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REAL-TIME TRAFFIC SIGNAL CONTROL FOR MAXIMIZING NETWORK THROUGHPUT

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A Thesis Submitted in Partial Fulfillment of the Requirements

for the Degree of Master of Engineering Program in Civil Engineering

Department of Civil Engineering

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การศึกษามุ่งหมายที่จะพัฒนาวิธีการควบคุมระบบสัญญาณไฟจราจรแบบที่ปรับเปลี่ยนได้ตามสถานการณ์เพื่อจัดการสภาพการจราจรหนาแน่นได้อย่างมีประสิทธิภาพโดยที่ไม่เพียงพิจารณาผลกระทบในปัจจุบันต่อพื้นที่ใกล้เคียงแต่ยังได้พิจารณาผลซึ่งส่งไปถึงทั้งระบบในอนาคตอันใกล้และทดสอบผลลัพธ์ของวิธีควบคุมนี้กับผลที่ได้จากวิธีการควบคุมแบบอื่น วิธีการควบคุมนี้จะเน้นการทำให้มีอัตราการผ่านระบบโดยรวมสูงสุด ขอบเขตการวิจัยนั้นครอบคลุมสองส่วนอันได้แก่การพัฒนาวิธีการควบคุมและการทดสอบวิธีควบคุมกับระดับการจราจรที่แตกต่างกัน วิธีการควบคุมนี้ได้รับการพัฒนาขึ้นและถูกทดสอบบนโครงข่ายเส้นทางหลัก (corridor) ที่มีแยกสัญญาณไฟ 4 แห่งและได้ทำการทดสอบกับสภาพการเดินทางคล้ายคลึงกับการเข้ามาสู่พื้นที่ของการจราจรในช่วงเช้าโดยทดสอบบน 2 กรณีศึกษาของสภาพการจราจรในแต่ละกรณีศึกษามีระดับการจราจร 5 ระดับตั้งแต่รถเบาบางจนถึงรถหนาแน่นมาก ซึ่งกรณีศึกษาเหล่านี้ถูกจำลองขึ้นในคอมพิวเตอร์เพื่อเปรียบเทียบกับการควบคุมสัญญาณไฟแบบเวลาคงที่ทั้งที่มีและไม่มีการป้องกันการสูญเสียช่วงเวลาไฟเขียว โดยเครื่องมือที่ใช้ในการทดสอบโปรแกรมจำลองสถานการณ์ระดับจุลภาคชื่อ Paramics ผลการทดสอบได้ถูกนำมาพิจารณาโดยใช้ตัวชี้วัดได้แก่ ปริมาณรถในระบบ ช่วงเวลาไฟเขียวเฉลี่ยที่แต่ละขาได้ในชั่วโมงเร่งด่วน การถึงที่หมายของยวดยาน อัตราการผ่านโครงข่ายในแต่ละเวลา ความเร็วเฉลี่ยของยวดยานในแต่ละเวลา เวลาในการเดินทางรวม และอัตราการผ่านระบบสูงสุดที่เกิดขึ้น ผลการทดสอบนั้นแสดงให้เห็นว่าวิธีการควบคุมดังกล่าวนั้นให้อัตราการผ่านระบบโดยรวมสูงมากในกรณีที่มีความต้องการเดินทางแบบเกินอิ่มตัว และยังให้ค่าที่ค่อนข้างดีในกรณีของต้องการเดินทางแบบอิ่มตัว แต่วิธีควบคุมแบบนี้กลับไม่มีประสิทธิภาพในกรณีที่มีความต้องการน้อยในระดับที่ไม่อิ่มตัวเทียบกับความสามารถในการรองรับประมาณการจราจรของระบบโดยรวม

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สาขาวิชา วิศวกรรมโยธา.................... ลายมือชื่ออาจารย์ที่ปรึกษา.......สรวิศ นฤปิติ.............
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The study aims to develop an actuated traffic-signal-control method that has ability to efficiently manage traffic, especially oversaturation, by considering not only present situation in only small area but also considering the effect to the whole network in nearly future and to compare effectiveness with some other methods. This method focuses on maximizing overall throughput of the system. The scope of study covers two parts which are developing and testing the method in some different levels of traffic. The method is developed and it is tested on a computer-simulated corridor with four intersections compared to fixed-time method with and without prevention of green-time loss. The tool used in the test is a micro-simulation program, named Paramics. The results, which are number of vehicles containing in the system, average green time given to each link during a peak hour, destination arrival, real-time throughput, real-time average speed in the system, total travel time, and peak throughput, are statistically considered. The results show that the method really gives very high total throughput to the system in the condition of having oversaturated demand. It also works well with the saturated one but this method works inefficiently for the case of having too low demand that is considered to be less than theoretical capacity of the system.

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CHAPTER I

INTRODUCTION

1.1 General

Recently, cost of fuel in the world market seems not to be so stable, caused by the truth that it is one of necessities of life for modern society and it is valued as a scarce and depleting resource. The increase in demand of energy in households, industries, and transportation is gradual while cost of energy increases drastically. For industrial side, production of goods and transport activities can be considered as cost of risk. This cost is very important in setting the price of each product. If the total cost including the cost of risk is high, the competition ability in the global market will decrease. For passenger travel, the increase in transport cost hampers the ease of travel in many ways. The travelers may limit their travel according to transport expenditure. Travel decision, frequency, destination, mode, and route would be affected by the increasing cost. The impacts of the increasing fuel price on the industrial goods transport and passenger travel are also correlated, and always lead to the pressure to optimize the travel. The aim of transport of both passengers and freights is the same; to reduce the cost of travel (and thus to reduce the amount of energy consumed). This is also the goal for sustainable transportation.

A large city normally experiences traffic congestion, especially during peak hours. The congestion does not only mean increasing travel time, but also the waste of fuel amount per vehicle-distance of travel compared to is the amount consumed by similar travel during non-congested period. If the congestion is efficiently controlled, the result on the decrease in fuel consumption can be expected.

Since the congestion causes excessive fuel-consumption and travel time, this implies that the congestion creates many negative effects to both economic and social sides. For economics, the time wasted in traveling can be used as economic capital for doing other valuable activities that give profit to travelers themselves and also their companies/society. Because the large amount of time is lost in traveling, travelers may have less time to be with their families. It causes many social problems, such as psychological and family-relationship problem, resulting in worse life qualities.

Congestion, especially in a large city, also creates pollution. Smoke and noise, which are produced by the traffic in a road network, can disturb both road users and city residents, especially for those who live near congested streets. In a congested area, pollution can reach a level that is much higher than the standard level. This pollution has an ability to harm both mental and physical conditions of the population, thus decreasing the overall welfare of the society.

From these problems caused by traffic congestion, it is eminent that solving a congestion problem, especially in a crowded large city with too much travel demand to be served by a limited capacity of the road network, is a necessary the solution should be sought as fast as possible. The solutions of this complex problem are grouped into two main practices according to the theory of demand and supply. The first part is to increase the supply or to improve serviceability of the road network. The other one is to control the demand under the capacity.

To solve the congestion by increasing an ability to serve excessive amount of users is sometimes costly and inefficient. One critical argument is that the improvement of system serviceability (capacity) attracts more demand for using the system. Also, this approach creates unawareness of users in the travel optimization. An example is the decision on when to travel and where to drive. If a new link is added into the system, most drivers tend to choose the route and departure time that give them the maximum utility without considering the spillover to overall system (or effects to others drivers). It causes unreliable ability of controlling the congestion and, sometimes, it is possible that the new link can worsen the problem by itself. The improvement normally requires a large budget to pay for labor, materials, and space to order to improve infrastructures. Another disadvantage of improving physical structures is that the project takes time for approval and much time for construction. From these disadvantages, this solution is appropriate only for a long-term project.

The changing or improving traffic control of an existing road system normally requires smaller budget for investment because it needs only the budget for developing or improving and adapting new effective method with a minimal change in road structures.

Focusing on controlling demand of road-network users, it includes the reduction in road-usage demand, the demand allocation to other modes of travel, and the distribution of the demand to appropriate points of time and parts of the system (route). The demand management and control is expected to improve the overall service of the network. Road-travel-demand control, especially demand reduction, seems to be effective but to change or reduce in demand in the peak period, which is the period that people make most of home-base-work and home-base-school trips, can be very difficult as these trips are mandatory to economical and educational activities. For the case of reducing road-travel demand by using travel cost as a barrier of using private vehicles may not be a good solution in several situations. An example of impracticality of using a cost barrier is the permission for too many vehicles to enter an expressway. This will reduce a moving rate on the link that is connected to the effect is extended to other exits of this expressway and many other links in non-expressway system. The last two ways for controlling the demand are to use other modes of travel and to cancel the trips. Some demand may decrease from travel mode shift and trip canceling but it also again represents the reduction in expected social activities, as the road network is not fully utilized.

Although an optimal solution to solve traffic-congestion problem for a city should be a proper combination of both demand-based and supply-based solutions, the option on better traffic signal control for network traffic management becomes one of effective solutions for the pressing congestion. A good signal control does not only increase ability of the network to transfer vehicles but also manages and distributes travel demand to some appropriate time and location. Road network in a large city normally consists of many road links forming a network and many intersections. They are all directly or indirectly connected to one another with different levels of influence due to locations and types of connections between links in the network, surrounding area, geometry of the roads, and other related factors. From this connection, traffic signal control at a point of time (and location) will affect future condition of the traffic in the whole network. From this reason, traffic signal control considering only a local situation at a moment is not sufficient to reach the real optimum. To optimize the system in all dimensions, place and time, traffic signal system should be dynamically controlled considering the effects to both individual link(s) and overall road network, both demand and supply of the system, and both present and predicted future by selecting appropriate of control setting that yields better control effectiveness.

The signal control method aforementioned is very interesting to investigate further into details. Many past and recent researches have been conducted during the past decades. The development of the signal control techniques starts with a consideration at a specific point in the network (isolated intersection) then extending the scope onto network consideration. Another advance topic to be studied is overall-network optimization over moving time and position which is much more complicated and very difficult to be manually calculated. In the past, it is almost impossible to do it efficiently. However, with fast development of computer and other information technologies, it becomes challenging for many researchers to study and evaluate the benefit of these developed control methods. Thus, it is reasonable to apply an advanced traffic analysis technique, such as traffic simulation, for this consideration. Traffic signal control method can be tested using the traffic simulation test bed and the results can be unbiasedly compared with the existing and other control methods.

This research mainly focuses on managing peak-period traffic in a large congested road network, especially in the morning peak period of weekdays with value of time normally higher than other peaks, using an appropriate traffic-control method. The concept of maximizing throughput or rate of removing vehicles out of the system is used with an assumption that the drivers have choices of delaying their departure time from home or waiting (in a car) in the network. The wasted time is valued equally. The proposed technique for maximizing throughput also causes the minimum level of total delay which is the basic goal of most traffic-system optimization.

To meet objective function, an algorithm for setting the traffic signal is proposed. Travel time of each links will be predicted by using the historical data with real-time adjustment. This history-based prediction should be effective in the case of having enough historical data from each step of control-method change. Each change should cover only a small range of time in order to get enough undisturbed historical data. To test this method of control, a simplified corridor will be simulated in the computer to find the efficiency of using it compared to some other selected methods. Since a corridor gives users no alternative route choice for travel from each origin to each destination, the route of each user may not be necessary to be considered. For the demand of users, its distribution along the time will be assumed as fixed because, on a weekday, the majority of demand comes from users that make trips to their workplaces or schools.

1.2 Objective

The main objectives of study are listed below:

1) To develop an appropriate method of traffic-signal control for managing the congestion during a morning-peak period in a crowded corridor, with the highest expected value of time in a day, to be used effectively in the changing demand situation.
2) To develop a test framework for testing traffic signal control using microsimulation.
3) To compare the results of the developed method to the results of other selected methods which may already be developed, tested, and adapted to be used in other researches or even the real situation.

1.3 Scope of Work

The scope of study will cover two main parts: developing the method of controlling signalized corridor and testing it. Firstly, a traffic-signal-control method for a morning-peak period of a weekday in a crowded city will be developed with help of queuing theory and knowledge in statistics for maximizing the expected throughput. Then, the developed method will be tested by using a computer-simulated road network and compared to the results of using some other selected methods to get some advantages and disadvantages of the usage. The test will be run with assumed fixed paths and fixed demands in some levels.

1.4 Procedure

The following is a sequence of research steps to be done in order to reduce complication and confusion of the study.

1) Study past knowledge, viewpoint, and some researches, which are directly related to traffic-signal control and indirectly related to the study such as basic of simulation, mathematics, statistics, decision making, computer programming, traffic engineering, and economics.

2) Roughly draw concepts, methods, theories, and possible problems for the study of throughput-maximization-based signal control and find the solution of those problems or adjust the method of controlling and testing to be more practical.

3) Study in details about how to use some computer programs, both fully-developed application programs and ones that require additional codes to increase their serviceability, to select the most appropriate computer program that is expected to be easy, fast, accurate, and convenient enough for running the simulation. The chosen simulation is Paramics with the addition of development of Application Interface Programming (API) capability.

4) Re-examine concepts, methods, theories, and possible problems details with studied knowledge of programming and develop the way to adapt the selected computer program to be efficient in simulating the problem. This includes the development of program code on API to do such desirable tasks, for instance to collect real-time data and the calculation of the developed signal control method.

5) Code a road network in a computer and also code the program in order to detect real-time condition and control the signals.

6) Run the simulation and collect some interesting results from each simulation run.

7) Analyze and compare some important results of simulation from both the developed method and other selected methods.

8) Make a conclusion and discussion for the result including and list some comments and suggestions.

9) Complete whole research papers.

1.5 Expected Benefit

After all the work is done and the result distributed to people working in the field of traffic management and those who are interested, some advantages will be gained by adapting it to be used in real road systems and using it as a base of other researches in the future. Here are some of the main expected benefits of this research.

1) To create an alternative for a method of traffic-signal-network control to efficiently manage a congested network in a crowded corridor in order to improve serviceability of the system.

2) To have a simulation environment that could be used to determine the efficiency of using the developed method of traffic control and also the model for testing other possible ways to increase the efficiency such as the improvement on geometry of a road network or other factors.

3) To have a method and a basic concept to be used in other studies and researches that require simulation API programming.

4) To have a method and a basic concept to be used in other studies and researches of traffic controlling, especially those are related to dynamic signal control and demand-distribution management by using the network of traffic signal to reduce instability of traffic conditions.

5) To encourage other researchers to study other new modern technologies and to use appropriate theories in developing some new methods not only in the field of traffic-signal management but also in other fields of transportation in the future.

CHAPTER II

LITERATURE REVIEW

Traffic signal control has been attended to by traffic engineers for many years since the invention of traffic signal control system in early 1900s. With the development of Area Traffic Control in late 1960s, the signal control method focuses on the real-time (adaptive) control that arguably yields higher performance and is responsive to the current flow condition. Lately, the advancement of traffic simulation provides researchers with a better tool to consider the better traffic signal control method, especially in studying the signal control performance in various traffic conditions. This chapter gathers concepts and past studies on the development of real-time traffic signal control. The highlight is placed on the signal control method in congested traffic condition.

The chapter is divided into four sections. First, the theory traffic signal control is described. Then some traffic management techniques related to the signal control are described, namely traffic demand and queue management. These techniques are utilized especially during saturated traffic condition. Some road user behaviors are presented in the next section. Lastly, the use of computer simulation is portrayed.

To find the way of solving the problem explained in chapter 1, some past studies are very valuable and useful in order to weight advantages and disadvantages of each solution including the limitation of usage. Other indirectly related theory and knowledge are also included in the study. Some parts of each study are selected to be reviewed and concluded in this chapter and grouped into four main topics, which are traffic signal control, travel-demand management, road-user behavior, and use of computer simulation in road-traffic engineering. In the end of this chapter, all topics will be summarized and concluded in an appropriate order that will reflect overall flow of idea.

2.1 Traffic Signal Control

2.1.1 Objective

"The Purpose of traffic control is to assign the right of way to drivers, and thus to facilitate highway safety by ensuring the orderly and predictable movement of all traffic on highways" is described by Nicholas J. Garber and Lester A. Hoel (2002). It seems to focus only on safety, which is the primary objective when the intersection-control is first introduced. Arguably, this definition does not cover all benefits gained from traffic signal control. In many studies, especially in recent years, traffic-signal control is also expected to give some profits in improving serviceability of an intersection or even the whole road network.

2.1.2 Levels of Traffic Signal Control

There are so many ways to control signalized intersections. Each method of traffic control at intersections yields different cost and effectiveness. In practice, each authority would select a proper traffic control that suits required effectiveness and desirable (affordable) cost. Traffic signal control is one of the methods of intersection control. Traffic signal control has many levels of sophistication. A more advanced system supposedly brings about higher performance (benefits). However, the more advanced signal control method is, the higher costs will be. In practice, the more advanced signal control system is selected and replaces the old-fashioned ones when the worthiness proof is made and the cost is affordable.

In some situations, spending too much money for an advance control system is not a practical alternative. For this reason, though there are a lot of advance control methods, the old traditional ones are still effective for many cases.

According to the paper "Development of Advance Traffic Signal Control Strategies for Intelligent Transportation Systems: Multilevel Design" by Gartner, Stamatiadis, and Tarnoff (1995), there are six levels of traffic control (including the base control).

Level 0 - Base control is defined as the control with a mix of fixed-time and traffic-actuated controllers and some arterial coordinated systems. Only some available historical volume data are required for this level of control. Some manual adjustments in field control are sometimes used for handling a situation such as the update of signal control plan due to changing traffic demand or incident.

Level 1 – The first level control is a little more systematic and centralized by the assistance of computer database. An arterial and network optimization package may be used. Normally this level considers results in a long term period.

Level 2 - For second level control, on-line optimization is used to plan the control pattern of each interval for five minutes to ten minutes. This shorter interval makes the control system become much more dynamic.

Level 3 - The third level control gives the full ability of adaptive control. For this level, no common cycle time is required. A whole system can be optimized together by considering performance of each sub-network.

Level 4 - The fourth level control is similar to the third one but it has some more intelligence which is an ability to interact with a dynamic traffic assignment module in the Advanced Traffic Management System (ATMS) to implement a proactive control, which is a dynamic ability to give priority to selected routes, a congestion avoidance strategy, and a congestion relief strategy.

Level 5- The fifth level control is the highest level of traffic control mentioned in the paper. This level possesses all abilities that the others have. The appropriate control strategy will be selected with some flexibility for facing any conditions automatically.

2.1.3 Signal Control at Isolated Intersection

The early traffic signal method is developed for controlling an isolated intersection. The method thus considers the best signal timing for this intersection given the traffic condition of the incoming flow into the intersection.

One of the most famous and widely-used methods is Webster's Method (1969). This method is expected to minimize delay occurring at an intersection. This delay consists of three main components: uniform delay component, stochastic delay component, and adjusted delay component. Parameters used in the consideration include approach flow rate, saturation flow rate, phasing diagram, start-up lost time and clearance lost time, which is the setting of amber period. The outcomes from the method are cycle length, red duration, and green duration. Moreover, the method provides the Webster's equation to estimate delay.

Webster's method estimates an optimum cycle length using the equation:

$$C_o = \frac{1.5L+5}{1-\sum_{i=1}^{\phi} Y_i} \tag{2.1}$$

Where C_o = optimum cycle length (sec)

L = total lost time per cycle (sec)

Y_i = maximum value of the ratios of approach flows to saturation flows for all the lane groups using phases I, i.e., q_{ij}/S_j

ϕ = number of phases

q_{ij} = flow on lane groups having the right of way during phase I

S_j = saturation flow on lane group j

To determine total lost time per cycle, equation 2.2 and 2.3 can be used to sum an effect of total lost time when vehicles start and stop moving and total all-red loss time during the cycle.

$$L = \sum_{i=1}^{\phi} l_i + R \tag{2.2}$$

$$l_i = G_{ai} + \tau_i + G_e \tag{2.3}$$

Where l_i = lost time for phase i

G_{ai} = actual green time for phase *i* (not including yellow time)

τ_i = yellow time for phase *i*

G_{ei} = effective green time for phase *i*

Definitions of effective green time and lost time are graphically described in Figure 2.1.

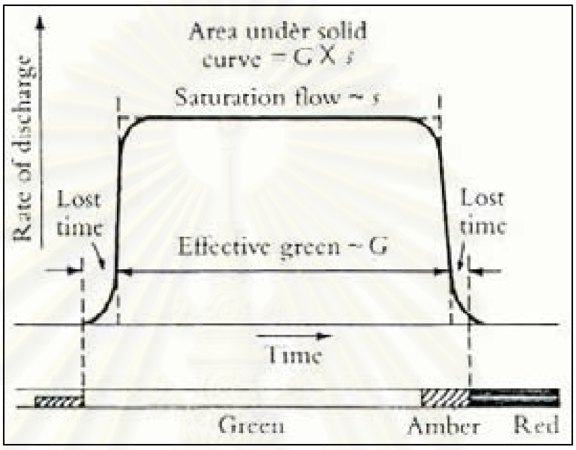


Figure 2.1 Discharge of vehicle at various times during a green phase (Garber and Hoel, 2002)

To allocate actual green time to each phase, total effective green time per cycle or G_{te} should firstly be calculated.

$$G_{te} = C - L = C - \left(\sum_{i=1}^{\phi} l_i + R \right) \tag{2.4}$$

$$G_{ei} = \frac{Y_i}{Y_1 + Y_2 + \ldots + Y_\phi} G_{te} \tag{2.5}$$

Where C is the cycle length

The actual green time for each phase, G_{ai}, can be calculated by using these sets of equations.

$$G_{ai} = G_{ei} + l_i - t_i \tag{2.6}$$

Webster's method is quite appropriate for controlling an individual intersection or an intersection that both gives and receives only little effect to other intersections.

2.1.4 Signal Control for Network

Nowadays a road network is built from many signalized intersections. Traffic entering an intersection is affected by the signal control at upstream intersections. Moreover, the effects of traffic spillover and queues are caused by the signal control at the downstream intersection. Traffic signal control considering the whole groups of intersections or even entire network seems to be more appropriate than independent isolated intersection control that does not consider the effect of surrounding signal operations.

Referring to "Adaptive Signal Control II" by Martin et al. (2003), Optimized Policy for Adaptive Control (OPAC), which is jointly developed by Parsons Brinkerhoff Farradyne Inc. and the University of Massachusetts at Lowell, divides a network into sub-networks. Each of them contains a group of intersections and links.

According to "Control Strategy for Oversaturated Signalized Intersection" written by Khatib and Judd (1995), phase sequences of each pair of connected intersections affect the ability to control the queue stored in each links. The size of this effect depends on the degree of saturation of both links. Another interesting topic in the paper is to divide movements into two groups: internal movements and external movements. Internal movements are movements in which vehicles use to pass each member of intersections under consideration while the external ones are movements that do not pass these intersections.

2.1.5 Dynamic Intersection Control

There are mainly two dimensions of the system control: place and time. The first one is to consider how to an intersection with or without considering other intersections and the other one is to focus on controlling an intersection with or without considering the changing of traffic situation while the time keeps moving forward.

One of the benefits of using dynamic signal control is to optimize stop delay. Rouphail (1991), the author of “Cycle-by-Cycle Analysis of Congested Flow at Signalized Intersection”, shows that, for light traffic volume, the delay will be minimized with no green time lost if a platoon of vehicles gets green light as soon as the first vehicle reaches an intersection. To make it possible, dynamic signal control and queue management are required.

According to the paper “Development of An Autonomous Adaptive Traffic Control System”, introduced by Tavladakis and Voulgaris (1999) from Technical University of Crete, traffic demand can be easily affected by variation of many parameters such as time, day, season, weather and unpredictable situations. This supports the necessity of controlling traffic-signal system dynamically. The research also mentions that cycle length, split, and also offset, that are fixed, are meaningless. From the papers, a dynamic traffic-control system requires a fault-detecting system. This system should be used for preventing accidents caused by conflicts in wrong phasing setting. In principle, two directions cannot be given right-of-way (green light) at the same time although it could improve serviceability.

