table 2-4 Comparison of Commercial Micro-simulation models

Model	**Functions**							**Network Elements**															
	Actuated traffic signals	Transit priority	Ramp metering	Variable message signs	Regional traffic information	Congestion pricing	Static/Dynamic route guidance	Vehicle detectors	Weather conditions	Parked vehicles	Commercial vehicles	Bicycles/ motorbikes	Pedestrians	Incidents	Public Transport	Traffic calming measures	Roundabouts	Reversible lanes	HOV lanes	Bus Only lanes	Toll Plaza	Urban Streets	Highway
AIMSUN2	√		√	√			√	√						√	√		√	√	√	√	√	√	√
FLEXSYT II	√	√	√					√			√	√	√	√	√	√	√		√	√	√	√	√
HUTSIM	√	√	√	√		√	√	√				√	√	√	√	√	√			√	√	√	
PARAMICS	√		√	√	√	√	√	√	√		√	√		√	√	√	√	√	√	√	√	√	√
SIMTRAFFIC	√							√			√		√									√	√
TSIS/CORSIM	√	√	√					√		√	√		√	√	√		√		√	√	√	√	√
VISSIM	√	√	√			√		√		√	√		√	√	√	√	√	√	√	√	√	√	√

Source: SMARTEST, 1997

Some important points, which can be observed from Table 2.4 (functions and network elements of commercial models) are:

- SIMTRAFFIC appears to be the commercial model with lowest capabilities although it is fair to say that the main focus of SIMTRAFFIC is the analysis and optimization of signal plans (through SYNCHRO); a feature that is lacking in other software.

- As can be observed, commercial software is very competitive due to its ability to reproduce most of the network elements and phenomenon observed in traffic streams. Pedestrian and transit modeling is commonly available in commercial models but that is not the case with bicycle and motorcycle modeling.
- Commercial models include both urban roads and highways and they are more flexibility in representing different types of infrastructure and operations (i.e. HOV lanes, roundabouts, and traffic calming).

Table 2-5 Comparison of commercial Microsimulation model

Model	**Output**												**Others**					
	Travel time	Speed	Congestion	Queue Length	Capacity	Delay	Stops per vehicle	Level of service	Emission/ Pollution	Noise Level	Number of Accidents	Interaction with pedestrians	Fuel Consumption	Default value of key parameter	User can modify key parameter	Graphical Network Builder	Graphical Animation of Results	3D Visualization
AIMSUN2	√	√		√		√	√		√		√			√	√	√	√	√
FLEXSYT II	√	√	√	√		√	√		√		√		√	√	√	√	√	
HUTSIM	√	√	√	√		√	√		√		√		√	√	√	√	√	
PARAMICS	√	√	√	√	√	√	√	√	√		√	√	√	√	√	√	√	√
SIMTRAFFIC	√	√		√		√	√					√	√	√	√	√	√	
TSIS/CORSIM	√	√	√	√		√	√		√		√		√	√	√		√	
VISSIM	√	√	√	√		√	√		√	√	√		√	√	√	√	√	√

Source: SMARTEST, 1997

The following points can be observed from Table 2.5:

- Commercial software has developed tools, which graphically support the network building process as well as to graphically show the simulation results. Some of these models can even show a 30 simulation. This is particularly useful in dealing with public hearings and discussions.
- Commercial models are still very poor in producing data and statistics to allow for the detailed analysis of different parameters for various transportation applications.
- As in the case of research models, users have total control over the key parameters controlling the simulation logic. These key parameters have default values that are normally calibrated for common conditions.

Some other important observations obtained from the literature and the analysis of Tables 2.2, 2.3, 2.4, and 2.5 are:

- Commercial models include more capabilities and features than research models because they must be responsive to both consulting and research groups interested in solving and analyzing a large variety of planning and operational issues related to transportation systems.
- A clear gap in both research and commercial models is the lack of pedestrian and bicycle/motorcycle modeling.
- PARAMICS and VISSIM seem to be the most complete models. These microsimulation tools stem from similar research backgrounds with car-following, lane changing, and behaviour simulation engines as core

components of the initial research. Although these packages are based on different algorithms, each is widely accepted within the academic and research community.

- Because most microsimulation models provide sensible default values and the capability for users to change key parameters, microscopic models may be adapted for representing the different traffic conditions existing in different countries, regions, roads, vehicles and driver populations.
- Most of the models provide indicators to measure speed and travel time and to a lesser extend congestion, travel time variability and queue length.
- Most packages use graphical displays showing a simulation, and therefore queue spill back and weaving can be observed.
- Parking issues, bicycles/motorcycles, pedestrians, and weather conditions can be considered for practical purposes as not included in microscopic models.
- Model developers need to improve the set of the existing model indicators and statistics so microscopic users can analyze traffic problems under different measures of effectiveness.

Most models are constantly being updated with the core logic and capabilities; therefore the data shown in Tables 2.2 to Table 2.5 may not reflect the current state of a model's functionality.

As of today, we were unable to find any report or study in which the microscopic model Quadstone PARAMICS has been used in the Kingdom of Saudi Arabia for traffic statistical analysis, traffic policy making and addressing their effect in the whole transportation system. Therefore, there is a potential prospect of using Quadstone PARAMICS extensively to address and solve few of the traffic related problems that the

Kingdom is encountering over the years. Therefore, based on the above discussion Quadstone PARAMICS was selected for this study. For simplicity Quadstone PARAMICS will be referred to as PARAMICS only in the following chapters.

2.7 LITERATURE REVIEW ON CALIBRATION AND VALIDATION OF PARAMICS

Several studies had been undertaken in the recent past and since the evolution of PARAMICS microscopic simulation model for calibration and validation of different network around the world by different researcher groups and companies. As the driver behavior and network geometry varies region to region around the world, there is an utmost need that the calibration process is conducted with different values of different input parameters other than using the default value. Some of the studies reported in the last decade are summarized below.

Zhe et al. (2010) in their paper proposed a systematic, practical procedure for microscopic simulation model calibration and validation. The validity of their proposed procedure was demonstrated via a case study in a freeway in Guangdong Province, China using microscopic traffic simulation model, PARAMICS. The simulation results compared against multiple days of field data to determine the performance of the calibrated model. They found that the calibrated parameters using the proposed procedure generated performance measures that were representative of the field conditions while the simulation results of the default parameters were significantly different from the field data. In this paper they presented a Generic Algorithm technique while using 2^{k-p} fractional factorial design for the calibration and validation procedure for microscopic simulation models. The validation was evaluated by comparing of simulation output to the multiple days of field data. The result shows that 2^{k-p} fractional factorial design was found to be useful in identifying the reasonable and appropriate ranges of calibration parameters. Conducting the sensitivity analysis and calibration, the researchers conclude that if the users can accept the relative lower simulation precision, a set of calibration parameters like, mean target headway and mean reaction time, are enough; however, to obtain a high simulation precision, the time step and aggression distribution should also be calibrated together. This study used travel time as the only one Measure of Effectiveness (MOE) for model calibration. They suggested that further research is recommended to include more MOEs in the calibration process.

Zhe et al. (2009), they presented a procedure for the calibration and validation of PARAMICS with toll data. They have identified important parameters of PARAMICS using 2k-p fractional factorial design and calibrated by using the detailed vehicle-by-vehicle toll data. A freeway in Guangdong Province, China, has been selected as test site. The simulation results after calibration and validation showed that the parameters like target headway, mean reaction time, simulation step, aggressive distribution affects the simulation precision most deeply, and the calibrated simulation model is able to adequately represent freeway traffic conditions.

Lee et al.Ozbay (2008) studied previous works on calibration and found that those studies generally focused on minimizing the sum of relative error between the observed data from a certain period of time in a typical day and the simulation output for the same period. They presented a static approach in this paper which can be explained as calibration with data obtained at one point in time. This paper proposes a calibration methodology based on the Bayesian sampling approach. Instead of a single demand matrix and corresponding observed traffic conditions that represent a specific point in time, this calibration methodology uses randomly generated demand matrices and corresponding traffic conditions from an observed statistical distribution of these variables. The goal of using input values generated from an observed distribution of demands is to accurately represent a wide range of all likely demand conditions observed at a facility. Moreover, a stochastic optimization algorithm, known as Simultaneous perturbation stochastic approximation (SPSA) algorithm is used in each iteration to re-estimate optimal parameters for the calibration. The proposed enhanced SPSA algorithm outperforms a simple SPSA algorithm based on several case scenarios studied as part of this paper. However, this type of calibration approach cannot capture a realistic distribution of all possible traffic conditions and may yield inaccurate calibration results.

Pinna (2007) used generic algorithm for selecting the input parameters while calibrating and validating the PARAMICS model for a highway traffic network between the sites of Veenendaal and Maarsbergen, in the province of Ulrecht, the Netherlands in his M.Sc project. He proposed an algorithm that regulates the flow of the vehicles on the network for the calibration of the input parameters. He found that by means of the algorithm for calibration with simulated data, the optimization routine pattern search has been selected as the most efficient for such a task, as it prevailed on fmincon (a function included in MATLAB's Optimization Toolbox which seeks the minimizer of a scalar function of multiple variables, within a region specified by linear constraints and bounds) and GA (Generic Algorithm). The accuracy on the results obtained with pattern search and with GA is quite high compared to the one obtained with fmincon.

Oketch et al. (2005) used PARAMICS model to calibrate and validate a small network in the city of Niagara Falls, Canada. Their calibration effort involved comparing the model results to the observed data with traffic volume and turning movement counts at intersections. They have also taken into account the measure of effectiveness such as travel time and approach queues in the calibration process. They found that there was an acceptable match between modeled and observed results with moderate calibration effort.

Chu et al., (2004) presented a systematic, multi-stage procedure for the calibration and validation of PARAMCIS simulation models. The procedure is demonstrated in a calibration study with a corridor network in the southern California, USA. While previous studies focused mostly on driving behavior model calibration to study a section of freeway, this study provides a general scheme of model calibration and validation for network-level simulation. The proposed procedure is demonstrated via a case network that involves multiple steps, and the calibrated model showed reasonable performance in replicating the observed flow condition. In their paper, they have used the default route choice model in PARAMICS as there is a close interaction between the route choice model and OD (Origin-Destination) estimation problem. In the network level model calibration/validation process, the problem gets more complicated due the inter-relationship between route choice and OD estimation, though it can be solved if one of the component is determined externally. This problem opens the door for further studies in the micro-simulation calibration/validation process.

Lianyu et al.(2004) proposed a calibration procedure for the PARAMICS microscopic simulation model. While most of the previous studies focused mostly on driving behavior model, they have attempted to describe a calibration procedure which took into account a broader aspect of traffic parameters and described a general calibration steps. The found that the PARAMICS model performed remarkably in replacing the observed condition while working on a network in the city of Irvine, Orange County, California.

Gardes et al. (2002) have evaluated freeway improvement strategies on Interstate 680 in the San Francisco Bay Area using the analysis produced by PARAMICS. The study mostly addressed the importance of calibrating the model and describing the process of developing a calibrated model in detail. The authors recommended four key components network characteristics, traffic demand, overall simulation configuration, and driver behavior factors need to be addressed when calibrating the model.

A research group from Portland State University (April, 2002) has applied PARAMICS in a diamond interchange at Wilsonville Road located in the City of Wilsonville, Oregon in submitting an official report to the state of Portland in USA. From this study they observed that there are negligible differences between the simulated interchange delay results from the PARAMICS model and delays described in HCM 2000 methodologies.

In traffic demand variation they found that for there was a substantial delay as large as 74.9 seconds for the ramps left turning movement in the interchange.

Ma et al., (2002) used GENOSIM, a generic traffic microsimulation parameter optimization tool that uses generic algorithms while implementing in the Port Area network in downtown Toronto, Canada. GENOSIM was developed as a pilot software as part of the pursuit of a fast, systematic, and robust calibration process. It uses the state of the art in combinatorial parametric optimization to automate the tedious and cumbersome task of hand calibrating traffic microsimulation models. The employed global search technique and genetic algorithms that can be integrated with any dynamic traffic microscopic simulation tool. In this research, GENOSIM was used in combination with PARAMICS. Genetic algorithms in GENOSIM manipulate the values of those control parameters and search for an optimal set of values that minimize the discrepancy between simulation output and real field data. Results obtained by replicating observed vehicle counts are promising.

Lee et al., (2001) described the importance of calibrating the PARAMICS model for local traffic conditions while working in a one-mile segment of Interstate 5 in Orange County, California. Real-time loop detector data had been collected and used and two field data sets in both calibration and validation processes. The authors stated that there are two key parameters used for calibration in the study were mean target headway and mean reaction time. They found that there is a significant difference in these calibrated parameter values between California drivers' behavior and the default values used in PARAMICS.

Stewart, P. (2001) described a study using PARAMICS to assess ramp meter control for eastbound traffic on Motorway 8 (M8) in Scotland. The author stated that traffic flows, speeds, travel times, and behavior over strategic sections of the M8 were compared with respect to the base model for evaluation. They found that the traffic simulation software helped confirm that the introduction of ramp metering has improved the flow of traffic on the M8.

As mentioned above several numbers of calibration and validation studies of PARAMICS microscopic simulation have been conducted in the past. Most of them have used mean target headway (MTH) and mean reaction time (MRT) to be the major calibration parameters for PARAMICS. Few of the also suggested that to get a greater match between the observed and simulated MOEs parameters such as driver Aggression and familiarity can also be used in addition to MTH and MRT. Final values of the calibrated parameters of few of those studies conducted are listed below for reference:

Table 2-6 Calibration of parameters in PARAMICS

Author Name	**Calibrated Parameter**	**Default Value (s)**	**Calibrated Final Value (Seconds)**	**Optimization Methodology**	**Objective function**
Ozbay, K (2003)	MTH	1.0	0.70	SPSA algorithm	Flow, density
	MRT	1.0	0.50		
Ma and Abdulhai (2002)	MTH	1.0	0.86	Genetic Algorithm	Volume
	MRT	1.0	0.71		
Gardes et al. (2002)	MTH	1.0	1.65	Not Available	Speed, Volume
	MRT	1.0	0.42		
Lee et al. (2001)	MTH	1.0	0.625	Not Available	Link Flow
	MRT	1.0	0.415		
Zhe Li (2010).	MTH	1.0	0.45	Genetic	Travel Time

Author Name	Calibrated Parameter	Default Value (s)	Calibrated Final Value (Seconds)	Optimization Methodology	Objective function
	MRT	1.0	0.43	Algorithm	
Jobanputra, R. et al. (2012)	MTH	1.0	0.50	Not Available	Flow and Turning Movement
	MRT	1.0	1.00		
Chu et al. (2004)	MTH	1.0	0.78	Manual Iteration	Flow and Travel Time
	MRT	1.0	0.66		

MTH- Mean Target Headway; **MRT**- Mean Reaction Time

CHAPTER 3

SOFTWARE DESCRIPTION

3.1 INTRODUCTION

PARAMICS is a microscopic urban and freeway traffic simulation software used to model the movement and behavior of individual vehicles on road networks. It is widely used in the United Kingdom and it is becoming more popular in North America and other regions of the World. PARAMICS was originally developed at the University of Edinburgh's Parallel Computing Centre (EPCC) in 1992, in partnership with a leading U.K. transportation consultant, SIAS Ltd. In 1996 several of EPCC's staff left to form Quadstone Ltd., a company specializing in the development and marketing of high performance software. Quadstone and SIAS formed a joint venture company to continue the development of PARAMICS but they separated in 1997 and since then have independently developed separate versions of the PARAMICS software. Both versions were originally the same but as time has passed they have become very different packages, although both include very similar features (Aldazaba, 2004).

This chapter will introduce the basic principles of Quadstone's PARAMICS 6.1. For simplicity it will be referred to as PARAMICS from now on. PARAMICS is a complex software package, yet can be easy to use. It includes many features, which the user can employ in testing transportation schemes and applications. It would not be a prudent idea to discuss here the entire functionality of the software and the reader can refer to the information included in the manual, on-line resources and various reports published by Quadstone for further details. Discussion about many of the topics would be omitted as well as those are beyond the scope and objective of this research. For example the ability of PARAMICS to model transit issues is very powerful but it will not be discussed here.

3.1.1 PARAMICS overview

The name PARAMICS is an acronym derived from PARAllel MICroscopic Simulation, which relates to the early developments at the Edinburgh Parallel Processing Centre. PARAMICS was developed as a result of six-year collaboration between specialists in high performance software, QUADSTONE and the traffic and transportation consultants, SIAS. The software was designed from the very beginning to take specific advantage of modern computer architecture. PARAMICS includes a sophisticated microscopic car following and lane changing model, dynamic and intelligent routing, inclusion of intelligent transport systems, and an ability to interface with other common microscopic data formats and real-time traffic input data sources. It takes full account of public transportation and its' interaction with other modes, particularly at bus stops and through bus priority measures.

There are five modules within the PARAMICS software package: Modeller, Processor, Analyser, Programmer, and Monitor. By using a Graphical User Interface (GUI), the Modeller module provides the ability to build, simulate, and visualize the road network. The Processor module also performs the same functionality as Modeller but with a faster speed as there is no visualization interface. The Analyser module uses output data generated by Modeller to present the results in tabular and graphical format for further off-line analysis. There is another supplementary module called Estimator, which actually capable of converting the traffic flow into separate O-D zones.

Movement of different types of vehicles can be modeled by PARAMICS. Vehicle type can be distinguished by physical characteristics such as length, height, width, weight, and maximum speed. In addition to cars and trucks, public transport or transit such as buses, light rail trains, and heavy rail trains can be modeled. At the signal-controlled intersections pedestrian interaction with the road network can be modeled through the provision of pedestrian phases. There are provisions for Bicycle traffic modeling within PARAMICS where the network allows for dedicated bicycle lanes.

The motion of vehicles in PARAMICS results from a combined process of a series of discrete steps, which, when strung together, result in the perception of movement. When viewed altogether on a computer screen, the "picture" is refreshed at each time step as the vehicle changes its position on the network. Reference is made to research conducted at the British Transportation Research Laboratory (TRL) that the two parameters: aggression and awareness can be used to describe driver behavior. PARAMICS randomly assigns aggression and awareness values to the driver of each vehicle on a scale of 1-8 that are active in the network. Using PARAMICS, the user can change the type of statistical distribution (i.e., Normal, Poisson) of the aggression and awareness parameters to reflect regional or local variations in driver behavior. Once the aggression and awareness parameters is assigned, three interacting models then control the movement of each vehicle: a vehicle following model, a gap acceptance model, and a lane changing model.

In order to represent reality PARAMICS divides the time into a sequence of sub one-second steps. The size of each step is configurable at each simulation but by default is 0.5 seconds. At each time step, each vehicle in the simulation is assessed, with regard to its situation with respect to its surroundings (other vehicles and the network). The model adjusts each vehicle's acceleration, location, right of way, required gaps, route, and lane targets in this time step.

Acceleration is basically determined by the desired headway, the speed difference between the leading and following vehicle, the maximum acceleration of the vehicle, the reaction time, and the distance between leading and following vehicles. Acceleration is affected (overwritten) by the following situations:

- A vehicle with a higher priority on or near the target junction
- A requirement to stop, turn, or reduce the speed at the next junction
- The requirement to obey traffic signals
- A need to adjust to the speed in order to realize a lane change
- A bus stopping
- The need to wait for a suitable gap
- Merging traffic

Once acceleration is calculated the vehicle speed is determined and its position is updated. The right of way is based on priority rules, which are based on a designation associated to each movement in the junction. A movement can be designated as:

Barred: means that no vehicle can make such a movement

Minor: means that the movement is opposed by more than one stream of traffic and minor traffic yields to medium and major traffic.

Medium: means that the movement is opposed by one stream of traffic and medium traffic yields to major traffic

Major: means that the movement is completely unopposed and that the other streams have to yield.

During every time interval, each vehicle assesses an appropriate target speed for crossing the next intersection on its route. The assessment includes the following rules:

- If the next link and lane is blocked back, set the target speed to zero
- If the priority is MAJOR, set the target speed to the maximum possible turning speed. The turning speed is calculated by considering the radius and angle of the movement.
- If the junction is clear, set the target speed to the turn speed.
- If the junction is not clear, set the target speed to half of the turning speed.

Vehicles adjust their acceleration to achieve the target speed at the end of the link. Every link in the network has a start and end point, which are known as stop lines. Between these two points PARAMICS operates a one-dimensional simulation (the car is just concern about its position in the lane). As the vehicle passes the end point (which corresponds to the end of the link), it shifts to a two-dimensional simulation in order to cross the intersection. The two dimensional model allows the vehicle to move through the intersection without the need of a specific lane.

On ramps an approaching vehicle will set its acceleration to stop at the end of the ramp, to fit in behind an offside vehicle, to get ahead of the offside vehicle, or to obey normal lane changing rules to merge into the traffic flow. Route choice is re- evaluated every time a vehicle moves onto a new link. The route choice is made from a route table which details the anticipated time from each turn at the end of the link to each destination zone. There is a route table for 'familiar drivers' (drivers that know the network very well) as well as a route table for 'unfamiliar drivers'. Familiar drivers have equal cost factoring for both major and minor routes. Unfamiliar drivers weight minor roads at twice the cost of major roads. The cost factor will be discussed in the following section. This emulates the fact that unfamiliar drivers prefer major roads because they don't know what to expect from minor roads. The number of route tables grows when the user defines restrictions in the network (i.e. minor streets don't allow heavy vehicles). Tables for familiar drivers are recalculated to reflect changes in the modeled delays. The frequency of this recalculation is defined by the user and is known as the feedback period. As feedback operates, vehicles may re-route as a result of congestion in the network.

Each type of vehicle has a routing tolerance referred to as 'perturbation', and at each route decision point the costs in the routing table are randomly varied by a factor up to the perturbation value. The resulting minimum value is the choice selected, which may vary for each vehicle. When a vehicle has determined its route choice, and therefore its next two turning movements, it determines the range of available lanes to keep to its route. If a lane change is required, this is made when a gap in the target lane is available and adjacent to the simulated vehicle. The change become progressively more urgent in preparation for a turn and a smaller gap becomes acceptable as lane choice becomes restricted.

At a junction, or as a vehicle passes, a 'hazard warning distance', a vehicle will send a 'scout' two junctions ahead. This scout will determine the lane range available to this vehicle based on the required turns and lane restrictions. Lane choice will then be made from this range. Less aggressive drivers will tend to the nearside lane and more aggressive drivers to the offside. The actual hazard warning distance is also dependent on the aggressiveness of the driver.

If a vehicle is in the wrong lane, or if it is caught in traffic and there is a less congested lane within its range, it will attempt to make a change. If the current lane is outside the lane range, an urgent lane change is requested and gap acceptance is reduced. When a car does not find a gap, it will crawl forward even if it is in a lane, which doesn't allow its intended turning movement at the end. Vehicles in the correct lane may reduce their speed to allow for a 'courtesy let in', but any single vehicle will only allow one such movement on a link. Thus, in congested conditions a vehicle may be seen to arrive at a turn in the wrong lane, hopefully looking for a gap in a similar way to how this may occur in reality.

The 'hazard warning distance' defines a point on the link from which a vehicle begins to be aware of any action required at the next node. Vehicle behaviour is significantly affected by the 'hazard warning distance' because it alerts them to get in lane for the next turn, and to re-assess their speed and lane range.

In PARAMICS the actions of individual vehicles are affected by its surroundings (geometries, controls and other vehicles) and it also influences the decisions of other vehicles. A simulation is then a complex combination of traffic patterns (origin-destination trips), individual behaviour, circulation rules, traffic controls (signals, stop signs, etc), congestion levels, and vehicle interactions.

3.1.2 Car following and Lane changing models in PARAMICS

The PARAMICS model was based on the research work undertaken by Hans- Thomas Fritzsche (1994) in Germany. The details of PARAMICS models are not openly available in order to maintain its leadership in the market. Though the car following and lane changing model has already been discussed in section 2.3.2.3 and 2.3.3.3 respectively, a brief general explanation of the main basis for these models is offered here again.

Car following models are based on the idea that each vehicle/driver has a target headway that varies according to:

- The presence of single lane highways (no lane changing is possible)
- Environment conditions (fog, rain, darkness)
- Proximity to a merging zone
- Proximity to a traffic signal
- Type of vehicles

- Vehicle aggression and
- Vehicle awareness.

A vehicle varies its speed to achieve its target headway. The drivers' reaction time is modeled by basing the calculation of the necessary acceleration on the speed at which the vehicle in front was travelling at some time in the past. The introduction of a reaction time results in the effective simulation of backward travelling shock waves. Vehicles change their speed according to the speed of the vehicle ahead. Speed changes are normally smooth but may be abrupt if the follower car perceives brake lights or a "notable" change of acceleration in the leading vehicle.

Acceleration and deceleration always depends on the speed difference between leading and follower vehicles but it gets more or less critical depending on what cruising situation the follower vehicle is experiencing. Lane changing models are based on a gap acceptance policy. A vehicle wishing to change lane (changing vehicle) first locates the lane where the driver wishes to be in (target lane). Once the 'changing vehicle' knows its target lane, the gap parallel to its current position is checked. When checking this gap, the changing vehicle will measure the following gaps:

- The imaginary gap between the projection of its front and the back of the leading vehicle in the target lane (front gap).
- The imaginary gap between the projection of its back and the front of the following vehicle in the target lane (back gap).

If both of these gaps are equal or more than a minimum expected value for more than few seconds the changing vehicle executes the maneuver. The minimum expected value for the front gap is different than the minimum expected value for the back gaps because these values depend on the speed differences between the related vehicles. This allows the model to take into account the speed differences between lanes and between vehicles.

3.1.3 Assignment and route choice model

Assignment and route choice are based on the following rules and tasks: A vehicle enters the network on a link whose centre point is in the vehicles zone of origin. Once on a link the vehicle determines it's next two turns based on the following criteria:

- A shortest path algorithm based on travel cost and the vehicle's destination. The travel cost is calculated by combining travel time and travel distance.
- Familiar drivers use the actual travel time, which is refreshed every "feedback period". The user defines this feedback period.
- Unfamiliar drivers use a travel time calculated from the free flow speed and link distance. This remains constant during the entire simulation.
- If feedback is disabled, familiar and unfamiliar drivers use the travel time calculated from the free flow speed and link distance.
- To spread traffic among paths having similar costs, the travel cost is modified by adding or subtracting a randomized value. This modification makes it possible to define the shortest path. This path may not in reality be the shortest path but is very close to it, in terms of cost. The variance of the randomized value is control by a 'perturbation factor'. The perturbation factor is defined and calibrated by the user.
- Based on its next turns, the vehicle changes to the appropriate lane, keeps going to the end of the link, and executes its first target turn.

- Once the vehicle reaches the new link, it recalculates again the next two turns. A vehicle keeps moving through the network in the same way until it reaches a link located in its destination zone. This link acts as a sink and the trip terminates.

It is important to notice that when a vehicle enters a network, the driver knows his/her destination zone but not the route or path to get there. The path is defined as the vehicle travels on the network.

3.2 MODEL PARAMETERS AND VARIABLES

PARAMICS is a software with multiple variables that makes it very complex. All the variables are listed and described briefly in the following sections.

Table 3-1 Vehicle Parameters

Parameter	Description
Type	Different types of vehicles that share the same characteristics
Proportion	The proportion of each type of vehicle.
Top speed	The maximum speed the vehicle can achieve.
Length, width, height and weight	These parameters define the dimensions of the vehicle.

Source: (Aldazaba, 2004)

Table 3-2 Road Parameters

Parameter	Description
Major/Minor	Roads can be classified as major or minor. This classification affects the way unfamiliar drivers decide on their route. Unfamiliar drivers perceive double the cost of roads classified as minor

Parameter	Description
Urban/Highway	Roads can also be classified as urban streets or highways. This affects the behaviour, with vehicles more likely to change lane in urban areas.
Category	Road links can be associated with a set of predetermined road features (speed, width, lanes, cost factor, major/minor, and urban/highway).
Width	Defines the width of the road.
Speed	Defines the posted speed.
Lanes	Defines the number of lanes
Restrictions	Restriction on the use of a road for all vehicles or for vehicles having specific characteristics can be specified, for example, weight, length, type.
Stay in Line	If enabled, it prohibits lane changing on the link
Overtaking	If enabled, vehicles are allowed to use opposing lanes for overtaking.
Gradient	Defines the gradient of the link, as a percentage.
Link Cost factor	Allows the user manipulate the perceived cost of a link. It is useful to compensate for situations that the model cannot reproduce, for example, driving conditions on a rough road.
Category cost factor	Similar to link cost factor but applies for all the links classified under a specific category.

Source: (Aldazaba, 2004)

Table 3-3 Junction Parameters

Parameter	Description
Priority	Turning movements in a junction can be classified as major, medium, minor or barred.
Signal Timing (Green/red/amber)	It allows the user to define the signal timing including offset and actuated signals.

Parameter	Description
Force Merge	If enabled, it allows vehicles on a low-priority link to force their way into slow moving traffic on a turn to the left, over riding the normal junction priorities. The forcing-in happens only after the vehicle has been stopped for many seconds.
Force Across	As for forced merges but more extreme. This allows vehicles to force their way across opposing streams of traffic to make a turn from a link.
Staking Left turn	This option allows left turning vehicles to queue in the centre of the junction, at a green light when opposing traffic prevents the maneuver.
End stop time	This forces vehicles to stop for a given number of seconds at the end of the link.
End speed	Sets the target speed at the end of the link. It is useful to simulate traffic calming measures.
Visibility	Sets the distance from the junction at which vehicles will begin to anticipate the available gaps in a major priority flow.

Source: (Aldazaba, 2004)

Table 3-4 Driver behaviour and route choice parameter

Parameter	Description
Aggression	This parameter is associated with the vehicle/driver's level of aggression. The level of aggression goes from 0 (no aggressive) to 8 (very aggressive). An aggressive vehicle/driver accepts smaller gaps, keeps a shorter headway, tends to change lanes more frequently, and tends to keep to the offside lane.
Awareness	This parameter is associated with a vehicle/driver's level of awareness. The level of awareness goes from 0 (no awareness) to 8 (very aware).
Familiar/unfamiliar	A vehicle can be classified as familiar or unfamiliar. Unfamiliar drivers weight minor roads at twice the cost of major roads so they mainly use major roads to reach their destination.