Blocking and queue spillback are also important. Mentioned in the paper by Abu-Lebdeh (2000), “Towards Semi-Automated Arterials: Dynamic Traffic Signal Control with Time-Dependent Variable Speed”, using an appropriate dynamic traffic signal control method can efficiently control the queue on each link and prevent intersection blocking. Because the blocking reduces a lot of capacity and worsens the effectiveness of intersection control, the dynamic signal control surely gives some advantages to overall traffic management. This work also explains that the dynamic traffic control can handle time-dependent variable speed, which is the changeable speed that keeps varying during a day.

The dynamic intersection control is commercially developed as “area traffic control (ATC) or urban traffic control (UTC) system. Currently many systems are in the market. Each system has its unique (and proprietary) control method (logic). In United States, Martin et al. (2003) summarizes the deployment of adaptive signal controls in the United States as shown in Figure 2.2.

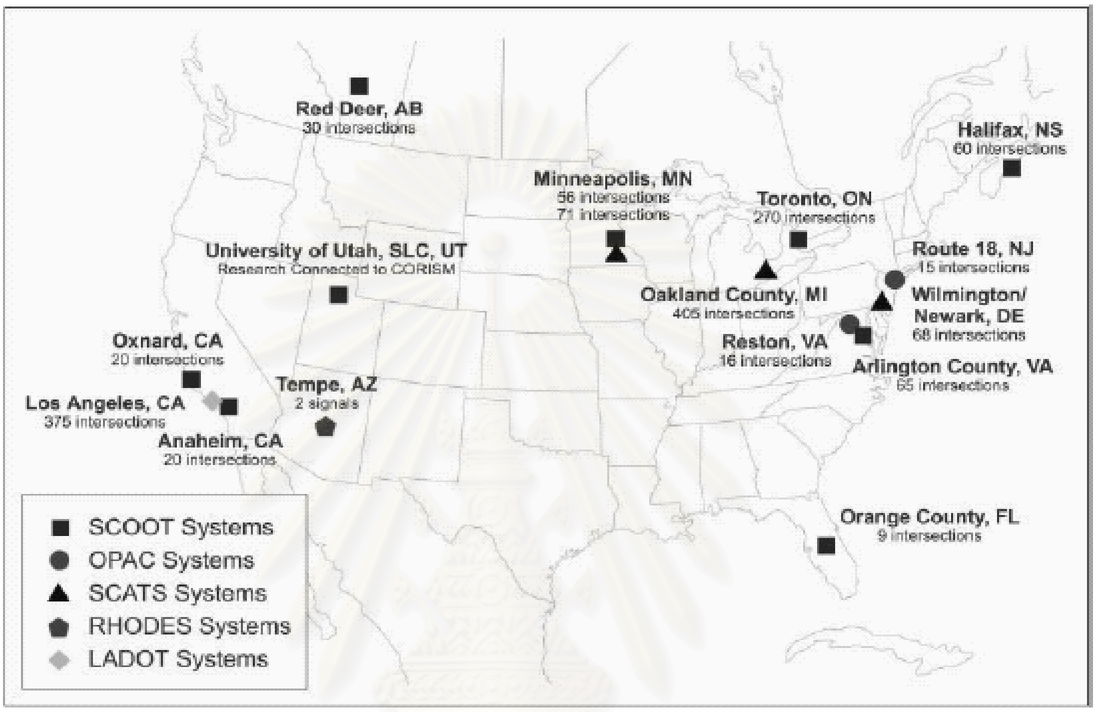


Figure 2.2 Adaptive traffic control system installation in North America (Martin et al., 2003)

Figure 2.2 implies that, the North America as an example, many different methods of adaptive traffic signal controls are used because of availability of knowledge, suitability to local conditions, and also the limitation of budget to be spent.

SCOOT (Split, Cycle, Offset Optimization Technique), is a kind of adaptive signal control systems developed by the Transportation Research Laboratory in the United Kingdom in the early 1980s mentioned by Hansen, Martin, and Perrin (2000). This method concerns the minimization of a performance index, PI, which is a function of average queue length and the number of stops at all approaches in the network. To optimize this performance function, all cycle lengths, splits, and offsets are adjusted dynamically using real-time traffic data collected from the road traffic detection system (mostly inductive loop detectors). The optimization takes place at the central processing unit and then all updated adjusted signal timing information is distributed to the local controllers.

Martin et al. (2003) introduce the combined use of adaptive signal control and fixed-time signal control. Adaptive signal control may be used only during the peak periods of a day and the fixed-time signal control may handle the signal system in other periods.

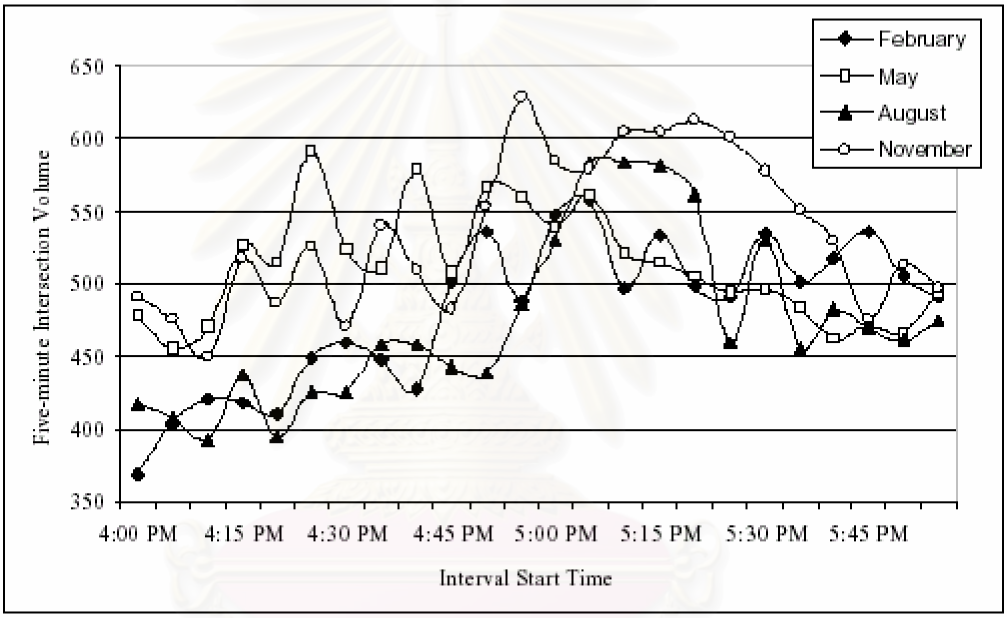


Figure 2.3 Daily flow variation in Murray, Utah in 2001 (Martin et al., 2003)

Figure 2.3 shows possibility of having more than 70 percent change in demand during five-minute duration in the peak period. This confirms that use of fixed-time traffic control in these peaks is not appropriate. But for dynamic control with changeable signal timing, every change in demand can theoretically be handled.

Table 2.1 shows some outcomes after the old-fashioned (fixed time or vehicle actuation) traffic-control systems are replaced by SCOOT around the world. It proves that most outcomes are significantly improved by the SCOOT system. Bear in mind that the Table shows only the benefits gained by the

SCOOT system but does not mention about the costs in addition to the previous systems.

Table 2.1 SCOOT Field evaluation results (Martin et al., 2003)

Location of SCOOT Installation	Previous Control Method	Year	% Benefit over previous control method	
			Delay	Travel Time
São Paulo, Brazil (ver. 2.4)	Fixed-time (TRANSYT)	1997	0 – 40	-
São Paulo, Brazil (ver. 3.1)	Fixed-time (TRANSYT)	1997	0 – 53	-
Nijmegen, The Netherlands (ver. 2.4)	Fixed-time	1997	25	11
Toronto, Canada (ver. 2.4)	Fixed-time	1993	17	8
Beijing, China (ver. 2.3)	Fixed-time (Uncoordinated)	1989	15 - 41	2 - 16
Worcester, UK (ver. N/A)	Fixed-time (TRANSYT)	1986	3 - 11	7 - 18
Worcester, UK (ver. N/A)	Isolated Vehicle Actuation	1986	7 - 18	15 - 32
London, UK (ver. N/A)	Fixed-time	1985	19	6 - 8
Southampton, UK (ver. N/A)	Fixed-time	1985	39 - 48	18 - 26
Coventry, UK - Foleshill Road (ver. N/A)	Fixed-time (TRANSYT)	1981	22 - 33	4 - 8
Coventry, UK - Spon End (ver. N/A)	Fixed-time (TRANSYT)	1981	0 - 8	0 - 3

Martin et. al. (2003) mentions about OPAC signal control. OPAC has two levels of control, fixed-time plans and the dynamic one. Though OPAC is a signal control at network level, it aims to maximize only number of vehicles passing each intersection. Nonetheless, it considers the effects by adjacent intersection control in terms of spillback and cycle length. Only the cycle length has to be globally calculated using a central computer. Other calculations can be done by the local computers. This gives some benefits when the central computer is out of function or when any emergency cases occur.

RHODES or Real-time Hierarchical Optimized Distributed and Effective System is another famous system of signal control method mentioned in the “Adaptive Signal Control II” by Martin et al. (2003). It has three models: dynamic network loading model, network flow control model, and intersection control model, for facing three levels of instabilities in demand. Estimation and prediction are used in all these three models in order to find optimum splits.

Los Angeles Adaptive Traffic Control System (LA-ATCS) is described by Peter T. Martin et al. (2003). Similar to some other controls, the road system must be divided into sections or groups of intersections. Traffic volume at each intersection is used to optimize the control by minimizing stops. The values to be verified are splits of each phase and offsets among intersections in each section. Cycle length for each section is determined from the highest-traffic intersection in the section. It is automatically considered to be the critical intersection and this intersection can be changed if traffic condition changes. To prevent too much inequity and inefficiency of the control, minimum and maximum cycle length should be limited but these limits may not be the same between different intersections. Approach flows are the main data for optimizing splits.

Another method suggested in “Toward Semi-Automated Arterials: Dynamic Traffic Signal Control with Time-Dependent Variable Speed” by Abu-Lebdeh (2000) is the use of Genetic Algorithms which provides an ability to handle a dynamic traffic situation. Speeds of vehicles are considered to be a variable in order to estimate appropriate green splits for each cycle. For this method, the algorithms aims to control not only lost time but also blocking and queue spillback, which can significantly worsen system capacity.

Sometimes, the green duration, which is mathematically and theoretically calculated, may not be practical. A report by Mirchandani and Lucas (2001), titled “RHODES-ITMS Tempe Field Test Project: Implementation and Field Testing of RHODES, A Real-Time Traffic Adaptive Control System”, suggests the use of green-time limitation. The green time should be limited between minimum and maximum value. This limitation of green time can efficiently prevent start-up lost time, a decrease in intersection capacity, and unbalanced travel time among users.

2.1.6 Use of Traffic Prediction in Dynamic Intersection Control

For dynamic signal control, there are two levels of consideration: controls to handle only the present situation and traffic controls that deal with expected short-term or long-term effects of each decision on future situations.

SCOOT also has this part of prediction. Real-time flow and occupancy information must be continuously collected by (loop) detectors. Then, the collected data are used to estimate/predict total delay and stops for each case of signal timings. The case that yields the minimum performance index, PI, will be chosen.

The prediction used in SCATS is much different from the prediction used in SCOOT because it does not need to make any complicated calculation to predict the outcome every round of the control update. This pre-calculation implies that SCATS would require less resource, especially computing time and computer hardware.

For OPAC, sixty-second prediction is calculated by only collected fifteen-second data with time-step (roll period) of 2-5 seconds. This prediction may not be far enough to reach the long-term optimum. The optimum point of this control system is determined just for a short-interval span.

2.1.7 Priority for Public Vehicles

Sometimes, the number of passengers carried is much more important than the number of vehicles to be served by a road network. Many studies aim to find some appropriate ways of giving priority to public vehicles, especially to buses.

This priority is described in “Fuzzy Public Transport Priority in Traffic Signal Control” by Niittymäki and Mäenpää, Helsinki University (2000). Public transport should be detected before it reaches each major intersection to provide them the priority of stop minimization. At least three kinds of detectors, namely pre-request detector, request-detector, and exit-detector, should be installed in order to improve the giving of the priority. A pre-request detector informs the control-system to prepare everything to give the priority as soon as a public vehicle approaches an intersection. Another detector, request-detector, reports the arrival of a public vehicle when it comes close to the intersection. When the vehicle reaches this point, the priority will be initiated. To get the time that the priority should be finished, the last detector should be used. This detector tells the controller when the bus leaves the intersection.

According to the papers introduced by Martin et al. (2003), a bus can transport more passengers for the same amount of space on a road way, thus it is reasonable to give those buses a priority by reducing travel time (intersection delay) and improve a reliability of the bus scheduling. SCOOT can adapt the signal to acommodate buses some priorities by detecting buses that nearly reach an intersection and extending green phase or shortening the time that the buses have to stop. The difficulty of doing this, introduced by the developers of Adaptive Signal Control II, is that buses sometimes have to stop at bus stops. It leads to misallocation of green time for the buses. Some other kinds of adaptive control system such as SCATS (Sydney Coordinate Adaptive Traffic System) and RHODES (Real-time, Hierarchical, Optimized, Distributed, and Effective System) also have some developed method of improving the service of intersections for buses and other kinds of public vehicles. Because, in many cases, buses can use the same lane or same roads as other vehicles, effect of giving priority should be controlled not to give too much effect to other traffic or the whole system. For the RHODES, BUSBAND algorithm can be used with its concept of considering not only the number of buses getting near the intersection but also the counted number of passengers in each bus if it is available.

Like ordinary traffic control, the control considering priority also needs prediction. An Approach Towards The Integration of Bus Priority, Traffic

Adaptive Signal Control, And Bus Information/Scheduling System written by Mirchandani et al. mentions about this point and also summarizes strategies for transit priority into three main groups: passive priority strategy, active strategy, and real-time strategy. The first one, passive priority strategy, covers adjustment of cycle length, phase splitting, area-wide timing plan, and metering priority. This group of priority uses the past data to plan how to change the control. The second set of strategies, called active priority strategies, focuses on changing the control as soon as the condition changes. The strategies included in this set are phase extending, early phase activation, special transit phase, phase suppression, unconditional priority, and conditional priority. Real-time priority strategy, the last group that consists of delay optimizing, intersection control, and network control, consider more dimensions. Some effects that can be distributed to other parts of the system and also other points of time are included in the consideration.

2.2 Travel Demand Management and Queuing System

Besides serviceability of the road network, management of travel demand is also very important in balancing the usage of the transport system, thereby increase the efficiency of the integrated transport system Sometimes, well-distributed demand can even improve the supply or serviceability by itself. If a road network is considered to be a queuing system with a variable serviceability as a dynamic function of amount, types, and behavior of each user, demand management in a term of queuing management will have a great role in congestion control.

As mentioned in Toward Semi-Automated Arterials: Dynamic Traffic Signal Control with Time-Dependent Variable Speed by Abu-Lebdeh (2000), queue length and spillback should be controlled in order not to block each junction. This implies that controlling demand can improve or even maximize serviceability of the system.

Shown in Figure 2.1 from Garber and Hoel (2002), Traffic and Highway Engineering, saturation flow-rate is quite constant. If there is less demand than this rate, some capacity will surely be lost.

For traffic control in the view of queue management, Hansen, Martin, and Perrin, Jr. (2000) explains the necessity of controlling queue length in the term of queue length ratio should not exceed the boundary of upper bound for queue length ratio and lower bound for queue length ratio which are the function of the average vehicle spacing within a standing queue, the length of subject arterial approach, the average number of cross-street vehicles per lane seeking service in one signal cycle for cross-street approaches, the proportion of total traffic on cross-street approach, the number of lanes, a safety factor to guard against spillback, and other related factors. This queue length and queue length ratio are said to affect the optimum offset between each connected pare of intersections in order to prevent blocking and lack of traffic for serving the capacity.

The spillback of vehicles is said to be the cause of congestion in A Control Scheme for High Traffic Density Sector, mentioned by Rathi (1988). When traffic density reaches the congested point, link travel time is mainly related to the number of vehicles in the queue and the spillback blocking each intersection should be carefully prevented. Many factors cause the blocking, including the inappropriate traffic signal control. If the offset of green time between two connected intersections is too long or inflow and outflow due to green split at each end are not balanced, spillback may occur and block the upstream intersection. For the case of having too small rate of entering flow compared to the link-leaving rate, serviceability of downstream intersection may not be effectively used. Considering both cases, to control the entering rate and leaving rate to be dynamically equal with an appropriate offset and acceptable queue length seems to be a key to control the congestion for a congested network or a crowed road system. In addition, the writer also considers the speed that each vehicle uses. Different speed and driving behavior can affect the setting of offset. For this reason, this consideration should necessarily be included in the offset calculation.

Not only the external blocking, which is the situation that stop vehicles from at least one of the links block others links, but a lack of waiting space of turning pocket can also cause an internal blocking as shown in Figure 2.5.

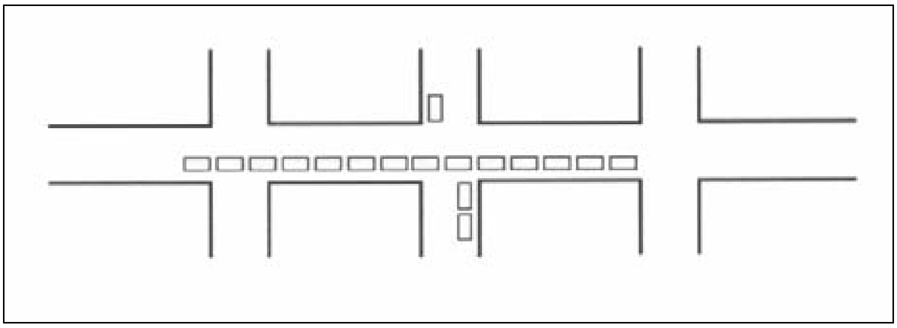


Figure 2.4 Spillback on the arterial (Khatib and Judd, 2001)

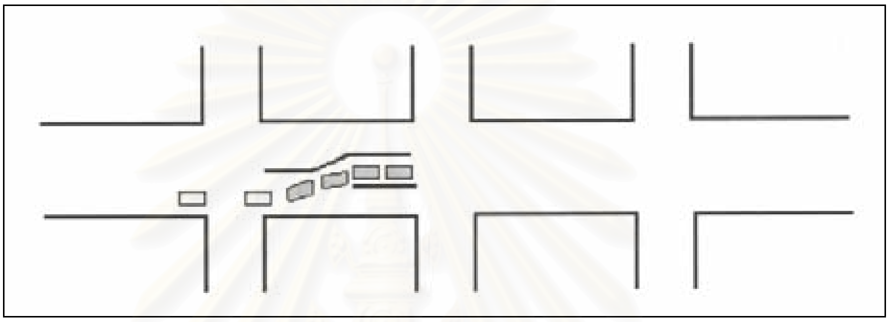


Figure 2.5 Starvation caused by left-turn spillback introduced
(Khatib and Judd, 2001)

Long pocket lane can prevent this internal blocking but it may take some cost and space. The more practical way to face this problem is to efficiently control the demand of through traffic and turning vehicles and the only way to do it dynamically is to efficiently control traffic-signal network so that the effect flow and queue are contained in the assigned area.

2.3 Road User Behavior

Studies dealing with human behavior are usually so complicated especially in the case of having interactions and reactions among many parties or many decision makers. This is strongly related to efficiency and reliability of prediction.

It is mentioned in Anticipatory Optimization of Traffic Control, researched by Zuylen and Taale (2000), the both traffic control and traveler's behavior have some influence to one another. This makes the problem become a bi-level optimization problem. All choices of route, mode, departure time, and other choices of making a trip can affect the control while a traffic control also affects what to be chosen. Because traffic condition tends to sway around an optimum condition, this optimum is used for forecasting effect of the control and finding the most appropriate control expected. Actually, there is a multiple stable situation. Van Zuylen and Taale (2000) suggests that traffic managers should be aware that an optimum point obtained from mathematically calculation may not be the real optimum point if the road users do not follow the presumed assumptions. Simulation considering user's behavior is suggested to be the best tool for finding this point and effectively managing the system. For decision making, decision makers seem to maximize their expected profit or to minimize their expected travel time used. The profit or cost to be used in consideration is not the real one they are going to face but the one that they think it is. This is strongly dependent on characteristic of each user.

Explained in General Method of Calculating Traffic Distribution and Assignment written by Ridley, University of California Berkeley (1968), every trip has its own origin and destination. For a large system, origins and destinations can be grouped into zones that may reduce the complication of traffic assignment. To make a trip from the origin to the destination, a traveler may have more than just one choice of route to be selected. The way to be chosen is judged by the profit it gives or the cost it takes by users. It can be said that road users usually choose the shortest (cost) path, which sometimes does not have the shortest length. The route that takes the shortest time or the smallest cost can be considered as the shortest path. The length, time, and cost to be used in the consideration is usually not the real values but users or drivers use the values they think it should be. This may ease the prediction of traffic managers to assign the traffic dynamically. For the case of an expressway, although the time used is reduced but drivers using the roadway have to pay some toll. Because of different value of time and behavior, users may have different choices of using or not using the toll way.

2.4 Use of Computer Simulation in Road-Traffic Engineering

In many cases of traffic studies, the real-world experiments are difficult or costly and calculations are too complicated to be efficiently done manually. Simulation, especially computer simulation, can be another alternative for traffic engineers to analyze the problem. It can be used in both the field of traffic study and the field of traffic control.

One of discernable benefits from analyzing traffic with simulation is the presentation of the results. Figure 2.6 is a snapshot from the visual interface of TRAFVU created by CORSIM, the computer-simulation program. This kind of user interface is much easier for users to understand than using numerical outcome. If any geometric condition is wrongly coded to the computer, the program can graphically notice the mistake. However, this presentation usually takes more calculation time than the numerical one does and it is usually disregarded when the simulation connects with other computer programs and with other controlling or computing hardware.

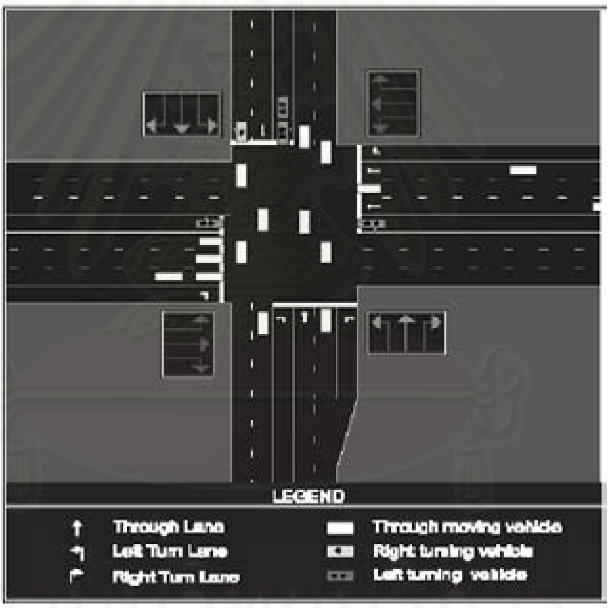


Figure 2.6 A TRAFVU animation snapshot (Martin et al., 2003)

Traffic simulation is not only used for analyzing traffic and control system independently, it is also used as a platform for generating and gathering traffic data and interface to other model/programs for accomplishing assigned tasks. Figure 2.7 shows that both structure of using SCOOT in a real system and a simulated CORSIM network should be quite the same in order to reflect what should happen in the real world. Some steps have to be changed to be appropriate for the simulation due to the limitation of programming but the sequence of real-time data collecting, analyzing, and controlling must be the same. Moreover, the simulation is used for gathering necessary results that reflect the status of traffic conditions. The results are commonly interpreted as Measure of Effectiveness (MOEs) for traffic (and control) performance. The examples of such indicators from SCOOT include congestion levels, defined by v/c.