Parameter	Description
Feedback period	Defines the interval at which the actual travel time is made available to familiar drivers in order to recalculate their route.
Feedback coefficient	This is the controlling coefficient that weights the influence of the actual travel time in the cost formulation.
Perturbation factor	It controls the maximum variance in the perceived cost of alternative routes so vehicles may spread themselves among routes offering similar travel cost.
Perturbation Algorithm	Allows the user two different ways of choosing perturbation
Distance/time cost factors	Distance cost factor and time cost factor weight the influence of the travel distance and the travel time in the calculation of travel cost.

Source: (Aldazaba, 2004)

Table 3-5 Car following and Lane changing Parameter

Parameter	Description
Mean Target Headway	Specifies the global mean target headway, in seconds, between a vehicle and a following vehicle. This will not necessarily be equal to the mean measured headway: the relationship between target and actual depends on traffic flow levels, driver behaviour and several other factors. The default value is 1.0 second.
Mean Reaction Time	The mean reaction time of each driver, in seconds. The value is associated with the lag in time between a change in speed of the preceding vehicle and the following vehicles reaction to the change. The default value is 1.0 second.

Source: (Aldazaba, 2004)

Table 3-6 Simulation control parameters

Parameter	Description
Seed	Sets the random seed generator, which is used to determine the release times, the randomization of the perturbation, and the random assignment of attributes such as aggression and awareness.
Steps per second	Defines how many times per second the model will recalculate the status of vehicles and network elements.
Simulation Start and· Simulation Duration	They define the period of the day that is being simulated.

Source: (Aldazaba, 2004)

3.3 PARAMICS MODEL BUILDING

Building a model in PARAMICS is in reality an easy task because of the powerful graphical interface that provides a user-friendly environment where the model looks very similar to what is observed in the reality. In order to explain the elements that are integrated in a model, this section is divided in subsections that group these elements into the following categories:

- Geometrics
- Traffic Operation
- Flow Generation (Zoning System)
- Vehicles/Drivers
- Calibration and Validation

3.3.1 Geometrics

The geometry of a road is defined by using traditional nodes and links. A node represents, a junction, an inflexion point in the network, or a point in the network where the number of lanes is modified. A link represents a section of the road, which has uniform features along its length. A link is defined by connecting two nodes. Figure 3.1 shows the relation between nodes, links, roads and junctions.

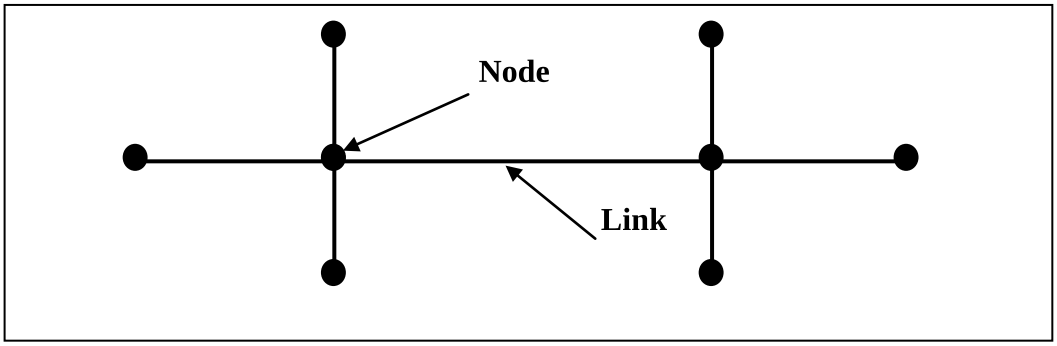


Figure 3-1 Network Representation

Once a link is defined the user indicates the category of the link, and sets up additional information affecting the functionality of this road. Link categories are associated with the following characteristics: number of lanes, width, major/minor road, rural/highway operation, speed, and cost factor. Additional information can also be defined by the user for each link, including:

- Enabling/disabling one-way operations, overtaking, stay in lane, bus-only road, force across, and forced merging actions.
- Setting values for visibility, stop time at the end, target speed at the end, and slip lane length.
- Defining vehicle restrictions.

Link length is calculated automatically based on the intrinsic information of its two related nodes.

Overlays allow the user to place a graphics file over the network display. Typically this file is a map image, for example from Google Earth, which is used as a starting point for the design of a network. The tool allows the user to select an existing graphics file and position/configure it as required. Overlays are typically available in the form of aerial photography and/or AutoCAD drawn vector image and is used as a starting point for the design of a network. PRAMICS supports the following type overlay files- BMP, JPG, PNG, TIF, SID, JP2, DXF, SHP, MIF/MID, DGN, ECW.

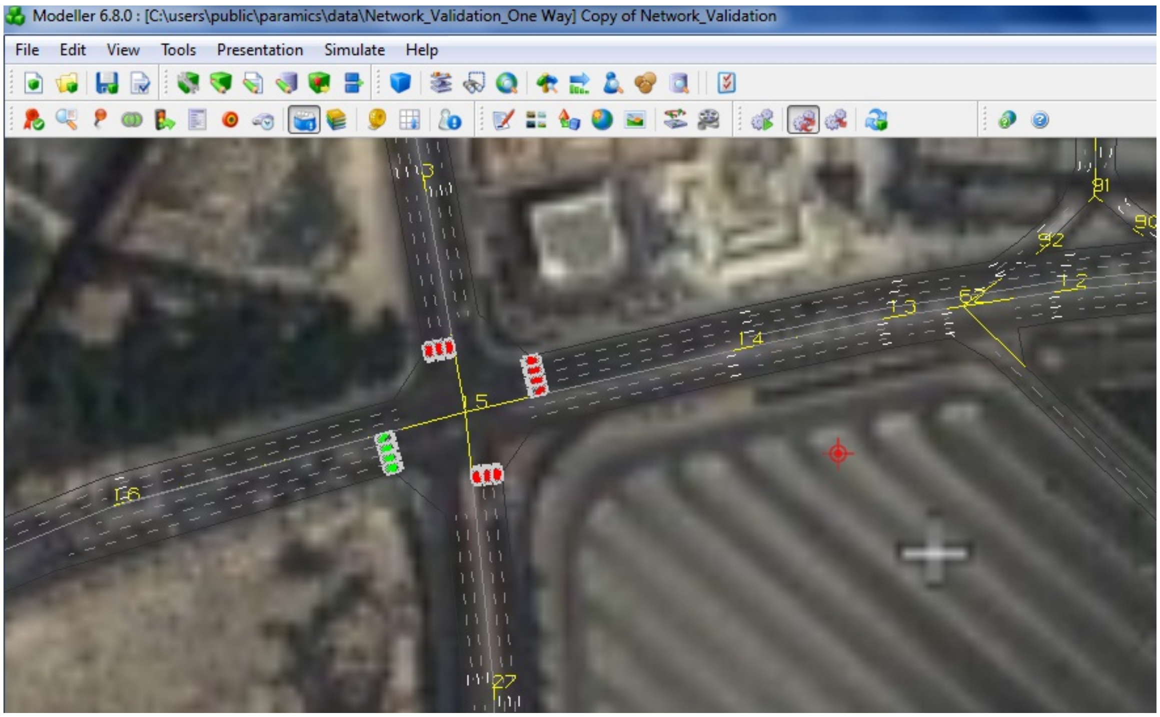


Figure 3-2 Road Network drawn on overlays (Google satellite image is used)

3.3.2 Traffic Operation

Traffic operations are defined by traffic signals, turn movements, priorities, kerbs (curbs), stop lines, and signposting distance. Signals are defined by the green, amber and all red times of a phase. The phase is also associated to the specific turning movements that are allowed during the green time of a phase. The user can define as many phases as required including pedestrian phases and set up the cycle length and offset if the signals are coordinated in case. Signals can be fixed time or vehicle actuated. To set up a signal the user only has to choose the node representing the intersection and a graphical interface will allow the user to define these parameters.

Turn movements (see Figure 3.3) are the definition of movements allowed in one intersection. PARAMICS automatically sets up all the possible movements in the junction when the links are set up. To modify the defaults from PARAMICS, the user only has to choose the node and a window will appear which allows the user to classify each turn movement as barred, major, medium, or minor.

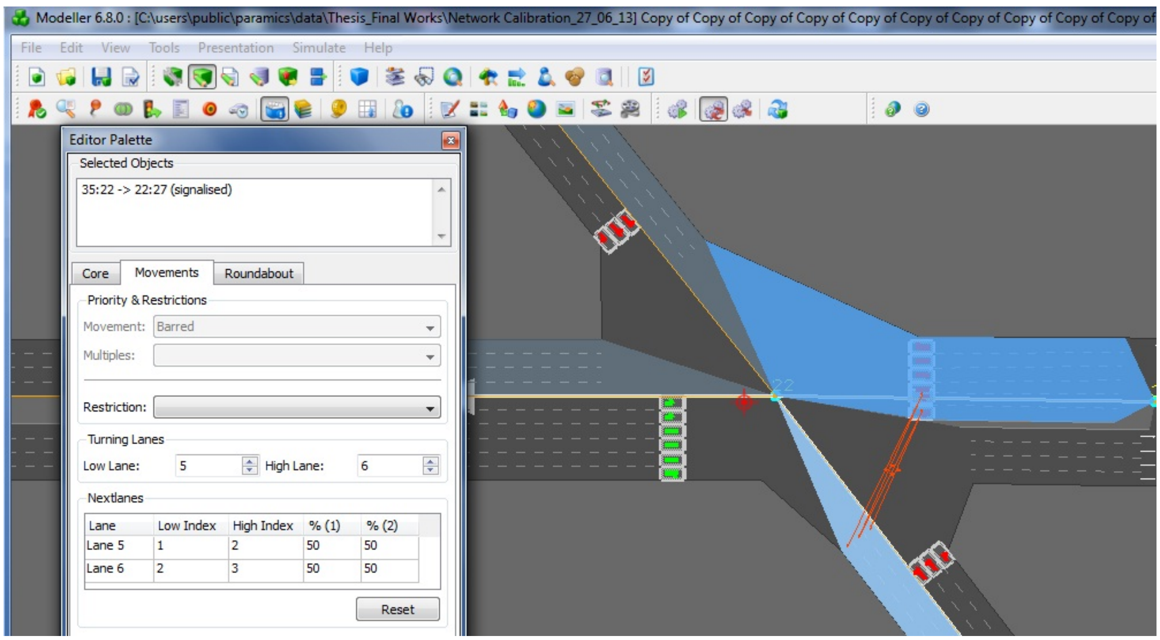


Figure 3-3 Turning movements and lane distribution

A barred status means that the movement is not allowed. Major, medium and minor statuses are levels of priority. Vehicles turning from a turn classified as minor have to yield to vehicles from medium and major turns. Vehicles turning from a turn classified as medium would yield to vehicles from a major turn.

Kerbs (curbs) (see Figure 3.4) are control points used to fine-tune the geometry and characteristics of the road, whose underlying structure is defined by nodes and links. A kerb point defines the edge of the road and also the default position of the stoplines. Moving a kerb indirectly affects the gap acceptance, the turning speed and the trajectory of the vehicle when traveling through an intersection.

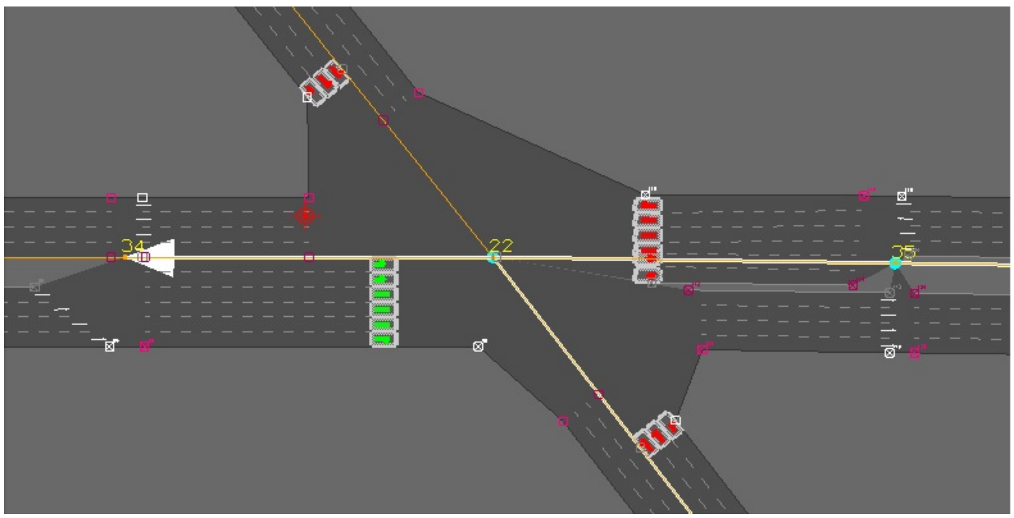


Figure 3-4 Kurbs or controlling points at the modeled intersection

Stop lines (see Figure 3.5) are points at the start and end of each link that vehicles must pass through. Vehicles always react to upcoming stop lines and adjust their behaviour in order to carry out a smooth and safe junction-crossing maneuver. The user is able to modify the angle, the position and the consecutive lane of a stop line. Changing the angle and the position of the stop line will modify the gap acceptance, the turning speed and the vehicle trajectory.

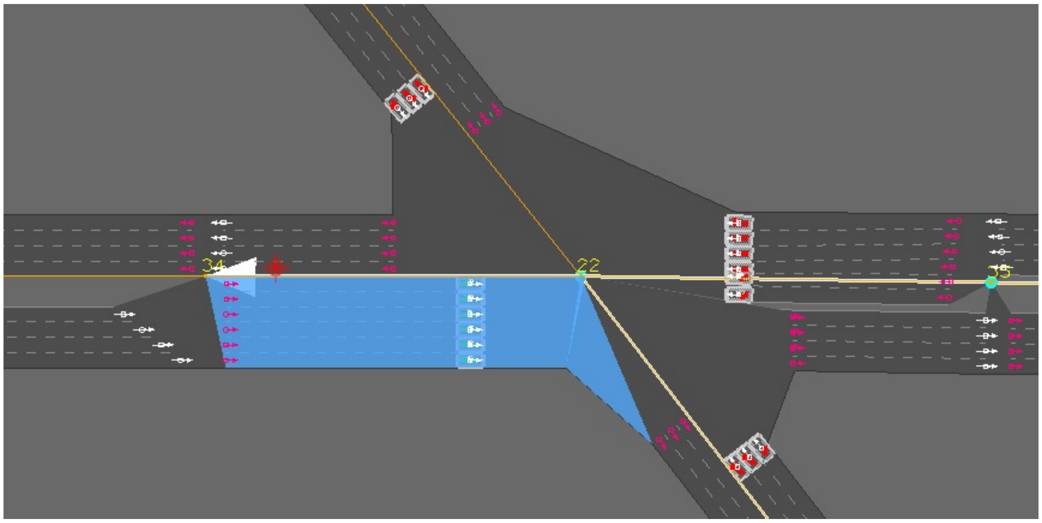


Figure 3-5 Stop lines in a modeled intersection

3.3.3 Trip Generation

Once the network is built up, the user must define a set of zones. These zones define locations where the vehicles will enter and leave the network. Each zone is associated with a number that relates it with an Origin-Destination matrix (trip table).The Origin-Destination (OD) matrix is automatically created when the user is setting up the zones. The OD matrix is later modified to define the number of trips between zones. The shape of a zone is immaterial. Links, whose mid-point are within a zone, can be used by vehicles to enter and leave the network. When a zone includes many links the distribution of origins and destinations is in proportion to the length and the number of lanes on each link. Figure 3.6 shows two zones and their corresponding links.

The travel demand in PARAMICS is defined by the initially created origin-destination matrix. However, traffic engineers usually collect data in the form of intersection turning movement diagrams. Therefore, a conversion from turning movements to an origin-destination matrix is required.

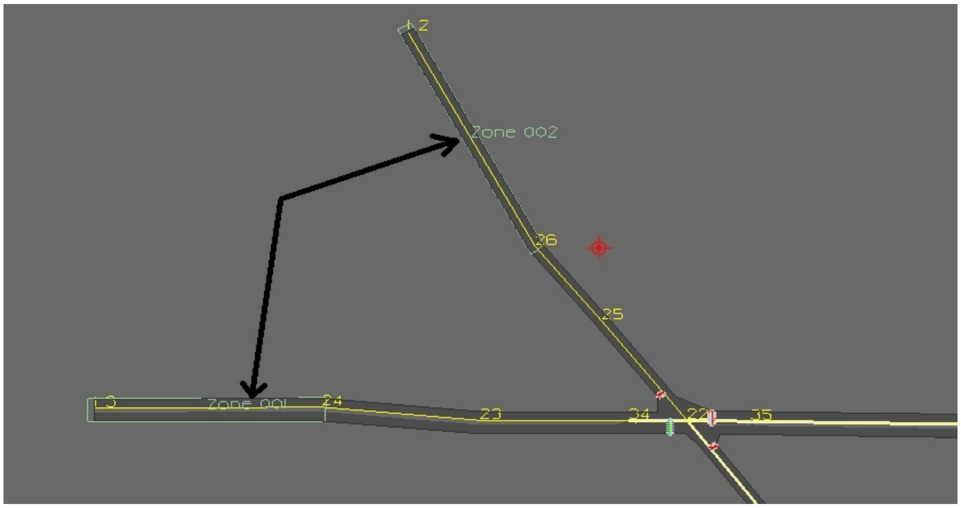


Figure 3-6 Zoning system in PARAMICS build network

Other than the fixed route vehicles or transits traffic assignment in PARAMICS can be calculated at each time step according to the following generalized cost function (Bertini, R.L., 2002)

Cost = a*T + b*D + c*P……………………………………………………………….(3.1)

Where:

a = Time coefficient in minutes per minute (default 1.0)

b = Distance coefficient in minutes per miles (default 0.0)

c = Toll coefficient in minutes per monetary cost (default 0.0)

T = Free-flow travel time in minutes

D = Length of the link in miles

P = Price of the toll in monetary cost units

Coefficients a, b and c can be changed to reflect conditions on the modeled network.

3.3.4 Vehicles/Drivers

To define the characteristics of vehicles in the traffic stream the user must define different types and proportions. PARAMICS defines a default set of vehicle types and proportions but the user can easily modify these values to better reflect real condition. Each vehicle type in the model is associated to parameters related to the shape (length, width, and height), the kinetics (weight, top speed, acceleration, deceleration, inertia) and routing (perturbation, familiarity, fixed route). The user can modify these parameters using the graphical interface shown in Figure 3.7

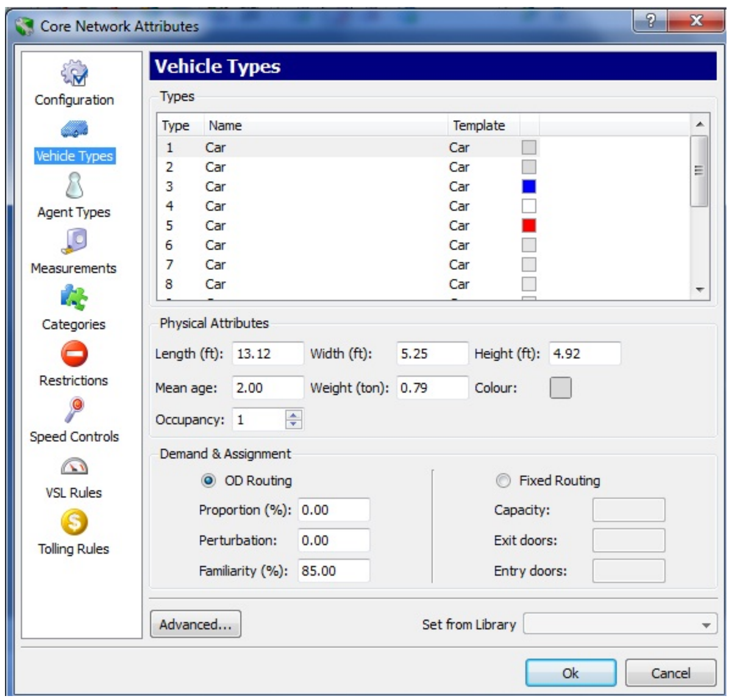


Figure 3-7 Graphic controlling interfaces of vehicle types

3.3.5 Calibration and Validation

The final steps in the network building process is the calibration and validation of the model, that requires an iterative process by which network and OD matrix are alternatively fine-tuned. The fine-tuning of the network can be divided into two tasks:

a) Improving network elements describing roads, signals and junctions.

b) Adjustment of parameters associated to car following, lane changing and routing algorithms.

Improving network elements requires the verification of the length of special turning lanes, position of stop lines and kerbs, setting up the points where left turning vehicles are stacked, changing signpost distances, checking turning movements and priority rules, and checking saturation flows. Most of these adjustments are easy to undertake because it normally only requires the verification of the model settings against what is observed in reality.

The adjustment of parameters associated with algorithms involves the manipulation of variables such as feedback period, feedback coefficient, proportion of familiar and unfamiliar vehicles, cost coefficients, Mean target headway, Mean reaction time and perturbation. Unfortunately, these parameters are not easy to measure and there are no defined procedures to adjust them so the users have to rely on their intuition and experience. The mean target headway and Mean reaction time is the most important parameters that have an significant effect in simulation run when it is changed.

OD matrix fine-tuning implies the addition or subtraction of trips to match the observed counts at intersections and middle-block locations. The adjustment of an OD matrix becomes more complex as more zones are defined but PARAMICS includes a module called Estimator that makes this task easier. Model validation is normally done by comparing model statistics to observations and measures from the field. Some of the more common ways of validating a model includes:

- Comparing travel times for specific routes

- Comparing average and maximum queues

Because of the stochastic nature of traffic, variations between the model and observed data is always expected and the onus is upon the model user to establish the desired reliability level and the validation effort required to achieve it. The calibration process for Paramics follows similar procedures to conventional traffic models with the implementation of a two phase process covering a thorough check of the input data and comparing modeled results with observed data. Comparison of modeled and observed data is possible for operational analysis where an existing system is being studied. Paramics applies the GEH statistic, that incorporates both relative and absolute differences, in comparison of modeled and observed volumes. The GEH formula is named after Geoffrey E. Havers, who invented it in the 1970s while working as a transport planner in London, England. Although its mathematical form is similar to a chi-squared test, is not a true statistical test. Rather, it is an empirical formula that has been proven to be useful for a variety of traffic analysis purposes. It is represented by the equation as below:

$$GEH = \sqrt{\frac{(M-O)^2}{(M+O)/2}}$$

Where, M is the modelled flow and O is the observed flow. (Source: UK design manual for roads and bridges, 1996)

Various GEH values give an indication of a goodness of fit as outlined below:

GEH < 5 Flows can be considered a good fit

5 < **GEH** < 10 Flows may require further investigation

10< **GEH** Flows cannot be considered to be a good fit

Using the GEH Statistic avoids some pitfalls that occur when using simple percentages to compare two sets of volumes. The traffic volumes in real-world transportation systems may vary over a wide range. If a common percentage error is accounted for, then the comparison can be misleading at times. For example, the mainline of a freeway might carry 5000 vehicles per hour, while one of the on-ramps leading to the freeway might carry only 50 vehicles per hour (in that situation it would not be possible to select a single percentage of variation that is acceptable for both volumes). For instance if we accept 10% deviation for both freeway and on ramps, the number of vehicle that we lose is 500 for the freeway which is relatively very high as compared to 5 vehicle for the on ramp. The GEH statistic reduces this problem; because the GEH statistic is non-linear, a single acceptance threshold based on GEH can be used over a fairly wide range of traffic volumes.

CHAPTER 4

RESEARCH METHODOLOGY

As discussed in the literature survey, Quadstone PARAMICS software has been selected to be used for this study. Attempt are taken to calibrate the model first in the same urban arterial where Olba (2007) had tried to calibrated two separate models SimTraffic and TRANSYT-7F. His endeavour to calibrate the models was only successful in the case of TRANSYT-7F. The same traffic data would be used to calibrate PARAMICS for the same network. However, another urban arterial with similar distinct traffic features would be used to validate the model in order to verify common calibration parameter values for the driving behaviour in Saudi Arabia. Since this two networks are different in terms of network setting, intersection arrangements and traffic features the two case study area would be referred to as Case Study-1 and Case Study-2 hereafter. The methodology adopted in these case studies to calibrate PARAMICS is illustrated in Figure 4.1 below. As attempts to calibrated SimTraffic and TRANSYT-7F has already taken place for the first study area, only PARAMICS would be used for calibration here. Calibration methods of the other two software are beyond the scope and objective of this study. After the successful calibration of PARAMICS a comparison would be drawn among the three models with different simulated measure of effectiveness.

For the second case study the calibrated parameter value would be used in case of PARAMICS to get a simulated output for validation. The same network coding would be done in SimTraffic and TRANSYT-7F with relevant traffic data to get another set of simulated output. The three simulated output would be compared again to find which model is more effective or suitable for local traffic condition assessment. Travel time and Queue length are the two selected measure of effectiveness (MOEs) that would be compared with the observed field data simply because they are easy to observed in field. As this study uses the data collected by Olba (2007), who also used these same two measure of effectiveness for an attempt to calibrate TRANSYT-7F and SYNCHRO being another reason of selecting these MOEs to ensure data compatibility. At the final stage signal timing plan would be optimized in SYNCHRO and TRANSYT and re used in PARAMICS to get different sets of simulated outputs. Comparison would be drawn again to identify which signal timing plan has resulted a better traffic condition.

After achieving all the above tasks conclusions and recommendations were drawn besides determination of the appropriate traffic simulation and optimization model for local traffic conditions, obtaining an optimal signal timing plans for the selected signalized intersections, investigating which parameters might be used as a yard stick in calibration process.

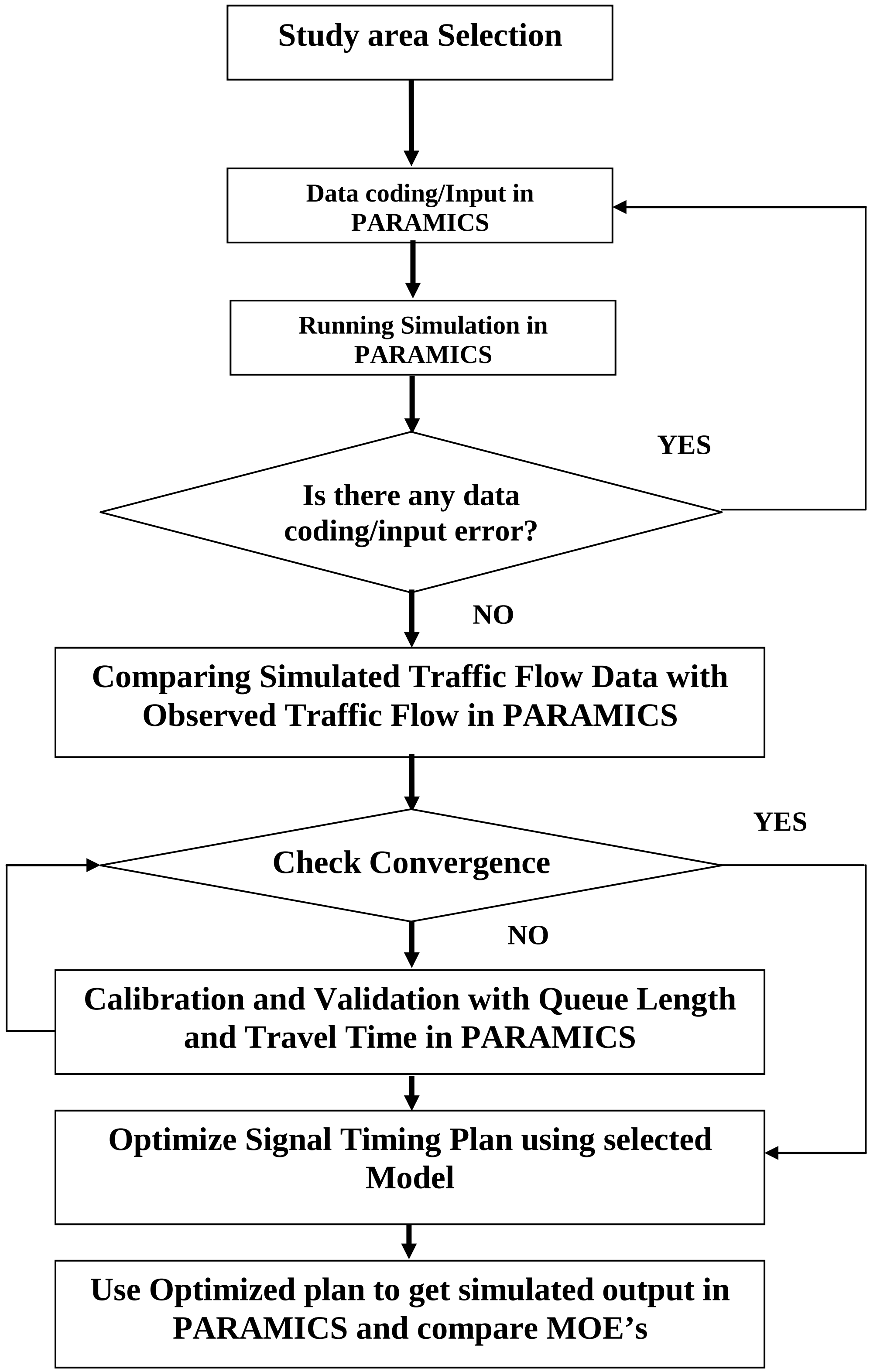


Figure 4-1 Research methodology flow chart

4.1 CASE STUDY-1

4.1.1 Study Area Selection

A suitable study area should consist of few signalized intersections in a metropolitan area that satisfy the study requirements and does not cause any complexity in data collection and model formulation for study. Olba (2007) studied city map of Al-Dammam and Al-Khobar to find a suitable study area. He found that King Abdullah Road is an urban arterial in Al Khobar area which was selected as it is the largest arterial in Al-Dammam and Al-Khobar cities and it is the main entrance of Al-Khobar city. This study area was selected based on the criteria that it operates in moderately high volume but not congested, an ideal geometry and less friction due to road side parking and pedestrian. Also, it has a common cycle length for the studied intersections with co-ordinated signaling system. The arterial consist of three signalized intersections connecting Makkah Street, Prince Homoud Street and King Fahd Road. It consists of four through lanes and two lane left turn storage bay in each direction, and it is located in mixed residential and commercial area.

Figure 4.2 shows an aerial photograph of the selected urban arterial. The geometric features of intersections are shown in the following Figures 4.3, 4.4 and 4.5 respectively.

Figure 4-2 Aerial photograph of Study Area (Photo source: Google Earth Satelite image)

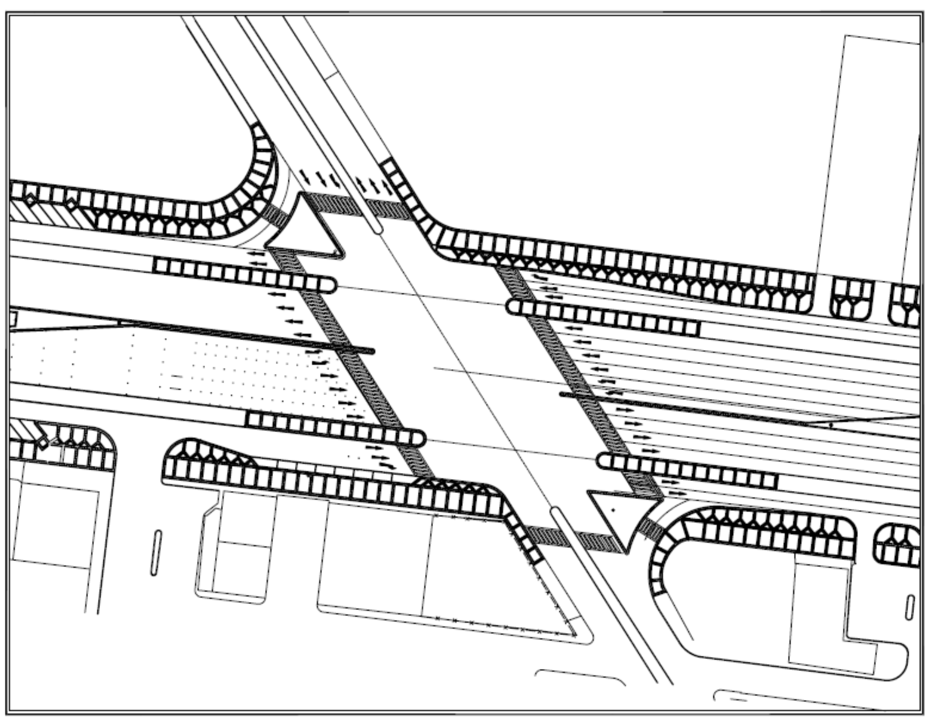


Figure 4-3 Intersection 1 (Node1) King Abdullah Road–Makkah Street (Olba, 2007)

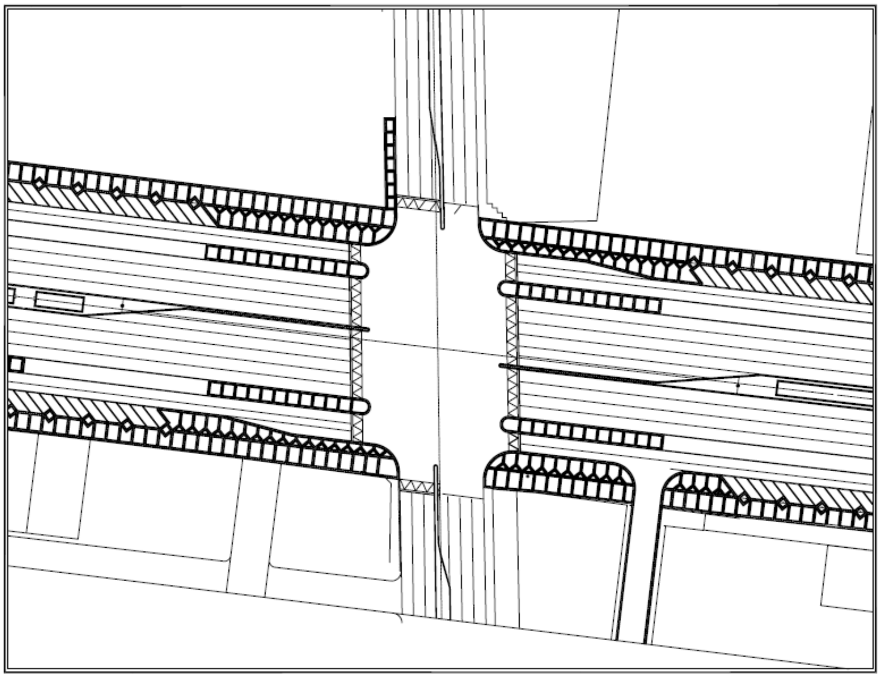


Figure 4-4 Intersection 2 (Node2) King Abdullah Road–Riyadh Street (Olba, 2007)

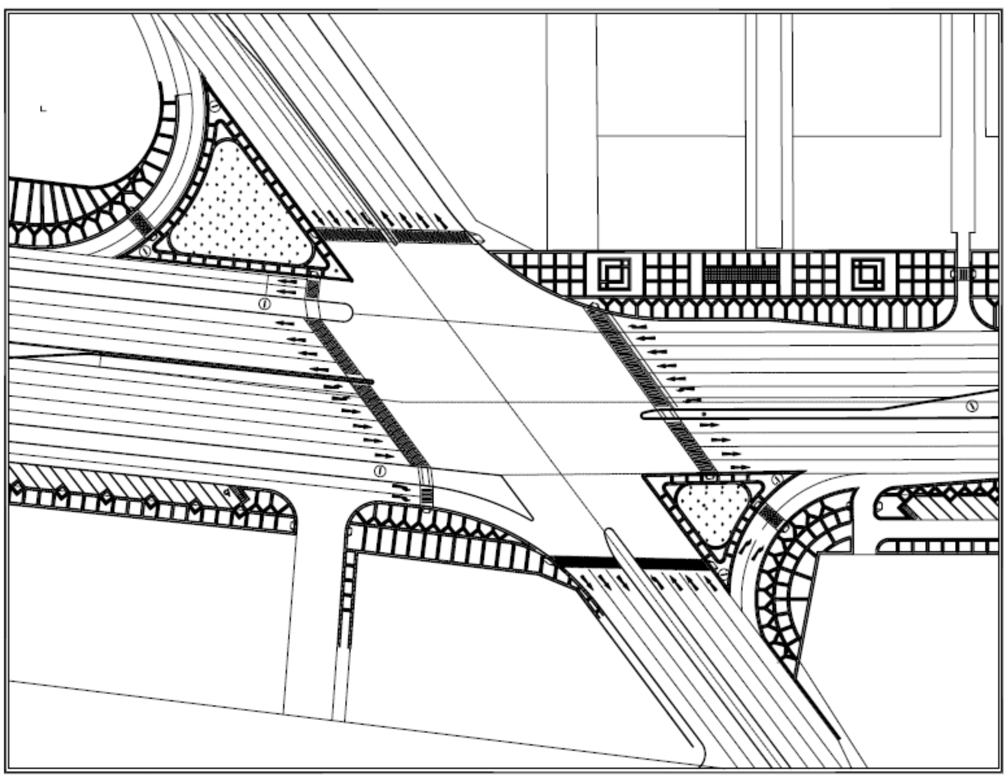


Figure 4-5 Intersection-3 King Abdullah- King Fahd road (Olba, 2007)

4.1.2 Data Collection

There are many ways of gathering traffic data from the field; the common way is to collect data from the field by data collectors using proper equipment and devices. As an alternative, other data sources may be considered and can be used successfully. One of these sources is live video detection where the traffic is monitored by live traffic-monitoring cameras. Yet another source of data is the data obtained from local departments of transportation.

The data source employed for this study by Olba (2007) is field observation. Using live video and video library sources was not possible since they were not available. Also, there was no reliable and updated database available at local transportation departments that could be used directly in the analysis.

To build a PARAMICS simulation model for this network and to calibrate it for the local traffic conditions, two types of data are required. The first type is the basic input data used for network coding of the simulation model. The second type is the observation data required for the calibration of simulation model parameters.

Basic Input Data: Basic input data include data of network geometry, traffic volume data, turning movements, vehicle characteristics, travel demands, vehicle mix, stop signs, signal timing plan, Origin Destination count etc.

Data for Model Calibration: The coded PARAMICS simulation network needs to be further calibrated to replicate the local traffic conditions. The calibration involves comparing the simulation results against field observed data and adjusting model parameters until the model results fall within an acceptable range of convergence. There are many measures of effectiveness such as delay, travel time, stops, fuel consumption and queue length that can be measured in the field and compared with simulated ones. Olba (2007) had selected Travel time and Queue length as the measure of effectiveness (MOE's) for his study and measured in the field and compared with corresponding simulated values. To maintain continuity and draw meaningful comparison we have also kept Travel time and Queue Length for PARAMICS model calibration and validation. Measuring Queue Length is easier than other MOE's, stops or fuel consumption. Olba deployed 4 proble vehicles to run throuth the network and collect travel time with a stop watch of good pricision in pivotal points. About 20 graduate students from King Fahd Univesity of Petroleum and Minerals (KFUPM) participated as data collectors to conduct data collection task. A practice session was arranged by Olba several days before conducting data collection to correct any undesirable mistake. A summary of data collected and used for Olba's study is shown in table 4.1 below

Table 4-1 Categorized data collection

Major Category	**Data Type**
Network Data	• Links with start and end points. • Link lengths. • Number of lanes. • Lane drops and lane gains. • Lane storage length for turning movements. • Connectors between links to model turning movements. • Position of signal heads/stop lines.
Traffic Volume Data	• Through and turning traffic volume counts • Vehicle composition • Vehicle length.
Speed data	• Link lengths. • Running time.

Major Category	Data Type
Signal timing control	• Cycle length • Offsets. • Splits • Phase sequence
Measured data used to compare with simulated results	• Queue length at beginning of green and travel time

4.1.2.1 Traffic Volume Study

Traffic volume is defined as the number of vehicles passing a point on a highway or lane during a specified period. It is the most basic of all parameters and the one most often used in planning, design and control, operation and management analyses. Since, volume is the most basic of all parameters, the observation and analysis of traffic volumes were done with utmost care and accuracy. Inaccurate volume information will compromise the accuracy and effectiveness of all analyses and improvements developed from it.

The two basic methods of counting traffic are manual and mechanical or automatic recording. Tally Sheets are the simplest means of conducting manual counts. The observer records each observed vehicle with a tick on prepared field form. A new form is used at the start of each interval. Mechanical Count Boards, which Olba (2007) used in his study, consist of various combinations of accumulating counters mounted on a board to facilitate the type of count being made. The counters used, have accumulating pushbuttons devices with three registers (for left, through and right or U-turn). Data were collected in 15 minutes interval. When the end of an interval is reached, the observer reads the counter, records the data on the field form, and resets the counter to zero. Electronic Count Boards operate in a fashion similar to that of mechanical count boards with a few important differences. They are lighter weight, more compact and easier to handle. They contain an internal clock that separates the data by whatever interval is chosen, therefore field forms becomes redundant.

Before Olba (2007) had conducted the traffic volume study, he collected a sample traffic count, through traffic only, at King Abdullah-Prince Homoud Intersection to identify the representative or desired traffic condition. The collected count periods were: 09:00–11:00 A.M., 01:30–03:30 P.M. and 07:30–10:30 P.M. The selected count period among this three period to conduct the traffic volume study was 01:15-02:30 P.M. Data was observed in 15 minutes interval throughout the count period, the first 15 minutes interval (01:15-01:30) was not included in the analysis since it was devoted to train the observers and make them familiar with the counting process. A principal reason behind the selection of this period is that the signal timing controller during the morning and evening periods (09:00-11:00 A.M. and 07:30-10:30 P.M.) was operated manually by traffic police officers at King Abdullah-Makkah and King Abdullah-King Fahd Intersections. This would have affected the study because of unstable cycle lengths.

All the intersections in the study area had four approaches. Four observers were assigned at each intersection and each observer was provided with mechanical count board. Since the signals were four phase signal systems, all approaches did not have the right-of-way simultaneously, two observers were assigned to count alternating movements for east approach and south approach as the signal phase changes while the other two observers counted movements for west approach and north approach. Duties were divided among observers in a way that one observer was responsible for counting through movement while the other observer was responsible for left and U-turn movements for the major approach (East-West). On the minor approach (North-South) one observer was responsible for counting through movement while other observer was counting the left and right turning vehicles.

4.1.2.2 Speed Study

Vehicle speed is directly related to travel time and delay and is also used to evaluate traffic and highway systems. Average or mean speeds can be computed in two different ways, Time Mean Speed (TMS) and Space Mean Speed (SMS), yielding two different values with differing physical significance. Time mean speed (TMS) is defined as the average speed of all vehicles passing a point on a highway over some specified time period. Space mean speed (SMS) is defined as the average speed of all vehicles occupying a given section of highway over some specified time period. In essence, time mean speed is a point measure or spot speed, while space mean speed is a measure relating to a length of lane. Space mean speed was computed to be used as an input for TRANSYT-7F and Synchro by Olba (2007). Running speed, which is the distance traveled divided by running time, is the speed input required for TRANSYT-7F and Synchro. Running time is the time a vehicle is actually in motion while traversing a section of the road.

A summary of Olba’s observed running time, mid block speed and computed running speed is appended below:

Table 4-2 Summary of Speed study (From west to east)

Run No.	Segment	Running Time (Sec.)	Distance (KM)	Running Speed (Km/h)
1	Makkah Int. to Homoud Int.	63	0.830	47.43
	Homoud Int. to Abdulaziz Int.	74	1.050	51.08
2	Makkah Int. to Homoud Int.	56	0.830	53.36
	Homoud Int. to Abdulaziz Int.	66	1.050	57.27
3	Makkah Int. to Homoud Int.	64	0.830	46.69
	Homoud Int. to Abdulaziz Int.	74	1.050	51.08
4	Makkah Int. to Homoud Int.	60	0.830	49.80
	Homoud Int. to Abdulaziz Int.	79	1.050	47.85
5	Makkah Int. to Homoud Int.	52	0.830	57.46
	Homoud Int. to Abdulaziz Int.	63	1.050	60.00
6	Makkah Int. to Homoud Int.	59	0.830	50.64
	Homoud Int. to Abdulaziz Int.	69	1.050	54.78
7	Makkah Int. to Homoud Int.	52	0.830	57.46
	Homoud Int. to Abdulaziz Int.	63	1.050	60.00
8	Makkah Int. to Homoud Int.	59	0.830	50.64
	Homoud Int. to Abdulaziz Int.	69	1.050	54.78

Table 4-3 Summary of Speed study (From east to west)

Run No.	Segment	Running Time (Sec.)	Distance (KM)	Running Speed (Km/h)
1	King Fahd Int. to Homoud Int.	76	1.050	49.74
	Homoud Int. to Makkah Int.	72	0.830	41.50
2	King Fahd Int. to Homoud Int.	68	1.050	55.59
	Homoud Int. to Makkah Int.	58	0.830	51.52
3	King Fahd Int. to Homoud Int.	62	1.050	60.97
	Homoud Int. to Makkah Int.	52	0.830	57.46
4	King Fahd Int. to Homoud Int.	72	1.050	52.50
	Homoud Int. to Makkah Int.	59	0.830	50.64
5	King Fahd Int. to Homoud Int.	68	1.050	55.59

Run No.	Segment	Running Time (Sec.)	Distance (KM)	Running Speed (Km/h)
	Homoud Int. to Makkah Int.	56	0.830	53.36
6	King Fahd Int. to Homoud Int.	68	1.050	55.59
	Homoud Int. to Makkah Int.	60	0.830	49.80
7	King Fahd Int. to Homoud Int.	62	1.050	60.97
	Homoud Int. to Makkah Int.	52	0.830	57.46
8	King Fahd Int. to Homoud Int.	71	1.050	53.24
	Homoud Int. to Makkah Int.	57	0.830	52.42

Both the saturation flow rate and start-up lost time are important parameters in signal timing and capacity analysis of signalized intersections. These two parameters can easily vary significantly between intersections and between times of the day. They are affected by the location of the intersection in the city, grade, driver characteristics and the geometric design of the intersection.

Saturation flow rate: Saturation flow rate was collected Olba (2007) during the period 1:30 to 2:30 P.M.; two observers were placed at each approach with two stopwatches. One observer was responsible for measuring saturation flow rate for the through movement while the other observer was measuring saturation flow rate for the left turn movement. The observer started the stopwatch when the rear axle of the fourth vehicle in the queue which is waiting for the green signal crosses the stop line. The observer stoped the watch when the rear axle of the seventh, eighth, ninth or tenth vehicle crosses the stop line. If the queue is longer than ten vehicles, the measurement was stopped when the tenth vehicle rear axle crosses the stop line and the rest of vehicles were ignored. This was done for convenience since it is usually hard to observe a queue longer than ten vehicles (Olba, 2007). Any vehicle that joins the queue after the start of the green was ignored in these calculations. Queues which are shorter than seven vehicles was also ignored because such queues provide highly unstable saturation rate values. Mean saturation flow rate was estimated by calculating an average number of seconds consumed per vehicle (i.e., headway) and converting that into a number of vehicles per hour. Table 4.4 summarizes the mean saturated flow rate observed in the field by Olba (2007)

Table 4-4 Observed saturation flow rate by Olba (2007)

Intersection No	Location	Approach Direction	Mean Saturation Flow rate (vph)	
			Through Movement	Left turn movement (Left turn bay)
1	Makkah	East Approach	1975	1590
		West Approach	1914	1561
		North Approach	1961	1779
		South Approach	1961	1779
2	Hamud	East Approach	1914	1561
		West Approach	1874	1521
		North Approach	1961	1779
		South Approach	1961	1779
3	King Fahd	East Approach	1892	1572
		West Approach	1914	1561
		North Approach	1961	1708
		South Approach	1961	1708

Start-up Lost Time: The start-up lost time was determined by measuring the time between the start of the green indication up to the moment when the rear axle of the first vehicle in a standing queue crosses the stop line. The lost time is then computed as follows

Average time: Av = S_t / No of observations; S_t = the sum of times for all observation

Lost time: L = Av – (3600/Saturation flow rate)

Table 4-5 Start up lost time study conducted by Olba (2007)

Serial No	**Movement Type**	**Location**	**Start up Lost Time (Seconds)**
1	Through	Major Approach	2.5
2	Left Turn (Left turn Bay)	Major Approach	2.1
3	Through	Minor Approach	3.8
4	Left Turn	Minor Approach	3.6
5	Left Turn	Minor Approach	3.4

As shown in the above table, the value of the start-up lost time for the minor approaches was high. This is due to the aggressive drivers when they use the middle or the right lanes to make left turn. When there are no or less vehicles in the middle or right lanes, those lanes are attracting aggressive drivers, who want to turn left, to use them and cross the stop line which will make them unable to see the green light and therefore take more time to start moving and delay the other vehicles.

4.1.2.3 Signal Control data

Signal control data consists of cycle lengths, phases, offsets and extension of effective green. Signal control data of each intersection were recorded using stopwatches. Cycle length is the time required for one complete sequence of signal indications (phases), i.e., the time from green indication to gain green indication. Usually it is measured in seconds. Phase is defined as the part of a cycle length allocated to any combination of one or more traffic movements simultaneously receiving the right of way during one or more intervals. Cycle length for all the signalized intersections of King Abdullah Road were recorder as 135 seconds. Also, all red time for each approach was found 2 seconds.

Table 4-6 Signal Timing Information

Time (hr)	Direction	All Red (s)	Yellow (S)	Green (S)	Cycle Length (S)
1:15 - 2:30 pm	Eastbound	2	3	36	135
	Westbound	2	3	34	135
	Northbound	2	3	25	135
	Southbound	2	3	20	135

Offset is the time difference between the start of the green indication at one intersection for a specific direction as related to the start of green indication at another intersection for the same direction or from system time base. Olba (2007) observed that the offset between Makkah Intersection and Prince Homoud Intersection is 50 seconds while the offset between Makkah and King Fahd intersection is 120 seconds.

4.2 CASE STUDY-2

The second study area was selected only 4.1 kilometers away from the first study network. Likewise the first network this one is also an urban arterial of ideal geometry and less friction due to pedestrian and parking. The mainline street is Prince Faisal Bin Fahd Road with three signalized intersections connecting Dhahran highway with a diamond intersection, Abu ubaidah street and King Saud road. The mainline street consist of three through lanes and one left turning bay at the middle intersection. The network is located in a commercial zone with sufficient parking facilities for each zone of trip attraction.

Figure 4.6 shows the selected study network drawn over a google satellite image with proper scaling. The network was carefully drawn in CAD to better reflect the geometric features.

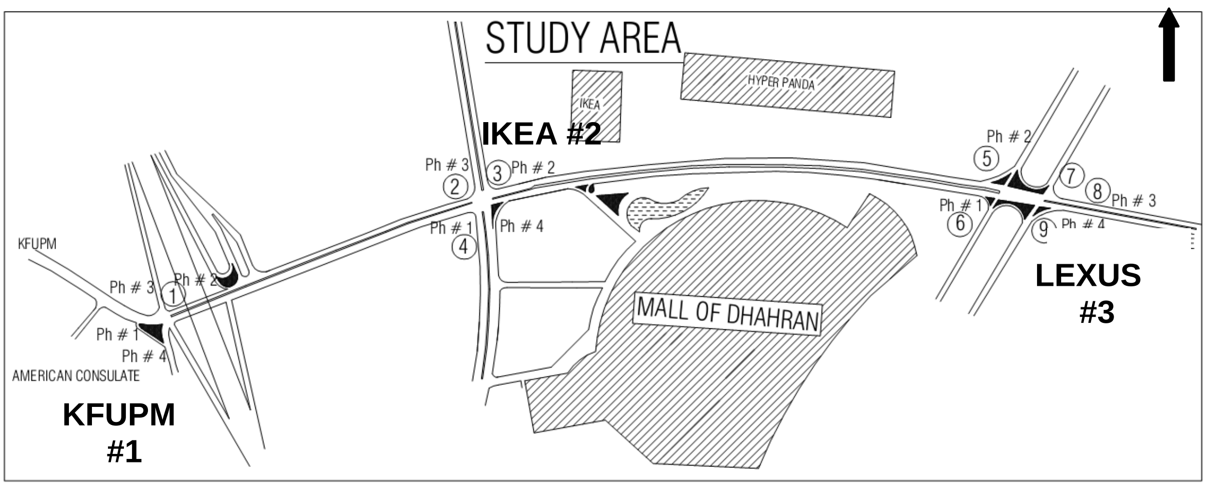


Figure 4-6 Selected Study Network drawn in CAD (Scaled on google satellite image)

4.2.1 Data Collection

The data source employed for this study is field observation. Likewise the first case study the basic input data of network geometry, traffic volume data, turning movements, vehicle mix, stop signs, signal timing plan, etc were observed directly from the study area.

To match the both the studies and validate the model with same measure of effectiveness travel time and queue length were observed at the intersections. 4 proble vehicles were deployed to run throuth the network repeatedly and collect travel time with a stop watch. About 14 graduate and undergraduate students from King Fahd Univesity of Petroleum and Minerals (KFUPM) were deployed to collect queue length and turning volume counts at the intersections with manual counters. As before, A practice session was conducted to demonstrate the data collection process several days before data collection to avoid errors.

4.2.1.1 Traffic Volume Study

In order to incorporate travel demand in PARAMICS getting traffic volume from the field is not indispensible. It can be done through Origin Destination (OD) counts also. Since the OD counts are more tedious and there is a builtin tool called ESTIMATOR in PARAMICS to convert intersection volume counts to OD matrix, we choose to observe turning traffic counts at the intersections. Sufficient care is given in volume data collection in order to get accurate and precise information from the model outcome.

Mechanical Count Boards are used in this study,as it was used by Olba also. A new form is used at the start of each interval of 15 minutes counts by the observers. When the end of an interval is reached, the observer reads the counter, records the data on the field form, and resets the counter to zero to proceed for next interval count.

Before conducting the traffic volume study, the site was visited to determine reasonable study period for use in later analysis. A sample of traffic count, through traffic only, was done at Prince Faisal bin Fahd-Abu Ubaidah Intersection to identify an ideal traffic conditions from different period of the day. The count periods were: 07:00–11:00 A.M., 01:30–03:30 P.M. and 07:00–10:00 P.M. The selected count period to conduct the traffic volume study is 08:15-09:45 A.M. Data was observed in 15 minutes interval throughout the count period, the first 15 minutes interval (08:15-08:30) was not included in the analysis since it was attributed to data collectors training to get accustomed with the whole proces. A principal reason behind the selection of this period is that the traffic volume at this time is not very high neither very low with no congestion and friction due to roadside parking.

In addition to the manual turning count at the intersections the study was aided by p-neumatic tube based automatic counter. The p-neumatic tubes were laid on the road at the mid night two days before the candidate day with very low traffic to avoid any undue risk of casualities. Appropriate safety measures were taken while installing the automatic counter. The volume data is given in the appendix. Few pictures taken at the time of installing automatic counter are presented below.

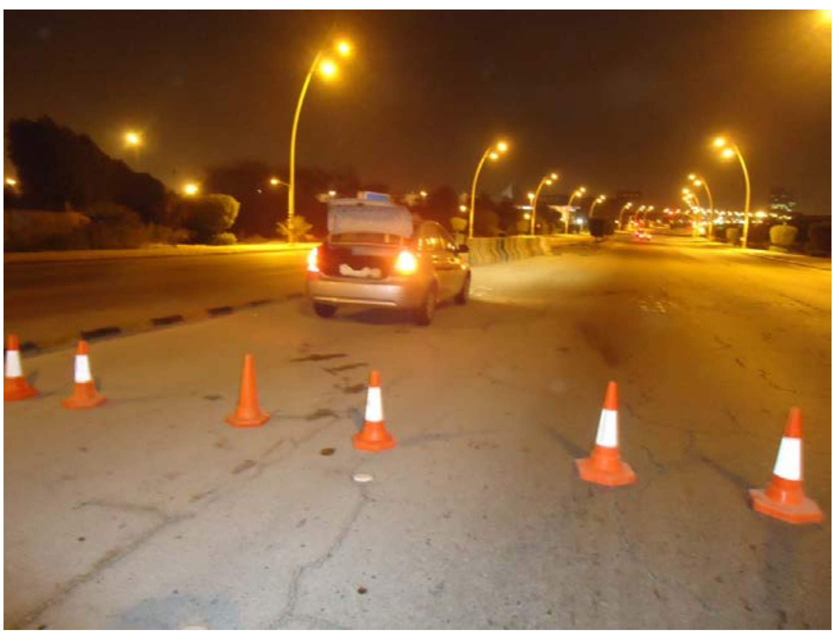


Figure 4-7 Traffic Volume data collection in Prince Faisal bin Fahd Road

Figure 4-8 Embedment of pneumatic tube on the street.

Figure 4-9 Setting up Automatic vehicle counter with the pneumatic tubes attached

4.2.1.2 Speed Study

Space mean speed data was collected using 4 probe vehicles. The travel time to traverse a section from one intersection to the downstream intersection was recoded while the vehicle was in motion. Running speed, which is the distance traveled divided by running time, is the speed input required for TRANSYT-7F and Synchro.

A summary of observed running time and computed running speed is given below

Table 4-7 Summary of Speed study (From Wast to Eest)

Run No.	Segment	Running Time (Sec.)	Distance (KM)	Running Speed (Km/h)
1	American Consulate Int. to IKEA Intersection	52	0.65	45.00
	IKEA Int. to LEXUS Int.	74	1.02	49.62
2	American Consulate Int. to IKEA Intersection	49	0.65	47.76
	IKEA Int. to LEXUS Int.	68	1.02	54.00
3	American Consulate Int. to IKEA Intersection	46	0.65	50.87
	IKEA Int. to LEXUS Int.	73	1.02	50.30
4	American Consulate Int. to IKEA Intersection	46	0.65	50.87
	IKEA Int. to LEXUS Int.	62	1.02	59.23
5	American Consulate Int. to IKEA Intersection	41	0.65	57.07
	IKEA Int. to LEXUS Int.	64	1.02	57.38
6	American Consulate Int. to IKEA Intersection	42	0.65	55.71
	IKEA Int. to LEXUS Int.	67	1.02	54.81

Table 4-8 Summary of Speed study (From Eest to Wast)

Run No.	Segment	Running Time (Sec.)	Distance (KM)	Running Speed (Km/h)
1	LEXUS Int. to IKEA Int.	75	1.02	48.96
	IKEA Int. to American Consulate Int.	50	0.65	46.80
2	LEXUS Int. to IKEA Int.	74	1.02	49.62
	IKEA Int. to American Consulate Int.	49	0.65	47.76
3	LEXUS Int. to IKEA Int.	70	1.02	52.46
	IKEA Int. to American Consulate Int.	53	0.65	44.15
4	LEXUS Int. to IKEA Int.	66	1.02	55.64
	IKEA Int. to American Consulate Int.	54	0.65	43.33
5	LEXUS Int. to IKEA Int.	67	1.02	54.81
	IKEA Int. to American Consulate Int.	59	0.65	39.66
6	LEXUS Int. to IKEA Int.	68	1.02	54.00
	IKEA Int. to American Consulate	59	0.65	39.66

Both the saturation flow rate and start-up lost time are important parameters in signal timing and capacity analysis of signalized intersections. This values were taken from the first case study with an assumption that there is minimum variation in this two parameters as both the networks are very closeby with similar attributes.

4.2.1.3 Signal Control data

Signal control data were collected from the field consisting cycle lengths, phases and sequence of phases. The intersections of this newtork were not co-ordinated but each intersection was facilated with digital coundown signal timer. Signal timing data were collected from the signal timer. All red time for each approach was found 2 seconds with 3 seconds of amber/Yellow time.