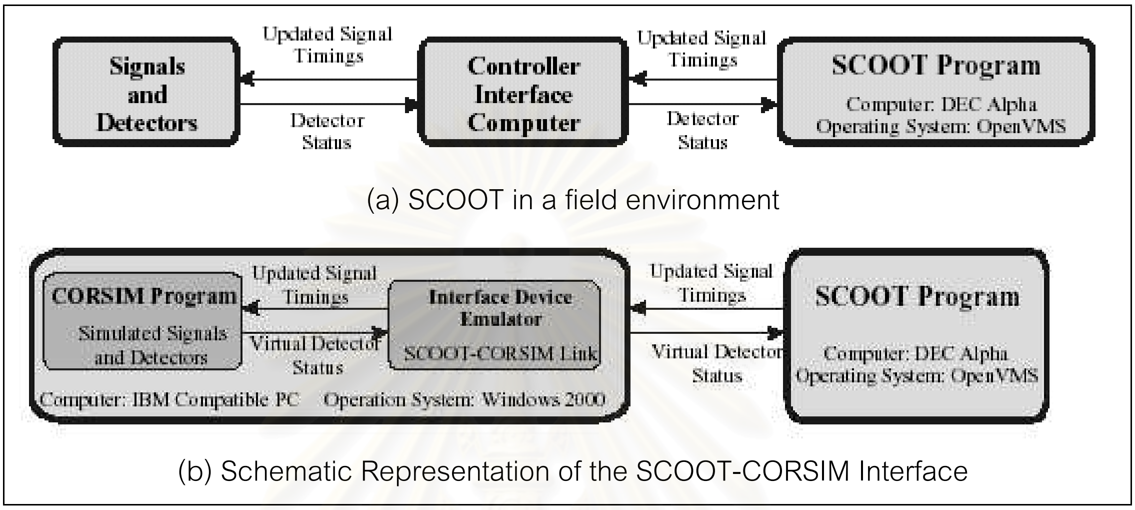


Figure 2.7 SCOOT system operation for (a) standard and (b) simulation environment (Martin et al., 2003)

Mirchandani and Lucas (2001) from The University of Arizona said in RHODES-ITMS Tempe Field Test Project: Implementation and Field Testing of RHODES, A Real-Time Traffic Adaptive Control System that before testing or using in the field, simulation, especially computer simulation, should be used to predict what will happen if any traffic-control strategies are used. This use of simulation can give a traffic manager some more reliability in making a decision.

Simulation optimization for uncertain situation is quite complex, while it is also very important to reflect what will happen in the real system. To optimize or stabilize the simulation, some related variables and also some random numbers, used for simulating uncertainty, have to be verified. Memon and Bullen (1996), Multivariate Optimizaton Strategies for Real-Time Traffic Control Signal, introduce the use of minimum and maximum values of each variable as an upper bound and a lower bound for alteration. The limits give much more possibility to vary the values because just a little change in steps of increasing or number of variables can double the number of simulation runs.

Because (amount of) users have some interaction with the control, each run of simulation needs a predictive response by users as a result of the change of traffic control and other variables.

For a large simulated network, it is impossible to easily ask any road users what they think about the service provided by the road-traffic manager. The use of appropriate MOEs is suggested to be the solution in Assessment of A Stochastic Signal Optimization Method Using Microsimulation, the research done by Park (2001) from Nation Institute of Statistical Sciences and NC State University. Some appropriate MOEs should effectively represent overall outcome with no major biasness that cause by systematic error. In practice, MOEs would reflect network performance, not individual accomplishment.

A computer program named Paramics is used in this study. From its computerized manual, Paramics is said to be the tool for modeling read system microscopically. There are three main programs in Paramics V4.2: Paramics Modeller, Paramics Processor, and Paramics Analyser. Paramics Modeller is the simulation program with graphical illustration. Paramics Processor works like the Modeller but it runs in batch mode without any graphical mode. It has some advantages in fasten the simulation because graphic mode takes a lot of computer resource. Paramics Analyser works differently. It is not the simulation program itself but it provides the tool for reading the simulated results and also analyses them. It is very useful, especially to analyze the result from Paramics Processor that cannot show the simulated result graphically. Paramics also let users be able to use some additional plugin for advance controlling and detecting some hidden result. The plugin can be coded by using Visual C++ computer and Java languages and added into each run. This is the powerful tool for users to manage the simulation in details.

2.5 Summaries and Conclusion

Many studies investigate the solution for the urban traffic congestion, especially for the large crowded city. Some of them focus on how to manage a traffic control system effectively. Some other past studies indirectly explain some characteristics of the system and also its users.

Signal control methods can be grouped in to a number of levels according to how complicate and dynamic they are. The complication surely comes with some extra cost (computational effort, additional hardware, better system requirement, etc). Each method of signal control should be carefully adapted by engineers if it would be used in any city or any area. The use of costly computer system can help the managers reduce a lot of the complication in calculation and data analysis. For an isolated-intersection control considers only on an individual intersection. The objective value to be minimized is usually the delay at the intersection. For this reason, this kind of traffic-signal control works well for only the case where there is no other intersection close to the subject intersection and affects the serviceability of each other. For a city with plenty of links and junctions, multi-intersection control considering effects between all intersections should be the better alternative. By including uncertainty and inconstancy to the consideration, dynamic signal control becomes the advanced choice of control that can manage the system appropriately at all time in a variety of traffic situations. A lot of dynamic multi-intersection control methods are continuously developed, tested, and used in all parts of the world. No method can be said to be the best because much more experiments are needed to yield the unified conclusion. Even in the same country, there can be more than just one strategy used for the control in practice.

To optimize the system, the controller needs to predict what will happen if any control pattern is applied. There are many different ways to predict outcome based on some different assumptions. Normally, they use historical data and real-time data to estimate the results. By the way, there are some limitations of the prediction, which are usually related to the time of prediction. The longer time of prediction needs more information or else it may give less accuracy. For this reason, each method of control has it own appropriate prediction interval, depending on how long (period of time) optimization it needs and how much information is available.

To make the priority vehicles travel faster, public vehicle's priority should be considered in the setting of traffic signal control. This policy should carefully be adopted in every type of traffic control. Some detectors should be in installed at an intersection. Each detector has it own function. Some may detect the vehicle before it enters the intersection while some others may be used to detect the leaving of the same vehicle. By the way, the priority always comes with some effect to overall system. It is also possible that giving a priority to one public vehicle may increase travel time of many other public vehicles in other directions. To give an appropriate level of priority by including the effect into the selected control method should be the solution of this problem but it also causes much more complication to the calculation.

Because the serviceability of a road network has some relations with the number of users on each link, management of demand and queue length can be considered as a primary goal for maximizing the serviceability of an existing system. Blocking at an intersection is the main problem that is caused by inefficient queue management. Not only the external blocking but also the internal blocking decreases the total amount of flow traversing the network. One of the ways to manage both of them is to limit amount of vehicles on targeted links on the network using traffic signals and detectors. Too small queue may not cause the blocking but it still cause another kind of problem, the lack of demand to use the existing supply. If there are no vehicles waiting to use a green interval, some green time may not be effectively used. For this reason, a lower bound of queue length should also be limited. Both upstream and downstream signals should be used as a tool for controlling the queue.

Then, the necessity of considering user behavior is discussed. Users of various transport systems usually have their own characteristics. All users and the systems always give some influence to each other. This makes the optimization become very complicated, especially for prediction and simulation, and also causes multiple-stable situation that may lead to non-optimum solution. The way to reduce this complication is to simplify some behaviors. For example, instead of trying to identify all choices of all users, the assumption of choosing the shortest path should be applied as an assumption for user behaviors

Advanced information technology gives traffic engineers the power to simulate situations. The simulation can ease some direct calculation and sometimes reflect some uncertainty effects. The simulation usually requires less cost compared to the case of field-testing. Another advantage of using a computer simulation is ability to graphically investigate the movement of each vehicle. Some hidden outcomes can be notified by this visual interface. By the way, because of the uncertainty and variations of some parameters, many runs of simulation that use a lot of computer's resource may be required. To reduce the usage, the smaller number of runs with less efficiency may be used. A good structure of calculation can also significantly reduce calculation time of the simulation. This well organized structure of control method algorithm does not harm any efficiency and it is the way to prevent some error caused by wrong programming.

Finally, the use of computer simulation in a prediction will be valuable if the appropriate MOEs are selected to describe effectiveness of each alternative of traffic-signal control. Not only the complication but also the use of too many MOEs or wrong MOEs cause the less reliability of the prediction. For this reason, traffic engineers should carefully choose the MOEs that truly can reflect everything needed in a calculation.

CHAPTER III

METHODOLOGY

In this chapter, some important details about each control method and the testing method were presented. The methodology was developed to prove if the newly-developed method, throughput-maximization method, had more efficiency to manage each level of morning-peak traffic.

Starting from scenario building, the description of network of and demand will be described with the reasons of selecting them. Then, some information about traffic control methods to be tested was presented. Finally, this chapter explained the testing method on how we could use both scenarios and control methods to test the throughput maximizing method.

3.1 Scenario Building

To setup all scenarios for the test, the network had to be selected by considering both generalization and limitation of program coding. After choosing the network, traffic demand had to be applied. In the consideration, both views of quantity and distribution had to be concerned. The results were some appropriate scenarios that fitted the purpose of testing traffic control in this study.

3.1.1 Network

A hypothetical corridor with four signalized intersections connected by three 1000-meter links was selected to be used in the simulation. Other legs of each intersection were two kilometers in length as shown in Figure 3.1. The dimension (length) of the road network was considered together with the level of demand (traffic volume) and resulting traffic situations. With a proper traffic demand, the network displayed saturated traffic condition where spillback developed onto upstream intersection. The reason of using 1000-meter length for inner links was that a queue could be created or dissipated in not too long or too short time. Moreover, the network should be able to accommodate the oversaturated situations where too long queue developed and blocked vehicles from arriving each link. These oversaturated situations were used to observe the effectiveness of traffic control in terms of lost of green time which affected total delay.

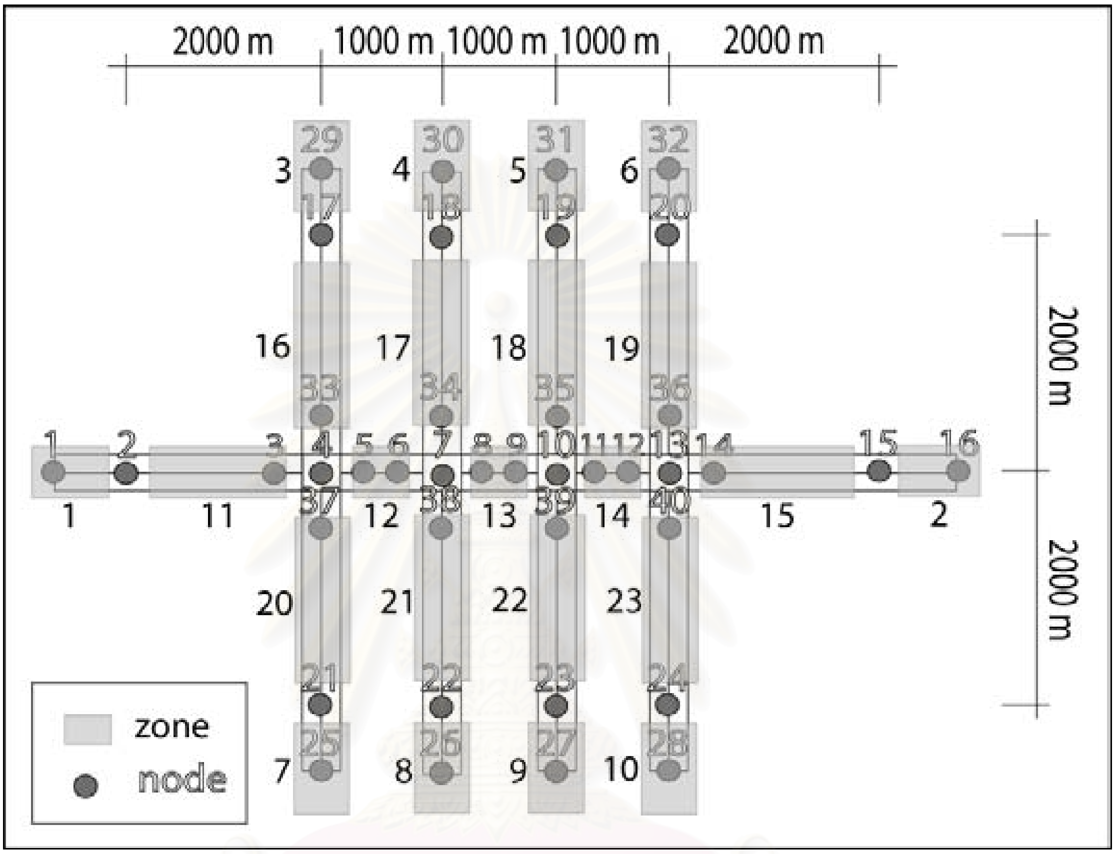


Figure 3.1 Schematic diagram of a simulated network

Zones 1 to 10 (outer zones) produced traffic into the network. Both zones 1 to 10 (outer zones) and zones 11 to 23 (inner zones) received the traffic. In other word, the outer zones were source (origin) and sink (destination) while zones 11 to 23 were only sinks. This represented the situation responsible to morning period when majority of traffic come to downtown (along the corridor with zones 12, 13, and 14). An origin zone could produce traffic at any position of the link that intersected the zone. On the other hand, as a limitation of Paramics, a vehicle leaved the system as soon as it reached the boundary of its destination zone.

Each link had three lanes and it was expanded to have five lanes at 100-meter distance from each intersection. Those two added lanes acted as pocket lanes, one left-turn pocket and one right-turn pocket, for improving serviceability of an intersection. Each lane had the width of 3 meters. All links were straight and have no lane-change restriction. All turning restrictions were assigned by considering the traffic moving from the link to each downstream link.

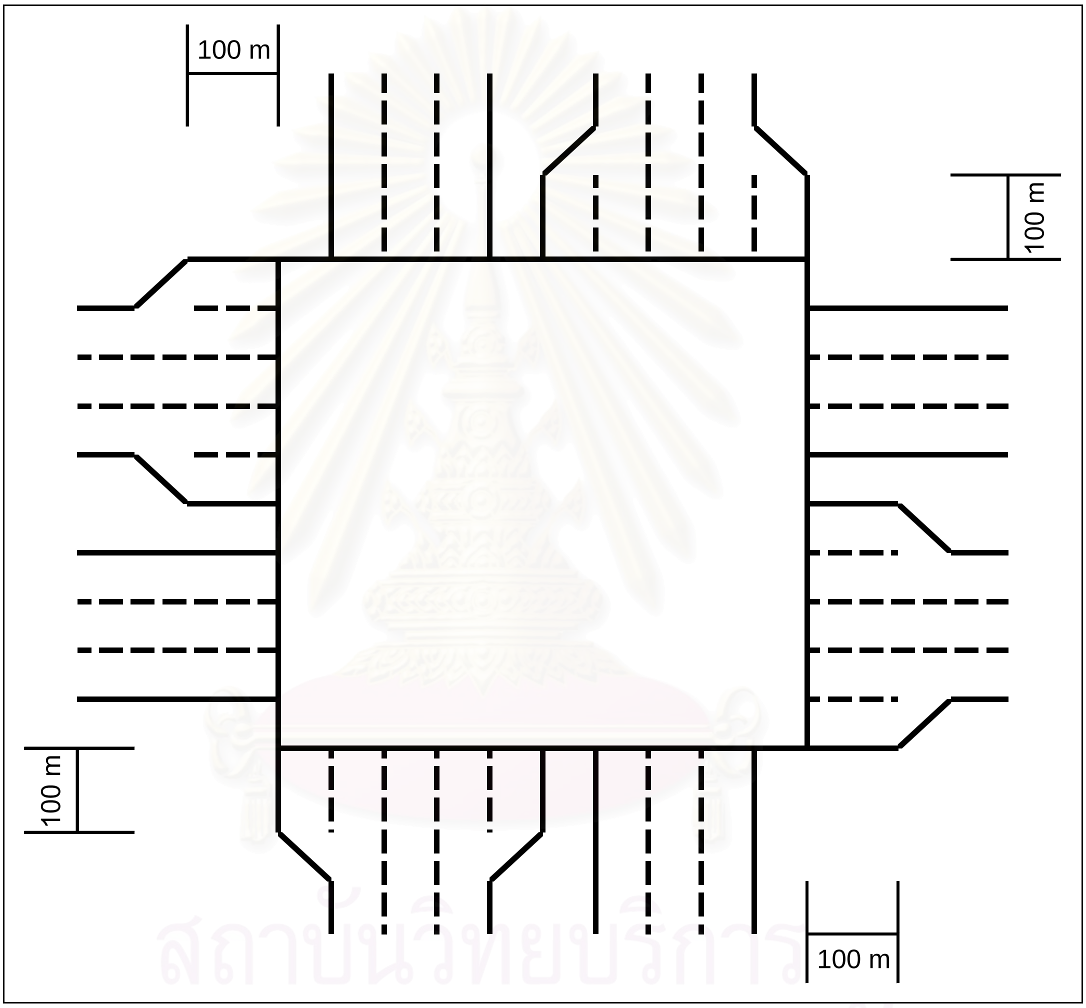


Figure 3.2 Lane expansion

At each intersection, blocking was prevented by setting the restriction in Paramics. It meant that though a queue of vehicles could reach an upstream intersection but it could not be long enough to block the intersection. Then, other approaches could use an intersection as soon as it got green signal if the downstream link was not full. Each intersection was identified by an ID number of the node corresponding to its position and each link was identified by ID numbers of the upstream node and the downstream node respectively. For example, link 3:9 started at node 3 and ends at node 9.

Left-hand driving was chosen to make the network similar to the network in Thailand.

Table 3.1 showed movement restrictions at each intersection. The number of lanes in each approach was selected so that the prospect capacity of each traffic movement matched traffic demand and thus yielding desirable levels of service. Each pattern of demand was described in the following part.

Table 3.1 Number of lanes for each movement at each intersection

Intersections	Upstream nodes	Upstream-node locations	Number of lanes (demand pattern 1)			Number of lanes (demand pattern 2)		
			Left	Through	Right	Left	Through	Right
4	17	North of an intersection	3	1	1	3	1	1
	7	East of an intersection	2	1	2	2	1	2
	25	South of an intersection	1	1	3	1	1	3
	2	West of an intersection	1	3	1	1	3	1
7	18	North of an intersection	2	1	2	2	1	2
	10	East of an intersection	1	3	1	1	3	1
	26	South of an intersection	2	1	2	2	1	2
	4	West of an intersection	1	3	1	1	3	1
10	19	North of an intersection	2	1	2	2	1	2
	13	East of an intersection	1	3	1	1	3	1
	27	South of an intersection	2	1	2	2	1	2
	7	West of an intersection	1	3	1	1	3	1
13	20	North of an intersection	1	1	3	1	2	2
	15	East of an intersection	1	3	1	1	3	1
	28	South of an intersection	3	1	1	3	1	1
	10	West of an intersection	2	1	2	2	1	2

3.1.2 Traffic Demand

Theoretically, there are three main levels of congestion, unsaturated level, saturated level, and oversaturated level.

In this study, the saturated demand for each pattern of demand was estimated by multiplying the demand with different values by keeping the ratio fixed based on the pattern to get the highest level of demand that the peak throughput was still cover the demand. The starting demand was calculated from the expected saturation flow rate. This demand was adjusted with the step of ten percent until the situation switched from the oversaturated condition to the unsaturated condition or from the unsaturated condition to the oversaturated condition. Then the value moved back for five percent to get closer to the real saturated condition.

In the study, only five levels of demand were applied to the system. They were 60-percent saturation, 80-percent saturation, 100-percent saturation, 120-percent saturation, and 140-percent saturation. The first two levels could be grouped into the unsaturated and the last two could be grouped into the oversaturated one. The saturation was determined on the network with the fixed time control (type-1 control) that was to be described in the next part.

For all patterns, demand in the first hour and in the third hour was set to be half of the peak one in the second hour to make the simulation smoother and there was no additional demand in the fourth hour to let the system clear itself.

Practically, many factors affected the real capacity of the network. Instead of calculating theoretical capacity to get a proper level of saturated demand, this study tested the resulting traffic condition from varying traffic demand to find the point of saturation. The saturation occurred when the demand and the capacity were the same. T-test with significant level of 0.01 was used to confirm the equality.

By the way, with random seed numbers, the program should make the demand inconstant for each run but there was only little difference.

3.1.3 Traffic Demand Patterns

To generalize the test, two demand patterns which represented two extreme distributions of travel demand were deployed in this research. Each of them was expected to be only a case study. These two patterns were assumed to be the real demand that could be any value without any systematic assignment.

The first one was set to be well distributed demand that distributes demand from ten origin zones to other destination zones. In this case, an inner link attracted demand, two times of that was attracted by an outer link.

In the second case, the cross demand was added in order to represent the network that had a main highway cut through the center of the city. Amount of this cross demand was equal to the demand of one inner destination zone.

Both patterns of demand were numerically shown in the two tables in Appendix A. These tables showed only the base demand to be multiplied by some different positive values to make the demand reached different level of congestions. A base value of 41 with origin zone 6 and destination zone 10 seemed to be extremely different from others but, in fact, it should be compared to the summation of all traffic entering the intersection.

Table 3.2 showed how traffic demand is distributed to each intersection.

Table 3.2 Unit traffic volume at each intersection from the OD table

Intersections	Upstream nodes	Upstream-node locations	Traffic volume (demand pattern 1), unit/hour			Traffic volume (demand pattern 2), unit/hour		
			Left	Through	Right	Left	Through	Right
4	17	North of an intersection	26	2	2	26	2	2
	7	East of an intersection	14	14	14	14	14	14
	25	South of an intersection	2	2	26	2	2	26
	2	West of an intersection	2	26	2	2	26	2
7	18	North of an intersection	18	2	10	18	2	10
	10	East of an intersection	10	50	10	10	50	10
	26	South of an intersection	10	2	18	10	2	18
	4	West of an intersection	6	54	6	6	54	6

Table 3.2 Unit traffic volume at each intersection from the OD table (continued)

Intersections	Upstream nodes	Upstream-node locations	Traffic volume (demand pattern 1), unit/hour			Traffic volume (demand pattern 2), unit/hour		
			Left	Through	Right	Left	Through	Right
10	19	North of an intersection	10	2	18	10	2	18
	13	East of an intersection	6	54	6	6	54	6
	27	South of an intersection	18	2	10	18	2	10
	7	West of an intersection	10	50	10	10	50	10
13	20	North of an intersection	2	2	26	2	42	26
	15	East of an intersection	2	26	2	2	26	2
	28	South of an intersection	26	2	2	26	2	2
	10	West of an intersection	14	14	14	14	14	14

To confirm that the turning restriction was reasonable, unit traffic volume per lane and expected v/c ratio after the unit demand of each pattern had been multiplied by an appropriate value in order to reach the saturation were shown in the tables. The expected v/c ratio is calculated by using Synchro 5 program with the unsynchronized fixed time control.