Table 4-9 Signal Timing Information

Name of the Signal	Time Duration	Direction	Red (s)	Yellow (S)	Green (S)	Cycle Length (S)
American Consulate	8:15 AM to 9:45 AM	Eastbound	114	3	25	142
		Westbound	98	3	41	142
		Southbound	98	3	41	142
		Northbound	124	3	15	142
IKEA	8:15 AM to 9:45 AM	Eastbound	87	3	45	135
		Westbound	97	3	35	135
		Northbound	112	3	20	135
		Southbound	117	3	15	135
LEXUS	8:15 AM to 9:45 AM	Eastbound	107	3	20	130
		Southbound	107	3	20	130
		Westbound	77	3	50	130
		Northbound	107	3	20	130

CHAPTER 5

DATA ANALYSIS

5.1 INTRODUCTION

This chapter presents the process of modeling PARAMICS for both of the case studies and Synchro/SimTraffic and TRANSYT-7F for the second one only. This includes preparing the models for the existing conditions and adjusting the model parameters in order to make the models replicate actual traffic conditions. The next task was to develop signal timing plans using Synchro and TRANSYT-7F for the selected signalized intersections and preparing to run PARAMICS again with the optimized signal plan. Then the simulated output of PARAMICS was compared with the observed value. Conclusions and findings are given in the next chapter.

The chapter is divided into two sections representing two separate case studies. For the first case, network data coding and calibration of PARAMICS would be presented and calibrated results from SYNCHRO/SimTraffic and TRANSYT-7F would be employed from previous studies (Olba, 2007) for comparison in which the same data set was used. The first part of the each of the case studies presents the data input/ network coding in PARAMICS and The second section deals with the Calibration and validation process employed to adjust the selected model parameters in order to obtain a reasonable convergence between the observed and simulated measure of effectiveness (MOE).

5.2 CASE STUDY-1

5.2.1 Calibration of PARAMICS

Calibration is defined as the process of adjusting the parameters used in the model to ensure that it accurately reflects the input data. Validation is defined as the process of running an independent check on the calibrated model.

As noted previously, there are no universally accepted procedures for conducting a calibration and validation for a network like this one. The responsibility lies with the modeler to implement a suitable procedure which provides an acceptable level of confidence in the model results. In this study, the first step in the calibration and validation process involved choosing suitable model parameters like vehicle characteristics, aggressiveness, awareness, target headways and reaction times that provided realistic results.

Model calibration involved three main processes, calibration of the network elements, calibration of origin/destination (OD) matrix and the calibration routing and driver behaviour parameters. These three processes are described in the following sections. It is important to mention that although these activities are presented sequentially they are part of an iterative process in which results from one process sometimes obligates adjustments in the others.

5.2.1.1 Network Calibration

Network calibration is the process by which the network elements, such as, number of lanes, signal timing, stop signs, speed limits etc. are adjusted to reflect reality. This process required a provisional origin-destination (OD) matrix capable of creating traffic with similar characteristics to the one observed in field. This traffic flowing in the network helps to identify locations where:

1. Traffic behaviour does not reflect reality
2. Queues do not reflect reality
3. Gridlock occurs
4. Congestion locations do not correspond to actual problem locations.

The network geometry in PARAMICS is represented through nodes, links, stop bars, curbs, and curves. As the basic layout of the study network, the relative coordinates of the PARAMICS nodes were calculated using link lengths that were originally measured from overlay images. Further geometric details, including locations of curbs, locations of stop bars, turning radii at intersections, were unavailable from the field dataset. Therefore, these characteristics were modeled and matched against the overlay image using the models visualization tool modeler.

Furthermore, where the above mentioned problems were detected, the following actions were taken

- Changing the length of special turning lanes.
- Verifying signal times and signal progression
- Setting up dedicated and double turning lanes to reflect reality
- Verifying and setting up places where specific turning movements are prohibited.
- Preventing lane changes in specific locations.
- Checking number of lanes and design speed
- Verifying siging distances.

After completing the the necessary adjustment of links and nodes and kerb positions the network is drawn on the same scale as of the overlying satelite image. The scaling was done carefully so that the model link length reflects what is prevailed in reality.

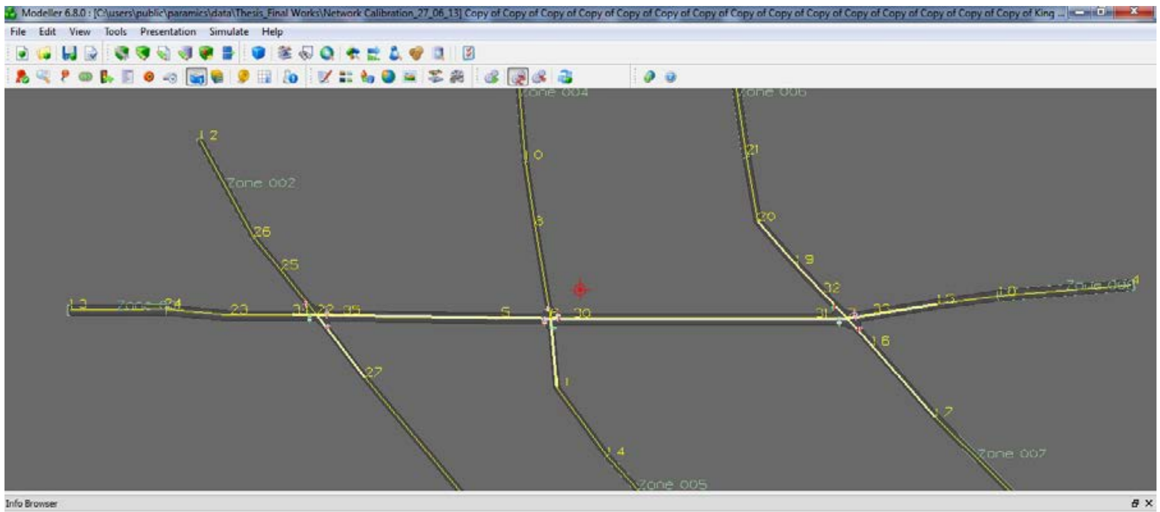


Figure 5-1 Modelled Network with geometry in PARAMICS

The Network comprises of 8 distinct zones and 35 nodes. Since PARAMICS do not have a function to simulate turning-pockets, a network link with a turning pocket was modeled by connecting two adjoining sections that had different numbers of lanes, within which lane-changing regulations were defined. Most of the network elements were modeled at the beginning of the calibration process but some of them were refined later as an improved OD matrix was obtained.

The PARAMICS default traffic control methods are based on a British urban traffic environment. As there were no un-signalized intersections in this study, we didn't have to model the actual stop and yield signs using priority controls.

5.2.1.2 Calibration of Routing and Driving behaviour

The manipulation of parameters related to route choice and driver behaviour are called for when the ESTIMATOR cannot further refine the OD matrix to achieve the established goodness of fit criteria. Interaction between the matrix calibration and route choice·calibration was more intense than the one involving the network calibration. This part of the calibration process was based on intuition because there was no information available that could help in the process.

As stated earlier traffic assignment in PARAMICS is done by the equation 3.1. The travel cost for each vehicle to reach its destination is calculated at each time step according to the cost function based on assiened time, distance and toll coefficient.

The following assignment techniques can be implemented in PARAMICS:

- All-or-nothing assignment method – assumes that all drivers are traveling with the same knowledge base for route choice and there is no congestion effect. Link costs do not depend on the flow levels.
- Stochastic assignment method – emphasizes the variability in drivers' perceptions of costs and the composite measure that they try to minimize (distance, travel time, generalized cost).
- Dynamic feedback assignment – assumes that the drivers who are familiar with the road network will reroute if information on current traffic conditions is provided to them.

For the area that we studied was a small network with only one route possible between each origin and destination, therefore the all-or-nothing technique was chosen.

Driver behaviour characteristics are represented by aggression, awareness and familiarity factors. These factors influence a driver's gap acceptance and lane changing characteristics, amongst others. A normal distribution of behaviour is typical of most environments and the PARAMICS default has been assumed within this model.

In terms of a driver's familiarity with the local road network, the PARAMICS default is set relatively high (85%) of vehicles/drivers being completely familiar with the network. From the literature it was found common to reduce this percentage to around 60% (Aldazaba, 2004), but as the vast majority of drivers within the selected study network are likely to live within the local area and therefore be familiar with the local road netwok system we stick to the default value.

Saudi Arabias transportation system is mainly based on cars and there are limited opportunities that passengers take transit within this network. Therefore, to keep consistency with the previous study done by Olba (2007) the proportion of vehicles is assumed as, 90% of the vehicles are cars and the rest 10% proportion is assigned to LGVs (Light good vehicles).

5.2.1.3 Calibrated Parameters

The PARAMICS model contains over 50 adjustable/ user defined parameters. A number of these are switches between one type and another or on/off values such as: a random number generator type, seed number, turning penalty and visibility. These variables were set at default values and were left un-amended throughout the calibration. Many of these parameters are based on logical and simple statistics (vehicle weight, vehicle height, large vehicle, etc.) and they do not need to be revalidated or recalibrated.

Despite these reductions, it can be seen (Table 5.1) that the number of parameters, their respective ranges of values and the combination of parameters that can be used for calibration is still significant. Moreover, some parameters affect the simulation on a 'global' basis and some on a 'local' link basis and many of the parameters are continuous values rather than discrete (Park and Schneeberger, 2005).

Table 5-1 Major variable Parameters in PARAMICS

Parameter	Default Value	Feasible Range	Effect
Mean Target Headway	1s	0.35-5s	Car following distances/ aggression
Mean Driver Reaction Time	1s	0.5-3s	Car following/ Lanechanging / awareness
Minimum Gap	2m	1-3m	Queue Lengths
Feedback Period	5min	1-10min	Assignment
Compliance Levels	100%	0-100%	Pedestrian behaviour and thus vehicles at crossings
Acceleration	2.5m/s2	1-8m/s2	Driver reaction time
Deceleration	4.5m/s2	1-8m/s2	Driver reaction time
Speed Memory	3	1-75	No. of timesteps/driver reaction time
Signpost Range	250m	1-300m	Driver behaviour
Link Headway factor	1	0.5-2s	Driver behaviour – link specific
Link Reaction Factor	1	0.5-2s	Driver behaviour – link specific
Category Headway Factor	1	0.5-2s	Driver behaviour – link category specific

In PARAMICS when a vehicle catches up with another vehicle or reaches an obstacle, such as a junction or bottleneck, a car following and lane changing algorithm takes effect. Several algorithms determine how the (trailing) vehicle will respond to the current circumstances. The three implemented individual vehicle movement models in PARAMICS (car following, gap acceptance and lane changing) are strongly influenced by two key user specified parameters (Gardes et al, 2002): the Mean Target Headway (MTH) and Mean Reaction Time (MRT). Moreover, based on the experience of PARAMICS users, the model includes the parameters awareness and aggressiveness (on which PARAMICS distinguishes itself from other models).

Increasing or decreasing the Mean Target Headway (MTH) changes the overall behaviour of the model. The default value of the MTH is set at one second and has been calibrated against UK traffic conditions. Decreasing the MTH value will result in an increased number of vehicles on the road, due to the acceptance of smaller gaps.

Similar to the MTH, the Mean Reaction Time (MRT) influences the three individual movement models. The default value of the MRT is set at one second as well. A decrease in the MRT implies that drivers are more aggressive and less aware. Probably, this results in more lane changing and lower anticipation of obstacles (Vreeswijk, 2004). The MRT is also used to obtain the correct volumes and speeds on specific links.

The visibility distance on the approach link will influence the lane changing behaviour of vehicles on a road and especially with turning movements at intersections. When the visibility distance is increased, vehicles will anticipate obstacles sooner. There was no reason to change default PARAMICS settings for this study.

Signposting distances have the same theory as visibility distance and driver familiarity. It provides information about the obstacles on the road (such as intersections). An increase in the signposting distance makes drivers more aware of the upcoming obstacles that they can now expect earlier. For urban arterial the standard minimum signposting distance is 400 meters (Aldazaba, 2004). The default value of PARAMICS was not adapted.

More time steps per second increase the number of calculations per second on which the detail of vehicle movements increase. Especially in congested situations, vehicles will see more opportunities for lane changing because of the more developed and visible gaps between the vehicles. For this study time step of 3 was considered.

Every PARAMICS model can be influenced by varying its ‘seed’ value. This value controls variation or randomness of a wide range of vehicle and driver behariour parameters, but within pre-defined settings. In order to reflect the real world variation of local road newrodk operations, it is common practice to vary the seed value between multiple model runs and then average the results to determine overall performance. To ensure the robustness of the calibration and validation of the model each criteria was therefore derived from an average of 5 model runs, each of which used a different random seed value.

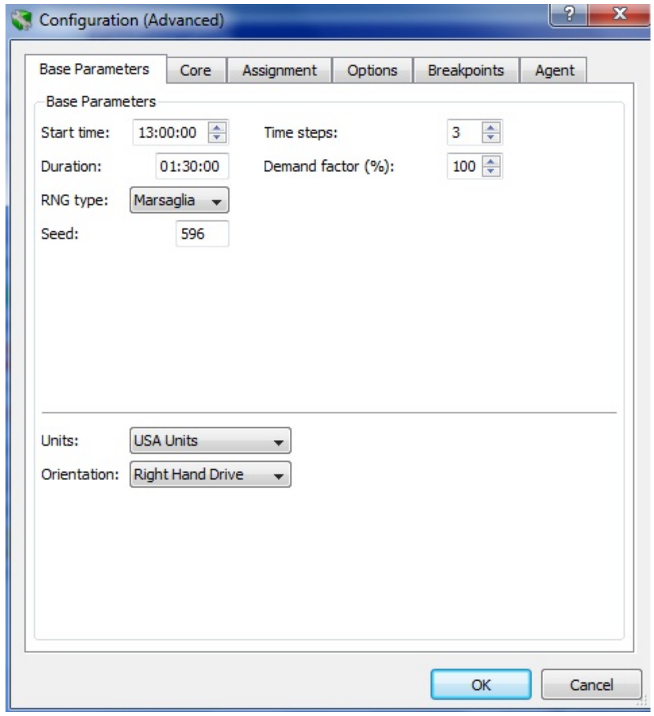


Figure 5-2 Configure settings before calibration in PARAMICS

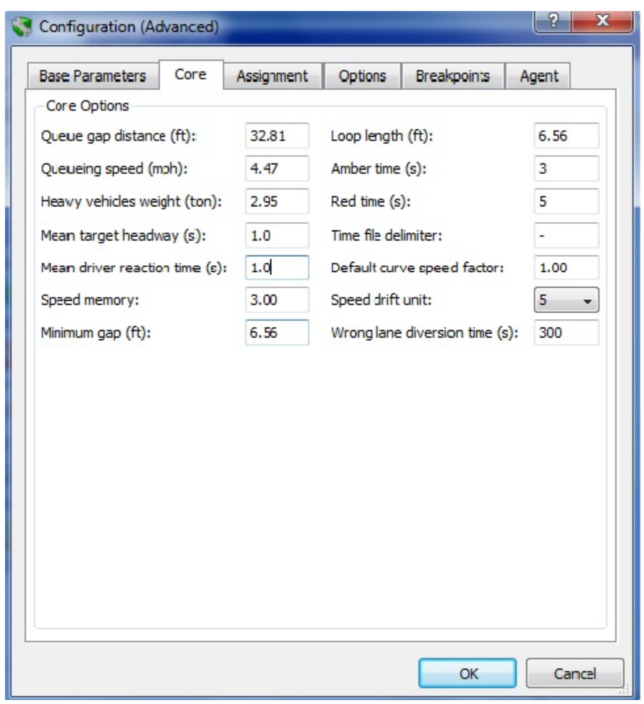


Figure 5-3 Core configuration settings before calibration

5.2.1.4 Demand Calibration

Demand calibration in the calibration process of PARAMICS requires an understanding of traffic patterns in the study area. A large portion of this process was supported by the software itself, which is included in PARAMICS, known as ESTIMATOR. ESTIMATOR uses the proportion of vehicles from one zone to another that are making a specific turn movement (commonly known as P_{ija} values) to calculate a new distribution that better fits the observed counts. P_{ija} values are obtained after running the model for the period that is being analyzed. For our case the model was run for 1 hour. Additional to the P_{ija} values the following data is required to allow ESTIMATOR to work.

- Observed turn counts and/or observed mid-block counts.
- A default seeding Origin destination matrix (OD) used as the departing point to find a better solution.

ESTIMATOR estimates the OD matrix in an iterative process where an improved OD matrix is used to run the model and obtain new P_{ija} values. These new P_{ija} values are fed into the estimator to obtain a refined OD matrix. The process keeps going until the user obtains the desired level of fit. Demand calibration is a long process that leads constantly to the recalibration of parameters related to routing and driver behaviour.

Since there was no prior information about the O-D matrix, neither it was possible to observe from the study area, it was generated from the observed traffic volumes and turning movements using the matrix estimation module (ESTIMATOR) of the software. The procedure involved estimation of the OD trips on the basis of observed link volumes and turning movement counts at the intersections. For that reason, it was necessary to balance the observed data ensuring that sums of all incoming (destination) and outgoing (origins) were the same. Independent link volumes data and approach volumes obtained from turning movement counts at intersections were also balanced to ensure consistency.

In PARAMICS a turning movement is represented by three nodes. In order to develop a robust OD matrix that complies closely with the turning movement a calibration criteria for hourly flow was set according to few previous studies (Jobanputra, 2012). As a general rule the following benchmarks were targeted as part of the calibration effort:

- Target 1: Achieve GEH value of 5.0 or less in the overall network
- Target 2: Achieve GEH value of 5.0 or less for at least 80 percent of all link locations, approach and turning movement flows considered.
- Target 3: Verify that no significant link, intersection approach or turning movement flows had a GEH value of greater 10.0

After putting all the turning values in the ESTIMATOR tool it developed a priliminary demand matrix. Since micro-simulation is a stochastic process in which every computer run represents a single observation, a complete experiment consisted of five computer runs and the results were averaged for each parameter. The simulation was run for 1 hour. The result obtained from the estimator is given in table 5.2 in the following page. A,B and C under Turning Links refers to the nodes involved in the turning movement.

Table 5-2 Comparison of observed and modeled flow

OBSERVED FLOW (vph)	Turning Link			MODELED FLOW (vph)	GEH
	A	B	C		
187	8	6	5	183	0.29
413	8	6	11	398	0.74
266	8	6	30	258	0.49
180	11	6	30	202	1.59
506	11	6	8	471	1.58
392	11	6	5	345	2.45
1770	5	6	30	1724	1.10
544	5	6	8	463	3.61
1410	30	6	5	1371	1.05
328	30	6	11	294	1.93
314	16	7	33	293	1.21
564	16	7	32	550	0.59
310	16	7	31	350	2.20
1168	31	7	33	1188	0.58
956	31	7	32	931	0.81
139	32	7	31	146	0.59
510	32	7	16	444	3.02
425	32	7	33	377	2.40
1340	33	7	31	1224	3.24
757	33	7	16	707	1.85
347	25	22	34	369	1.16
343	25	22	27	343	0.00
261	25	22	35	217	2.85
300	27	22	35	310	0.57
338	27	22	25	356	0.97
404	27	22	34	426	1.08
1831	34	22	35	1675	3.73
817	34	22	25	785	1.13
1711	35	22	34	1735	0.58
174	35	22	27	144	2.38
				Average	1.53

Since the GEH value is much less than the first target, we did not have to run the estimator again but as we were mainly concerned with the flow of major King Abudllah Street, visual inspection was carried out to those corresponding turning movements to identify if there is any larger value of GEH than 5. In that case there were no such values.

Demand profile matrix, another type of matrix was built to avoid sub hourly traffic variation and to make sure that the vehicle release from the model is close to reality during the simulation. The shape of the demand profile therefore affects the peaked nature of traffic across the network. The modelled one hour simulation duration was divided into 4 equal parts of 15 minutes as our observed data was also collected at an interval of 15 minutes.

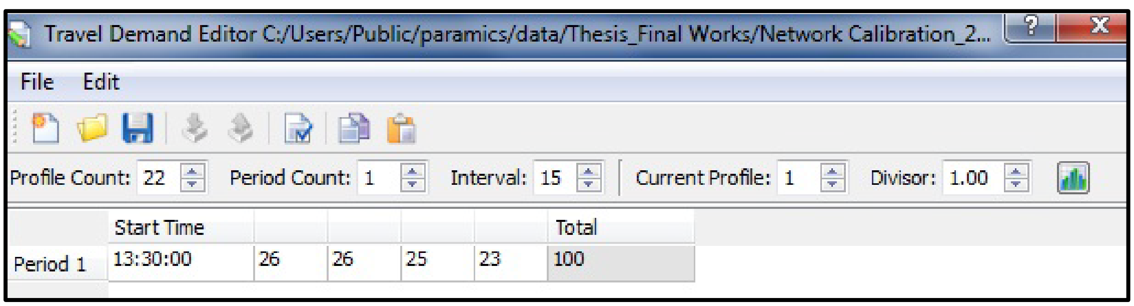


Figure 5-4 Traffic Demand Profile

The demand profile is translated as the profile number is 22 and during the one hour of simulation period 26 percent of the hourly flow vehicle would be released in the first interval and the subsequent release of vehicle would be 26, 25 and 23 respectively. Finally, another matrix is build that comprises the profile count of all the zones. The number in the profile matrix (Table 5.3) refers to the Profile count number.

Table 5-3 Demand Profile Matrix

	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Zone 6	Zone 7	Zone 8
Zone 1	1	2	1	1	1	1	1	1
Zone 2	3	1	4	5	1	5	1	5
Zone 3	6	7	1	8	8	8	8	8
Zone 4	9	9	9	1	10	11	11	11
Zone 5	12	12	12	13	1	14	14	14
Zone 6	15	15	15	15	15	1	16	17
Zone 7	20	20	20	20	20	18	1	19
Zone 8	22	22	22	22	22	1	21	1

After doing slight manual tuning in the ESTIMATOR generated OD matrix it takes the following form (Table 5-4). Having achieved the calibration target with hourly flow in the first attempt we have stopped the calibrating process for volume here and moved

Table 5-4 Initial estimated OD matrix in PARAMICS

Zone	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	Sum
1	0	817	0	270	0	686.00	0	1070	2843
2	347	0	343	93	0	93.00	0	101	977
3	404	338	0	104	0	101.00	0	109	1056
4	168	0	33	0	413	110.00	0	200	924
5	400	0	45	506	0	81.00	0	125	1157
6	89	0	34	0	70	0	510	425	1128
7	190	0	57	0	98	564.00	0	314	1223
8	1270	0	83	0	190	0.00	757	0	2300
Sum	2868	1155	595	973	771	1635	1267	2344	**11608**

forward to other model parameter adjustments for Travel Time(TT) and Queue Length (QL). The estimated OD matrix is appended in table 5.4. After the sensitivity analysis of Travel time and queue length for the two parameters final OD matrix would be developed using estimator module again.

5.2.2 Model Calibration for Travel Time and Queue Length

Model calibration for Travel Time and Queue Length also involves an iterative process. The sensitivity of the two key parameters Mean Target Headway (MTH) and Mean Reaction Time (MRT) that directly affects the model embdded theories are analyzed in order to set a reasonable match between modeled and observed values. Later model validation would be carried out for the second case study with a different set of data of similar traffic conditions. This is regarded as a final stage to investigate whether each component is adequate enough to reproduces observed travel characteristics independently.

5.2.2.1 Sensitivity Analysis for Travel Time

A sensitivity analysis was performed by varying the two parameters Mean Target Headway (MTH) and Mean Reaction Time (MRT); this was done by making changes to one parameter and keeping the other constant at default values. The simulated measure of effectiveness that was compared to the observed one was Travel Time (TT) first. As there were three intersections in the study area, both Eastbound and Westbound travel time was compared starting from the first intersection (Dhahran-Makkah) to the following intersections. Based on the experience of previous studies in different countries (table 2.6) an initial range for MTH and MRT was set as 0.5 to 1.5. When MRT was held constant at its default value of 1.0 the other parameter MTH was increased by 0.1 s interval. The effect of Mean Headway and Reaction time on the travel time can be seen in the following figures 5.5 to 5.12.

For the Eastbound simulated Travel time for 1 hour, 4 hours and with 1 hour warm up period were compared with the observed travel time between the interim intersections to identify if there is any variation in simulation time duration. Visually the variation identified due to simulation duration was very insignicant. Therefore, as described in the software manual we stick to simulation results with 1 hour warm up peroid. Firstly, Mean target headway was increased from 0.5 second to 1.5 seconds with an increment of 0.1 second while keeping Mean reaction time constant at its default value 1.0 and finally it was altered as mean reaction time was changing and mean headway remained stationary. When Mean Headway was made to change the optimum value of simulated travel time lies between 0.5 to 0.7 for the run from Makkah to Haumd Intersection while from Hamud to King Fahd Intersection the closest value lies between 0.6 to 0.8. In case of Reaction time the closest value was within 0.5 to 0.6 seconds for Makkah to Hamud Intersection and also 0.5 to 0.6 seconds for Hamud to King Fahd Intersection.

For the westbound, when Mean headway was made to change the optimum range of was between 0.5 to 0.6 for travel time among intermediate and external intersections. The value of tarvel time increased as the value of Mean headway was also increased. The average slope of all the trend line was positive and linear which reflects that there is a minimum chance that the simulated curve of travel time would hit or come close to the observed travel time straight line again. In case of changing the reaction time with mean headway being constant the same trend and domain was observed. Initially a general travel time calibration target was set to have 85% of the compared travel time of the network should fall within 85% of the observed value.

As we would also be calibrating the model in terms of Queue Length, we made a range or domain of the two parameters out of the figures (figure 5.5 to 5.12). The reason to find a domain for both the parameters is valid as we need to find a common value of the parameters that satisfies or closely matches both travel time and Queue length for simulated and observed values.

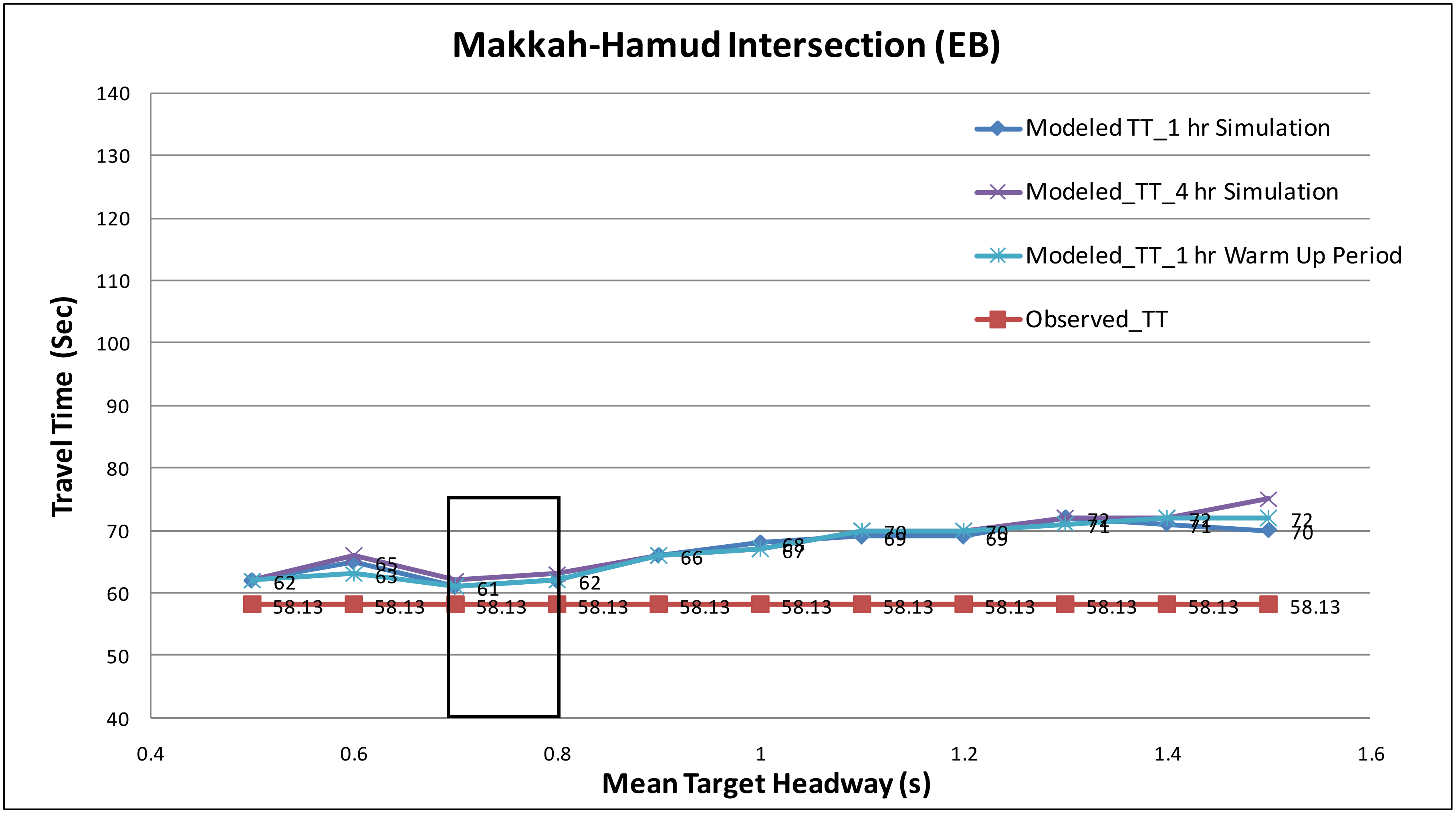


Figure 5-5 Travel Time comparisons with changing Mean Target Headway (s) Eastbound (Makkah to Hamud Intersection)