Table 3.3 Unit traffic volume per lane

Intersections	Upstream nodes	Upstream-node locations	Traffic volume (demand pattern 1), unit/hour/lane			Traffic volume (demand pattern 2), unit/hour/lane		
			Left	Through	Right	Left	Through	Right
4	17	North of an intersection	8.7	2.0	2.0	8.7	2.0	2.0
	7	East of an intersection	7.0	14.0	7.0	7.0	14.0	7.0
	25	South of an intersection	2.0	2.0	8.7	2.0	2.0	8.7
	2	West of an intersection	2.0	8.7	2.0	2.0	8.7	2.0
7	18	North of an intersection	9.0	2.0	5.0	9.0	2.0	5.0
	10	East of an intersection	10.0	16.7	10.0	10.0	16.7	10.0
	26	South of an intersection	5.0	2.0	9.0	5.0	2.0	9.0
	4	West of an intersection	6.0	18.0	6.0	6.0	18.0	6.0
10	19	North of an intersection	5.0	2.0	9.0	5.0	2.0	9.0
	13	East of an intersection	6.0	18.0	6.0	6.0	18.0	6.0

Table 3.3 Unit traffic volume per lane (continued)

Intersections	Upstream nodes	Upstream-node locations	Traffic volume (demand pattern 1), unit/hour/lane			Traffic volume (demand pattern 2), unit/hour/lane		
			Left	Through	Right	Left	Through	Right
10	27	South of an intersection	9.0	2.0	5.0	9.0	2.0	5.0
	7	West of an intersection	10.0	16.7	10.0	10.0	16.7	10.0
13	20	North of an intersection	2.0	2.0	8.7	2.0	21.0	13.0
	15	East of an intersection	2.0	8.7	2.0	2.0	8.7	2.0
	28	South of an intersection	8.7	2.0	2.0	8.7	2.0	2.0
	10	West of an intersection	7.0	14.0	7.0	7.0	14.0	7.0

Table 3.4 Expected v/c ratios calculated from Synchro 5

Intersections	Upstream nodes	Upstream-node locations	Expected v/c ratio (demand pattern 1)			Expected v/c ratio (demand pattern 2)		
			Left	Through	Right	Left	Through	Right
4	17	North of an intersection	0.6	0.2	0.2	0.6	0.2	0.2
	7	East of an intersection	0.4	0.8	0.4	0.4	0.8	0.4
	25	South of an intersection	0.2	0.2	0.8	0.2	0.2	0.9
	2	West of an intersection	0.2	1.0	0.2	0.2	1.1	0.2
7	18	North of an intersection	0.6	0.2	0.5	0.6	0.2	0.5
	10	East of an intersection	0.5	1.1	0.6	0.5	1.1	0.6
	26	South of an intersection	0.4	0.2	1.0	0.5	0.2	1.1
	4	West of an intersection	0.3	1.1	0.4	0.3	1.1	0.4
10	19	North of an intersection	0.4	0.2	1.0	0.5	0.2	1.1
	13	East of an intersection	0.3	1.1	0.4	0.3	1.1	0.4
	27	South of an intersection	0.6	0.2	0.5	0.6	0.2	0.5
	7	West of an intersection	0.5	1.1	0.6	0.5	1.1	0.6
13	20	North of an intersection	0.2	0.2	0.8	0.1	1.0	0.7
	15	East of an intersection	0.2	0.9	0.2	0.2	1.1	0.2
	28	South of an intersection	0.6	0.2	0.2	0.6	0.2	0.2
	10	West of an intersection	0.4	0.8	0.4	0.4	1.0	0.6

3.1.4 Summary of Scenarios

After saturated demand based on the unsynchronized fixed time control method for each pattern was gotten, five scenarios for demand-pattern 1 and five scenarios for demand-pattern 2 was developed by varying demand levels. There were the total of 10 scenarios to be tested. For each demand pattern, the five scenarios were different by 20% of the saturated demand level and the scenarios to be tested had levels of saturation, 60%, 80%, 100%, 120%, and 140%, as summarized in the table.

Table 3.5 Scenario summary

Scenarios	Demand-Patterns	% Saturation
1	1	60
2	1	80
3	1	100
4	1	120
5	1	140
6	2	60
7	2	80
8	2	100
9	2	120
10	2	140

Using Synchro 5 simulation program, expected levels of service for saturated condition in scenario 3 and scenario 8 were shown in the table as examples. The full list of LOS for each scenario was presented in Appendices. The method of getting each saturated level of demand was described in the part of test method.

Table 3.6 Expected levels of service for type-1 control (100% saturation)

Intersections	Upstream nodes	Upstream-node locations	Levels of service (demand pattern 1)			Levels of service (demand pattern 2)		
			Left	Through	Right	Left	Through	Right
4	17	North of an intersection	A	C	C	A	C	C
	7	East of an intersection	A	C	B	A	C	B
	25	South of an intersection	A	C	C	A	C	C
	2	West of an intersection	A	E	C	A	E	C

Table 3.6 Expected levels of service for type-1 control (100% saturation) (continued)

Intersections	Upstream nodes	Upstream-node locations	Levels of service (demand pattern 1)			Levels of service (demand pattern 2)		
			Left	Through	Right	Left	Through	Right
7	18	North of an intersection	A	C	C	A	C	C
	10	East of an intersection	A	D	B	A	D	C
	26	South of an intersection	A	C	E	A	C	F
	4	West of an intersection	A	E	C	A	E	C
10	19	North of an intersection	A	C	E	A	C	F
	13	East of an intersection	A	E	C	A	E	C
	27	South of an intersection	A	C	C	A	C	C
	7	West of an intersection	A	D	B	B	D	C
13	20	North of an intersection	A	C	C	A	E	C
	15	East of an intersection	A	D	C	A	F	C
	28	South of an intersection	A	C	C	A	C	C
	10	West of an intersection	A	C	B	A	E	C

3.2 Signal Control Method to Be Tested

To measure an effectiveness of the newly-developed control, there should be some other traffic control methods of for comparison. The selected method was the fixed-time method. The reason of choosing the fixed-time method was that it usually used in a lot of comparative study. This meant that comparing the test result to this method was to indirectly compare with many others.

By the way, it was not so fair to compare only a classic fixed-time control with a control method that avoided green-time loss. A modified method based on the fixed-time one was another method that should be more appropriate for the test. The following were three types of signal-control methods to be tested and compared.

1) Type-1 control, unsynchronized fixed-time control, was the control a method that used constant green-time for each phase, fixed cycle length, and also fixed phase order. The priority for each phase was graphically described in Figure 3.3. When each leg got green light, all movements of the leg got the priority to pass the intersection while all other movements of three other legs get red signal. Green time was determined by using Webster method with 3-second phase amber time, no all-red time, and the cycle-length upper limit of 400 seconds. For the case of having less demand then the capacity, cycle length was also determined using Webster method but it was set to be 400 seconds for the case that the intersection was oversaturated. Because there were some differences in demand at each intersection, optimal cycle length could vary among the four intersections and there were no fixed offset between each pair of intersections. Because it was one of the common controls that were compared to many other methods, it should be practical to use this kind of traffic signal control to evaluate advantages or disadvantages of this method.

2) Type-2 control, synchronized fixed-time control, was the control a method that used constant green-time for each phase, fixed “common” cycle length, and fixed phase order. Using common cycle length, it was possible to set an offset between each pair of connected intersections in order to improve flow rate of the major path. To get the value of green split, common cycle length, and offset, the computer program, Synchro 5, was selected to be the tool for optimize the control.

3) Type-3 control, fixed-time control with green-time-loss prevention, was the signal-control method that gets the base control from the first type but if there is no vehicle traveling on the leg or no vehicle was able to pass an intersection for a while, which is 10 seconds in this case, from the leg that was getting green signal, the signal for the link would change to red and that phase was ended though the actual green time given to the phase was

still less than that in the plan. This method tended to eliminate most of green-time loss caused by giving green signal to the leg that produced no throughput. For an appropriate condition, it removed most losses of green time and, with this loss reduction and fixed phase priorities, overall throughput mainly depended on only priority given to each leg.

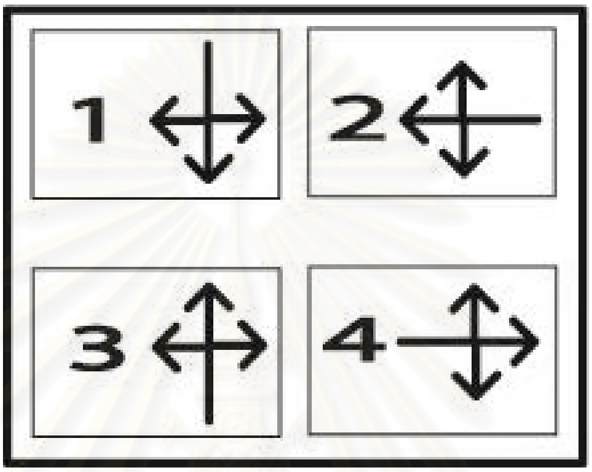


Figure 3.3 Phase setting and phase order to be used in the test

4) Type-4 control, throughput-maximizing control, was the newly-developed control method to be tested and compared with other three methods. It aimed to minimize total used time by giving the real-time appropriate non-major-conflict combination of priority allocation that gave the maximum expected throughput. Following equation showed an approach expected-throughput function in terms of two main parts of the throughput.

$$TP_i = TP1_i + TP2_i \quad (3.1)$$

where

TP_i = expected network throughput when a unit of green time is given to a movement i

$TP1_i$ = expected throughput when a unit of green time is given to movement i considering only the estimated time to arrive their destinations (throughput from the traffic departing from the movement *i*)

$TP2_i$ = expected throughput a unit of green time is given to movement i considering only the amount of upstream vehicles that arrive their destinations during the time that queue is being filled up and the expected time that those vehicles use for the leaving (throughput from traffic coming from the upstream nodes destined at a sink)

Total expected throughput for each combination was the summation of all expected values of approach throughput corresponding to the approach that got the priority of passing an intersection.

$$TTP_A = \sum TP_a \tag{3.2}$$

where

TTP_A = total network throughput when the movement combination (phase) *A* is given green time

a = a member of movements which traffic move in the same green period

For this network, there were many non-conflict combinations of priority and much more non-major-conflict combinations. Therefore, it was not practical to compare the result from testing type-4 control to results from other control methods without using the same limitation of arranging phase priority combinations.

So, each phase of type-4 control in this study would be limited not to differ from the set of 4 phases used with other three controls but there would be neither fixed timing nor fixed order and the green signal would stop if no vehicles could pass the link within 10 seconds.

Because left, right, and through movements gave different rates of transferring vehicles from an upstream link to a connected downstream link at an intersection, the factors of 0.85 and 0.95, recommended by Garber and Hoel (2002), were used for adjusting expected saturated flow of left and right movement respectively.

The average gap of around 6 meters (density of 0.15 vehicles per meter) was set to be the limit (critical gap) to identify if there was any effect of queue filling up. If there was a longer average gap than this critical gap, it meant that though no vehicle is released from the link, some others are still able to join the queue.

Without any restriction, it was possible to have the situation that the major legs got green every time there were only few vehicles arrived. The phase might change very quickly to release only few vehicles per green time. It meant that there would be so much lost time during the frequent phase changing and also some reduction of the saturated flow. This situation rarely occurred in the case of having fixed phase order likely in the case of using the first three types of control. To prevent this problem, the minimum limit of 25 queuing vehicles was set. The queue shorter than this tended not to get the next green time but if all legs had the queues shorter than this, the leg that was expected to produce the highest throughput will be released.

In this case, the maximum red time is set to be 400 seconds to prevent too long waiting time on each leg and the green time for each phase must be at least 10 seconds.

Finally, because the control was expected to work efficiently on a congested network or at least the network with some traffic, a fixed-time control was used during the starting period of each run. It meant that this type of control should be applied at the peak hour. In this test, the first method of control was used and it was change to the third one at the seventieth minutes of the simulation.

In details, the two parts of the throughput function could be described as following.

To calculate the first part, travel time for each link had to be estimated from link travel time used by last ten vehicles passing the link. Then, both the probability of a vehicle choosing each path to the destination after leaving the intersection and also travel time of the path had to be estimated. This possibility was estimated from the number of vehicles expected to choose each direction at each intersection and the number of vehicles that was expected to have each link as the destination. The function could be mathematically written as shown in the following equation.

$$TP1_j = \sum^{k} \frac{S_j PP1_{jk}}{PTT1_{jk}} \tag{3.3}$$

where

$PP1_{jk}$ = the probability that a vehicle chooses path k to the destination after leaving the intersection from movement j

$PTT1_{jk}$ = the travel time on path k to the destination after leaving the intersection from movement j

S_j = the saturation flow rate corresponding to movement j

$PP1_{jk}$ could be estimated using the probability of choosing each direction at each intersection to make the path connected. By multiplying them together, the result was the probability of choosing the path.

To determine the probability of choosing each direction at each intersection or terminate the trip, number of vehicles that was expected to make each move or exit the system by using each link had to be estimated by using the statistical data (in this case it was estimated from the OD table) and this statistical data had to be subtracted by the real number of vehicles that already passed an intersection or exit the system by using the link to be the number of vehicles waiting to make the move or terminate the trip by using each link respectively. Then, the ratio of this number of vehicles waiting to make the move or terminate the trip to the number of vehicle passing the point was estimated to be the probability of choosing each direction at each intersection or terminate the trip respectively.

Because it was possible to have some uncertainty, it was not so practical to use the constant value of the proportions. When a vehicle passed each intersection or arrived the destination on each link, the statistics would be updated to be real time. This meant that though the program seems to control signals dynamically every time step of 0.2 second but, in fact, it tried to change or maintain the control when there was a transferring of vehicle.

There were many different ways to estimate the value of $PTT1_{jk}$ which was the time used by a vehicle to exit the system with each path. This study used the link travel time averaged from the last ten vehicles passing each link.

For the second part, $TP2_{m}$, the travel time of each path and the possibility of choosing the path also had to be estimated but, for this case, there were some differences. The travel time in this case was not directly measured from the field. Because this part of throughput considered an effect of queue filling up, the time used in this case was the estimated time that shockwave of released vehicle moved backward from the end of a link to the tail of a queue. As described, if the lane had too short queue defined by the minimum free flow lane density, the throughput would be considered to be very low because there was no need to release any vehicle to create a space to be filled up. In this case, a vehicles was assumed to arrive its destination as soon as it reached the link corresponding to the destination zone because Paramics terminated a vehicle when it reached the boundary of the zone. The function could be mathematically written as shown in the following equation.

$$TP2_m = S_m\left(\frac{N2_m - \sum^n N2^n}{BTT2_m}\right) + S_m\left(\sum^n\left(\left(\frac{PP2^n_m}{1-PL2_m}\right)\left(\frac{N2^n}{BTT2_m + \frac{S_m N2^n}{TP2^n}}\right)\right)\right) \quad (3.4)$$

where

$N2_m$ = total number of vehicles expected to leave the system during the queue filling up due to a small unit of vehicles entering an intersection by using movement m

$BTT2_m$ = an expected time that an effect of releasing a vehicle from the head of the queue entering an intersection using movement m is transported to the back of the queue

$PP2_m^n$ = the probability that a filling vehicle comes to the link corresponding to movement m from movement n

$PL2_m$ = the proportion of vehicles that arrive the link corresponding to movement m have the destination zone located on the link

The first term represented the throughput from having an amount of vehicles leaving the system while filling up the link and the second one represented the throughput from having an amount of vehicles leaving the system while filling up the upstream links. It was the second part of throughput on the link corresponding to the movement. The second term transforms the throughputs of upstream links to a time to fill up the queues and transform it back after adding the time used on the link being considered. The value of $(PP2_m^n)(1 - PL2_m)^{-1}$ is used for distributing the available space to each approach. It also included an effect of having some vehicles leaving the system while the space was being filled up.

In this study a small unit of vehicle was set to be 1 vehicle. The number of vehicles expected to leave the system during the queue filling up, $N2_m$, could be estimated by using the following equation.

$$N2_m = \frac{1}{1-PL2_m} - 1 \quad (3.5)$$

$BTT2_m$ could be estimated by using the following equation

$$BTT2_m = \frac{queue\ length\ corresponding\ to\ the\ movement\ with\ an\ index\ of\ m}{average\ shockwave\ speed} \quad (3.6)$$

To estimate $PP2_m^{\ n}$, number of vehicles that was expected to make each move could be estimated by using the statistical data extracted by the detected number of vehicles that already made each move. The probability that a filling vehicle came to the link from each upstream link could be estimated from the ratio of number of vehicles that came from this upstream link by the approach with index of m to the total number of vehicles that were expected to come from all upstream links.

For $PL2_m$, it could also be estimated by using the statistical data extracted by the detected number of vehicles that already exited the system. It was calculated by dividing the value of number of vehicle waiting to exit the system on the link by the total number of vehicles waiting to enter the link.

For other complicate cases, these functions to estimate throughput should be adapted according to the network characteristic.

Figure 3.4 showed the flow of control using the throughput-maximizing method.

3.3 Sample of Calculation for Throughput-Maximizing Traffic Control Method

Because throughput-maximizing traffic control method was the new method to be tested, it was important to have a sample of calculation in order to make it easy to understand. To simplify the sample, a small network with two connected link was selected. Some different condition was set up to show how this type of traffic signal control did to the system.

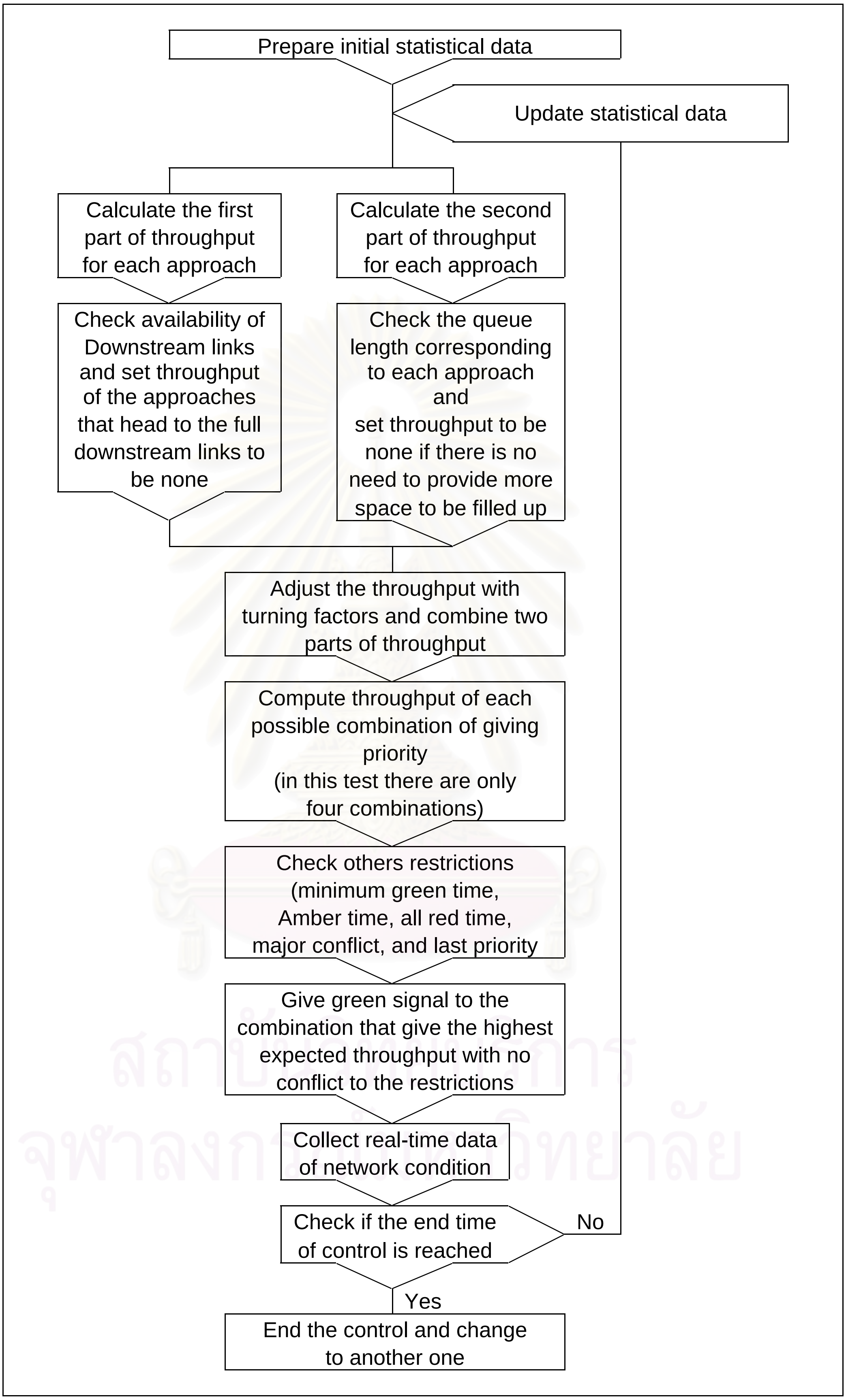


Figure 3.4 Flow of control using the throughput-maximizing method

3.3.1 Network

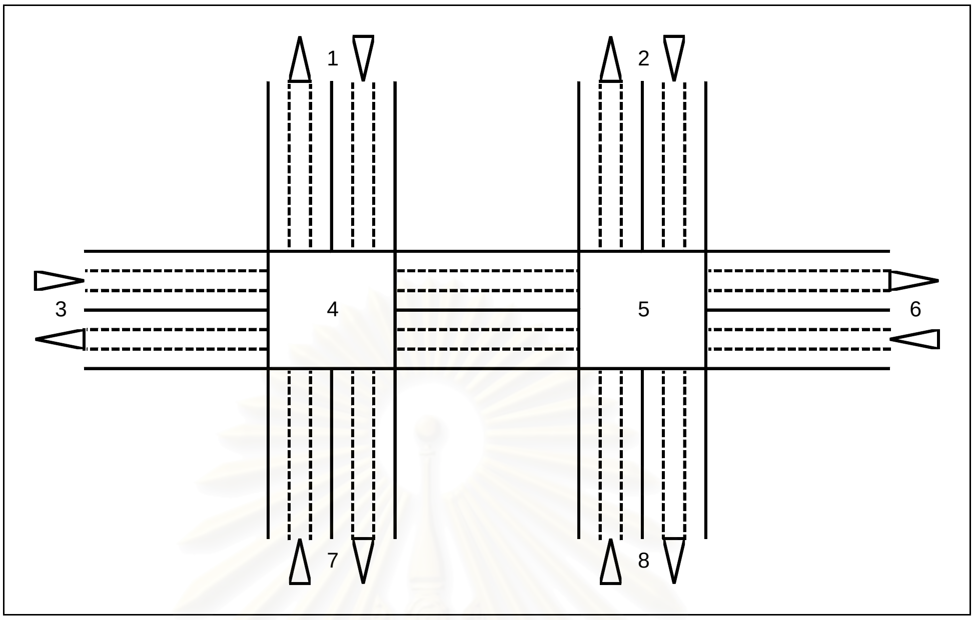


Figure 3.5 Network for sample of calculation

A road system with two intersections was appropriate for this sample because it was a simple road section.

Each link had 3 lanes with the length of 1 kilometer. At an intersection, the left lane, the middle lane, and the right lane were for left, through, and right movement respectively. The numbers in the figure were node names. Each link used the name of the starting node and the ending node for its identification.

On each link, there was only one exit point near the upstream node of the link. Averagely, after entering a destination link, a vehicle traveled on the link for 5 more second before exiting the link. There was no exit point at any node.

The capacity of each link was estimated to be 500 vehicles. Amber time was 3 seconds. There was no all red time in both intersections. Shockwave speed (back of the queue) of a stationary queue was 1000 vehicles per hour.

Left-turn saturation flow, through saturation flow, and right-turn saturation flow were 1600 vehicles per hour, 1900 vehicles per hour, and 1800 vehicles per hour respectively.

All vehicles entered the system from only the outer nodes, 1, 2, 3, 6, 7, and 8.

3.3.2 Traffic Demand

Figure 3.6 showed turning volume in vehicles per hour. The number shown in each lane of each leg was the traffic volume corresponding to the turning restriction of the lane.

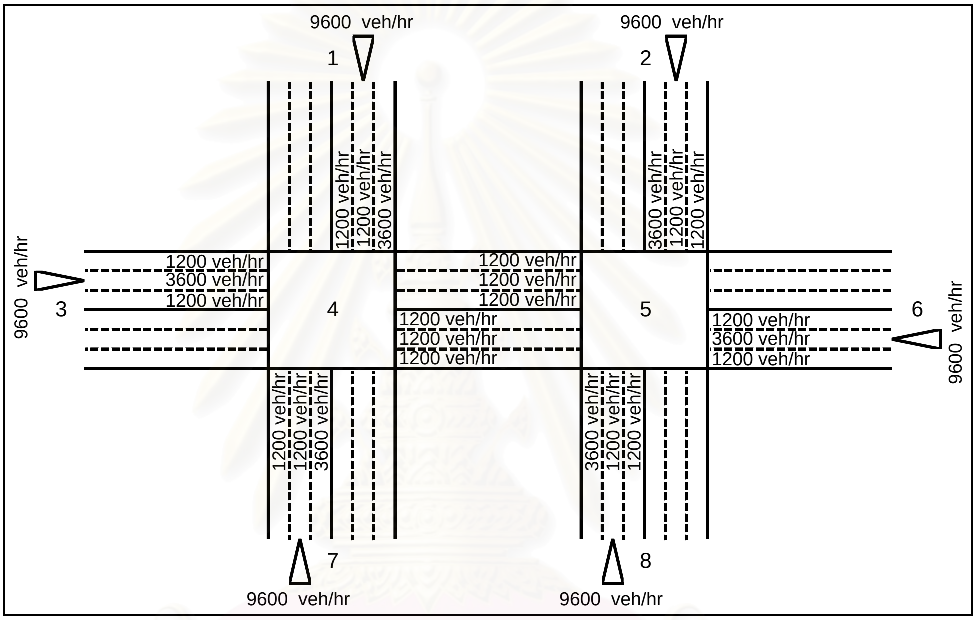


Figure 3.6 Turning volume

From the turning volume and the restriction, hourly number of vehicles that were expected to exit the system on each link was calculated in Figure 3.7.

For example, hourly number of vehicles that were expected to exit the system on link 4:5 was equal to (3600 + 3600 +3600) veh/hr – (1200 + 1200 + 1200) veh/hr.

3.3.3 Initial Condition

The initial condition was defined only to show the simplified calculation. It might not reflect any real situation.

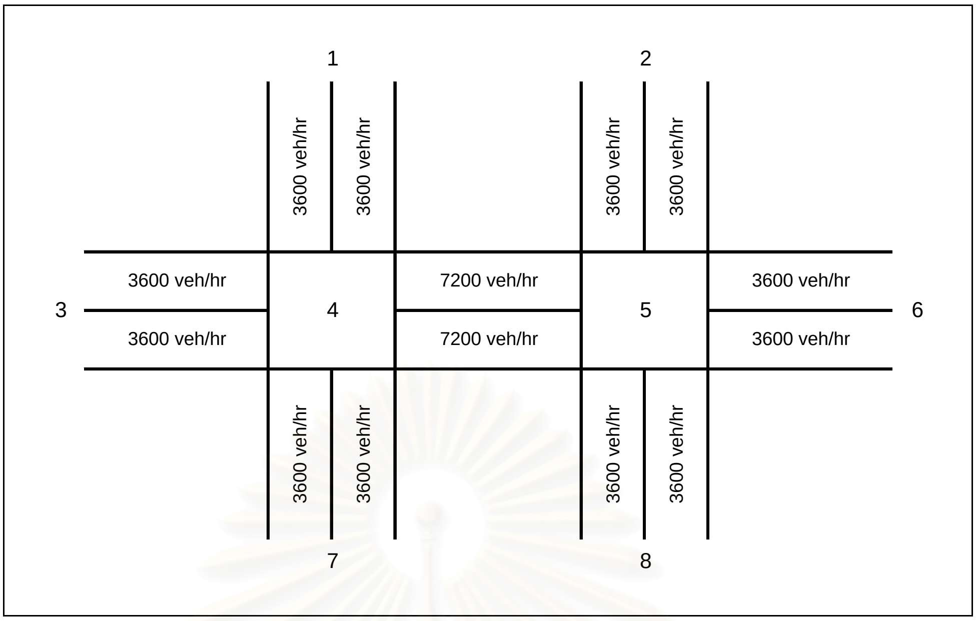


Figure 3.7 Rate of vehicles that are expected to exit the system on each link

With this initial condition, number of vehicles passing each intersection was 0. The initial time was also 0 second.

3.3.4 Calculation

As an example, one simplified case was setup in order to show the simplified steps of calculation.

The condition in this case was shown in the table.

Table 3.7 Traffic condition

Upstream nodes	Downstream nodes	Time (second)	Queue length (vehicles per lane)	Link travel time (s)	Number of vehicle that have exited the link since the start time			
					Left	Through	Right	Sink
1	4	1200	100	600	900	200	300	600
5	4		160	800	300	200	300	1200
7	4		100	600	300	200	900	600
3	4		100	600	300	600	300	600
2	5		160	500	300	200	900	600
6	5		100	500	300	600	300	600
8	5		100	500	900	200	300	600
4	5		100	500	300	200	300	1200
4	1		0	70	-	-	-	600
4	7		0	70	-	-	-	600
4	3		0	70	-	-	-	600
5	2		0	70	-	-	-	600
5	6		0	70	-	-	-	600
5	8		0	70	-	-	-	600

The first step was to find the average time that a vehicle used in exiting the network after it had been released. After one vehicle had been released from each approach of each leg, it could choose many paths with different probabilities. Total amount of vehicles that was expected to make a turn could be estimated from the turning volumes and the rate of vehicles that was expected to exit the system in each link. The probabilities could directly be calculated from the amount of vehicles that were waiting to go straight or turn at each intersection and the amount of vehicles that were waiting to exit the system on each link. Then, the possibility of choosing each connected path was calculated by using these proportions. Each path travel time was the summation of all link travel times along the path. By weighting the path travel time with the proportion, average time that a vehicle used in exiting the network after it had been released could be calculated.

Table 3.8 Decision of road users to exit each link

Links	Number of vehicles expected to exit the link for the case of having no delay				Number of vehicles waiting to exit the link				Proportions			
									At an intersection			From entered traffic
	Left	Through	Right	Sink	Left	Through	Right	Sink	Left	Through	Right	Sink
1:4	1200	400	400	1200	300	200	100	600	0.500	0.333	0.167	0.500
5:4	400	400	400	2400	100	200	100	1200	0.250	0.500	0.250	0.750
7:4	400	400	1200	1200	100	200	300	600	0.167	0.333	0.500	0.500
3:4	400	1200	400	1200	100	600	100	600	0.125	0.750	0.125	0.429
2:5	400	400	1200	1200	100	200	300	600	0.167	0.333	0.500	0.500
6:5	400	1200	400	1200	100	600	100	600	0.125	0.750	0.125	0.429
8:5	1200	400	400	1200	300	200	100	600	0.500	0.333	0.167	0.500
4:5	400	400	400	2400	100	200	100	1200	0.250	0.500	0.250	0.750
4:1	-	-	-	1200	-	-	-	600	-	-	-	1.000
4:7	-	-	-	1200	-	-	-	600	-	-	-	1.000
4:3	-	-	-	1200	-	-	-	600	-	-	-	1.000
5:2	-	-	-	1200	-	-	-	600	-	-	-	1.000
5:6	-	-	-	1200	-	-	-	600	-	-	-	1.000
5:8	-	-	-	1200	-	-	-	600	-	-	-	1.000

For example, the number of vehicles expected to turn left at the end of the link 1:4 for the case of having no delay was equal to (3600 veh/hr)(1200 seconds) (1 hr / 3600 seconds).

Number of vehicles waiting to turn left at the end of the link 1:4 was equal to (1200 – 900) vehicles.

The proportion of number of vehicles waiting to turn left at the end of the link 1:4 to the total amount of vehicles waiting to enter the intersection 4 was 300 veh / (300 + 200 + 100) veh.

Table 3.9 Probability of choosing each path after exiting downstream intersection of each starting link

Starting	Ending links													
links	1:4	5:4	7:4	3:4	2:5	6:5	8:5	4:5	4:1	4:7	4:3	5:2	5:6	5:8
1:4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.750	0.000	1.000	1.000	0.063	0.125	0.063
5:4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.000	1.000	1.000	0.000	0.000	0.000
7:4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.750	1.000	0.000	1.000	0.063	0.125	0.063
3:4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.750	1.000	1.000	0.000	0.063	0.125	0.063
2:5	0.000	0.750	0.000	0.000	0.000	0.000	0.000	0.000	0.063	0.063	0.125	0.000	1.000	1.000
6:5	0.000	0.750	0.000	0.000	0.000	0.000	0.000	0.000	0.063	0.063	0.125	1.000	0.000	1.000
8:5	0.000	0.750	0.000	0.000	0.000	0.000	0.000	0.000	0.063	0.063	0.125	1.000	1.000	0.000
4:5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.000	1.000	1.000
4:1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4:7	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4:3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5:2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5:6	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5:8	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

For example, the probability of choosing each path to exit the system on the link 5:2 after exiting downstream intersection of the link 1:4 was equal to (1.000-0.750) (0.250) (1.000).

Table 3.10 Travel time of each path, second

Starting	Ending links													
links	1:4	5:4	7:4	3:4	2:5	6:5	8:5	4:5	4:1	4:7	4:3	5:2	5:6	5:8
1:4	0	0	0	0	0	0	0	5	0	5	5	505	505	505
5:4	0	0	0	0	0	0	0	0	5	5	5	0	0	0
7:4	0	0	0	0	0	0	0	5	5	0	5	505	505	505
3:4	0	0	0	0	0	0	0	5	5	5	0	505	505	505
2:5	0	5	0	0	0	0	0	0	805	805	805	0	5	5
6:5	0	5	0	0	0	0	0	0	805	805	805	5	0	5
8:5	0	5	0	0	0	0	0	0	805	805	805	5	5	0
4:5	0	0	0	0	0	0	0	0	0	0	0	5	5	5
4:1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4:7	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4:3	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5:2	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5:6	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5:8	0	0	0	0	0	0	0	0	0	0	0	0	0	0

For example, travel time of the path from the link 1:4 to the link 5:2 was equal to (500 + 5) seconds.

Table 3.11 Average time that a vehicle uses in exiting the network after it is released

Links	Average time that a vehicle uses in exiting the network after it is released (second)		
	Left	Through	Right
1:4	130	5	5
5:4	5	5	5
7:4	5	5	130
3:4	5	130	5
2:5	5	5	205
6:5	5	215	5
8:5	205	5	5
4:5	5	5	5
4:1	0	0	0
4:7	0	0	0
4:3	0	0	0
5:2	0	0	0
5:6	0	0	0
5:8	0	0	0

For example, average time that a vehicle used in exiting the network after it had been release from the link 1:4 and turned left was equal to (0.750) (5 seconds) + (0.063) (505 seconds) + (0.125) (505 seconds) + (0.063) (505 seconds).

The second step was to estimate the first part of throughput. The first part of throughput could be calculated by dividing the saturation flow rate by the time that it was expected to use in exiting the system.

For example, the first part of throughput for letting 1 vehicles turn left was equal to 1600 vehicles per hour / 130 seconds.

Table 3.12 The first part of throughput

Links	The first part of throughput (vehicles/hour per second)		
	Left	Through	Right
1:4	1.1	380.0	360.0
5:4	320.0	380.0	360.0
7:4	320.0	380.0	1.3
3:4	320.0	1.3	360.0
2:5	320.0	380.0	0.7
6:5	320.0	0.8	360.0
8:5	0.6	380.0	360.0
4:5	320.0	380.0	360.0

The third step was to estimate the second part of throughput. In this example, there was only two links, 2:5 and 5:4, that had enough density which, in this case, was set to be more than 0.15 vehicles per meter. With the queue length of 160 vehicles per lane and the shockwave speed of 1000 vehicles per hour, it took 576 seconds to transfer effect of releasing a vehicle from an intersection to the back of the queue.

If one vehicle was released to an intersection the amount of vehicles to fill up the link from an upstream node should averagely be a little more because it was possible to have some destination arrival as soon as it entered the link.

Table 3.13 Decision of road users to enter each link

Links	Number of vehicles waiting to exit the link				Number of vehicles waiting to enter the link				Proportions		
	Left	Through	Right	Sink	Left	Through	Right	Total	Left	Through	Right
1:4	300	200	100	600	-	-	-	-	-	-	-
5:4	100	200	100	1200	300	600	300	1200	0.250	0.500	0.250
7:4	100	200	300	600	-	-	-	-	-	-	-
3:4	100	600	100	600	-	-	-	-	-	-	-
2:5	100	200	300	600	-	-	-	-	-	-	-
6:5	100	600	100	600	-	-	-	-	-	-	-
8:5	300	200	100	600	-	-	-	-	-	-	-
4:5	100	200	100	1200	300	600	300	1200	0.250	0.500	0.250
4:1	-	-	-	600	100	200	100	400	0.250	0.500	0.250
4:7	-	-	-	600	100	200	100	400	0.250	0.500	0.250
4:3	-	-	-	600	100	200	100	400	0.250	0.500	0.250
5:2	-	-	-	600	100	200	100	400	0.250	0.500	0.250
5:6	-	-	-	600	100	200	100	400	0.250	0.500	0.250
5:8	-	-	-	600	100	200	100	400	0.250	0.500	0.250

For example, the proportion of vehicles waiting to enter the link 5:4 by turning left is (300 vehicles) / (300 + 600 + 300) vehicles.

For the link 2:5, 50 percents of vehicles had this link as a destination. To fill a gap of 1 vehicle, it averagely took 2 vehicles from the upstream node. It meant that from this gap filling up, 1 vehicle was terminated.

This 1 vehicle took 576 seconds to be terminated. Because there was no upstream link, the second part of through put for this leg was 2.8 vehicles/hour per second ((1600 vehicles per hour)(1 vehicle / 1 vehicle) / 576 seconds), 3.3 vehicles/hour per second ((1900 vehicles per hour)(1 vehicle / 1 vehicle) / 576 seconds), and 3.1 vehicles/hour per second ((1800 vehicles per hour)(1 vehicle / 1 vehicle) / 576 seconds) for left, through and right movements respectively.

For the link 5:4, 75 percents of vehicles had this link as a destination. To fill a gap of 1 vehicle, it averagely took 4 vehicles from the upstream node. It meant that from this gap filling up, 3 vehicles were terminated. It produced the throughput equal to 8.3 vehicles/hour per second ((1600 vehicles per hour)(3 vehicles / 1 vehicle) / 576 seconds), 9.9 vehicles/hour per second ((1900 vehicles per hour)(3 vehicles / 1 vehicle) / 576 seconds), and 9.4 vehicles/hour per second ((1800 vehicles per hour)(3 vehicles / 1 vehicle) / 576 seconds) for left, through and right movements respectively.

Because these 4 vehicles per one-vehicle space filling up link 5:4 could come from link 2:5, link 6:5, and link 8:5 with the probabilities of 0.25, 0.50, and 0.25 respectively, 1 vehicle per one-vehicle space, 2 vehicles per one-vehicle space, and 1 vehicle per one-vehicle space came from link 2:5, link 6:5, and link 8:5 respectively. Because only the link 2:5 had enough queuing vehicles the other two was expected not to produce throughput.

With 1 vehicle from link 2:5 to fill up the one-vehicle space on link 5:4, 1 more vehicle was terminated in link 2:5. This termination occurred at 576 seconds after an effect had been transferred to the link or 1152 seconds (576 seconds + 576 seconds) after a vehicle had been released from link 5:4. It produced additional throughput of 1.4 vehicles/hour per second ((1600 vehicles per hour)(1 vehicle / 1 vehicle) / 1152 seconds), 1.6 vehicles/hour per second ((1900 vehicles per hour)(1 vehicle / 1 vehicle) / 1152 seconds), and 1.6 vehicles/hour per second ((1800 vehicles per hour)(1 vehicle / 1 vehicle) / 1152 seconds) for left, through and right movements at link 5:4 respectively.

The second part of throughput for link 5:4 was the summation of the second part of throughput occurred in itself and the second part of throughput occurred in all other upstream links. The value of this second part of throughput was 9.7 vehicles/hour per second (8.3 vehicles/hour per second + 1.4 vehicles/hour per second), 11.5 vehicles/hour per second (9.9 vehicles/hour per second + 1.6 vehicles/hour per second), 11.0 vehicles/hour per second (9.4 vehicles/hour per second + 1.6 vehicles/hour per second).

Total throughput was the summation of the two parts as shown in the table. One vehicle per hour per second throughput means that if one second of green time is given to the leg, the terminating rate of 1 vehicle per hour is expected to be produced.

Table 3.14 Total throughput produced by releasing 1 vehicle to an intersection

Links	Throughput (vehicles/hour per second)									
	The first part			The second part			Total			
	Left	Through	Right	Left	Through	Right	Left	Through	Right	Total
1:4	1.1	380.0	360.0	0.0	0.0	0.0	1.1	380.0	360.0	741.1
5:4	320.0	380.0	360.0	9.7	11.5	11.0	329.7	391.5	371.0	1092.2
7:4	320.0	380.0	1.3	0.0	0.0	0.0	320.0	380.0	1.3	701.3
3:4	320.0	1.3	360.0	0.0	0.0	0.0	320.0	1.3	360.0	681.3
2:5	320.0	380.0	0.7	2.8	3.3	3.1	322.8	383.3	3.8	709.9
6:5	320.0	0.8	360.0	0.0	0.0	0.0	320.0	0.8	360.0	680.8
8:5	0.6	380.0	360.0	0.0	0.0	0.0	0.6	380.0	360.0	740.6
4:5	320.0	380.0	360.0	0.0	0.0	0.0	320.0	380.0	360.0	1060.0

The result shown that throughput was expected to be maximized if green signals were given to leg 5:4 and leg 4:5 of intersections 4 and 5 respectively.

To change or not to change the phase was up to the present phase and other restrictions.

3.4 Test Procedure

To systematically test the traffic control methods, the test procedure was setup. The test was performed on simulation using Paramics micro-simulation program. Some assumptions and expected result were also included in this part because they were directly related to the test conditions and their efficiency.

3.4.1 Assumptions

Due to the complexity of traffic environments, this study posted some necessary assumptions and focused on the details of environments that affected the traffic signal performances. The assumptions were mainly associated with limited time and resource. There were

1) There was only one type of vehicles which was the passenger car.
2) Using default setting provided by Paramics program, the network required no calibration because the network was assumed to be a real network with some specific characteristics.
3) The time that travelers spent for waiting to enter the network had the same value as that the travelers spent when they are already in the network and throughput effectively reflected overall serviceability of the network.

4) Cost of time was the most important cost to be considered and other major costs were also directly related to this cost.

5) A road system with high level of congestion became a queuing network that the queuing time mainly contributed on each link travel time.

6) Real-time link travel time was equal to the time that the last set of vehicles traversed each link.

7) To make the system acted the same as an urban network in morning peak periods, all origin zones were located at the ends of outer links and the main destinations was located on some inner links. Some outer links connected to more than one inner link while other links were minor.

8) Result from testing in a simplified corridor could represent tendency of what should be derived from applying the same traffic-signal control method in other complex networks.

3.4.2 Testing Method

Firstly, the simulated network had to be filled with each level of demand and saved in different files ready to be used with any types of controlling. Then, each control method was applied to each network to get the effectiveness of each method for different level of congestion.

Testing period of 4 hour was used with one starting hour, one peak hour, and two ending hours (one with a half of peak demand and another one with no traffic demand). During the peak hour, which was the second hour, peak demand which was the target demand was produced. During the first hour and the third hour, entering rate was half of the peak demand. And in the last hour, there was no more traffic added into the road network. For demand-level 5 (140% of saturation), there was much more demand than the expected capacity, estimated from the case of using Webster method. In this case, some start up demand and ending demand were installed to produce some waiting queue and reflected the real behavior of queue development. But for other cases, there was smaller amount of peak demand which seemed not to be much more than the capacity and if there was any start up or ending demand, it should be so small and easily be terminated without causing any major change to the result.

One of the main MOEs selected to be directly collected was the relative saved loss-time during the 4-hour period of simulation. Using total lost time or delay was not practical because there was high possibility to had some vehicles trapped in the system when applying high level of demand and their delays could not be measured without keep running the simulation until all vehicles was gone. This not only took a lot of simulating time for each run but also caused more instability of outcomes resulted from this too long simulation period. Instead of collecting time that vehicles used, the better way was to assume that if vehicle could not get out of the system during the five-hour simulating period it losed the whole five hours and if they could get out before the referent time, the time that each vehicle saved can together be the relative saved lost-time that effectively reflected the output of the network using each type of traffic signal control.

Some other values were also collected every minute in order to represent real-time condition of the system for each control of each demand level. These values were number of vehicles that already leaved the system, average inner-link speed, and average outer-link speed.

From number of vehicles that already leaved the system, hourly throughput could be calculated. Peak-hour throughput was another main MOE. It could prove if the method really gave large throughput during the peak period as it aimed to.

Five simulation runs at each level of demand for each demand pattern using each type of traffic-signal control method would be used for collecting necessary data. If each set of data showed acceptable variation, no more experiment was required.

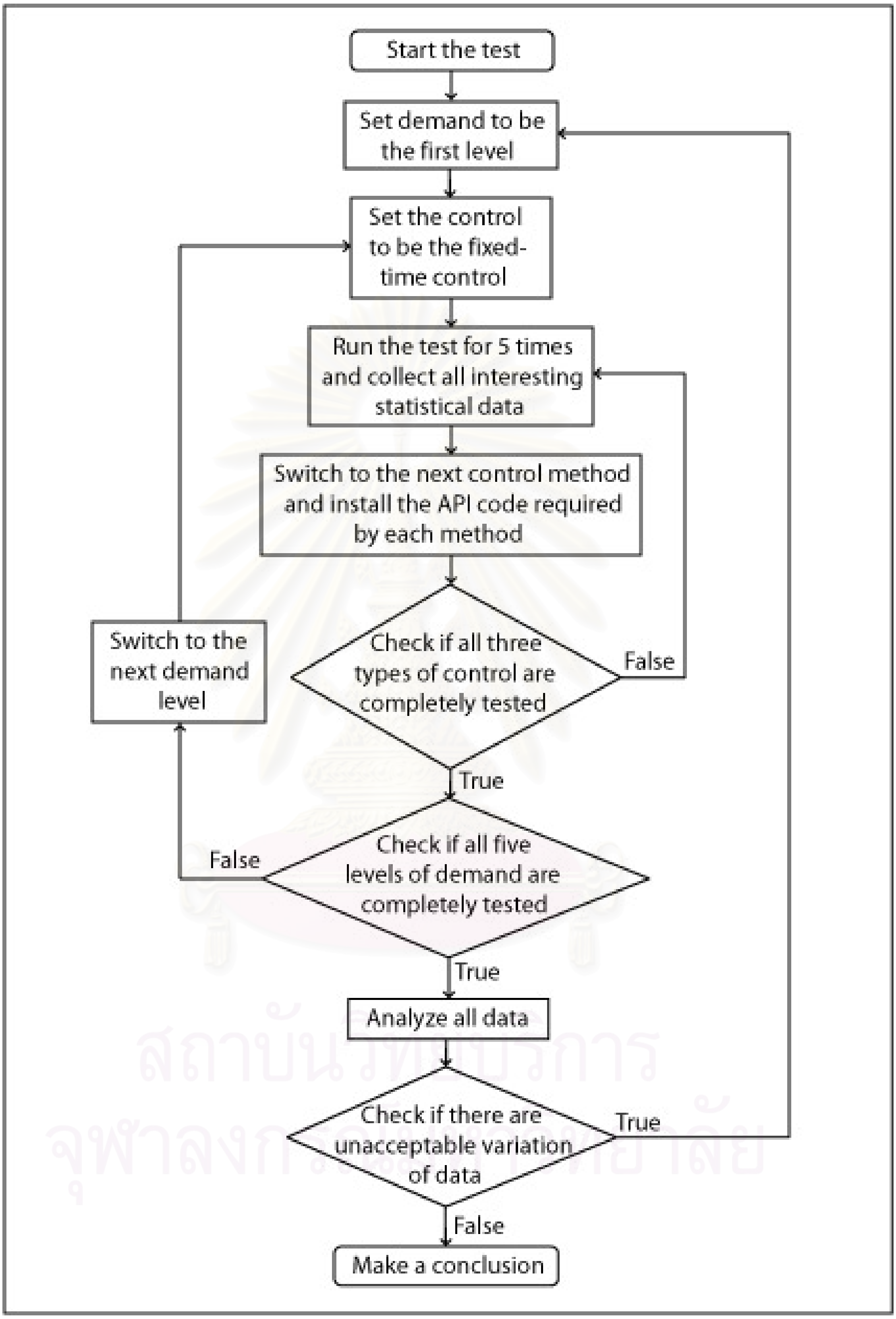


Figure 3.8 Flow of testing

Then, all data would be saved in text files with some systematic formats that could be opened by using MS excel program.