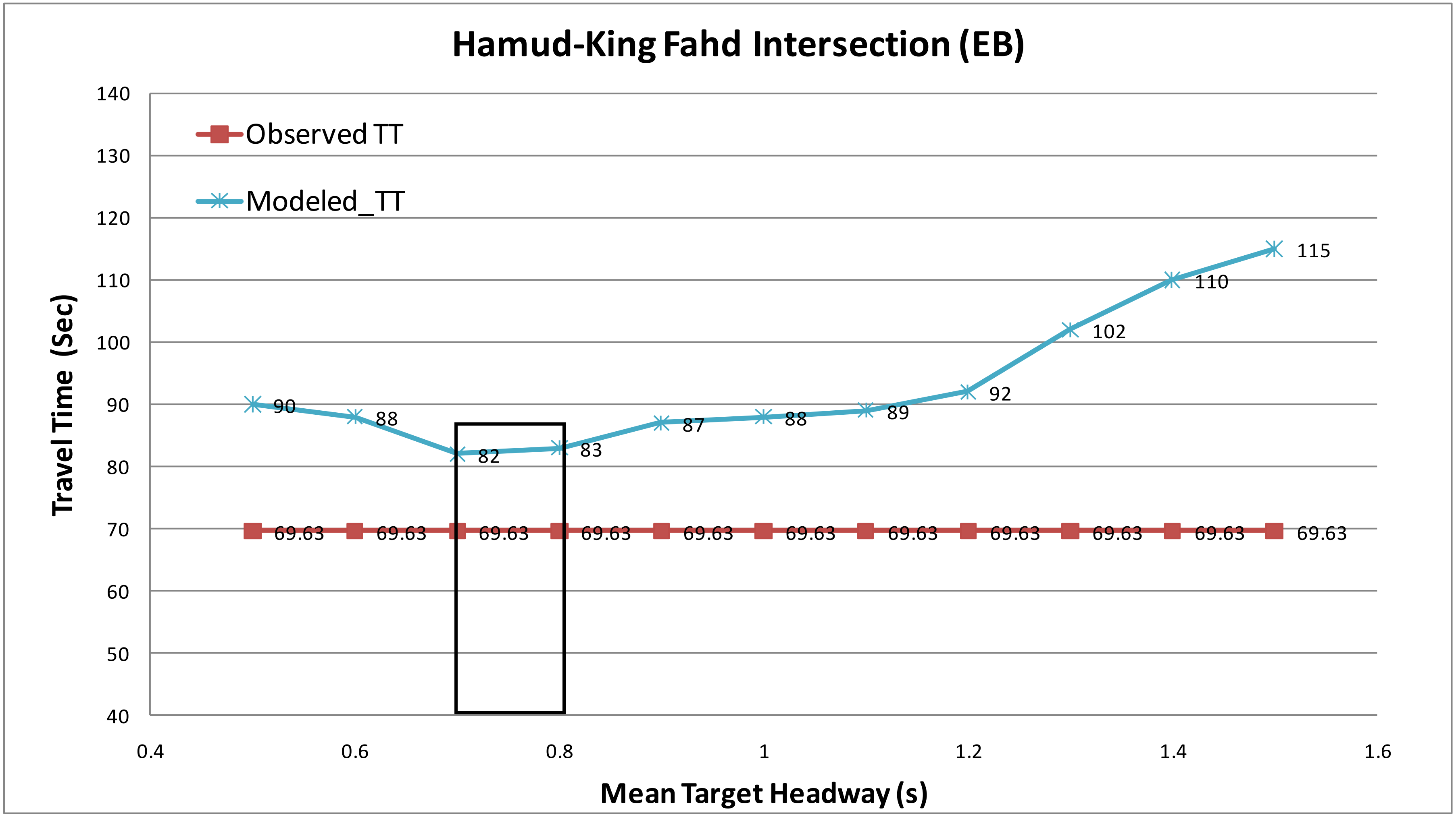


Figure 5-6 Travel Time comparisons with changing Mean Target Headway (s) Eastbound (Hamud to King Fahd Intersection)

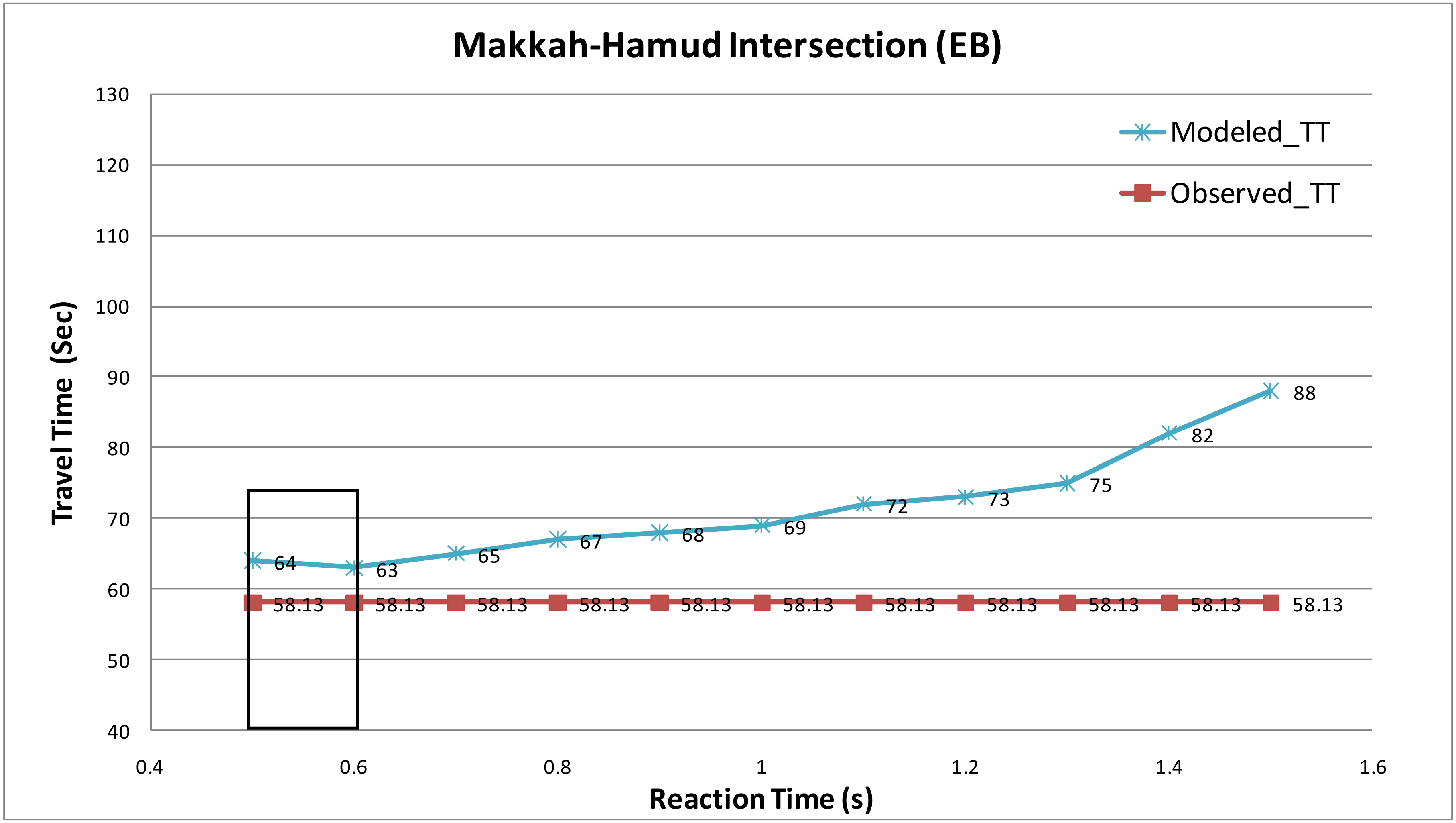


Figure 5-7 Travel Time comparisons with changing Mean Reaction Time (s) Eastbound (Makkah to Hamud Intersection)

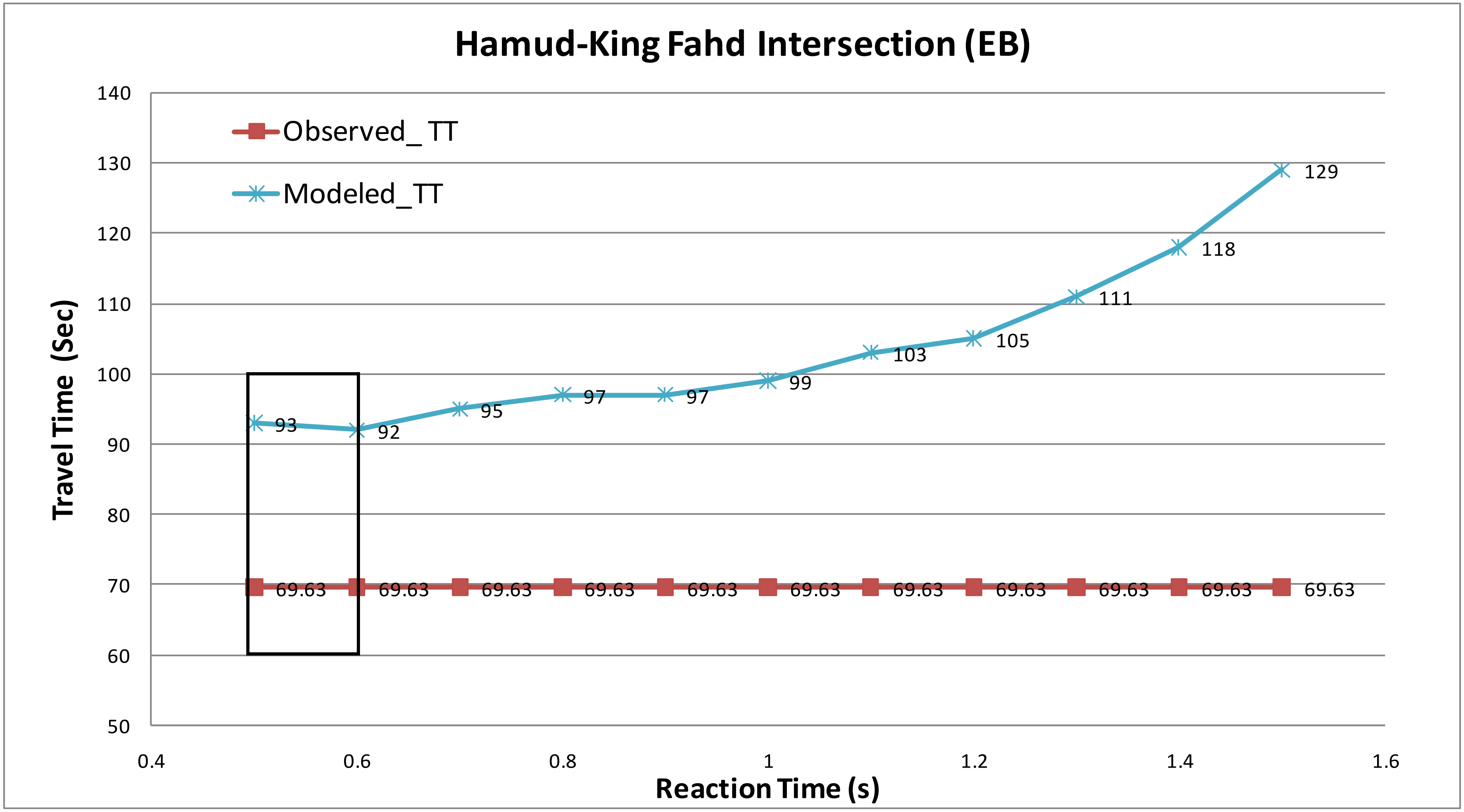


Figure 5-8 Travel Time comparisons with changing Mean Reaction Time (s) Eastbound (Hamud to King Fahd Intersection)

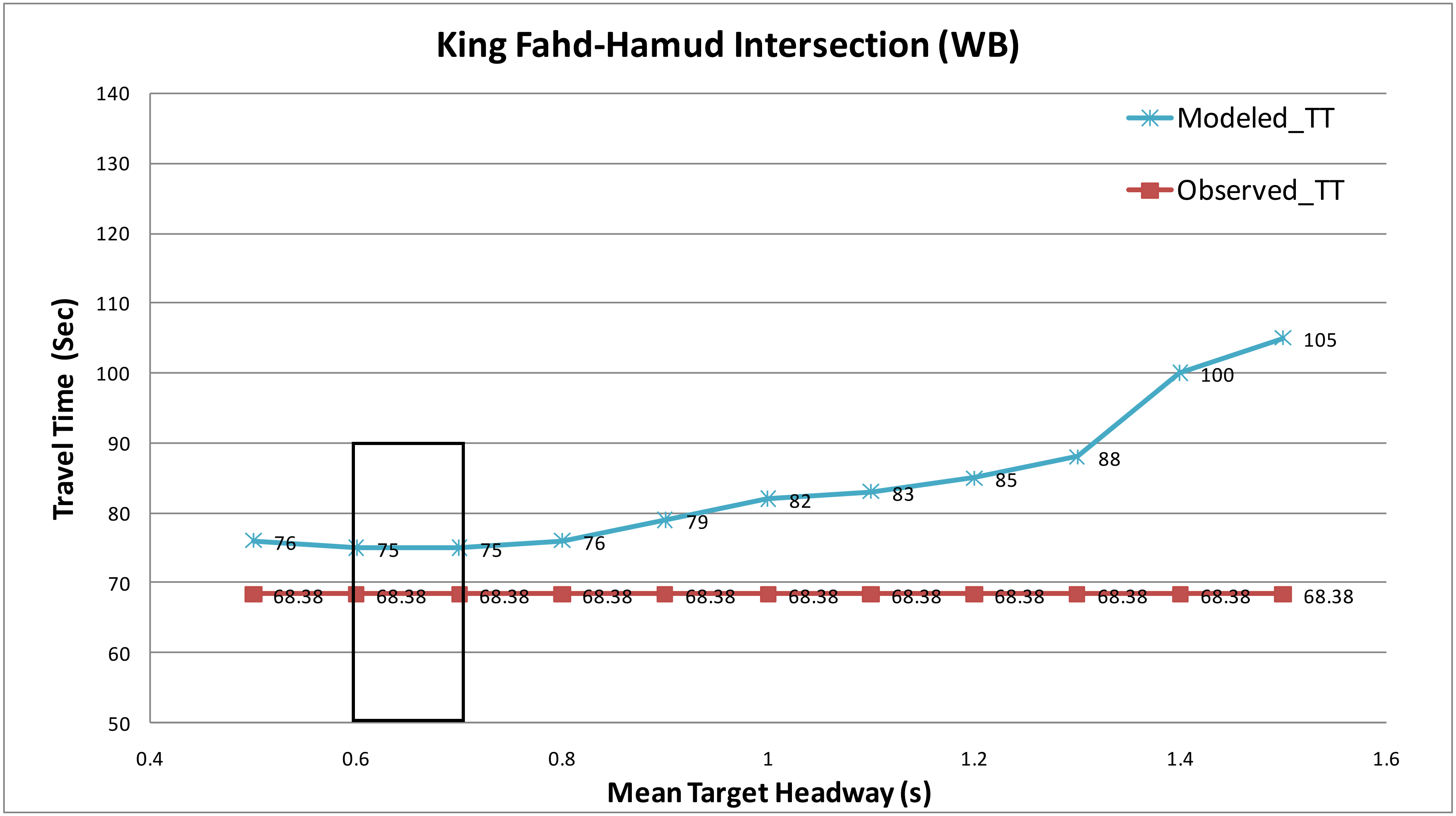


Figure 5-9 Travel Time comparisons with changing Mean Target Headway (s) Westbound (King Fahd to Hamud Intersection)

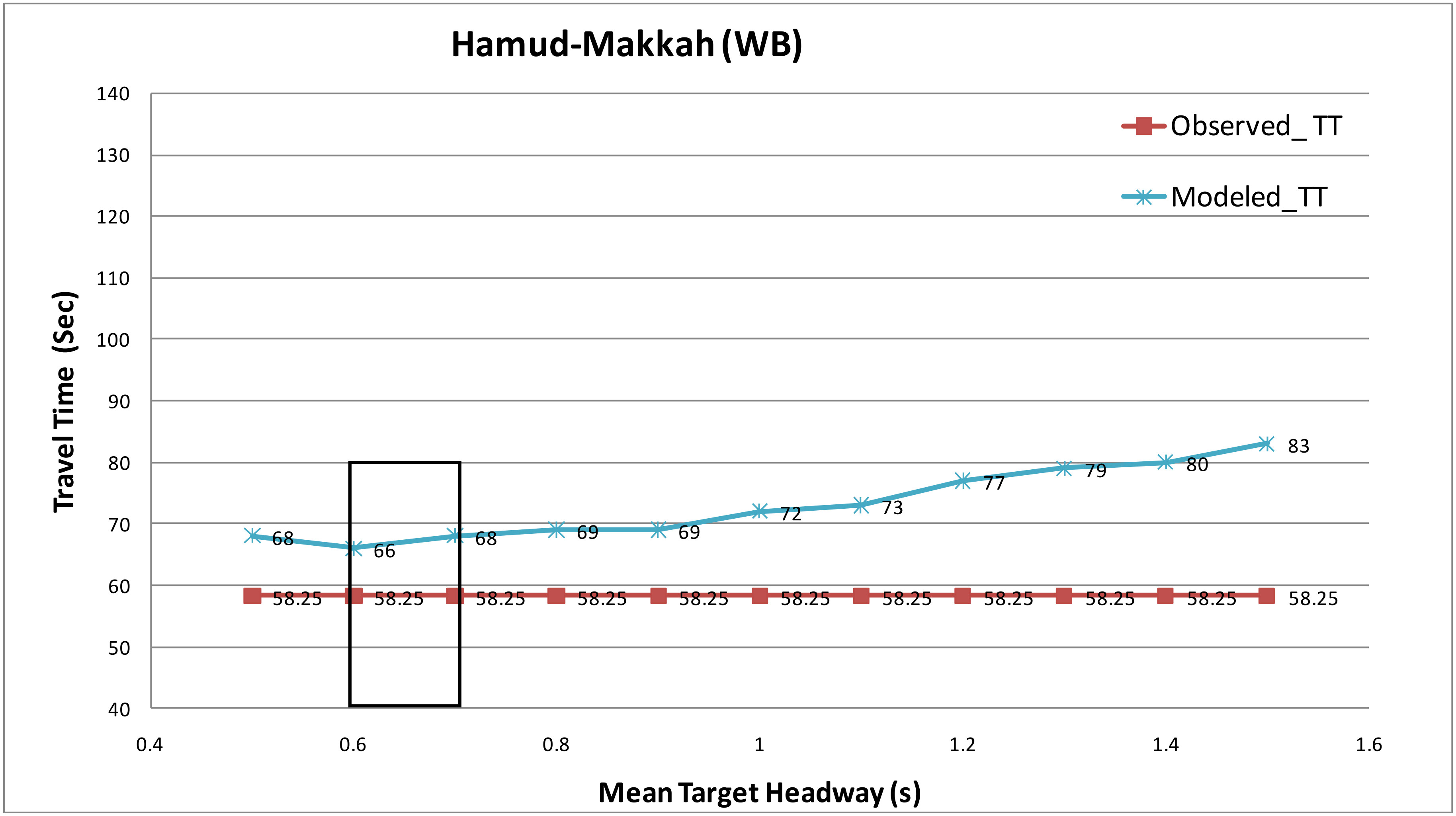


Figure 5-10 Travel Time comparisons with changing Mean Target Headway (s) Westbound (Hamud to Makkah Intersection)

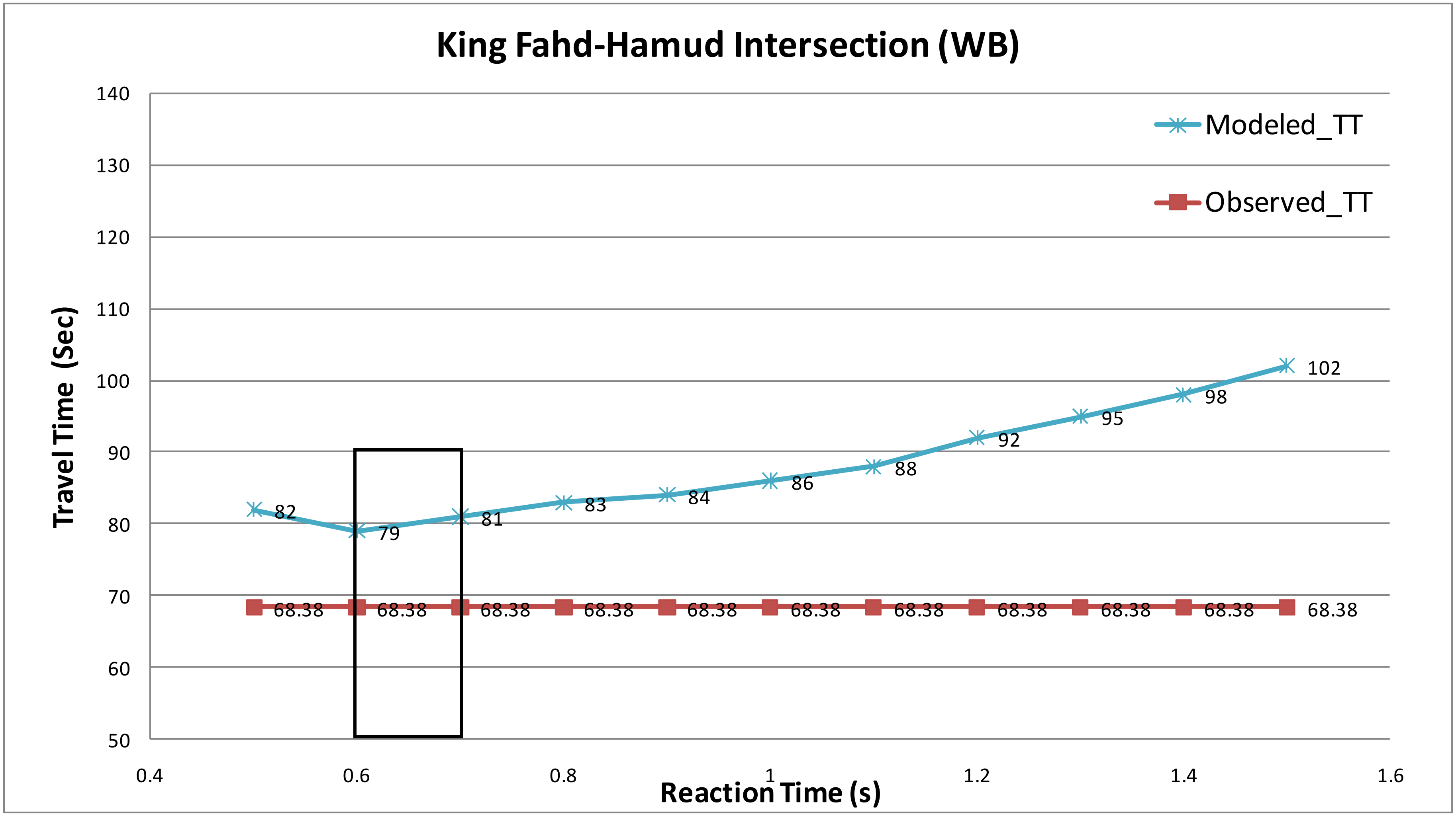


Figure 5-11 Travel Time comparisons with changing Mean Reaction Time (s) Westbound (King Fahd to Hamud Intersection)

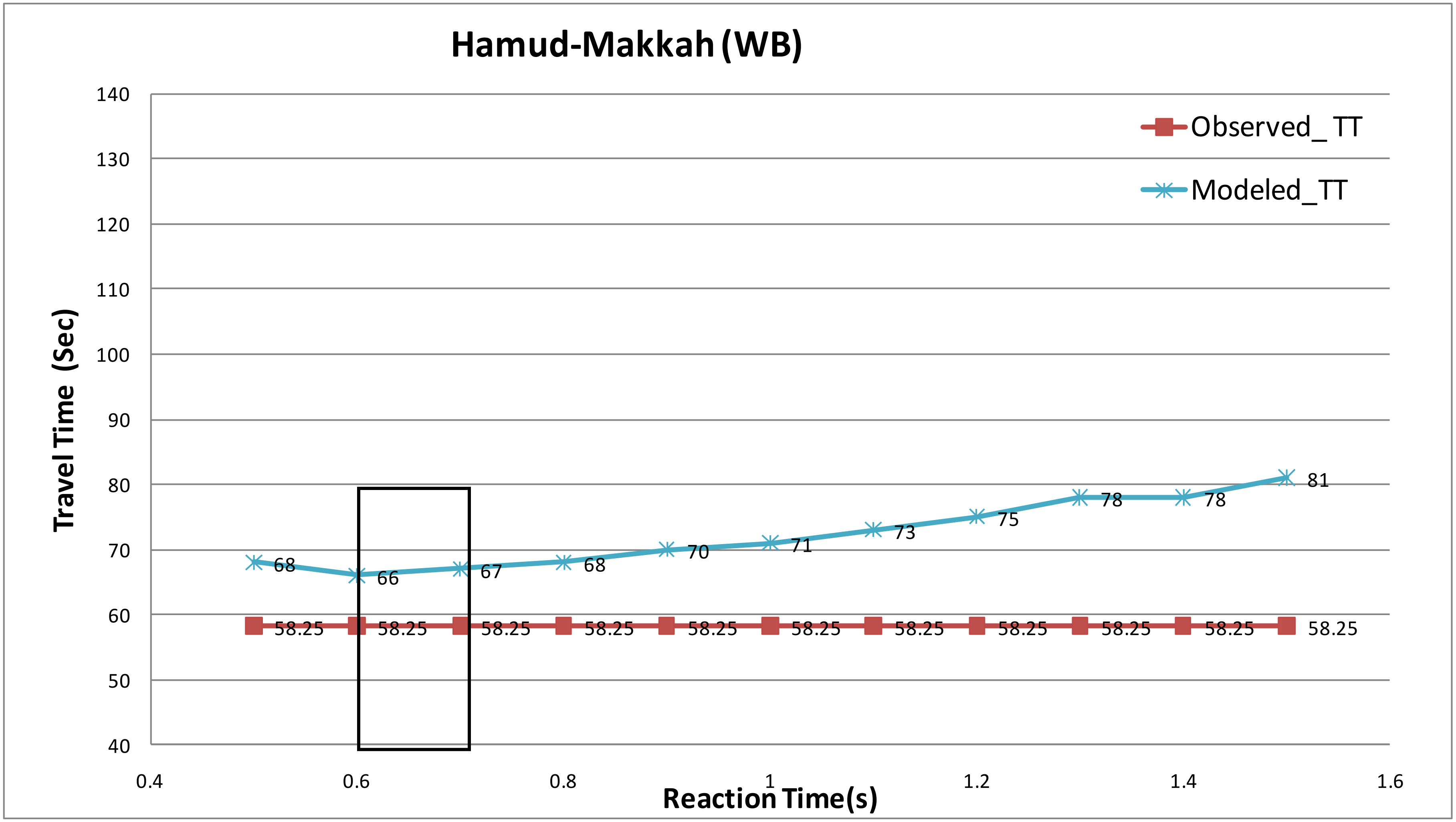


Figure 5-12 Travel Time comparisons with changing Mean Reaction Time (s) Westbound (Hamud to Makkah Intersection)

concurrently. So a trades off needs to be done between the accuracy of calibrated travel time and Queue length.

By visual inspection over the developed graph it was found that the model gave best results when it was run for 1 hour with 1 hour warm up period. The domain of Mean Target headway and Mean reaction time is shown in the following table.

Table 5-5 Ranges of Mean Target Headway and Mean Reaction Time

Mean Target Headway	0.5 to 0.7 Seconds	Default 1.0 Second
Mean Reaction Time	0.5 to 0.6 Seconds	Default 1.0 Second

A table is appended below for an initial comparison of travel time along the networks.

Table 5-6 Comparison of travel time through the network for Mean Target Headway (MTH) and Mean Reaction Time (MRT) domain

Direction	**Intersection Name**	**Observed Travel Time**	**Travel Time (s)**							
			Combination 1		**Combination 2**		**Combination 3**		**Combination 4**	
			MTH 0.7 and MRT 0.6	**Cumulative GEH**	**MTH 0.7 and MRT 0.5**	**Cumulative GEH**	**MTH 0.5 and MRT 0.6**	**Cumulative GEH**	**MTH 0.5 and MRT 0.5**	**Cumulative GEH**
Eastbound	Makkah to Hamud	58.13	61	1.79	62	2.78	62	2.57	62	2.78
	Hamud to King Fahd	69.63	82		90		88		90	
Westbound	King Fahd to Hamud	68.38	75	1.77	75	2.01	75	1.77	82	2.80
	Hamud to Makkah	58.25	66		68		66		68	
Total =				3.56		4.79		4.34		5.58

5.2.2.2 Sensitivity analysis for Queue Length

As a part of the calibration process simulated Queue length was also matched with the observed value. To do that the same procedure was adopted for the sensitivity analysis as it was done for travel time. The main idea was to simulate the model with a gradual change in Mean Target Headway (MTH) whilst keeping Mean Reaction Time (MRT) in its default value and reversing the same process for the two parameters. When the simulated results in terms of Queue Length are plotted for each of the parameters, they would converge or come very close to the plotted line that refers to observed Queue Length. But the process was not so easy as we had observed queue length for left turning lanes and through lanes and wanted to compare both of them separately. Among the three signalized intersections there were four cases where queue length were attempted to be matched for both Eastbound and Westbound direction. The results showed that in some cases the model output was very inspiring for few intersections and some of it gave wayward results.

The analyzed domain of MTH and MRT was 0.5 second to 1.5 second with an interval of 0.5 second. Therefore, nine different combination of 0.5, 1.0 and 1.5 for MTH and MRT were tried for simulation run with varying seed numbers for each combination. Co-relation coefficient was used in this case to choose between this combinations of MTH and MRT that showed closest match to observed value.