Finally, all data would be statistically analyzed and compared to get the reasonable conclusion of the whole work.

3.4.3 Result Comparison and Data Analysis

There were two parts of the analysis which were the part of real-time data analysis for tracking overall system condition and the part of statistically analyzing two main MOEs.

For the first part which was considered to be the less important, each of cumulative number of out vehicles leaving the network, average outer-link speed, average inner-link speed at each point of time with each demand level and each control, and some others would be averaged and plotted in the same chart with others tested with the same demand level and also data type. These charts would graphically show what happened to the system at each demand level when applying each type of control. For the start-up period, some link velocity would be 0 because there was no vehicle using the link. Then, at that period and also the period that most queues were cleared, it was not appropriate to compare the conditions by using these speed values without considering the effect of the lack.

Real-time throughput could also be estimated by considering destination-zone arrival during a small period, which was selected to be ten minutes.

For the second part, analyzing MOE statistics, both total travel time and also peak-hour throughput would be statistically analyzed to compare each pair of control's MOEs. For each demand level, the control type that gave less total travel time and higher peak-hour throughput tended to be appropriate for that level of demand. If the test of two MOEs showed confliction of results, total travel time was considered to be the major measure because the queue needed time to be developed so it was possible to have unsaturated condition at the beginning period of a peak hour.

For MOEs testing, F-test was used to test equity of sample variances and Student's t-Test was used for testing equity of the means with the same significant level of 0.01.

After the separately analysis, some relation among each set of interesting statistics also had to be considered in order to be able to describe the reason of what really happened during the simulation and why the result became different to what it should be if there was any unexpected result.

By the way, if there was any unacceptable result or the lack of data was proved to exist, some more additional test would be run with either old method or the adapted one.

3.4.4 Expected Result

From the objective of this project, type-4 control should work effectively on the oversaturated network.

But for other levels of demand, the basic concept of the throughput-maximizing control might not work better than others because it was designed for the congested road network.

CHAPTER IV

COMPARISON OF SIGNAL CONTROL METHODS

4.1 Introduction

In this chapter all, results from the tests described in Chapter III were analyzed to compare the effectiveness of signal control methods. Because there were two basic patterns of demand, the analyses were separated to two main parts according to demand types, which were the scenarios without cross demand and scenarios with cross demand.

Many indicators were used both to show network conditions occurring in each scenario and to compare the results.

Starting from the network conditions, some images showed the base condition for applying each scenario to the network with the fixed time control to graphically confirm the level of congestion due to each demand level at the end of the peak hour. The end of peak hour was selected because it was the time that was expected to have the highest number of vehicles containing in the system. Then, the number of vehicles containing in each of two parts of the system, inner links and outer links, at each point of time was shown to represent the average queue length and density on each part. The reason that both of them was not directly considered was that in some scenarios the queues could exceed outer links and this made it impractical to use those factors as indicators. Because having overall efficiency did not mean that all road users received the benefit, some of them might have to pay for others. It was important not to consider only overall benefit but also the equity given to all group of users. This equity could be reflected in the ratio of giving green time to each leg at an intersection. Theoretically, as we knew that the Webster's method distributed demand equally to all legs by considering the degree of saturation on those legs, other two methods could be compared to the result from this type of control to evaluate the equity resulted from them. Phase changing frequency on each intersection due to each type of control method was also in the consideration because it could represent both lost during amber time and the way that each method control the system.

Then, the main analysis started in the part of graphical analysis. Number of vehicles that already arrived at their destination at each point of time was graphically shown together with cumulative number of vehicles that already entered the system to show the rate of terminating traffic and the change in amount of trapped vehicles at each point of time. The weighted average real-time throughput was another MOE that directly reflected efficiency of each control. It could show the change in network capacity along the time line of simulation. While throughput showed the serviceability in the term of terminating rate, average speed showed it in the view of transferring rate. In this view, the difference was expected to be more noticeable than the small difference of throughputs. Average speed on inner links and average speed on outer links were shown separately to determine an effect of each control method to each part of the road system.

Finally, all methods of traffic signal control were statistically compared by using the value of total travel time during a peak hour and the value of peak throughput gotten form the maximum 30-minute throughput. The reason that the peak 30-minute throughput was selected was there were some transition periods before and after a peak hour. It was not practical to include these periods into the consideration.

Number of vehicles containing in each part of the system represented the policy that each type of traffic signal control used. Some control methods might give priority to inner links while some others might give it to the outer or both of them equally.

Average green time given to each link during a peak hour reflected the equity of green-time allocation. Though green time should be given to each leg by considering the demand ratio but it was possible for some control method to give additional priority to some legs.

Real-time throughput was assumed to be the rate of destination arrival. It was an important indicator because throughput was not constant during each period.

Real-time average speed was averaged from the speed of all vehicles on each type of links.

Total travel time was the summation of the time that all vehicles used in passing the network.

Peak 30-minute through put was the highest rate of exiting the system computed from each 30-minute interval. The reason of using 30 minute instead of an hour was that it was possible to have some effect of period transition especially for the case of type-4 traffic control method that used fixed-time control at the first hour.

4.2 Results for Scenarios without Cross Demand (Patern-1 Demand)

This section presented the results from the case of the first pattern of demand for the test.

4.2.1 Network Condition

Firstly, for the unsaturated level, the road systems filled with each same amount of demand were graphically illustrated in each of Figure 4.1 a) and 4.1 b).

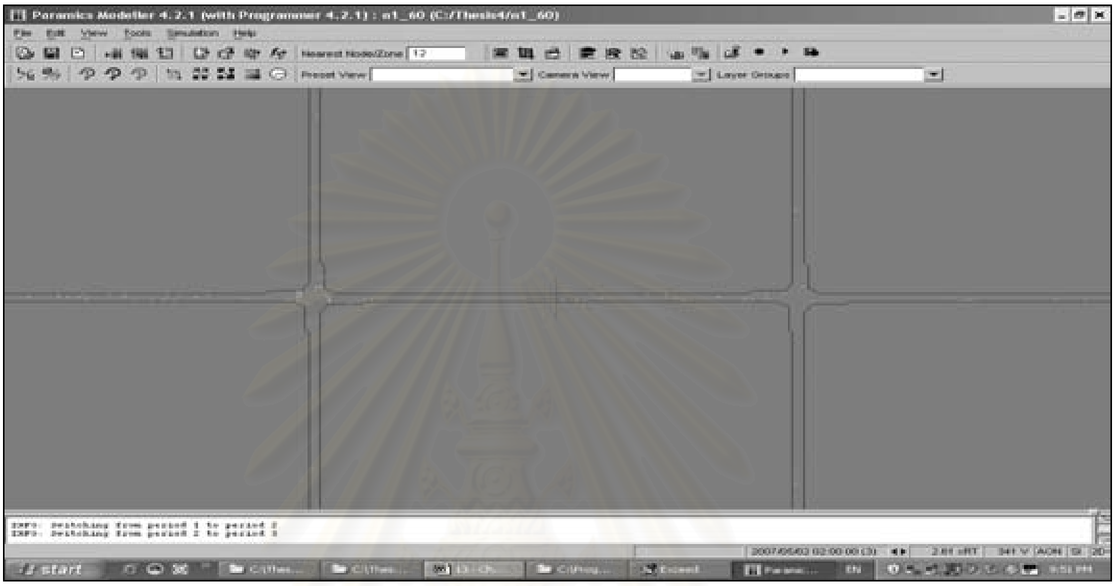
Level-1 and level-2 demands were planned at 60 percents and 80 percents of the saturated one in the peak hour, respectively. There were some very short queues occurring in both of them. Note that these scenarios reflected the condition that the demand was less than the capacity, an unsaturated condition.

Then, the system filled with each saturated level of demand was graphically shown in Figure 4.1 c).

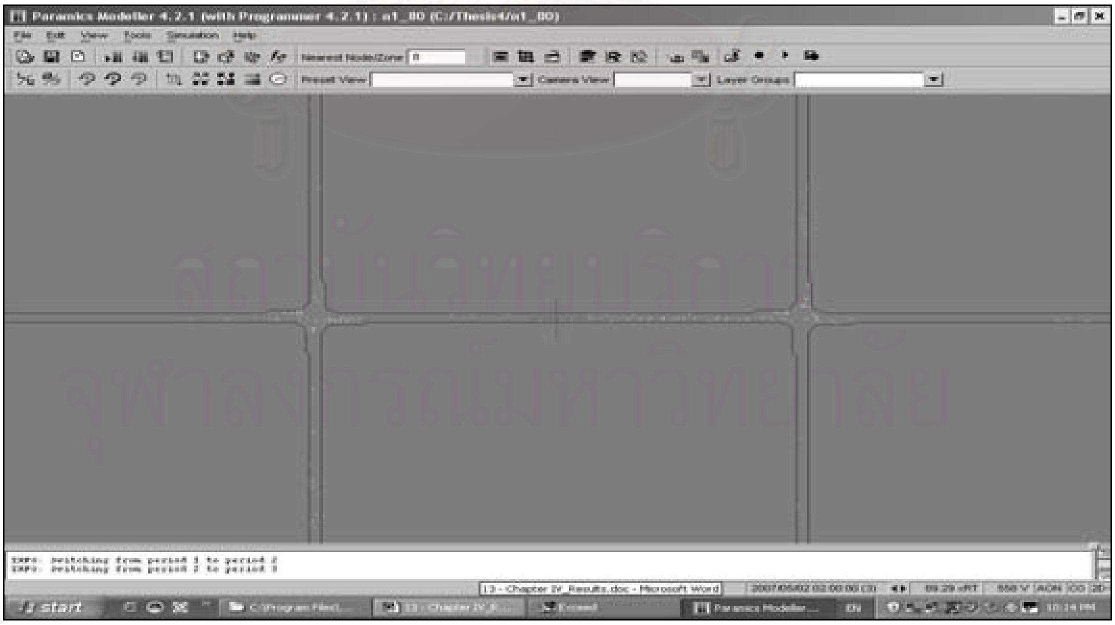
Level-3 demand was the saturated one. Some queues waiting for the next green time could be seen. This condition reflected the condition that the demand was almost equal to the capacity, a saturated condition.

The systems filled with oversaturated levels of demand were graphically shown in each of Figures 4.1 d) and 4.1 e).

Level-4 and level-5 demands were 120 percents and 140 percents of the saturated one in the peak hour, respectively. Some long queues were experienced both cases. The longer queues were seen in the latter case. Note that the queue grew and reached the highest length approximately at the end of the peak periods. The figures illustrated the condition that the demand exceeded the capacity, an oversaturated condition.

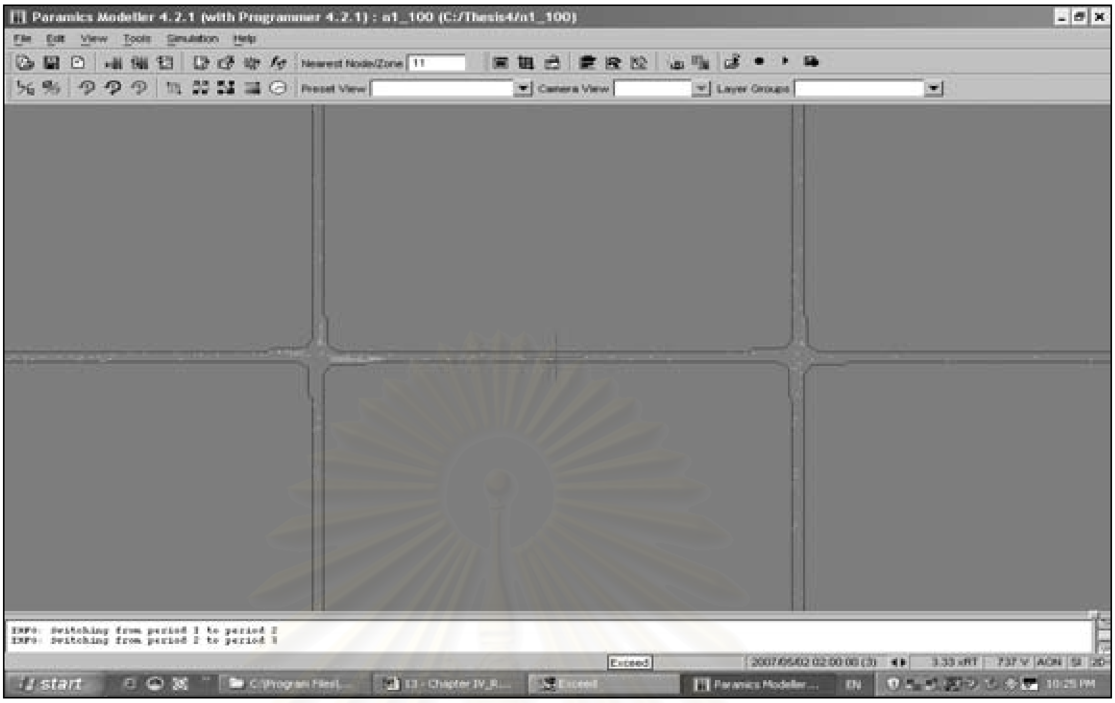


a) Demand-level 1 (unsaturated traffic condition)

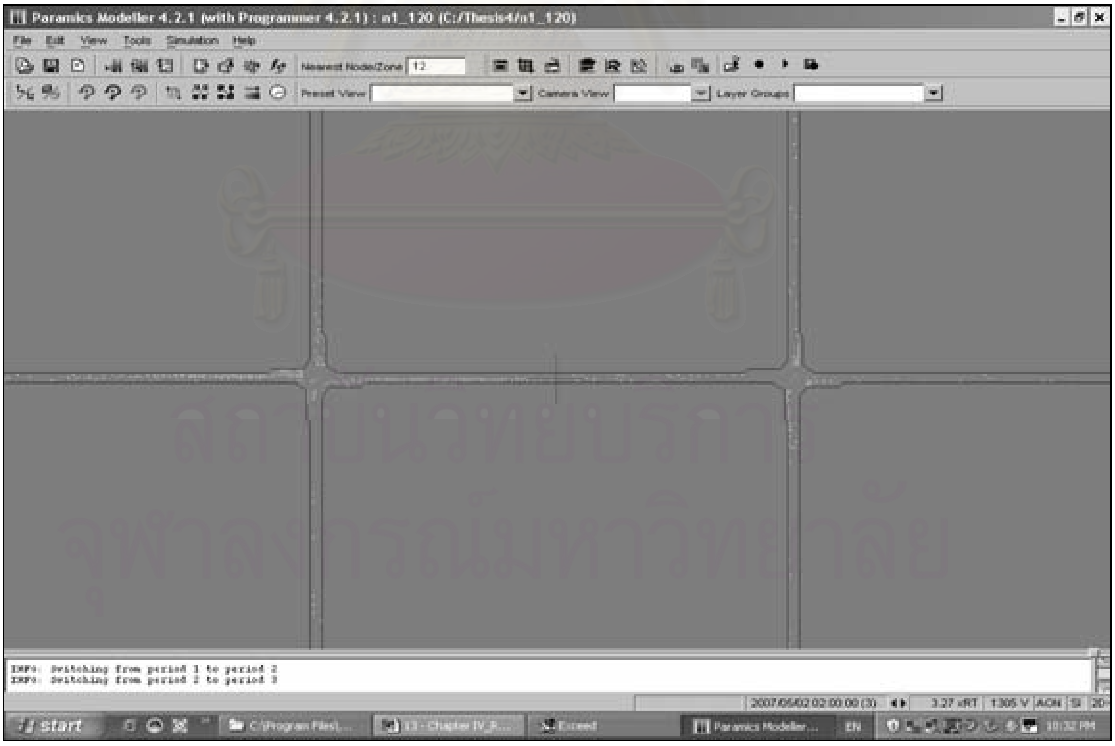


b) Demand-level 2 (unsaturated traffic condition)

Figure 4.1 Network conditions (pattern-1 demand)

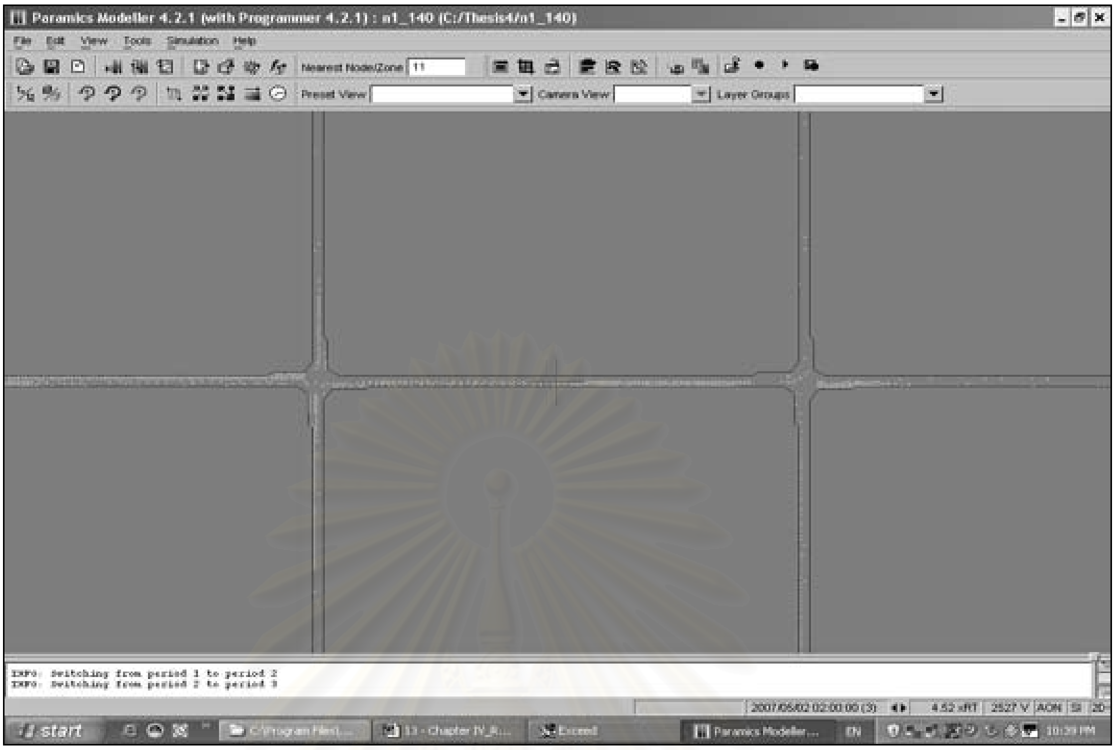


c) Demand-level 3 (saturated traffic condition)



d) Demand-level 4 (oversaturated traffic condition)

Figure 4.1 Network conditions (pattern-1 demand) (continued)



e) Demand-level 5 (oversaturated traffic condition)

Figure 4.1 Network conditions (pattern-1 demand) (continued)

Figure 4.1 was captured at the end of the peak hour when it was expected to have the highest average length of queues.

Figure 4.2 displayed the overall pictures of the performances of each control method, expressed by the ability to release (hold) the vehicles out of the road system. Figures 4.2 a) and 4.2 b) illustrated the number of vehicles in the road network, which was one of principle measures of effectiveness. The figures showed that type-4 control made the whole system with these unsaturated levels of demand carry the highest amount of vehicles, while the other three controls performed similarly. The figures showed that with low demand, the dynamic control (type-4) did not work so effectively, as the control held vehicles on the road longer and yielded a large number of vehicles on the network, not only in the peak period but also after the peak period when the queue accumulated during the peak time dissipated. Generally, the effectiveness was lower when the demand was lower (Figure 4.2 a) compared to Figure 4.2 b)).

Figure 4.2 c) showed the traffic control performance when the road was loaded at approximately capacity. The test controls performed differently from the unsaturated scenarios, in terms of where to hold the traffic and overall performances. It was seen from the figure that the type-1 and type-3 control allowed the greatest number of vehicles to store on the inner links during the peak hour. On the outer links, the type-2 and type-4 controls let more vehicles stay on the links than the other two methods during the peak hour. During the queue dissipation period, controls type-1, type-2, and type-3 performed similarly while control type-4 held many vehicles on both inner and outer links. Note that the control type-4 had a poor ability to release the traffic out of the links during the queue dissipation period, as observed on both inner and outer links.

Figures 4.2 d) and 4.2 e) showed the effectiveness of each control method in releasing traffic out of the road network in the oversaturated conditions. During these conditions, the effect of traffic control and effect of oversaturation (queue spillback) made different traffic patterns from the unsaturated and saturated traffic conditions, as indicated by different shapes of traffic congestion formulation on both inner and outer links. Compared to Figures 4.2 a) to 4.2 c), the number of vehicles on the links were increasing from the beginning until the end of peak time period.

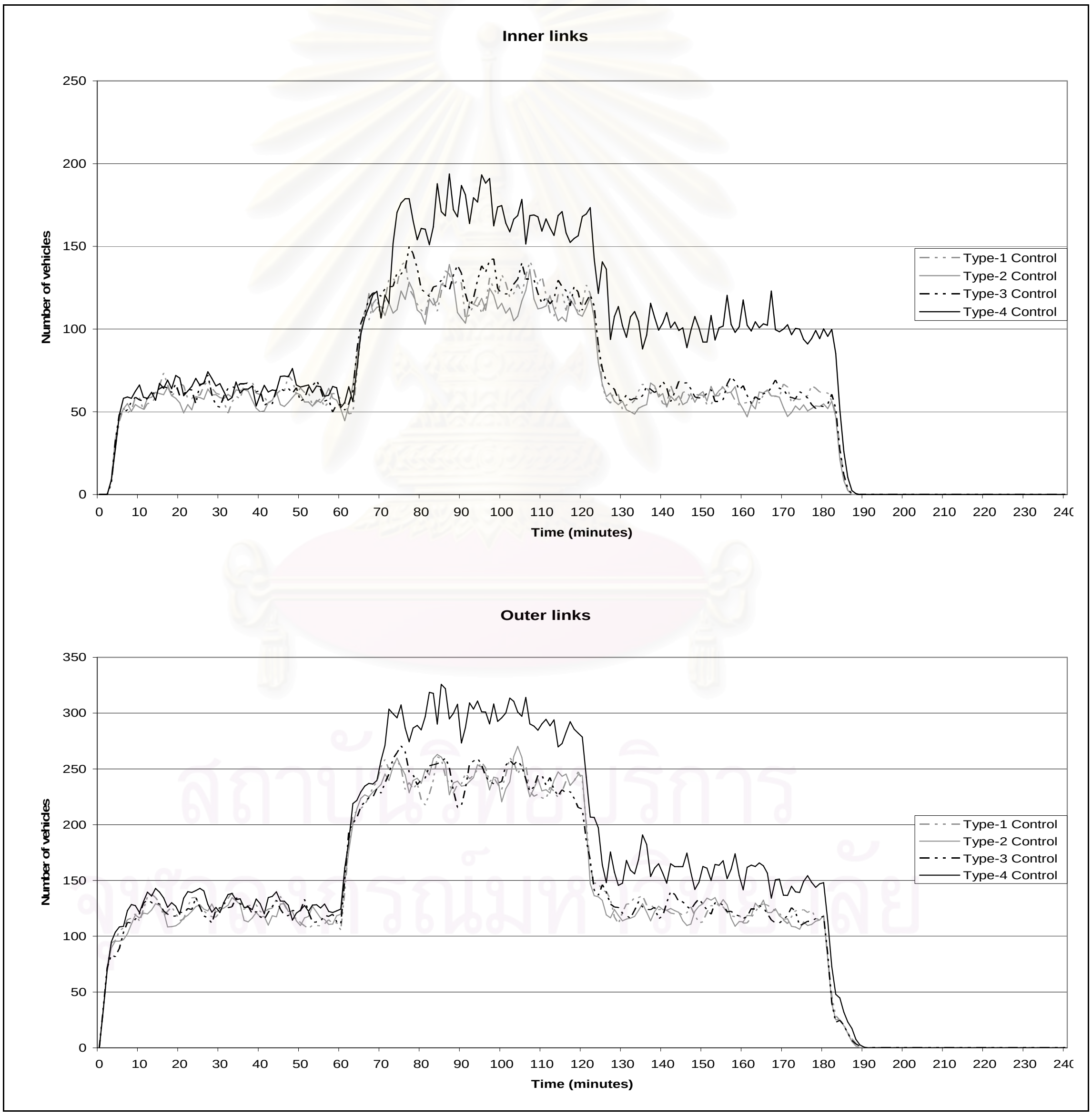
In these oversaturated traffic conditions, the differences in the effectiveness of traffic control were clearly seen. From the Figures, the type-1 and type-3 controls held a larger number of vehicles on the inner links, while the type-2 and type-4 controls let more vehicles stay on the outer links than the other two methods. The order of the control methods holding the vehicles on the inner links (from highest to lowest) was type-3, type-1, type-4, and type-2, respectively (both oversaturated demand levels from Figures 4.2 d) and 4.2 e)), while the order of the control methods holding the vehicles on the outer links were type-2, type-4, type-1, and type-3 in the 120 percent oversaturation case and type-4, type-2, type-1, and type-3 in the 140 percent oversaturation case. It implied from the figures that the types of traffic control placed significant effects on levels of congestion on each link. The type-3 and type-1 controls held a large number of vehicles on the inner links, making these links very congested during the peak period. The number of vehicles holding on the inner links was approximately as twice of those from type-4 and type-2 controls at the end of the peak period. Type-2 and type-4 controls held a larger number of vehicles on the outer links. Note that type-2 control held a larger number of vehicles on the outer links than type-4 control in the case of 120 percent oversaturation, but less number of vehicles for the 140 percent oversaturation. The amount of the vehicles that were held by the type-2 control was approximately 60 percent greater than those by type-1 and type-3 controls at the end of the peak period.