Table 5-7 Comparison of Queue Length with different MTH and MRT combination

MTH	MRT	Node	Direction	Left Turn Links					Through Links				
				OBS QL	**SIM QL**	**GEH**	**Total GEH**	**R**	**OBS QL**	**SIM QL**	**GEH**	**Total GEH**	**R**
0.5	**0.5**	1	WB	7.3	4	1.39	8.83	0.86	32.3	38	0.96	3.48	0.75
		2	EB	8.9	5	1.48			18.3	17	0.31		
		2	WB	14.1	5	2.94			32.9	25	1.47		
		3	EB	16	6	3.02			19.6	23	0.74		
0.5	**1**	1	WB	7.3	4	1.39	8.47	0.84	32.3	91	7.48	11.89	0.53
		2	EB	8.9	5	1.48			18.3	18	0.07		
		2	WB	14.1	5	2.94			32.9	22	2.08		
		3	EB	16	7	2.65			19.6	31	2.27		
0.5	**1.5**	1	WB	7.3	4	1.39	9.33	0.71	32.3	97	8.05	13.90	0.48
		2	EB	8.9	4	1.93			18.3	17.5	0.19		
		2	WB	14.1	4	3.36			32.9	22	2.08		
		3	EB	16	7	2.65			19.6	39	3.58		
1	**0.5**	1	WB	7.3	4	1.39	8.12	0.82	32.3	72	5.50	11.07	0.43
		2	EB	8.9	5	1.48			18.3	18	0.07		
		2	WB	14.1	5	2.94			32.9	16	3.42		
		3	EB	16	8	2.31			19.6	30	2.09		
1	**1**	1	WB	7.3	4	1.39	8.47	0.84	32.3	88	7.18	14.13	0.5
		2	EB	8.9	5	1.48			18.3	12	1.62		
		2	WB	14.1	5	2.94			32.9	16	3.42		
		3	EB	16	7	2.65			19.6	29	1.91		
1	**1.5**	1	WB	7.3	4	1.39	8.20	0.66	32.3	85	6.88	13.34	0.53
		2	EB	8.9	5	1.48			18.3	12	1.62		
		2	WB	14.1	4	3.36			32.9	20.5	2.40		
		3	EB	16	9	1.98			19.6	32	2.44		
1.5	**0.5**	1	WB	7.3	4	1.39	9.11	0.39	32.3	57	3.70	10.65	0.29
		2	EB	8.9	5	1.48			18.3	33	2.90		
		2	WB	14.1	2	4.26			32.9	23	1.87		
		3	EB	16	9	1.98			19.6	30.5	2.18		
1.5	**1**	1	WB	7.3	5	0.93	8.42	0.4	32.3	87	7.08	13.17	0.45
		2	EB	8.9	5	1.48			18.3	28	2.02		
		2	WB	14.1	4	3.36			32.9	19	2.73		
		3	EB	16	7	2.65			19.6	26	1.34		
1.5	**1.5**	1	WB	7.3	4	1.39	8.41	0.51	32.3	80	6.37	9.01	0.6
		2	EB	8.9	6	1.06			18.3	24	1.24		
		2	WB	14.1	5	2.94			32.9	26	1.27		
		3	EB	16	6	3.02			19.6	19	0.14		

From Table 5.7 it is conspicuous that the combination of MTH 0.5 and MRT 0.5 is the best among all of the combinations as it has the highest correlation coefficient 0.86 and 0.75 respectively. When the MTH was held at 0.5 and MRT was increased from 0.5 to 1.5, a decrease in R value was observed for both left turn and through links. But keeping MTH constant for 1 second and 1.5 second, an increase in MRT from 0.5 to 1.5 resulted in an increase in R value. On the other hand when MRT was held constant at 0.5 and 1.0 second and an increase in MTH from 0.5 to 1.5 had attributed to decrease in R value for both Left Turn and Through Links. Only stationary value of 1.5 for MRT with a gradual change in MTH resulted a decrease and an increase in R value for Left Turns and Through Links respectively.

Therefore, it is obvious from the above table that the optimum solution to calibrate the model solely in terms of queue length should be around the combination of MTH 0.5 and MRT 0.5 second.

Table 5-8 Final model calibration in terms of queue length and travel time (Eastbound).

Mean Target Headway	**Mean Reaction Time**	**Queue Length (QL)**				**Travel Time (TT)**			
		Observed QL	**Simulated QL**	**GEH**	**Total GEH**	**Observed TT**	**Simulated TT**	**GEH**	**Total GEH**
0.7	0.6	27.1	25	0.411	2.613	58.13	61	0.372	1.793
		35.6	50	2.201		69.63	82	1.421	
0.7	0.5	27.1	26	0.213	2.131	58.13	62	0.499	2.779
		35.6	48	1.918		69.63	90	2.280	
0.6	0.55	27.1	27	0.019	1.047	58.13	62	0.499	2.355
		35.6	42	1.027		69.63	86	1.856	
0.5	0.5	27.1	22	1.029	2.191	58.13	62	0.499	2.779
		35.6	29	1.161		69.63	90	2.280	
0.5	0.6	27.1	21	1.244	1.311	58.13	62	0.499	2.569
		35.6	36	0.067		69.63	88	2.069	
0.53	0.5	27.1	23	0.819	0.920	58.13	61	0.372	2.120
		35.6	35	0.101		69.63	85	1.748	

Table 5-9 Final model calibration in terms of queue length and travel time (Westbound).

Mean Target Headway	Mean Reaction Time	Queue Length (QL)				Travel Time (TT)			
		Observed QL	Simulated QL	GEH	Total GEH	Observed TT	Simulated TT	GEH	Total GEH
0.7	0.6	47.0	21	4.459	7.523	68.38	75	0.782	1.765
		39.3	61	3.064		58.25	66	0.983	
0.7	0.5	47.0	20	4.665	7.603	68.38	75	0.782	2.009
		39.3	60	2.938		58.25	68	1.227	
0.6	0.55	47.0	22	4.256	6.937	68.38	76	0.897	2.245
		39.3	58	2.681		58.25	69	1.348	
0.5	0.5	47.0	30	2.740	3.163	68.38	82	1.571	2.798
		39.3	42	0.423		58.25	68	1.227	
0.5	0.6	47.0	28	3.103	3.526	68.38	75	0.782	2.009
		39.3	42	0.423		58.25	68	1.227	
0.53	0.5	47.0	26	3.476	3.587	68.38	75	0.782	1.765
		39.3	40	0.111		58.25	66	0.983	

The final target was to minimize the error for both queue length and travel time for a single combination of MTH and MRT within the optimized domain. GEH statistc is used again to find the minimum, that would better reflect the closest match.

5.2.3 Comparison of Queue Length among different calibrated model and PARAMICS

Olba (2007) had attempted to calibrate Macroscopic Simulation model TRANSYT-7F and Microscopic simulation model SimTraffic for the same arterial and using the same data set (Figure 4-2). He was successful in calibrating TRANSYT-7F but failed to calibrated SimTraffic as the model output type do not match with the process how the Queue Length is actually measured in the the field. Even though each of the model has it's own inbuilt logic and attributes, they are different in many cases but it worths to compare their selected MOE's from simulated output to find the adequacy of the model to the local traffic behaviour. As we were able to calibrate PARAMICS with an acceptable level of accuracy for the first study network, simulated output of both TRANSYT-7F and PARAMICS can be compared in terms of Queue Length. The Queue Length recorded in SimTraffic simulated output is different than TRANSYT-7F and PARAMICS in terms of time interval. Therefore, it could not be included in the comparison.

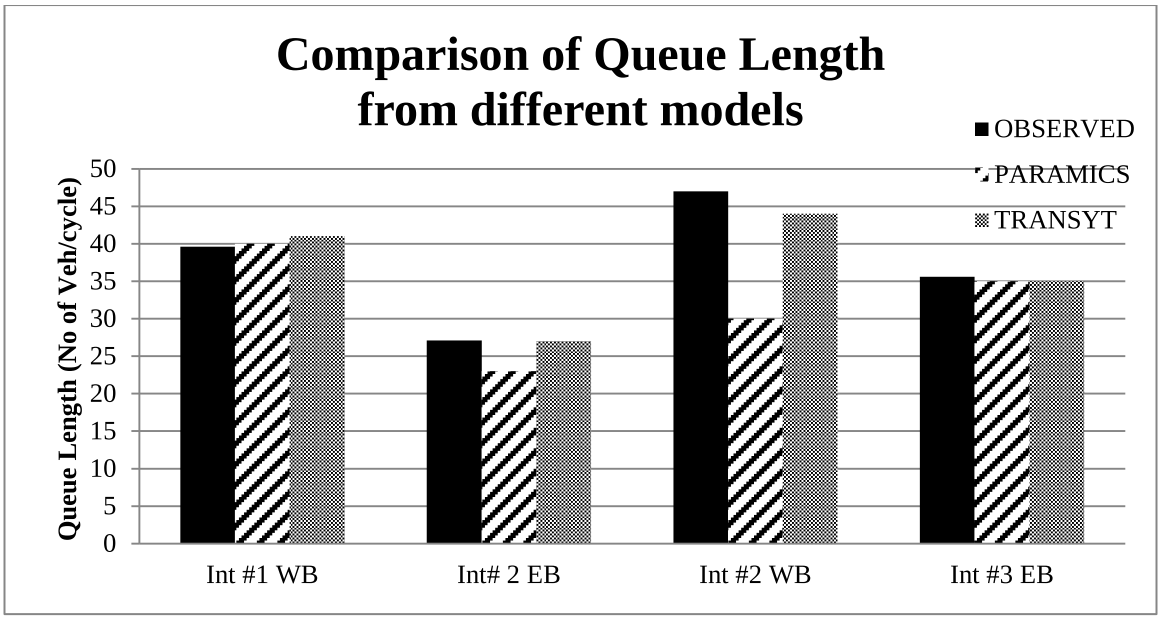


Figure 5-13 Comparison of Queue Length from different simulation model after calibration

From the above figure it can be found that TRANSYT-7F is showing slightly better performance than PARAMICS model when it comes to comparison of calibrated model queue length.

5.3 CASE STUDY-2

Another study area was selected in the same city of Al Khobar which is only 4.1 km away from the first study area. A new set of traffic data was obtained from this site as mentioned in section 4.2. The main purpose of selecting the second study area was to assess and identify if the value of calibrated model parameters for the first site in PARAMICS sufices the requirement of the second site when selected MOE’s are compared with the observed value.

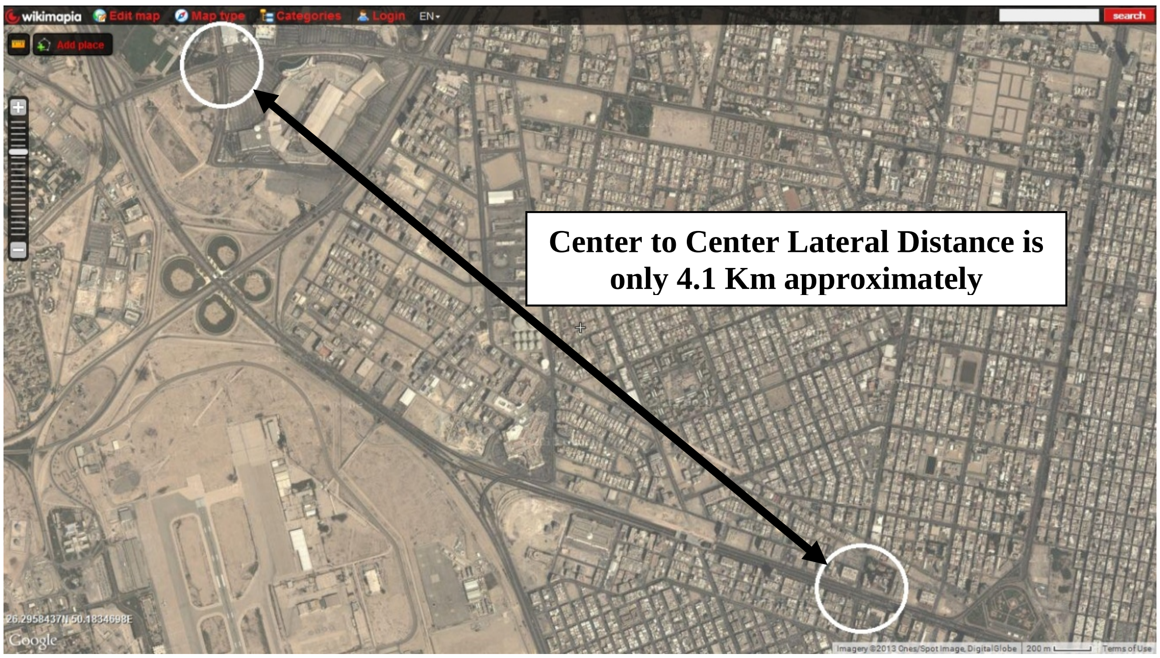


Figure 5-14 Location of the First and Second Study Area

The new network would be developed again in PARAMICS maintaining all the criteria that was adopted for the first study network. However, the model will not be calibrated again in terms of driving behaviour for this new network. The value of the calibrated parameter obtained from the first study would be used here for a fullscale 1 hour simulation run. The simulated output results would be compared to the observed MOE’s. In addition to that the nework would be modeled in TRANSYT-7F and SimTraffic/SYNCHRO for this study area for comparison. Based on the observed traffic volume and existing signal timing plan both of the aforementioned models would be used to optimize the Signal timing plan for all of the intersections for further analysis in PARAMICS.

5.3.1 Network Building and comparison of MOE's in PARAMICS

Network elements, such as, number of lanes, signal timing, stop signs, speed limits etc. was adjusted to reflect reality in order to calibrate the network in PARAMICS. In PARAMICS an initial estimate of origin-destination (OD) matrix is indispensible in creating traffic of similar characteristics to the observed one in reality.

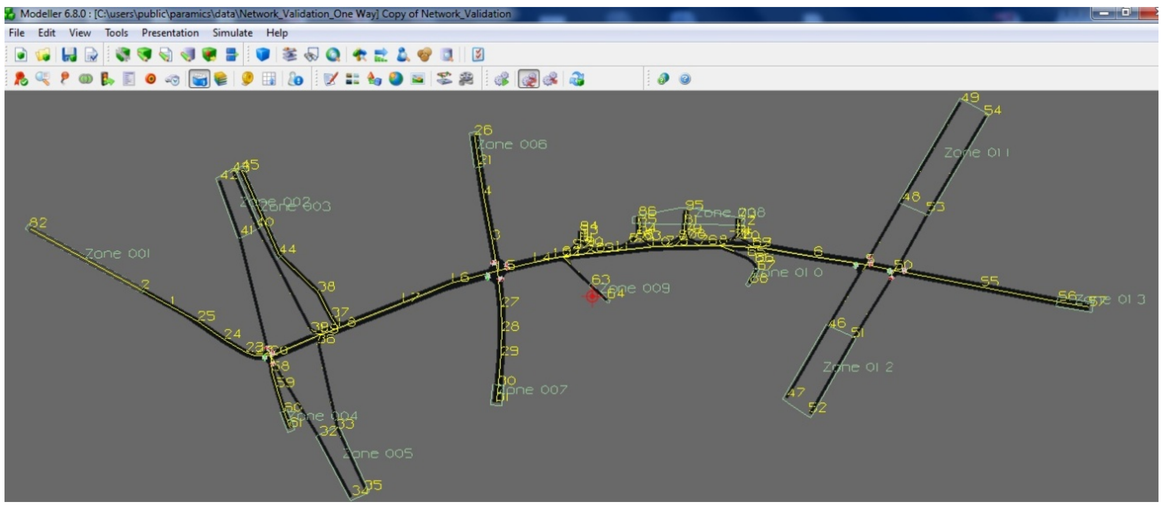


Figure 5-15 Drawn network in PARAMICS

The network has 13 distinct zones and it was built with the help of 95 nodes. There are 3 signalized intersections and the signals are not co-ordinated. Unlike the first study area this area is in a mixed commercial zone with shopping malls around it. Only the middle intersection has a left turning bay from Eastbound and Westbound. Please refer to Figure 4-6 for intersection numbering arrangement.

The driver familiarity to the network was asumed as 85% and the vehicle proportion was also kept the same to keep conformity with previous study (90% are cars and 10% are

LGV's). Ther purterbation has been kept at its default value. The simulation will be run for 1 hour from 8:30 AM to 9:30 AM to match the data collection period of field data. All other calibration parameters were kept to either default or similar to the values assumed in the first case study.

The incipient function of PARAMICS would be to build a OD matrix using the turning movement and keeping conformity to other calibration criteria. Therefore, first target is to have 85% of the GEH values should be below 5 (See section 3.3.5). The following turning movement was used and simulated in Estimator to get the GEH statistics. As we have already calibrated the PARAMICS model with an acceptable accuracy, we would use the same value of Mean Target Headway (MTH) 0.53 and Mean Reaction Time (MRT) 0.50 seconds for this study. 5 separate run were made to run with 5 different seed number and the average was taken in the analysis.

Table 5-10 Turning movement and GEH estimation

OBSERVED FLOW	**TURNING MOVEMENT**			**MODELED FLOW**	**GEH**
	A	**B**	**C**		
1203	22	20	19	1128	2.2
882	41	20	19	540	12.83
634	19	20	22	540	3.88
422	19	20	32	348	3.77
1555	16	15	14	1440	2.97
530	16	15	3	540	0.43
73	3	15	14	108	3.68
1320	14	15	16	888	13
79	17	18	37	120	4.11
816	6	5	50	696	4.36
271	5	50	53	300	1.72

OBSERVED FLOW	TURNING MOVEMENT			MODELED FLOW	GEH
545	5	50	55	504	1.79
1119	55	50	5	864	8.1
392	50	5	46	384	0.41
904	50	5	6	792	3.85
177	51	50	5	312	8.63
129	48	5	6	192	4.97
116	80	77	10	216	7.76
1165	77	10	11	1248	2.39
1003	69	8	11	960	1.37
1056	18	19	20	1176	3.59
185	18	19	39	276	5.99
1033	5		7	1020	0.41
1241	17	18	19	1236	0.14
				Average	4.26

As the average GEH value is less than 5, the first target of validation is achieved. Following to the GEH estimates a corresponding OD matrix is also generated in Estimator which is exported to Modeler for simulation run.

After the simulation was set to run in Modeler, a visual inspection was done to identify if there is any anomaly in the traffic movement. There were no cases where such discrepencies were observed. Similar to the previous study an Origin was attirbuted to only one destination and therefore, all or nothing traffic assignment method was adopted. The all or nothing traffic assignment assumes that driver will always follow minimum travel cost route under free flow condition. The following OD matrix was developed (Table 5-11) in ESTIMATOR.

The cell containing zero represents that no trip has been made to this combination of zones. The next step is to compare the Travel time and the Queue Length. After completing the simulation run for one hour with calibrated value of MTH and MRT, the model was made to run again with the default value of MTH and MRT of 1.0 second. Then a comparison in terms of Queue Length and Travel Time between intersections are drawn with the default value and the calibrated value of MTH and MRT.

Table 5-11 Developed OD matrix in PARAMICS for second study network

ZONE	***1***	***2***	***3***	***4***	***5***	***6***	***7***	***8***	***9***	***10***	***11***	***12***	***13***	***Total***
1	0	0	0	10	10	16	460		179	110	65	166	84	1100
2	0	0	0	10	10	30	100		25	137	75	92	97	576
3	112	10		11	85	0	0	0	0	0	0	0	0	218
4	10	0	0	0	10	530	10	0	13	68	44	10	60	755
5	0	0	0	0	0	10	10	0	13	68	44	40	60	245
6	102	20	10	12	85	0	10	0	11	15	18	10	24	317
7	102	20	10	10	85	10	0	0	105	52	34	23	50	501
8	10	116	10	73	10	59	116	0	0	0	0	0	0	394
9	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0
11	62	10	19	42	10	10	47	10	0	0	0	10	115	335
12	61	48	10	10	59	47	47	10	0	0	10	0	10	312
13	79	63	62	75	72	66	66	21	0	0	10	392	0	906
Total	538	287	121	253	436	778	866	41	346	450	300	743	500	**5659**

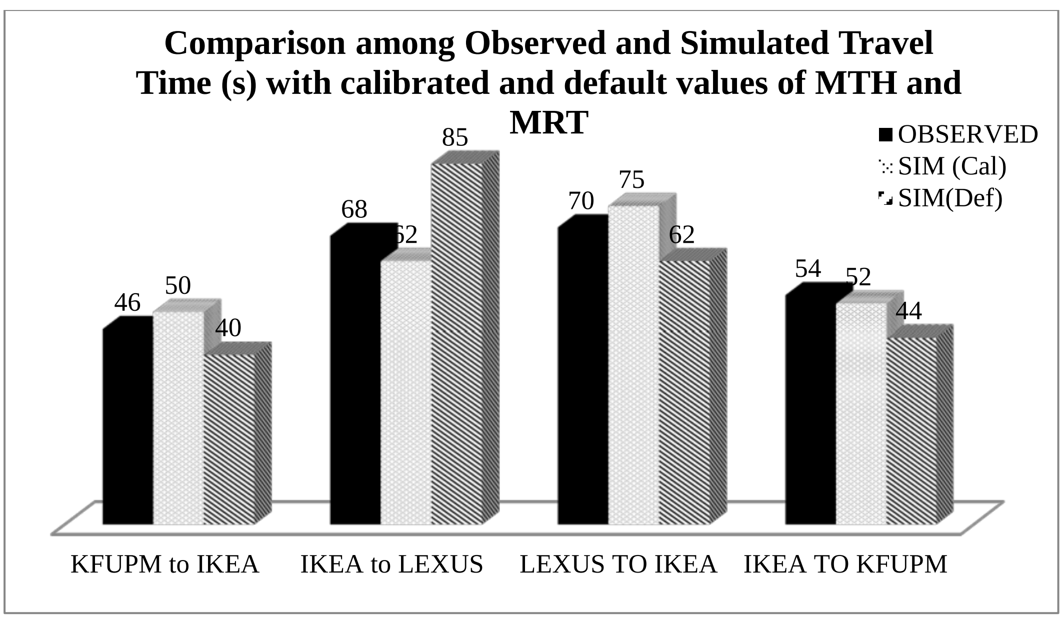


Figure 5-16 Comparison of Travel Time with calibrated and default values of Mean Target Headway (MTH) and Mean Reaction Time (MRT) in PARAMICS

As it can be seen from the above figure that the simulated Travel time from Ikea to Lexus intersection (see figure 4-6 for intersection name and numbering arrangement) using the default value of MTH and MRT was very high compared to the observed value, even though all other values are reasonably closer. However, the simulation with calibrated value gave more closer result to the observed one. Thus decision can be taken that default value of MTH and MRT warrants a change to better reflect reality.

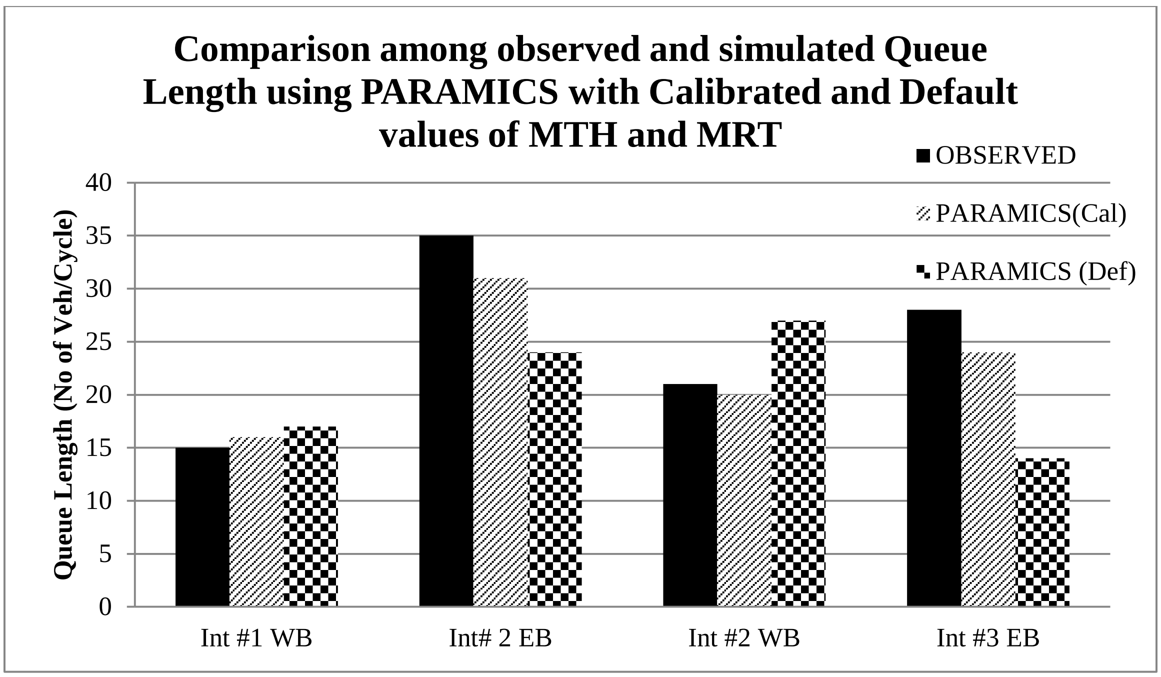


Figure 5-17 Queue Length comparison with observed and simulated value in PARAMICS

When Queue Length was compared (See Figure 4-6 for intersection number), the same trend can be observed that the simulation run with default value is not sufficient to reproduce Queue Length close to the observed value. The difference of queue length with calibrated parameter to the measured Queue Length in reality was within a range of 6% to 15% for all the intersections.

5.3.2 Network Building and comparison of MOE’s in TRANSYT-7F

For the new study area the network was coded in TRANSYT-7F. A brief illustration of the network coding process is appended below.

On opening a new file, a dialog screen illustrating some appropriate settings for beginning the new file appears in TRANSYT-7F (T7F). Once the "OK" button has been clicked to create the new data file, it is now necessary to code the remaining data. There are five main edit screens in the order they appear in T7F Edit menu. Next, the overall network layout was established using map view. All nodes will appeared in the middle of the screen and it needed to be placed at the right location laid upon an overlay. The user can then drag them to any desired location on the screen. Figure 5.18 below shows the coded network of this study.

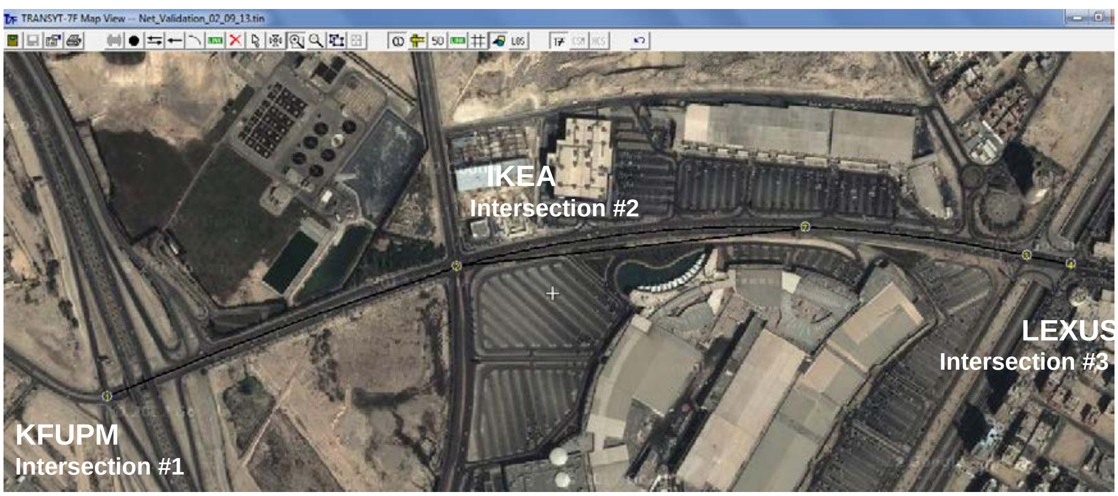


Figure 5-18 Coded network in TRANSYT-7F

At this stage, lane configuration and volume data need be coded for all four approaches, of the three intersections. Figure 5.19 below shows an example of coded lane configuration and volumes.

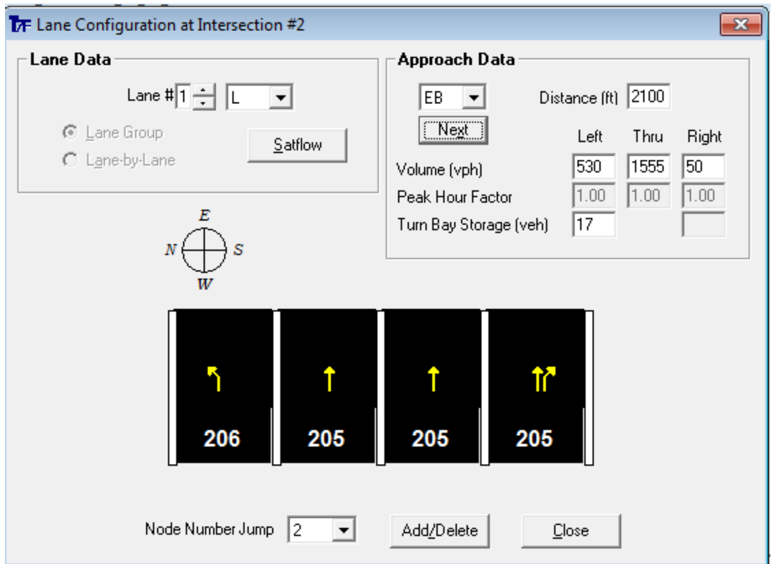


Figure 5-19 Lane Configuration and Volume Screen in TRANSYT-7F

After coding the lane configuration and volumes for all the intersections, the next step was to go to the traffic screen to review volumes and other traffic-related data. In traffic screen, the default values for link length, mid-block source volume, start-up lost time, and extension of effective green time were modified to the field measured values.

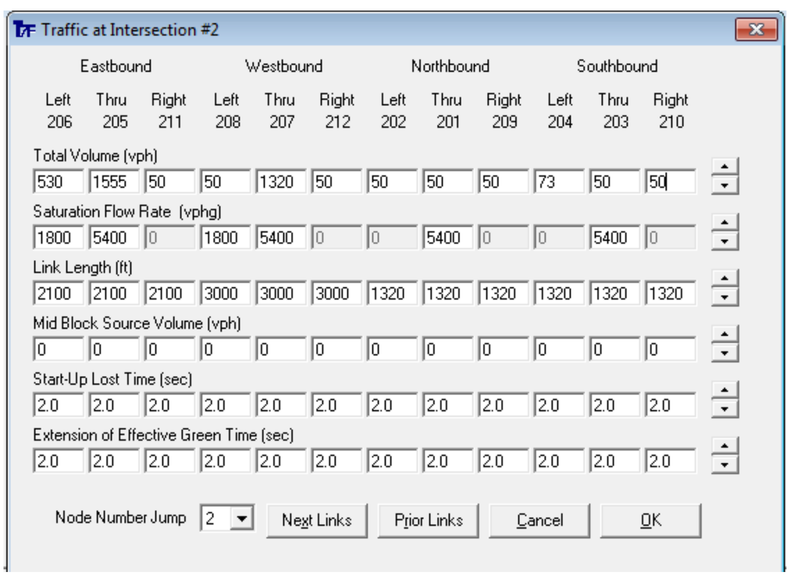


Figure 5-20 Traffic coding screen in TRANSYT-7F

As the traffic data coding is finished now, the next step is to specify intersection timing data on the timing screen. The timing plan is pre-timed at all three intersections. As the intersections are not co-ordinated it is not required to code any offset value. A sample of the timing screen is shown in Figure 5.21.