It was observed from the Figures 4.2 d) and 4.2 e) that, at the end of peak time (120 minutes after the beginning of the simulation), the accumulation of vehicles on the links still continued, especially the inner links. Type-1 and type-3 controls still created the vehicle accumulation until reaching the highest number of vehicles some times later (approximately 10 minutes after the peak time in the 120 percent oversaturation and 20 minutes after the peak time in the 140 oversaturation case). This was one important consideration since, the more accumulation and the later in time, the more difficult on congestion dissipation.

After the end of peak time, the number of vehicles stored in each part of the system was decreasing until it reached stability. From Figures 4.2 d) and 4.2 e), the rates of reduction in the number of vehicles stored on each link type were various, depending on the amount and time (the highest number of vehicles stored) and the effectiveness of the traffic control to dissipate the queue. From Figure 4.2 d), type-2 and type-4 controls had an ability to dissipate traffic on the inner links and reached the stability level sooner than type-1 and type-3 controls, although the rates of reduction were less than those of type-1 and type-3 controls. It was clearly seen from Figure 4.2 e) that type-2 and type-4 controls made traffic reach stability approximately 20 minutes sooner than type-1 and type-3 controls (approximately 10 minutes sooner in the 120 percent oversaturation case in Figure 4.2 d)). The slow rate of reduction in the number of vehicles on the inner links meant the type-2 and type-4 controls managed the traffic in the congested links under the existence of waiting demand from the outer links.

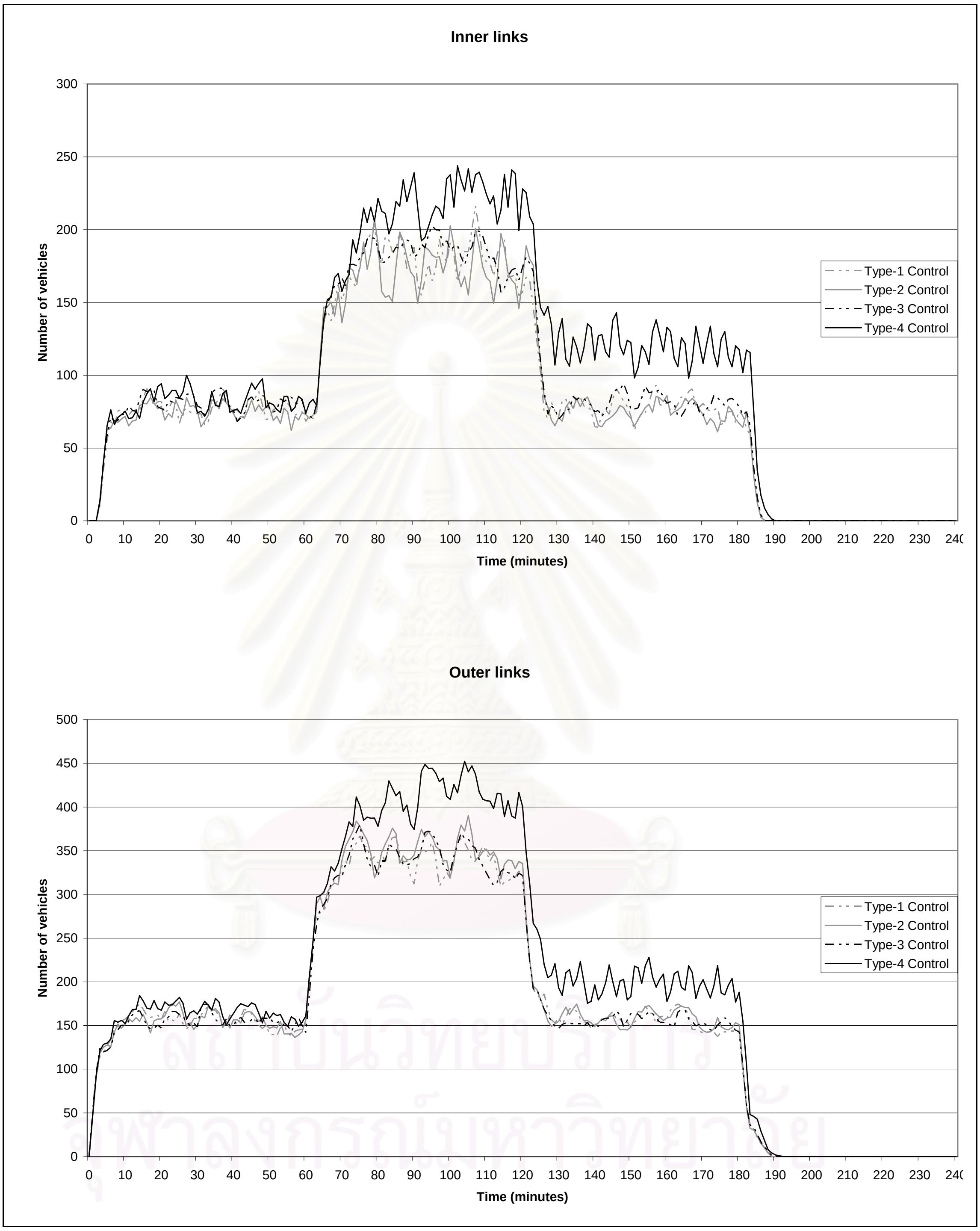
It could be viewed that during the dissipation period (the time period after the peak time when the vehicles were released from the links and the traffic conditions turned to normal congestion level (stability)), the outer links where the excessive demand were stored during the peak period released the traffic onto the inner links. The rate of the decrease in the vehicle accumulation was dependent to the ability of the inner links to accept these amounts of traffic. Figure 4.2 d) illustrated that each control method took different time period to get back to normal (stability). The type-2 control, which developed the highest congestion on the outer links, took longer time to be back to stability. The same trend could be observed in Figure 4.2 e). Note that type-4 control could perform better than type-2 control in bringing the level of congestion (number of vehicles on links) down to stability. However, the type-2 control was not as good as type-1 and type-3 controls, in terms of the time to reach stability and the level of traffic at stability (a larger number of vehicles was maintained even during the stability period). It was remarked that the type-4 control maintained the level of traffic at stability until the end of demand period (the 180th minute in the simulation). Compared with the type-2 control, type-4 control held the traffic on the outer links more than type-2 control during the 170th-180th minute of the simulation period (Figure 4.2 d)). Type-2 control created the longest dissipation period, as clearly observed in Figure 4.2 e) on both inner and outer links. It took 30 to 40 minutes longer to clear the entire traffic out of the network at the end of simulation study.

The effectiveness of the traffic control came from the ability to allocate “proper” green time, both in terms of duration and timing. In principle, considering an average green time period given to each link during a peak hour, if the first control type was said to equally give the green time to each leg due to the v/c ratio on the leg, type-4 control aimed to give more priority only to the legs that seemed to produce the maximum throughput. The type-2 and type-3 controls could be inferred to the methods described in Chapter III. Thus, it was important to learn about the green time allocation and this consideration could be viewed as the total green time period allocated to each link. The examples of green time allocation on selected links were shown in Figure 4.3 for pattern-1 demand. The figure showed only the legs of the intersections on the main corridors.



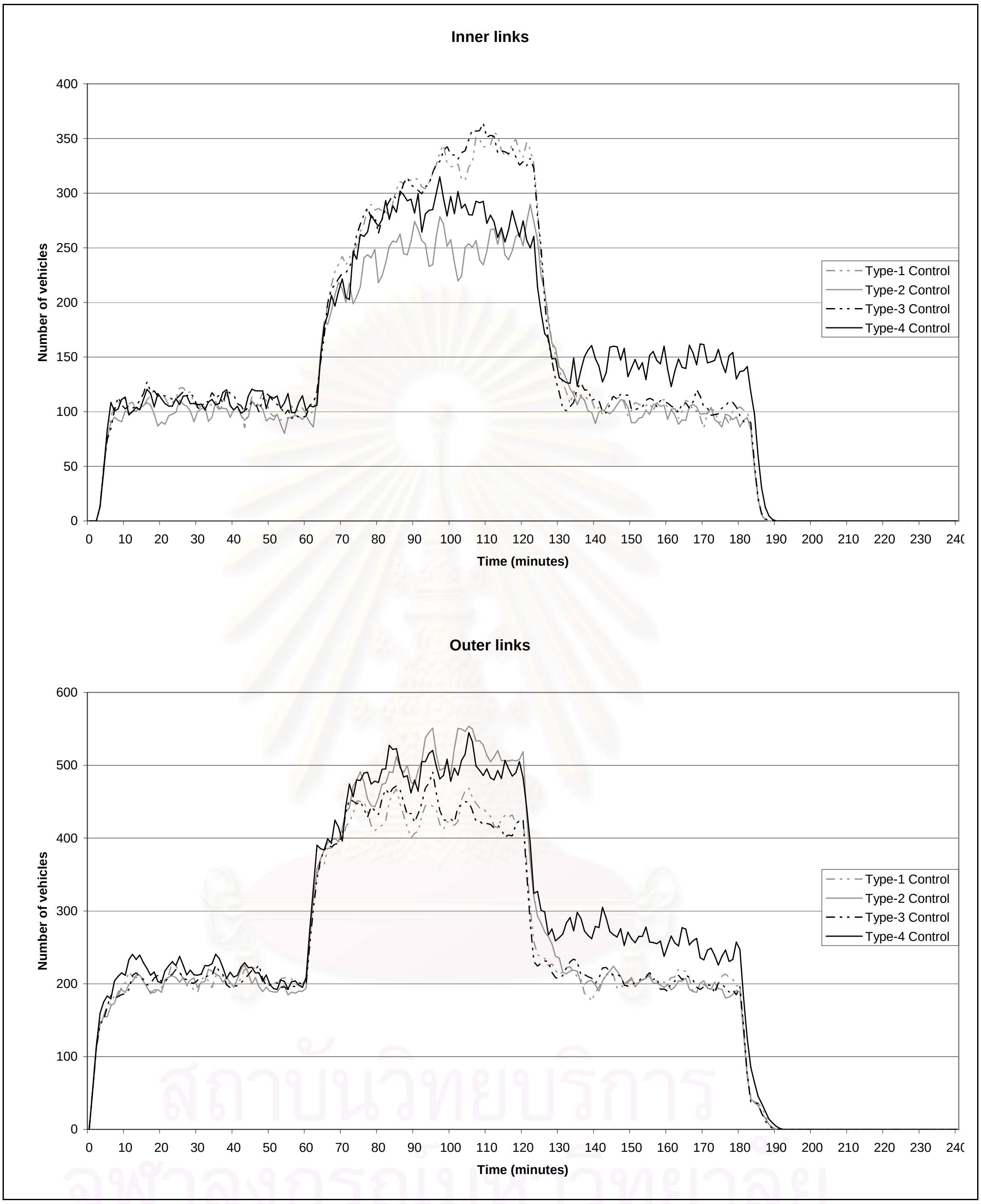
a) Demand level 1 (unsaturated traffic condition)

Figure 4.2 Number of vehicles containing in the system (pattern-1 demand)



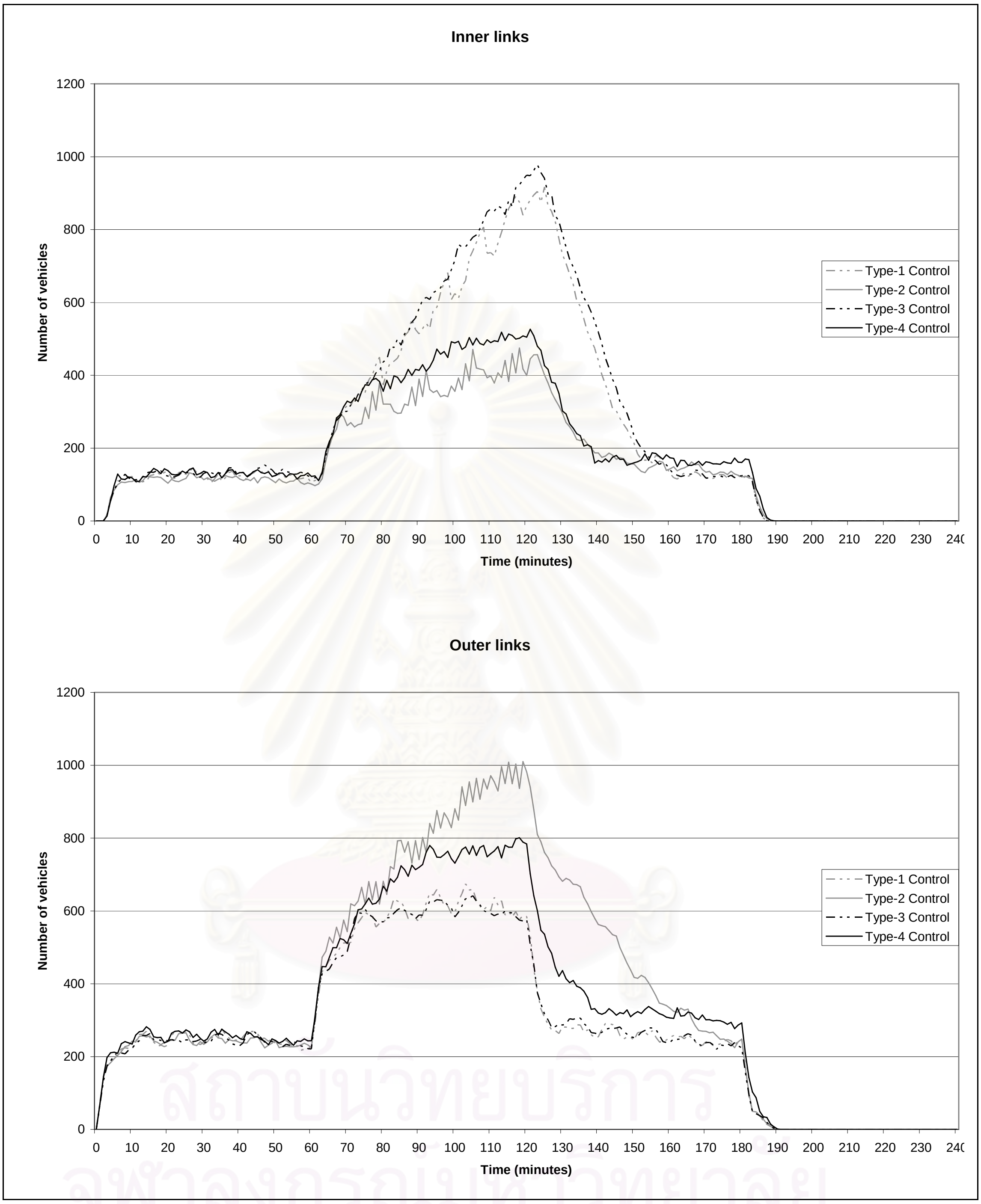
b) Demand level 2 (unsaturated traffic condition)

Figure 4.2 Number of vehicles containing in the system (pattern-1 demand) (continued)



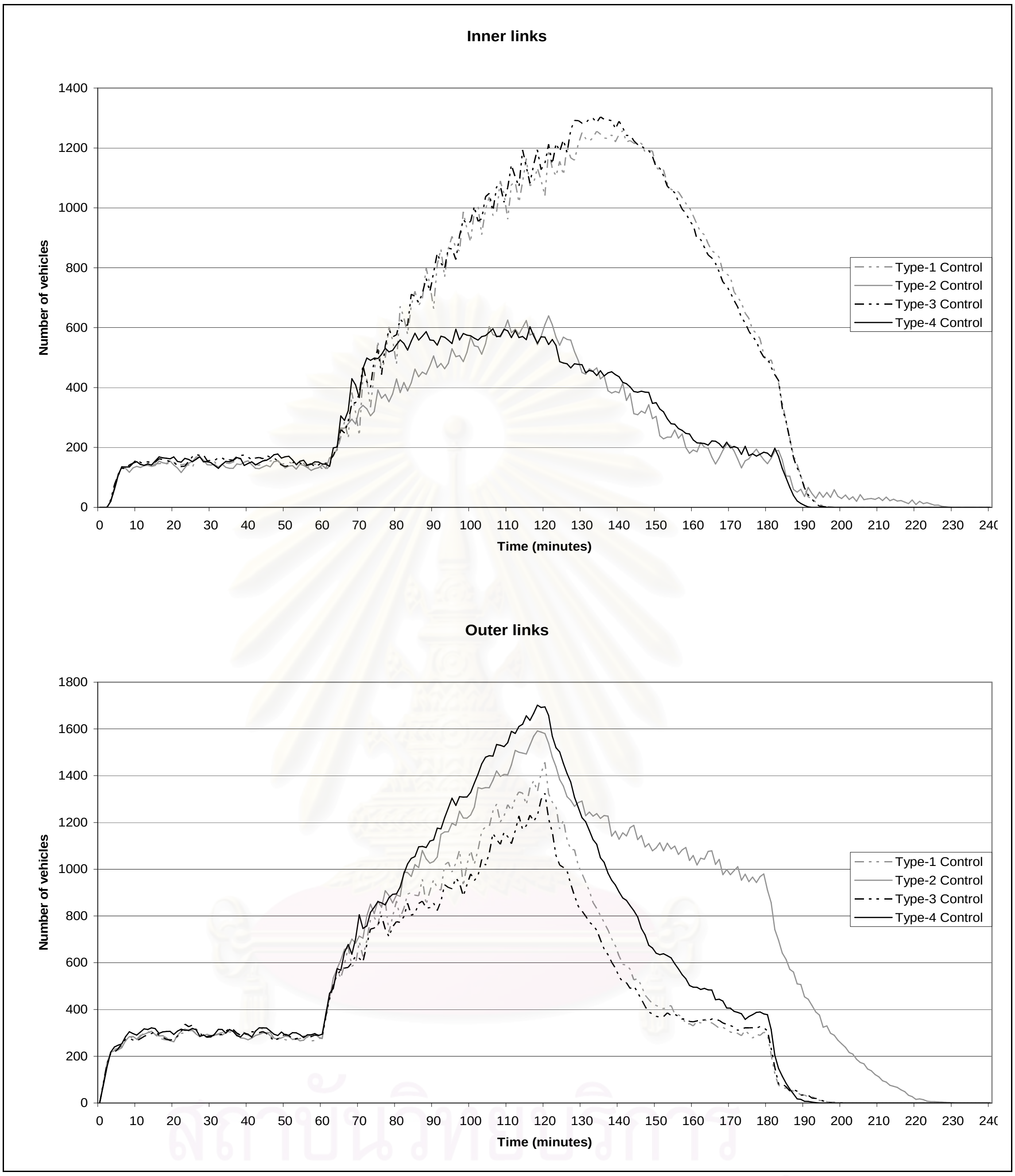
c) Demand level 3 (saturated traffic condition)

Figure 4.2 Number of vehicles containing in the system (pattern-1 demand) (continued)



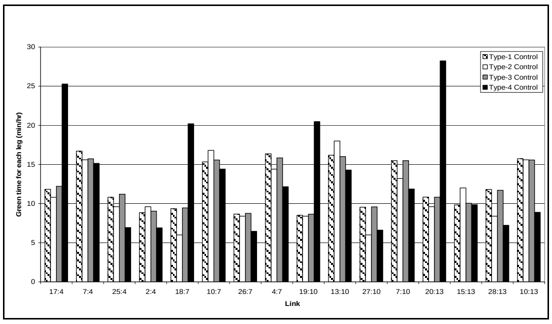
d) Demand level 4 (oversaturated traffic condition)

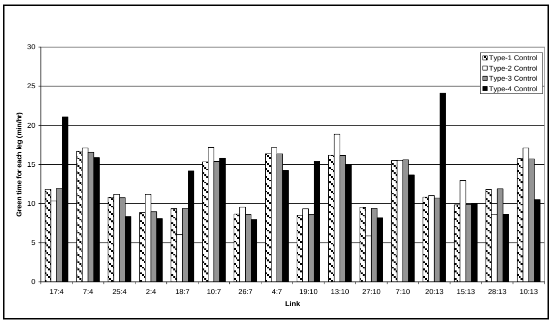
Figure 4.2 Number of vehicles containing in the system (pattern-1 demand) (continued)



e) Demand level 5 (oversaturated traffic condition)

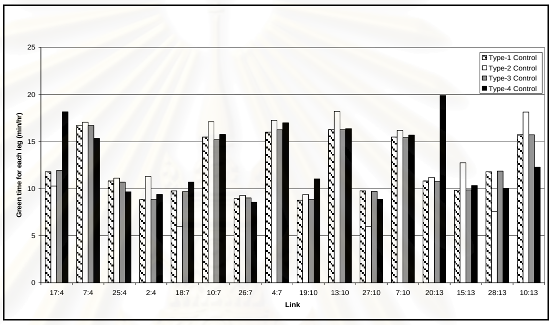
Figure 4.2 Number of vehicles containing in the system (pattern-1 demand) (continued)





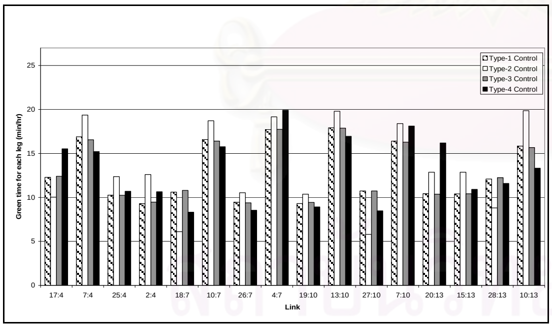
a) Demand level 1 b) Demand level 2

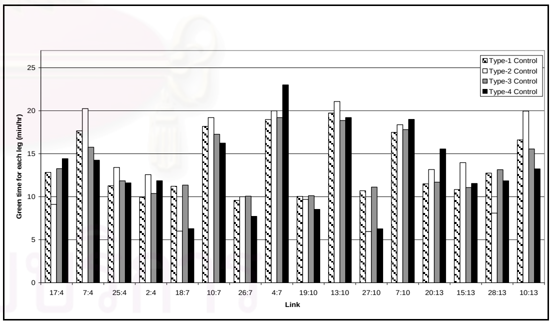
(Unsaturated traffic condition)



c) Demand level 3

(Saturated traffic condition)





d) Demand level 4 e) Demand level 5

(Oversaturated traffic condition)

Figure 4.3 Average green-time given to each link during a peak hour (pattern-1 demand)

As seen from Figure 4.3, the green time allocation (averaged from several simulation runs for the same scenarios) under different control methods varied. However, since this figure presented the total green time period, it could not determine the proper “timing” of the allocation. Table 4.1 summarized the range of green time during the peak period for each leg. It indicated that each control method gave different green time periods. As different performances of the control were seen from the Figure 4.2, it implied that the allocation of green time was attributable to the effectiveness of the overall control.