At this stage the data on the feeders screen should be entered. Input data on this screen are primarily applicable to internal links having an upstream intersection that may be affecting traffic flow patterns. The link connection information specified here affects simulation of platoon dispersion, as well as fuel consumption and travel time measurements. The information specified here also affects simulation of queue spillback, when step-wise simulation is used.

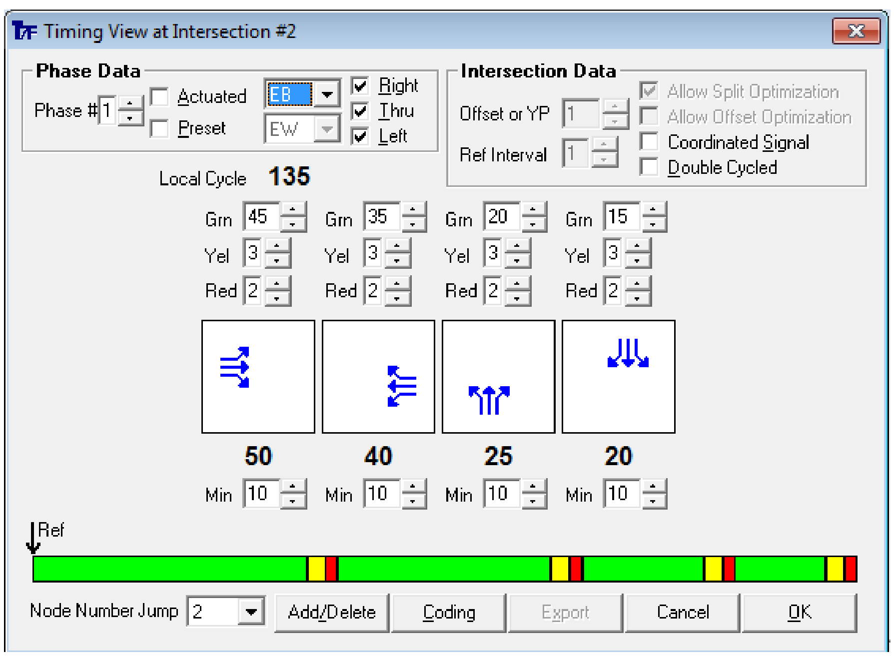


Figure 5-21 Timing and Phase Sequence for Node 2 in TRANSYT-7F (Prince Faisal Ibne Fahd Rd.–Abu Ubaidah Road, IKEA intersection)

After entering all the necessary data in the five screens above, the global data screen was opened and network parameters such as network-wide platoon dispersion factor and average vehicle spacing was set. Average vehicle spacing was used to code the jam spatial headway (default value 25 feet or 7.6 m) and the optimal spatial headway (default value 75 feet or 22.9 m). Jam spatial headway is the space a vehicle occupies when standing in the queue. Optimal spatial headway is the space a vehicle occupies when departing from a queue. From the TRANSTY-7F manual it was seen that the PDF value directly affects the queue length in intersections. A PDF value of 35 is the default and it signifies no friction in the road. As we are not calibrating the model we kept this value as it is.

Now we moved to the analysis screen (illustrated in figure 5.20 below), to specify all simulation run instructions. The screen below indicates multi-cycle step-wise simulation, with the analysis period of 60 minutes.

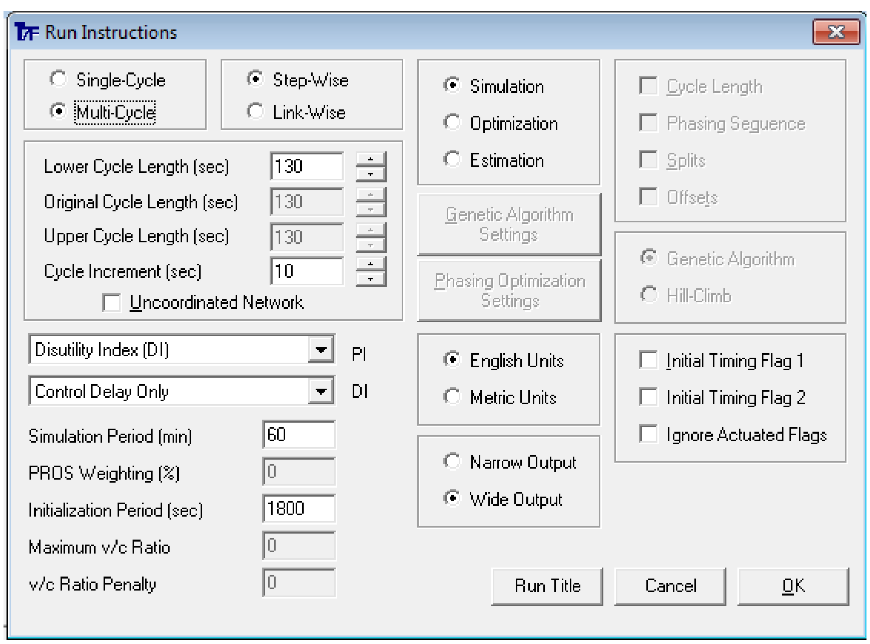


Figure 5-22 Analysis Screen in TRANSYT-7F

The initial timing flags should be deactivated so that the coded timing plan can be explicitly simulated. The disutility index is selected as the objective function for any upcoming optimization runs. Disutility index values are also reported for simulation-only runs, but this measure of effectiveness is more useful and meaningful in the context of optimization. At this time, after saving the data on the analysis screen, TRANSYT-7F was made to run without getting any fatal errors.

After setting all the parameters TRANSYT model was made to run for 3 times with PDF value of 20, 35 and 50 respectively to observe if the model is sensitive to the changes. It was found that the model changed few of its output parameters when a change is made to PDF value.

Only to identify how good the model TRANSYT-7F works a comparison was drawn between the observed and simulaetd Queue length. The following figure 5-23 shows the comparison of observed and simulated queue length in TRANSYT-7F

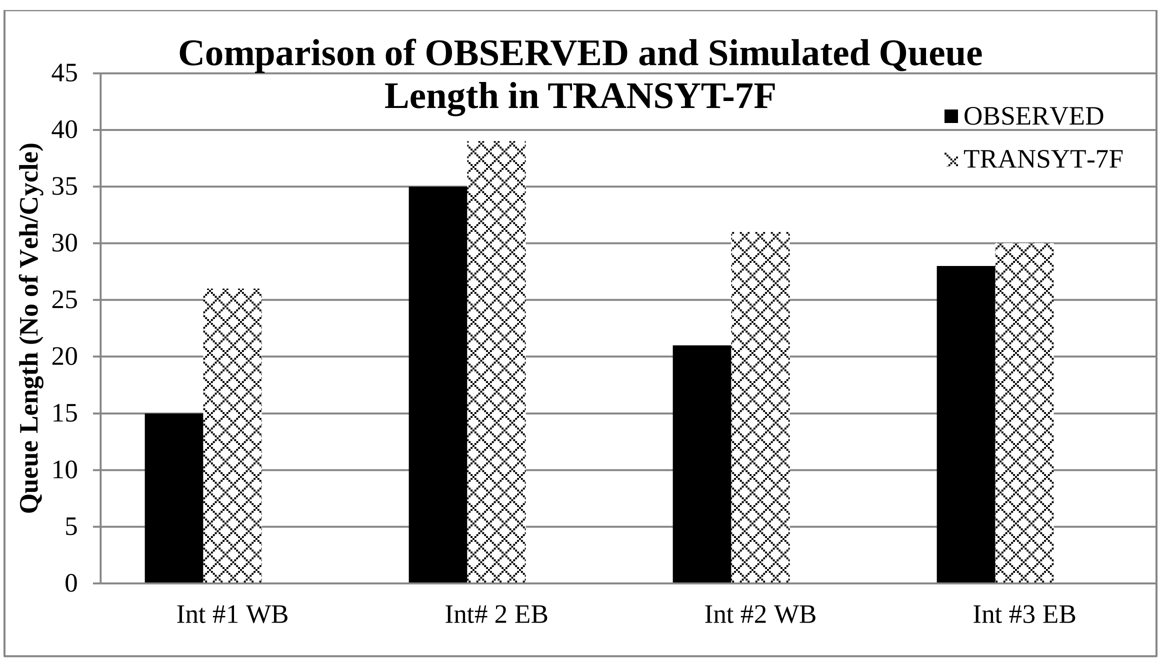


Figure 5-23 Comparison of Queue Length with simulated and Observed value

The next step was to develop an optimized signal timing plan in TRANSYT-7F. As the network was not co-ordinated in the real field, we chosed to optimize the intersection signal timing plan without offset in order to make the model compatible to the observed real field plan. Therefore, No offsets of the intersections were derived from TRANSYT-7F.

Table 5-12 Optimized signal timing plan developed in TRANSYT-7F

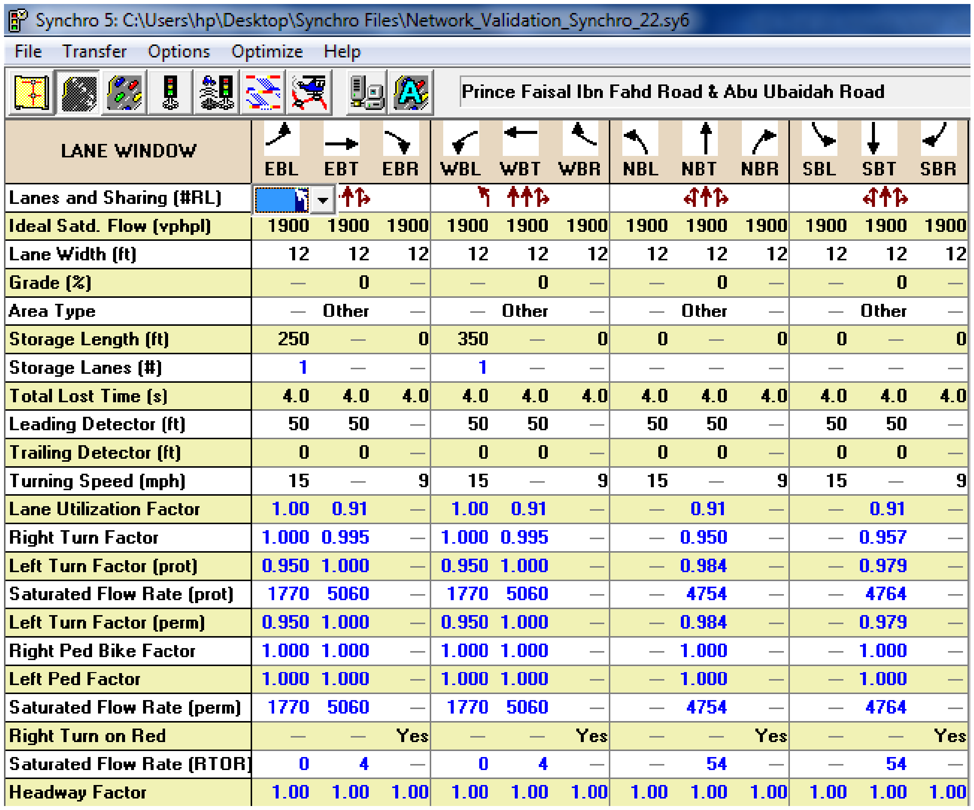
			OBSERVED SIGNAL PLAN					OPTIMIZED SIGNAL PLAN				
Time (hr)	**Intersection #**	**Direction**	**Phase No**	**Red (S)**	**Yellow (S)**	**Green (S)**	**Cycle Length (S)**	**Phase No**	**Red (S)**	**Yellow (S)**	**Green (S)**	**Cycle Length (S)**
8:30 to 9:30 Am	1 (Exit from KFUPM)	NB	4	124	3	15	142	4	104	3	13	120
		EB	1	112	3	25	142	1	96	3	21	120
		SB	3	124	3	15	142	3	80	3	37	120
		WB	2	98	3	41	142	2	88	3	29	120
		All Red		**2**				**All Red**	**2**			
8:30 to 9:30 Am	2 (Prince Faisal-Abu Ubaidha)	NB	3	112	3	20	135	3	115	3	12	130
		EB	1	87	3	45	135	1	84	3	43	130
		SB	4	117	3	15	135	4	118	3	9	130
		WB	2	97	3	35	135	2	81	3	46	130
		All Red		**2**				**All Red**	**2**			
8:30 to 9:30 Am	3 (Prince Faisal-King Saud-1)	NB										
		EB	1	107	3	20	130	1	99	3	13	115
		SB	2	107	3	20	130	2	89	3	23	115
		WB	3	52	3	75	130	3	48	3	64	115
		All Red		**2**				**All Red**	**2**			
8:30 to 9:30 Am	3 (Prince Faisal-King Saud-2)	NB	3	107	3	20	130	3	94	3	18	115
		EB	1	82	3	45	130	1	71	3	41	115
		SB										
		WB	2	77	3	50	130	2	71	3	41	115
		All Red		**2**				**All Red**	**2**			

5.3.3 Network Building and comparison of MOE's in SYNCHRO/SimTraffic

Similar to TRANSYT-7F, the data required in SYNCHRO for network coding are: traffic volumes, traffic roadway conditions and signal phasing and timing (phase sequence cycle lengths, splits and offsets). Input data entered in SYNCHRO through entry screens that include lane, volume, timing/signing, phasing and simulation windows. Data input and network coding in SYNCHRO is easier than TRANSYT-7F.

Creating street network in SYNCHRO is fast and convenient. Simply drawing of two intersecting links in SYNCHRO automatically creates a full intersection where vehicles can make multiple maneuvers (i.e. left-turns, right-turns, etc.). To draw the infrastructure network, base map (aerial photo from Google Earth) in JPEG image format was imported and used to exactly trace the study network in SYNCHRO.

After coding the network, the lane and geometric information were entered in the lane settings window. This information include lanes and sharing, traffic volume, link distance, link, speed, ideal saturated flow rate, lane width, storage length etc. Cares were taken when overriding the link distance. The field distance was taken within 20% to the map distance: otherwise, the simulation software rejects the data because map coordinates are used to simulate runs in SimTraffic. Few of the input factors are calculated by SYNCHRO automatically in the input screen. User can override their values; the overridden values appear in red. Figure 5.24 illustrates an example of lane settings window.

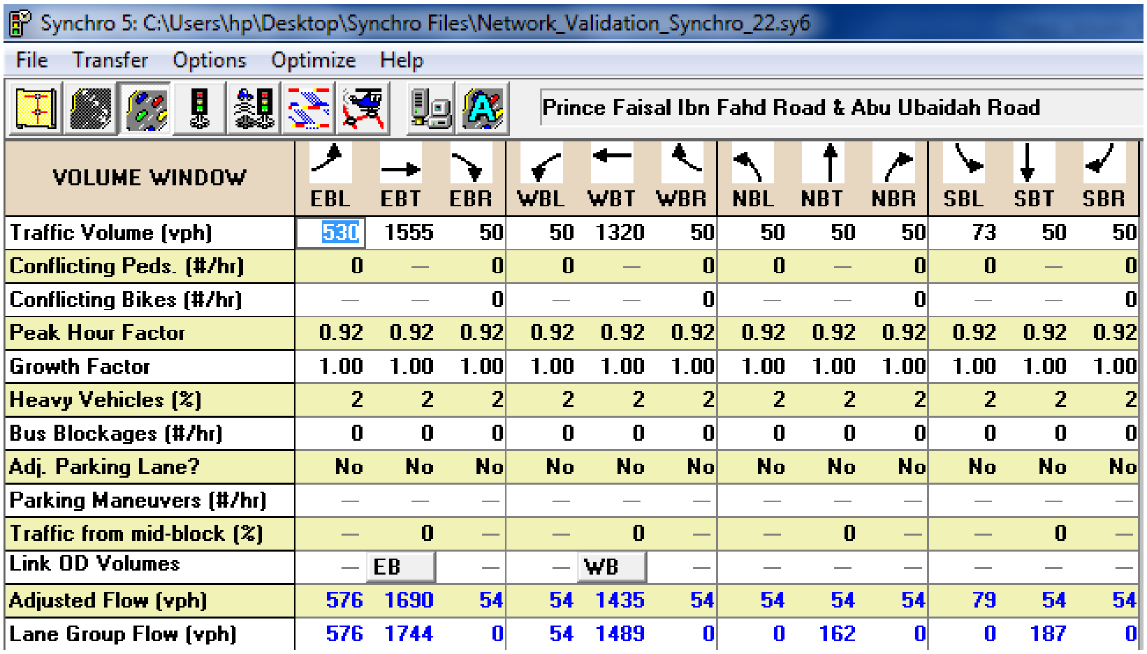


LANE WINDOW	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lanes and Sharing (#RL)												
Ideal Satd. Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width (ft)	12	12	12	12	12	12	12	12	12	12	12	12
Grade (%)	—	0	—	—	0	—	—	0	—	—	0	—
Area Type	—	Other	—	—	Other	—	—	Other	—	—	Other	—
Storage Length (ft)	250	—	0	350	—	0	0	—	0	0	—	0
Storage Lanes (#)	1	—	—	1	—	—	—	—	—	—	—	—
Total Lost Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Leading Detector (ft)	50	50	—	50	50	—	50	50	—	50	50	—
Trailing Detector (ft)	0	0	—	0	0	—	0	0	—	0	0	—
Turning Speed (mph)	15	—	9	15	—	9	15	—	9	15	—	9
Lane Utilization Factor	1.00	0.91	—	1.00	0.91	—	—	0.91	—	—	0.91	—
Right Turn Factor	1.000	0.995	—	1.000	0.995	—	—	0.950	—	—	0.957	—
Left Turn Factor (prot)	0.950	1.000	—	0.950	1.000	—	—	0.984	—	—	0.979	—
Saturated Flow Rate (prot)	1770	5060	—	1770	5060	—	—	4754	—	—	4764	—
Left Turn Factor (perm)	0.950	1.000	—	0.950	1.000	—	—	0.984	—	—	0.979	—
Right Ped Bike Factor	1.000	1.000	—	1.000	1.000	—	—	1.000	—	—	1.000	—
Left Ped Factor	1.000	1.000	—	1.000	1.000	—	—	1.000	—	—	1.000	—
Saturated Flow Rate (perm)	1770	5060	—	1770	5060	—	—	4754	—	—	4764	—
Right Turn on Red	—	—	Yes	—	—	Yes	—	—	Yes	—	—	Yes
Saturated Flow Rate (RTOR)	0	4	—	0	4	—	—	54	—	—	54	—
Headway Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Figure 5-24 Lane Settings Window in SYNCHRO

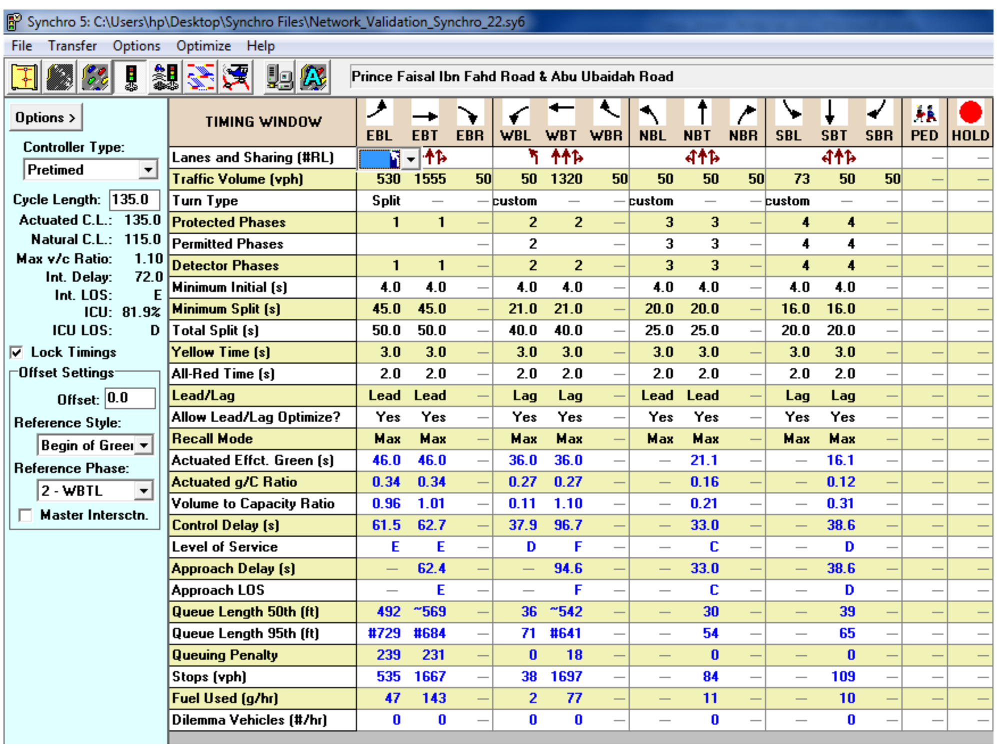
The volume information such as peak hour factor and percentage of heavy vehicles were entered in the volume settings window. Percent of heavy vehicles was left at its default value (2%). When opening the volume window, the lanes and sharing and traffic volumes entered in the lane settings will appear in this window. See figure 5.25 below.

The next step is to enter the signal timing data, all information related to the timing was entered in the timing/signing settings window. Timing data include cycle length, offsets, total splits, yellow time, all-red time, turn type, etc. Near the bottom of timing settings window, a splits and phasing diagram is displayed. Timing window is illustrated in figure 5.26 below. For detailed information about phase settings, phase setting window (figure 5.27) includes a column for every phase that has been set in the timing settings.



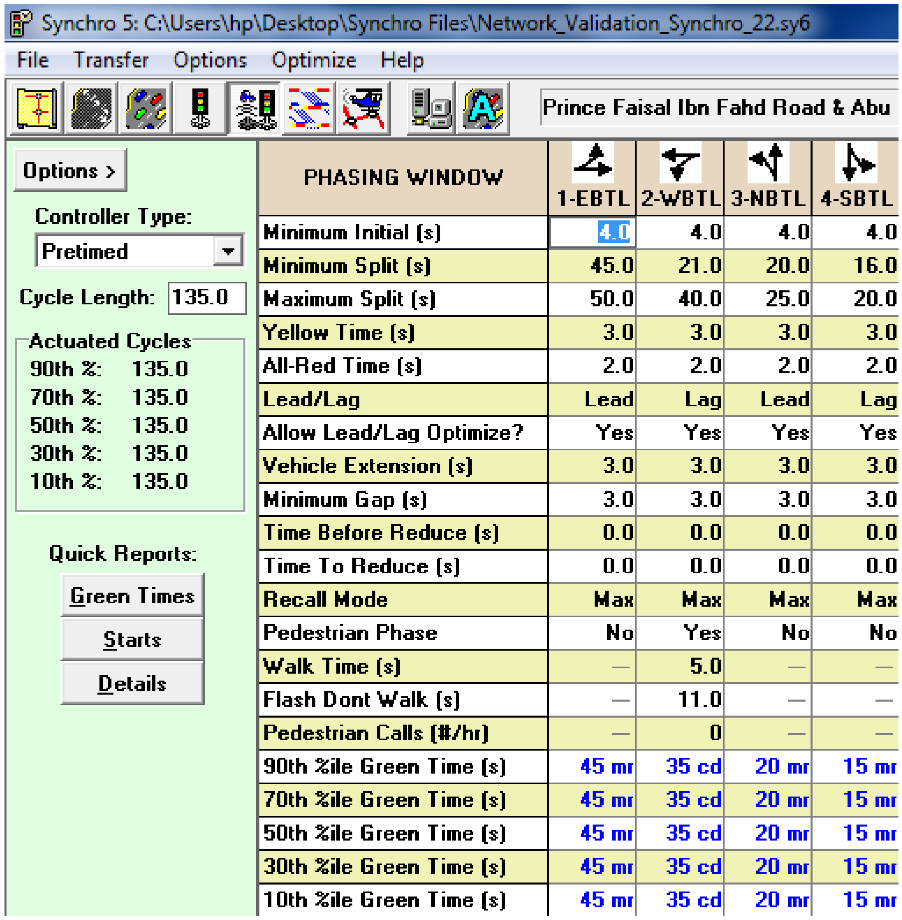
VOLUME WINDOW	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Traffic Volume (vph)	530	1555	50	50	1320	50	50	50	50	73	50	50
Conflicting Peds. (#/hr)	0	—	0	0	—	0	0	—	0	0	—	0
Conflicting Bikes (#/hr)	—	—	0	—	—	0	—	—	0	—	—	0
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Growth Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Bus Blockages (#/hr)	0	0	0	0	0	0	0	0	0	0	0	0
Adj. Parking Lane?	No	No	No	No	No	No	No	No	No	No	No	No
Parking Maneuvers (#/hr)	—	—	—	—	—	—	—	—	—	—	—	—
Traffic from mid-block (%)	—	0	—	—	0	—	—	0	—	—	0	—
Link OD Volumes	—	EB	—	—	WB	—	—	—	—	—	—	—
Adjusted Flow (vph)	576	1690	54	54	1435	54	54	54	54	79	54	54
Lane Group Flow (vph)	576	1744	0	54	1489	0	0	162	0	0	187	0

Figure 5-25 Volume Settings Window in SYNCHRO



TIMING WINDOW	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	PED	HOLD
Lanes and Sharing (#RL)													—	—
Traffic Volume (vph)	530	1555	50	50	1320	50	50	50	50	73	50	50	—	—
Turn Type	Split	—	—	custom	—	—	custom	—	—	custom	—	—	—	—
Protected Phases	1	1	—	2	2	—	3	3	—	4	4	—		
Permitted Phases			—	2		—	3	3	—	4	4	—	—	—
Detector Phases	1	1	—	2	2	—	3	3	—	4	4	—	—	—
Minimum Initial (s)	4.0	4.0	—	4.0	4.0	—	4.0	4.0	—	4.0	4.0	—	—	—
Minimum Split (s)	45.0	45.0	—	21.0	21.0	—	20.0	20.0	—	16.0	16.0	—	—	—
Total Split (s)	50.0	50.0	—	40.0	40.0	—	25.0	25.0	—	20.0	20.0	—	—	—
Yellow Time (s)	3.0	3.0	—	3.0	3.0	—	3.0	3.0	—	3.0	3.0	—	—	—
All-Red Time (s)	2.0	2.0	—	2.0	2.0	—	2.0	2.0	—	2.0	2.0	—	—	—
Lead/Lag	Lead	Lead	—	Lag	Lag	—	Lead	Lead	—	Lag	Lag	—	—	—
Allow Lead/Lag Optimize?	Yes	Yes	—	Yes	Yes	—	Yes	Yes	—	Yes	Yes	—	—	—
Recall Mode	Max	Max	—	Max	Max	—	Max	Max	—	Max	Max	—	—	—
Actuated Effct. Green (s)	46.0	46.0	—	36.0	36.0	—	—	21.1	—	—	16.1	—	—	—
Actuated g/C Ratio	0.34	0.34	—	0.27	0.27	—	—	0.16	—	—	0.12	—	—	—
Volume to Capacity Ratio	0.96	1.01	—	0.11	1.10	—	—	0.21	—	—	0.31	—	—	—
Control Delay (s)	61.5	62.7	—	37.9	96.7	—	—	33.0	—	—	38.6	—	—	—
Level of Service	E	E	—	D	F	—	—	C	—	—	D	—	—	—
Approach Delay (s)	—	62.4	—	—	94.6	—	—	33.0	—	—	38.6	—	—	—
Approach LOS	—	E	—	—	F	—	—	C	—	—	D	—	—	—
Queue Length 50th (ft)	492	~569	—	36	~542	—	—	30	—	—	39	—	—	—
Queue Length 95th (ft)	#729	#684	—	71	#641	—	—	54	—	—	65	—	—	—
Queuing Penalty	239	231	—	0	18	—	—	0	—	—	0	—	—	—
Stops (vph)	535	1667	—	38	1697	—	—	84	—	—	109	—	—	—
Fuel Used (g/hr)	47	143	—	2	77	—	—	11	—	—	10	—	—	—
Dilemma Vehicles (#/hr)	0	0	—	0	0	—	—	0	—	—	0	—	—	—

Figure 5-26 Timing/Signal Settings Window in SYNCHRO



PHASING WINDOW	1-EBTL	2-WBTL	3-NBTL	4-SBTL
Minimum Initial (s)	4.0	4.0	4.0	4.0
Minimum Split (s)	45.0	21.0	20.0	16.0
Maximum Split (s)	50.0	40.0	25.0	20.0
Yellow Time (s)	3.0	3.0	3.0	3.0
All-Red Time (s)	2.0	2.0	2.0	2.0
Lead/Lag	Lead	Lag	Lead	Lag
Allow Lead/Lag Optimize?	Yes	Yes	Yes	Yes
Vehicle Extension (s)	3.0	3.0	3.0	3.0
Minimum Gap (s)	3.0	3.0	3.0	3.0
Time Before Reduce (s)	0.0	0.0	0.0	0.0
Time To Reduce (s)	0.0	0.0	0.0	0.0
Recall Mode	Max	Max	Max	Max
Pedestrian Phase	No	Yes	No	No
Walk Time (s)	—	5.0	—	—
Flash Dont Walk (s)	—	11.0	—	—
Pedestrian Calls (#/hr)	—	0	—	—
90th %ile Green Time (s)	45 mr	35 cd	20 mr	15 mr
70th %ile Green Time (s)	45 mr	35 cd	20 mr	15 mr
50th %ile Green Time (s)	45 mr	35 cd	20 mr	15 mr
30th %ile Green Time (s)	45 mr	35 cd	20 mr	15 mr
10th %ile Green Time (s)	45 mr	35 cd	20 mr	15 mr

Figure 5-27 Phase Settings Window in SYNCHRO

After entering all the required data, the simulation options need to be set. In simulation settings window, the SimTraffic simulation specific information such as taper length, median width, crosswalk width and turning speed were entered. Other information such as traffic volume, storage length, no of storage lanes, lane width were automatically synchronized with SYNCHRO input. Since the turning radii at the studied intersections is large and to improve capacity in SimTraffic, the U-turning speed was set to be 25 km/hr and the left turning speed 45 km/hr to match the prevailing condition. Parameters like link speed, turning speed was adjusted by driver speed factor. Vehicle length in SimTraffic is the bumper to bumper length of a vehicle. SimTraffic assumes a distance between stopped vehicles of 1.5m. The average length of vehicles in meter including the space between them in the network settings in SYNCHRO was set to be 7m. Therefore, the vehicle length in SimTraffic was set as 5.5 m for cars and carpool.

After entering the data properly in SYNCHRO, it should be possible to run SimTraffic without any fatal errors. After loading the file, the network map created in Synchro appeared in the map view. Then network was seeded to have vehicles in the network when simulation begins. After that, the simulation was recorded for animation, reports and statistic graphics. The seeding and simulation recoding durations can be changed. 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 maximum cycle length. The seeding time used in this study was 10 minutes and the simulation recording duration was set to be 60 minutes.

The main objective of using both SYNCHRO and TRANSYT-7F is to optimize Signal timing plans for all the intersections in the network and simulating PARAMICS model using the optimized plans for further analyses. The existing signaling plan was not coordinated for all the intersections in this specific study (Case study-2). Thus the option to find an offset was turned off while optimizing the signal plans. Two (KFUPM and IKEA) out of the three intersections did not warrant all red based on the width of intersection and posted speed, yet it was provided only to conform with the existing signal plan. The optimized Signal Timing Plan is shown in the following page

Table 5-13 Optimized Signal Timing Plan in SYNCHRO

			OBSERVED SIGNAL PLAN					OPTIMIZED SIGNAL PLAN				
Time (hr)	**Intersection #**	**Direction**	**Phase No**	**Red (S)**	**Yellow (S)**	**Green (S)**	**Cycle Length (S)**	**Phase No**	**Red (S)**	**Yellow (S)**	**Green (S)**	**Cycle Length (S)**
8:30 to 9:30 AM	1 (Exit from KFUPM)	NB	4	124	3	15	142	4	131	3	16	150
		EB	1	112	3	25	142	1	110	3	37	150
		SB	3	124	3	15	142	3	103	3	44	150
		WB	2	98	3	41	142	2	114	3	33	150
		All Red			**2**			**All Red**		2		
8:30 to 9:30 AM	2 (Prince Faisal-Abu Ubaidha)	NB	3	112	3	20	135	3	97	3	15	115
		EB	1	87	3	45	135	1	72	3	40	115
		SB	4	117	3	15	135	4	101	3	11	115
		WB	2	97	3	35	135	2	83	3	29	115
		All Red			**2**			**All Red**		2		
8:30 to 9:30 AM	3 (Prince Faisal-King Saud-1)	NB										
		EB	1	107	3	20	130	1	49	3	16	68
		SB	2	107	3	20	130	2	57	3	8	68
		WB	3	52	3	75	130	3	36	3	29	68
		All Red			**2**			**All Red**		2		
8:30 to 9:30 AM	3 (Prince Faisal-King Saud-2)	NB	3	107	3	20	130	3	57	3	8	68
		EB	1	82	3	45	130	1	36	3	29	68
		SB										
		WB	2	77	3	50	130	2	49	3	16	68
		All Red			**2**			**All Red**		2		

5.3.4 Comparison of Queue Length

Finally, a comparison of Queue Length (QL) was made among the simulated queue length in PARAMICS with existing signal plan, QL developed in PARAMICS using SYNCHRO optimized Plan and TRANSYT-7F optimized plan. The following figure 5-28 depicts the comparison

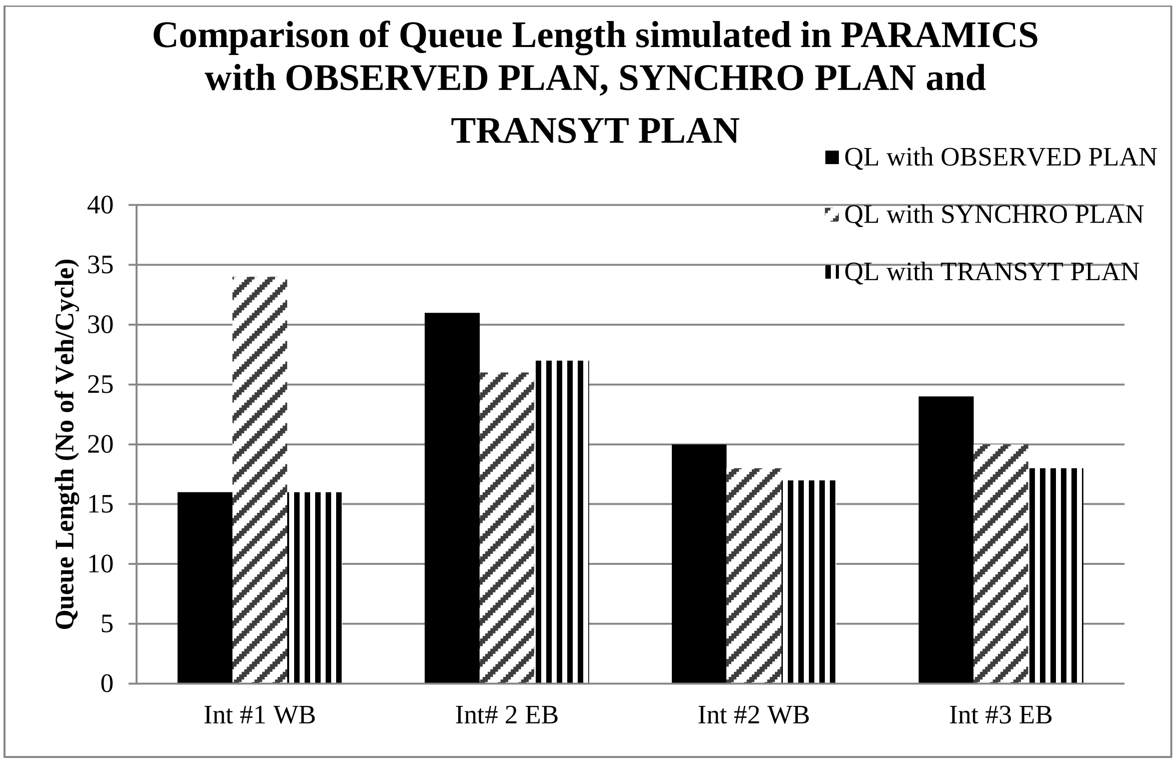
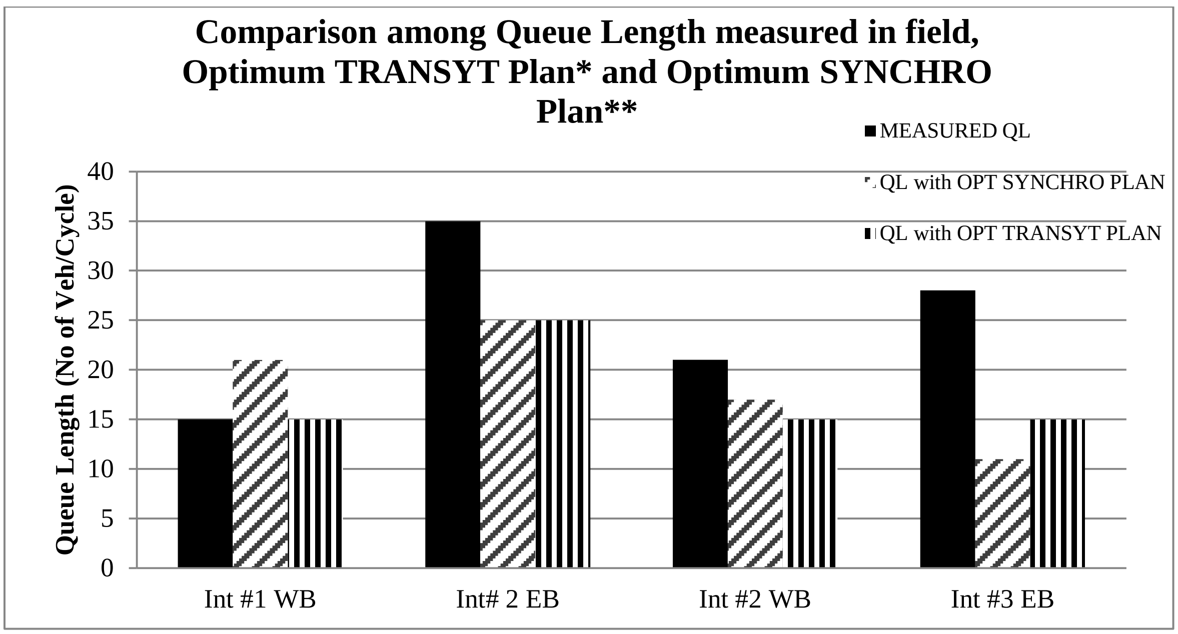


Figure 5-28 Comparison of Queue Length simulated with different signal timing plan in PARAMICS

PARAMICS simulation output using the TRANSYT optimized plan produced the minimum Queue Length for the intersections (See Figure 4-6 for intersection numbers). The comparison reasonably attains its validity as PARAMICS simulation was considered to be the common yardstick. In most cases the queue length with optimized signal timing in TRANSYT-7F was below the observed queue length with existing signal plan, which indicates a better traffic system has been achieved with more vehicles passing the stop line without being stopped. The SYNCHRO optimized timing plan performed better than TRANSYT-7F plan only in the third intersection. The third intersection was in fact a diamond interchange and TRANSYT-7F and SYNCHRO have different method of modeling it.

The optimized signal plans developed by both TRANSYT-7F and SYNCHRO was made to run in their respective simulation program and the Queue Length from the simulated outputs were compared with the infield measured Queue Length. Figure 5-29 below shows the comparison



* Simulated by TRANSYT-7F and ** Simulated by SimTraffic

Figure 5-29 Comparison of Queue Length with respective optimized plan in TRANSYT-7F and SYNCHRO/SimTraffic

Whilst comparing figure 5-28 and figure 5-29, it can be seen that both TRANSTY-7F and SimTraffic produced slightly better results when they are simulated with their respective optimized signal plans (Refer to figure 4-6 for intersection numbers).

5.3.5 Summary of Results

The results from this study are summarized below:

- By five iterations it was found to have GEH (A specific distribution used for traffic volume comparison) value less than 5 (which was the initial target) for most of the turning movements when constructing the Origin Destination (OD) matrix in PARAMICS.
- To enter traffic demand in PARAMICS an OD matrix can be derived using two different methods, namely a statistical fitting method and a stochastic assignment method. The statistical method was found to fit the model best for this specific study.
- When travel time was considered only as the objective Measure of Effectiveness, a domain of 0.5 to 0.7 second for Mean Target Headway (MHT) and 0.5 to 0.6 seconds for Mean Reaction Time (MRT) produced the closest match with the observed field data. On the other hand when Queue Length is solely considered, a combination of 0.5 seconds for MTH and 0.5 seconds for MRT produced closest fit.
- With regards to the Mean Target Headway and Mean Reaction Time, it was found that the parameter settings are much lower than the default setting of PARAMICS model, which is based on driving behaviour in the United Kingdom. The Final calibrated value of MTH and MRT was 0.53 and 0.50 seconds respectively.

- When the model was calibrated in terms of travel time or queue length only, the simulated model performed better but when both the Measure of Effectiveness (MOEs) were considered a compromise in accuracies between the two MOEs was accomplished in order to find a reasonable fit of the simulated and observed field data.
- The optimized signal timing plan produced in TRANSYT-7F performed better then the signal plan of SYNCHRO when both of them were simulated in PARAMICS.
- The optimized signal timing plan developed in TRANSYT-7F and SYNCHRO was used to simulate in their respective simulation program and showed slightly better results in terms of queue length comparison with the infield measured value.

CHAPTER 6

CONCLUSION AND RECOMMENDATION

6.1 CONCLUSION

This chapter contains general conclusions, recommendation and suggestions for further research. In general, the usefulness and weakness of this research has been discussed in conclusions, some valuable experiences in the process of calibrating PARAMICS model has been depicted in the recommendation. Suggestions for future research reflect some of the issues encountered during this analysis, which may complement this work.

The study detailed the calibration and validation efforts involving two separate urban arterial networks analyses using the PARAMICS microsimulation model. The efforts included comparison of flows at selected links and intersections as well as comparison of travel times along major streets and queue lengths at intersections within the study network. Specific benchmarks were set to guide the calibration effort in order to achieve results that corresponds to the observed data to an acceptable level of confidence. It was found that in most cases, the targeted benchmarks were achieved with moderate to high modeling efforts.

The main conclusions and findings of this study are summarized in the following points:

- A review of published literature considering the pros and cons, characteristics and uses of various transport related microsimulation packages showed that there is no one particular package that can be termed as the best overall. The choice of package depends on the function required.
- The PARAMICS model is developed on UK driving behaviour. The UK driving behaviour and driving conditions in Saudi Arabia can be found very different to those of the European countries. This statement is deduced from the fact that the default values of few of the model parameters needed a change in order to match the observed field data. A final calibrated value of Mean Target Headway of 0.53 second suggest that the Saudi drivers tend to leave a shorter distance between the preceding and following vehicle compared to the UK drivers with a shorter time to react to any change of speed of the preceding vehicle .
- Optimal signal timing plan resulted by TRANSYT-7F improves the system performance more than the optimal signal timing plan resulted by SYNCHRO when both plans were compared using PARAMICS simulation. Minimizing delay was the objective function in optimizing signal plan for TRANSYT-7F and SYNCHRO. Since both the models are deterministic, the difference in optimized plan can only be attributed to the core models.
- Microsimulation software packages need to produce useful information that allows the user to calibrate models more efficiently and logically. The time expended on the analysis of parameters for the calibration of the model was more than expected because the output information from PARAMICS had to be processed significantly in order to produce the graphs and tables. Some tables and graphs were created from

large output files that record every single event on the network or details of trips for every single vehicle. This complicated the output data processing and required much more time than was initially anticipated.

Few of the limitations of this study and PARAMICS model are mentioned below:

- As PARAMICS was developed maintaining European standard, it lacks some important functions such as modeling of turning bays and sign controls, and its vehicle and driver attributes needed to be carefully tuned to achieve reasonable performance.
- This study used two Measure of Effectiveness (MOEs), travel time and queue length for model calibration, the performance of other MOEs were not examined. Therefore, it does not guarantee that other MOE's from the modeled output would necessarily fit the field observed value.

6.2 RECOMMENDATION

The following recommendations are made based on the study conducted

- This study used only two MOEs, travel time and queue length for model calibration. Further research is recommended to include more MOEs in the calibration process.
- Calibrating the PARAMICS model by only calibrating the driving behaviour parameters, namely Mean Target Headway and Mean Reaction Time was sufficient for this specific study comprised of a relatively smaller network. If the size of the network is increased then considering only these two parameters may not suffice the calibration requirement. Additional parameters like driver aggressiveness, familiarity with the network may prove to be useful in that case.

- However, it would be a challenging task if more than two parameters are selected for calibration and the calibration is done only on trial and error basis. For some MOEs, such as delay and queue, it should be carefully noted that the method of recoding MOEs varies from one model to another. Therefore, comparison can only be done when the models record the MOEs in a similar format.

6.3 FUTURE RESEARCH ISSUE

A number of issues have been identified during this research which warrants further investigation. These include the following:

- In this study network there were no roundabouts, interchanges and complex geometries. Even in the signal plan, permissible phases and pedestrian phases were absent. Inclusion of such features may prove to be potential for future research.
- Awareness and Familiarity of drivers can be added as calibrating parameters to better reproduce the observed field data.
- Understanding the effects of different traffic composition that may include mixed traffic and pedestrian crossing.
- Understanding the effects of severe congestion in the release of vehicles into the network could be a useful research that may lead to recommendations about the number, location, and set up of zones. The release of vehicles into the network can also be controlled by profile matrix with specific intervals. Future researcher may wish to investigate the impact of shorter vehicle release intervals on the network system performance than the intervals used in this specific study.

REFERENCES

[1] Aldazaba, A.A., Calibration of PARAMICS microscopic simulation model. M.S. Thesis, University of Calgary, Alberta, Canada, 2004.

[2] Ahmed, S.A., Calibration of VISSIM to the Traffic Conditions of Khobar and Dammam, Saudi Arabia. M.S. Thesis, King Fahd University of Petroleum & Minerals, Saudi Arabia, 2005.

[3] Alexiadis, V., Jeannotte K., and Chandra A., Traffic Analysis Toolbox: Volume I:vTraffic Analysis Tools Primer. Publication FHWA-HRT-04-038. FHWA, U.S. Department of Transportation, 2004.

[4] Al-Gadhi, S.A.H, Microscopic modeling and simulation of Hajj vehicular traffic during Ifadha. The fifth Saudi Engineering Conference, Volume 3, Civil Engineering, March, 1999.

[5] Al-Ghamdi, A.S., Analysis of time headways on urban roads: case study from Riyadh. Journal of Transportation Engineering /july/august 2001 / 289

[6] Al-Ofi K.A., The Effect of Signal Coordination on Intersection Safety, Ph.D. Dissertation, King Fahd University of Petroleum and Minerals, Saudi Arabia, 1994.

[7] Al-Saif, A., et al., Investigation of the causes of traffic accidents in Makkah and eastern regions and their countermeasures. King Abdulaziz City of Science and Technology, Riyadh, Saudi Arabia, 1990.

[8] Al-Ghamdi, A.S., A statistical comparison between severe accidents and PDO accidents in Riyadh. Transactions on the Built Environment vol 33, 1998 WIT Press, www.witpress.com, ISSN 1743-3509.

[9] Bendak, S., An in-depth analysis of red light crossing problem in Saudi Arabia. Advances in Transportation Studies an international Journal Section B 25, 2011

[10] Barcelo, J. et al., Microscopic traffic simulation: A tool for the design, analysis and evaluation of intelligent transport systems. Journal of Intelligent and Robotic Systems, Volume 41, Numbers 2-3, January 2005, pp. 173-203(31)

[11] Bertini, R. L., Lindgren, R., and Tantiyanugulchai, S., Application of PARAMICS Simulation At a Diamond Interchange. A research report submitted to the Oregon Department of Transportation, USA, April, 2002.

[12] Bloomberg, L., and Dale, J., A Comparison of the VISSIM and CORSIM Traffic Simulation Models on a Congested Network, 2000.

[13] Boxill, S. A., and Yu, L., An Evaluation of Traffic Simulation Models for Supporting ITS Development, Center for Transportation Training and Research, Texas Southern University, 2000.

[14] Breiman, R. L., Goodwin, D., and Bailey, B., The statistical process of freeway traffic.'' Transp. Res., 11(1), 221–228, 1977.

[15] Chu, L., Liu, H.X., Oh, J., and Recker, W., A Calibration Procedure for Microscopic Traffic Simulation. Presented at the 83rd Annual Meeting of the Transportation Research Board, Washington D.C., January 2004.

[16] Contram. http://www.contram.com/. Accessed June, 2013.

[17] CORFLO. McTrans, University of Florida, Gainesville. http://mctrans.ce.ufl.edu/store/description.asp?itemID=441. Accessed February 15, 2007.

[18] D-H Lee, Yang, X., and Chandrasekar, P., Parameter calibration for PARAMICS using Genetic Algorithm. 80th Annual Meeting Preprint CD-ROM, Transportation Research Board, Washington, D.C., January 7-11, 2001.

[19] Druitt, S., An Introduction to Microsimulation. In Traffic Engineering and Control, Vol. 39, No 9. 1998.

[20] DYNASMART-P. McTrans, University of Florida, Gainesville. http://mctrans.ce.ufl.edu/featured/dynasmart/. Accessed March 8, 2007.

[21] Figueiredo, L., Machado, J. A. T., and Ferreira, J. R., Dynamical Analysis of Freeway Traffic. IEEE Transactions on Intelligent Transportation Systems, Volume 5, Issue 4. Institute of Electrical and Electronics Engineers, Parma, Italy, 2004.

[22] Fritzsche, H-T., Daimler-Benz, AG., A model for traffic simulation. Traffic Engineering and Control, Vol. 35, No. 5, 1994, pp. 317-321.

[23] Gao, Y., Calibration and comparison of VISSIM and Integration microscopic traffic simulation model, M.Sc Dissertation, Virginia Polytechnic Institute and state university, USA, 2008.

[24] Gardes, Y., May, D.M., Dahlgren, J., and Skabardonis, A., Freeway Calibration and Application of the PARAMICS Model. Transportation Research Board 81st Annual Meeting Pre-Print CD, 2002.

[25] Halati, A., Lieu, H., and Walker, S., CORSIM-Corridor Traffic Simulation Model in 76th Transportation Research Board Meeting, Washington D.C., 1997.

[26] Highway Capacity Manual 2000. Transportation Research Board. Washington, D.C., 2000.

[27] Jiménez, T., et al., A road traffic simulator: Car-following and lane-changing. Proceedings of the 14th European Simulation Multiconference on Simulation and Modelling: Enablers for a Better Quality of Life, 2004, pp 241-245

[28] Jobanputra, R., and Vanderschuren, M., Calibration and Validation of a Microsimulation model for a local arterial in Cape Town. Proceedings of Southern African Transport Conference (SATC), Pretoria, South Africa, 2012.

[29] Joines, J. A., Barton, R. R., and Kang, K., and Fishwick, P. A., eds. ITT Industries, Dept. of Intelligent Transportation Systems; National University of Singapore, Dept. of Electrical Engineering, Singapore. 2000, pp. 1143-1147.

[30] Kan. S., and Bhan, G., Evaluation of microscopic lane change models using NGSIM data. Proceedings of the 18th conference on Proceedings of the 18th IASTED International Conference: modeling and simulation, 2007, pp: 592 – 597

[31] Kwon, E., and Michalopoulos., P., Macroscopic Simulations of Traffic Flows in Complex Freeway Segments on a Personal Computer. Institute of Electrical and Electronics Engineers, Minneapolis, MN. http://ieeexplore.ieee.org/iel3/3948/11424/00518859.pdf?arnumber=518859. Accessed February 15, 2013.

[32] Lee, D-H., Yang, X., and Chandrasekar, P., Parameter calibration for PARAMICS using Genetic Algorithm. 80th Annual Meeting Preprint CD-ROM, Transportation Research Board, Washington, D.C., January 7-11, 2001.

[33] Lee, J., and Ozbay, K., Calibration of a Macroscopic Traffic Simulation Model Using Enhanced Simultaneous Perturbation Stochastic Approximation Methodology. Presented in Transportation Research Board's 87th Annual Meeting, Washington, D.C., 2008

[34] Li, Z., Liu, H., and Li, J., A Calibration and Validation Procedure for Microscopic Simualtion Model, 13th International IEEE Annual Conference on Intelligent Transportation Systems Madeira Island, Portugal, September 19-22, 2010.

[35] Li, Z., Hao, L., and Li, J., Calibration and Validation Procedure for Microscopic Simualtion Model , Annual Conference on Intelligent Transportation Systems Madeira Island, Portugal, September 19-22, 2010.

[36] Ma, T., and Abdulhai, B., Genetic Algorithm-Based Optimization Approach and Generic Tool for Calibrating Traffic Microscopic Simulation Parameters. Transportation Research Record: Journal of Transportation Research Board, No.1800, 2002, pp.6-15.

[37] May, A.D., Traffic Flow Fundamentals. Prentice Hall: Englewood Cliff, New Jersey, 1990.

[38] Moen, B., Fitts, J., Carter, D., and Ouyang, Y., A comparison of the VISSIM Model to Other Widely Used Traffic Simulation and Analysis Programs. Presented at the Institute of Transportation Engineers 2000 Annual Meeting and Exhibit, Nashville, TN, 2000.

[39] Oketch, T., and Carrick, M., Calibration and Validation of a Micro-Simulation Model in Network Analysis, Presented at the Transportation Research Board Annual Meeting, Washington DC, 2005.

[40] Olstam, J., and Tapini, A., Comparison of Car-Following Models, in VTI Publication M960A. Swedish National Road and Transport Research Institute: Linköping, Sweden, 2004.

[41] Owen, L.E., Zhang, Y., Rao, L., and McHale, G., Street and traffic simulation: traffic flow simulation using CORSIM, in Proceedings, Winter Simulation Conference, 2000, pp.1143-1147.

[42] Ozbay, K., Mudigonda, S., and Bartin, B., Development of a Simulation Model of an ITS Corridor, Final Report, NJDOT research project, New Jersey,USA. 2003, pp. 24

[43] Ozbay, K., and Bartin, B., Microscopic simulation and calibration of an integrated freeway and toll plaza model, Transportation Research Board 85th Annual Meeting, 2005.

[44] Panwai, S., and Dia, H., Comparative evaluation of microscopic car-following behavior, IEEE Transactions on Intelligent Transportation Systems, Vol. 6, No. 3, 2005.

[45] Paramics V5.0 Modeller Reference Manual, version 1.0, Quadstone Ltd., 2004.

[46] Paramics V5.0 Programmer Reference Manual, version 1.0, Quadstone Ltd., 2004.

[47] Park, B.B., and Schneeberger, J.D. Microscopic Simulation Model Calibration and Validation. Case Study of VISSIM Simulation Model for a Coordinated Actuated Signal System. Transportation Research Record 1856, Paper no. 03-2531, 2005.

[48] Prevedouros, P., Wang, Y., Simulation of a large freeway/arterial network with CORSIM, INTEGRATION and WATSim, presented at 78 TRB meeting, 1998.

[49] Quadstone Paramics (2011), [Online] Quadstone Paramics [Accessed September 2013].

[50] Rakha H., and Zhang Y., The INTEGRATION 2.30 Framework for Modeling Lane-Changing Behavior in Weaving Sections, Transportation Research Record: Journal of the Transportation Research Board, No. 1883, 2004, pp. 140-149.

[51] Ratrout, N.T., and Olba, M.A., Adequacy of TRANSYT-7F and Synchro Models Along a Major Arterial in Saudi Arabia, Canadian Journal of Civil Engineering, Vol. 36, 2009, pp. 95-102.

[52] Ratrout, N.T., Calibration of TRANSYT Platoon Dispersion Algorithm for Conditions Along Major Arterials in the Cities of Dammam & Al-Khobar, Saudi Arabia, AJSE, Vol. 21, No. 4A, October, 1996.

[53] SATURN. WS Atkins. http://www.saturnsoftware.co.uk/7.html. Accessed June, 2013.

[54] SMARTEST project deliverable D3, Review of Micro-simulation Models. Institute for Transport Studies, University of Leeds. www.its.leeds.ac.uklsmartest, 1997.

[55] Stewart, P., M8 PARAMICS RAMP METERING ASSESSMENT, A report submitted to the Scottish Executive Development Department, August, 2001.

[56] Sundaram, S., Development of a Dynamic Traffic Assignment System for Short-Term Planning Applications. Master's Thesis, Massachusetts Institute of Technology, Cambridge, 2002.

[57] TSS-Transport Simulation System, AIMSUN 5.1 Microsimulator User's Manual, 2006.

[58] UK Highways Agency, "UK design manual for roads and bridges," London, UK, 1996.

[59] Van Aerde, M., and Rakha, H., INTEGRATION © Release 2.30 for Windows: User's Guide Volume I: Fundamental Model Features. M. Van Aerde & Assoc. Ltd., Blacksburg, 2007.

[60] Vanderschuren, M.J., Intelligent transport systems for South Africa, Impact assessment through Microscopic Simulation in the South African context, PhD dissertation, University of Twente, Ensched, The Netherland, 2006.

[61] Van Vliet, D., and Hall, M., SATURN 10.3 user Manual. The institute for transport studies, The University of Leeds.

[62] Xiao, et al., Methodology for Selecting Microscopic Simulators: Comparative Evaluation of AIMSUN and VISSIM, University of Minnesota, Center for Transportation Studies, 2004.

[63] Yuhao, W., and Panos, D., Prevedourous, Synopsis of traffic simulation models. University of Hawaii. 1996.

[64] Zhe, L., Hao, L., and Ke, Z., Calibration and Validation of PARAMICS for Freeway Using Toll Data, Proceedings of the 12th International IEEE Conference on Intelligent Transportation Systems, St. Louis, MO, USA, October 3-7, 2009.

VITAE

Name : Imran Reza

Nationality : Bangladeshi

Date of Birth : 27th July, 1980

Email : reza.imran@gmail.com

Degrees:

Bachelor of Science in Civil and Environmental Engineering from Shahjalal University of Science & Technology, Sylhet, Bangladesh (March, 2005).

Master of Engineering in Water Resources Engineering from Bangladesh University of Engineering & Technology, Dhaka, Bangladesh (September, 2010).

Master of Science in Civil and Environmental Engineering from King Fahd University of Petroleum and Minerals, Dhahran, K.S.A. (October, 2013)

Research Experience:

- Alam, J.B., Chowdhury, R.K. and **Reza, I.**, "Adsorption dynamics of Cobalt [Co(ii)] on rubber granules" International Journal of Environment and Waste Management, Inderscience Enterprises Ltd., 2010.
- Alam, J.B., Dikshit, A.K., Banerjee, M., **Reza, I.,** and Rahman, M.A., "Study of sorption of 2,4-D on outer Peristaltic Part of Waste Tire Rubber Granules", Journal of Dispersion Science and Technology, 27:843–849, 2006.
- **Reza, I.**, Mahbub, A.Z. and Rahman, M.A., "Risk assessment and management of effluent from a textile industry at Narayanganj, Bangladesh"

published in international conference on "Management of Water, Wastewater and Environment: Challenges for the developing countries" held at Kathmandu, Nepal, September 13-15, 2006.

- Alam, J.B., **Reza, I.**, Mahbub, A.Z. and Islam, S., "A study on water quality parameters of Surma river" published in international conference proceedings on "Management of Water, Wastewater and Environment: Challenges for the developing countries" held at Kathmandu, Nepal, September 13-15, 2006.
- Ahmed, N., Khan, Z.H., **Reza, I.**, Mahbub, A.Z. and Alam, J.B., "Effectiveness of ISO-14000- a case study" published in international conference proceedings on "Environment and Development" held at Kolkata, West Bengal, India, December, 2003.