Table 4.1 Summary of network conditions for pattern-1 demand

Scenarios	Maximum Number of Vehicles Containing on inner links				% Difference			Maximum Number of Vehicles Containing on Outer links				% Difference			Average Green Time Range Among Legs of An Intersection During A Peak hour (Minutes)			
	Type-1 Control	Type-2 Control	Type-3 Control	Type-4 Control	Type-4 from Type-1	Type-4 from Type-2	Type-4 from Type-3	Type-1 Control	Type-2 Control	Type-3 Control	Type-4 Control	Type-4 from Type-1	Type-4 from Type-2	Type-4 from Type-3	Type-1 Control	Type-2 Control	Type-3 Control	Type-4 Control
1	141	139	150	194	38	39	29	261	270	271	326	25	21	20	7.29	9.00	6.66	16.75
2	216	207	202	244	13	18	20	367	390	380	452	23	16	19	7.29	9.85	6.98	10.88
3	355	290	364	315	-11	9	-14	469	553	491	545	16	-2	11	7.09	10.43	7.09	8.65
4	917	475	978	526	-43	11	-46	674	1010	642	801	19	-21	25	7.47	11.87	7.30	7.85
5	1258	639	1304	603	-52	-6	-54	1457	1592	1325	1702	17	7	28	8.16	13.03	6.93	9.12

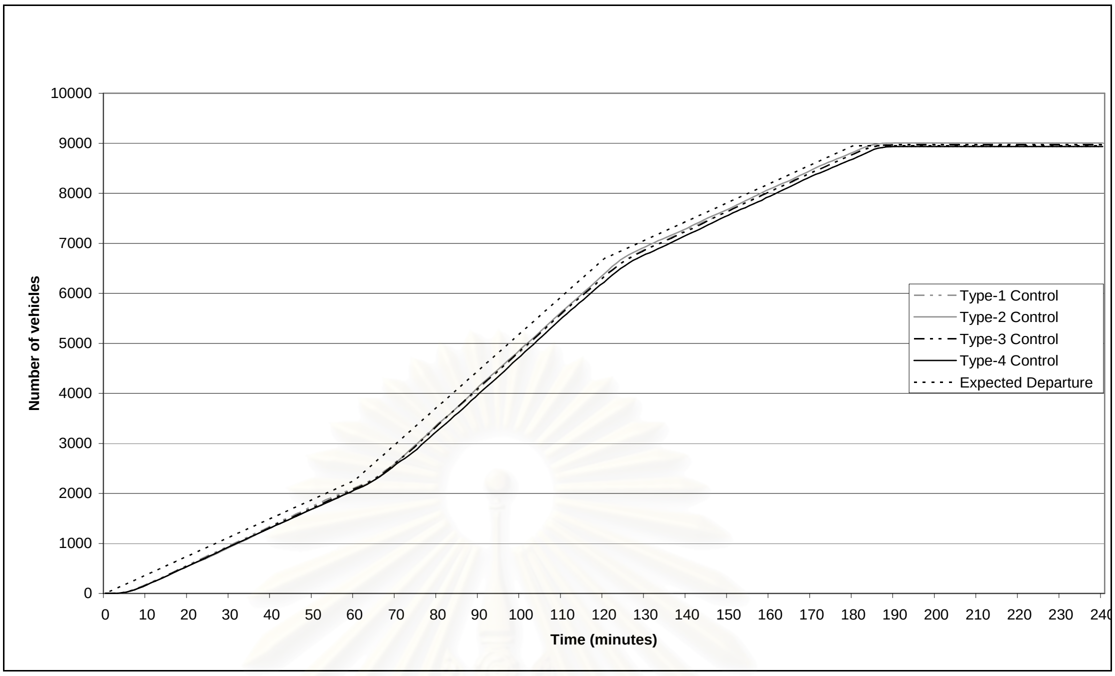
Table 4.1 also summarized the maximum number of vehicles containing on the inner links and outer links in each scenario due to traffic control types. It showed that when comparing the devised control method type-4 with others, the type-4 control maintained a higher “maximum” number of vehicles on the inner and outer links when the traffic condition was unsaturated. During oversaturation, however, the type-4 control held the vehicles on the outer links, creating a higher “maximum” number of vehicles at these outer links (except when compared to type-2 control). Under the 140 percent oversaturation, the type-4 control maintained the minimum amount of vehicles on the inner links.

4.2.2 Graphical Analysis on Throughput and Speed

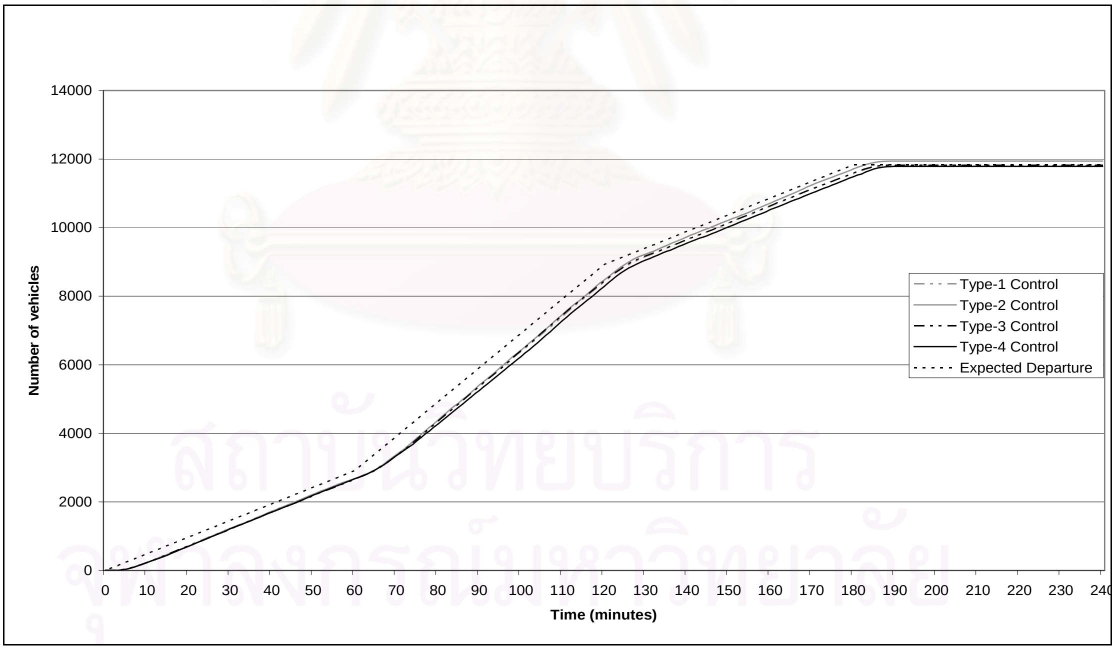
Cumulative plots of the number of vehicles arriving their destination over time were presented in Figure 4.4. The plots showed the ability of the control to let road users accomplish their goals and reach destinations. The sooner, the better. It also meant that the higher number of vehicles implied a better effectiveness. For an unsaturated condition, Figures 4.4 a) and 4.4 b) showed that the order of effectiveness (from high to low) during the peak and dissipation period was type-2, type-1, type-3, and type-4. Note that the type-4 control method had not produced much different effectiveness in handling the peak demand compared to other three methods. Actually, because the demand was less than the capacity, the result should be like this and the difference should mainly depend on the way that each control managed some short queues, lining up at each intersection.

For a saturated condition, the curves in Figure 4.4 c) showed that all control methods performed very similarly during the peak period. However, during the dissipation period, the order of effectiveness (high to low) was as follow: type-1, type-3, type-2, and type-4, respectively. Again note that type-4 control method had almost equal efficiency in handling the peak demand compared to other three methods.

For an oversaturated condition, the curves in Figures 4.4 d) and 4.4 e) showed that all control methods performed similarly during the peak time, except the type-1 and type-3 controls in the 140 percent oversaturation. However, during the queue dissipation, the control methods gave various effectiveness. At 120 percent oversaturation, type-4 and type-2 controls performed better at the beginning of the dissipation period but perform worse at the end of the period. Under 140 percent oversaturation, type-4 control method performed the best during the dissipation period. Type-2 control performed well at the beginning of the period, but became the worst control at the end of the period. It could also be seen that the type-2 control took a longer time to clear the entire vehicles out of the system at the end of simulation period. From Figure 4.4 e), the order of effectiveness (high to low) of the control methods in clearing the vehicles out of the network (seen at the end of the simulation period) was type-4, type-3, type-1, and type-2, respectively. The findings confirmed the conclusion on the control effectiveness from the consideration of the vehicles containing on the links in the previous section.

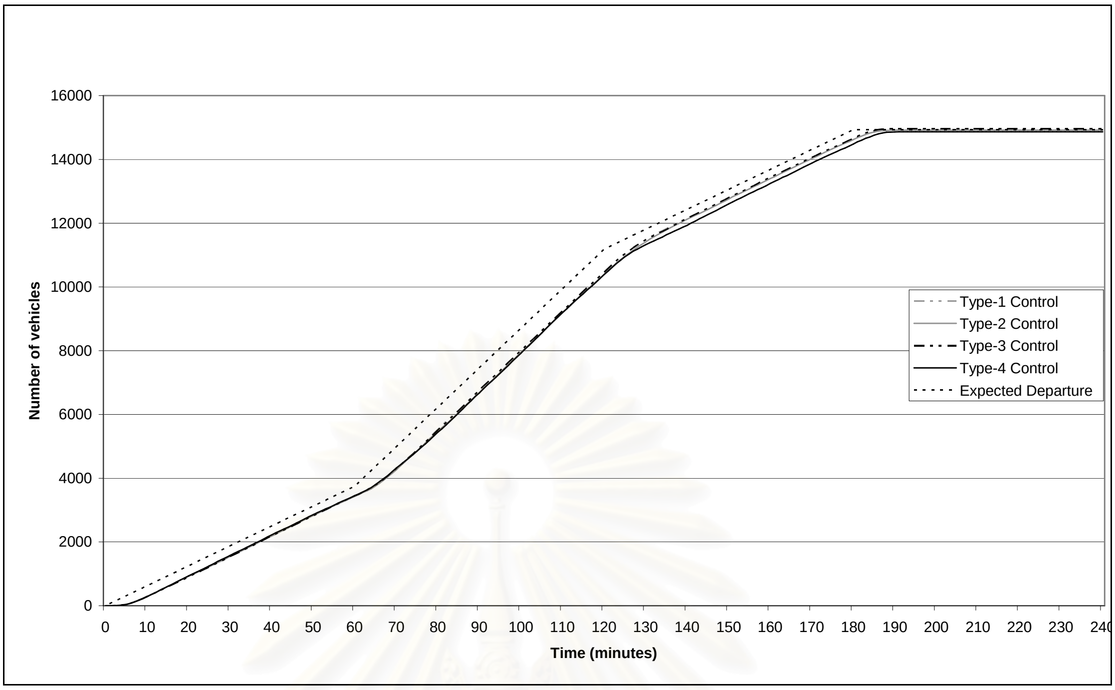


a) Demand-level 1 (unsaturated traffic condition)

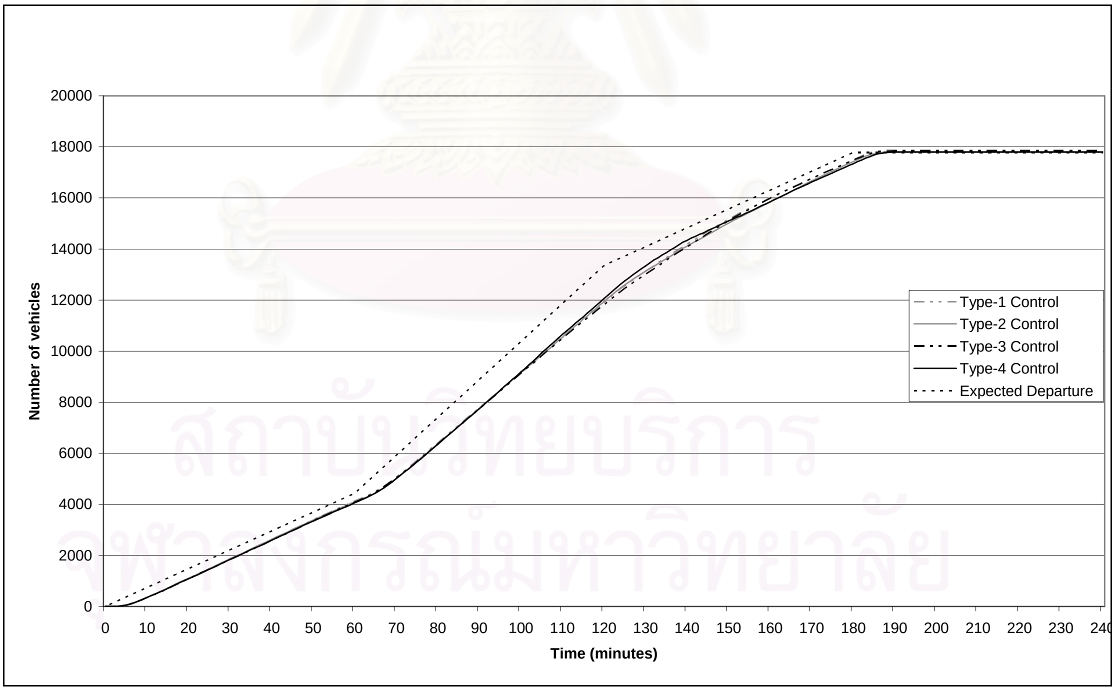


b) Demand-level 2 (unsaturated traffic condition)

Figure 4.4 Arrival at all destinations (pattern-1 demand)

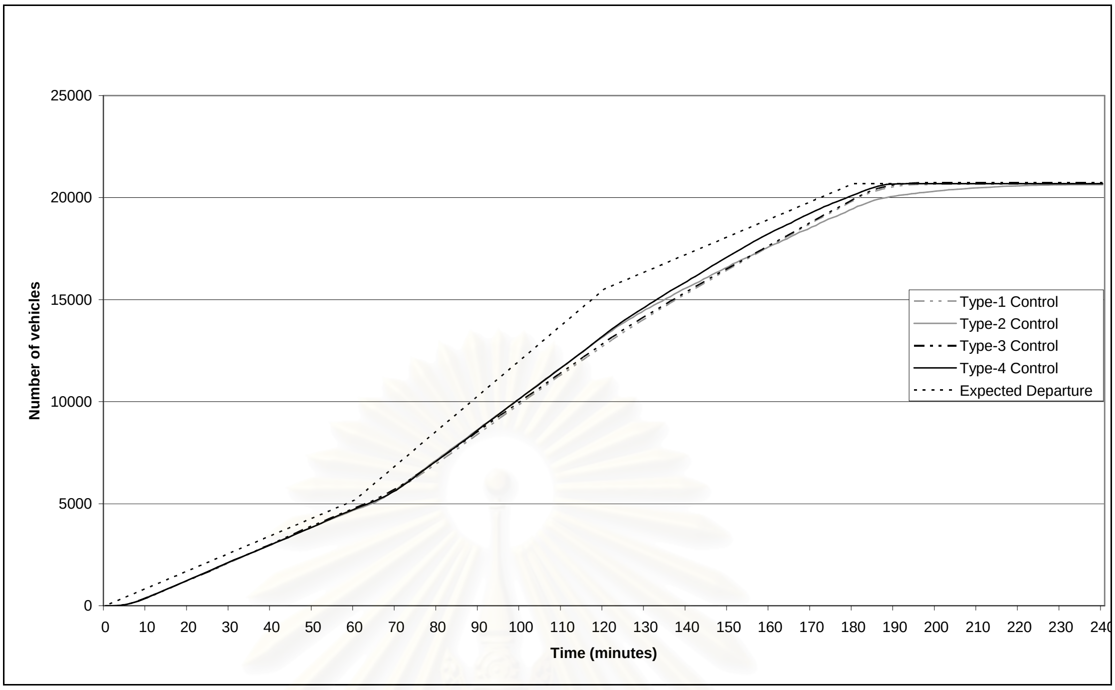


c) Demand-level 3 (saturated traffic condition)



d) Demand-level 4 (oversaturated traffic condition)

Figure 4.4 Arrival at all destinations (pattern-1 demand) (continued)

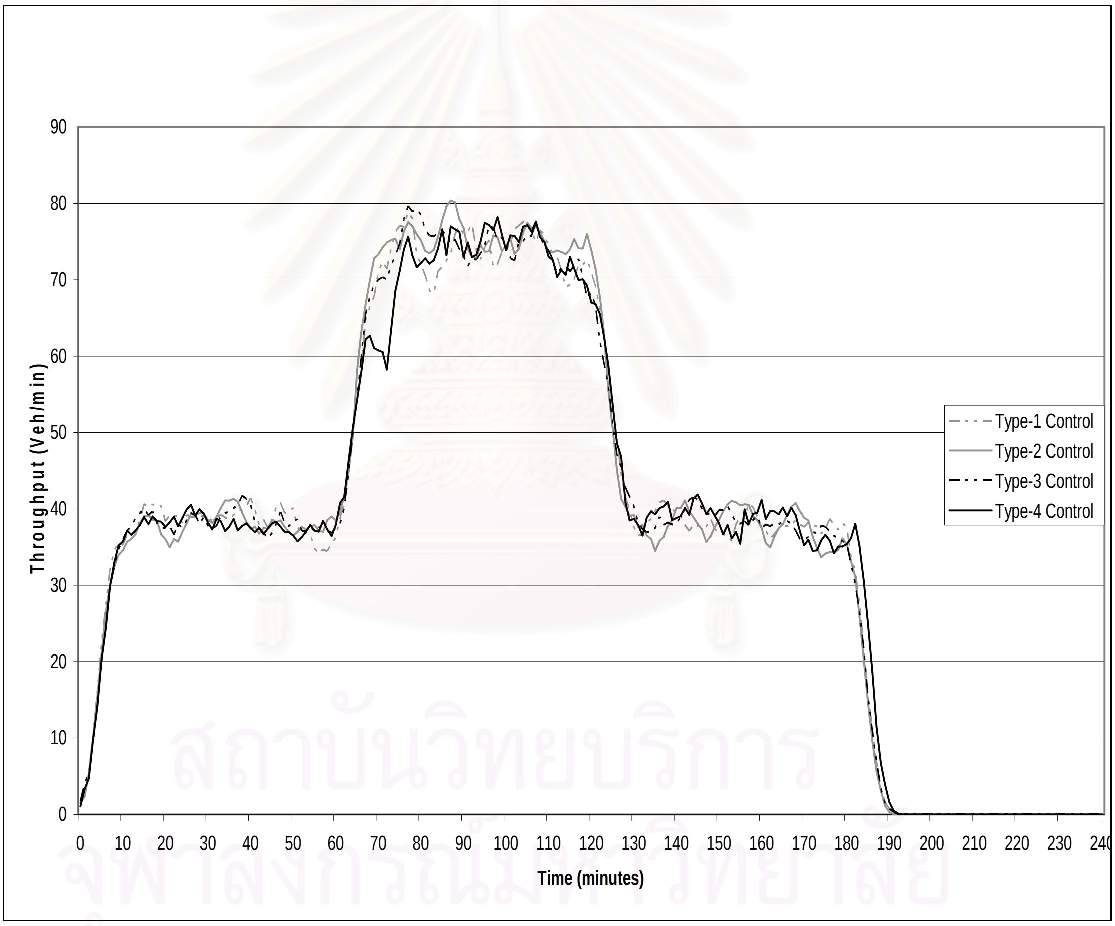


e) Demand level 5 (oversaturated traffic condition)

Figure 4.4 Arrival at all destinations (pattern-1 demand) (continued)

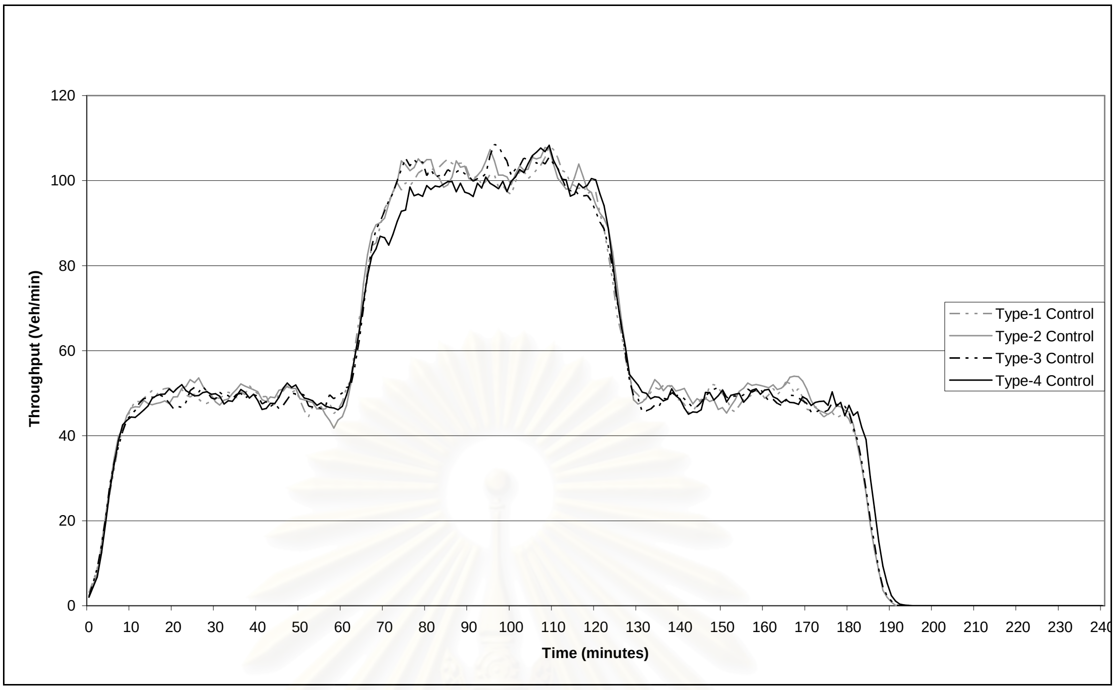
Apart from the cumulative plots, the “throughput” vs time could be presented to obtain the effectiveness of each control method at a particular time. Figure 4.5 presented the “throughput” or number of vehicles arriving their destination over time under various traffic conditions. This consideration was not the same as “throughput” of the intersection where the number of vehicles departing the stop-line was used as a measure of throughput.

Figure 4.5 yielded the same conclusion as those from Figure 4.4. However, the differences in control effectiveness during time periods were more discernable. In general, all control methods performed similarly during the peak period. The exception was at the high oversaturation (Figure 4.5 e)) where type-2 and type-4 produced high throughput during the peak period. The outstanding differences in control effectiveness could be seen from the ability to maintain the rate of throughput during the dissipation period and the time to completely release traffic out of the network. Focusing on type-4 control, the type-4 control performed similarly at unsaturated traffic condition, but when the traffic became saturated, the control had the least ability to maintain high throughput (120th to 180th minute). This resulted in the slowest method to release traffic out of the network. However, during the high oversaturated condition (Figure 4.5 e)), the type-4 control could maintain high rate of throughput during the end of the peak period and the beginning of the dissipation period. The outstanding effectiveness could be seen from the ability to release traffic out of the system sooner than any other control methods (195th minute). The slowest control method was the type-2 method (230th minute).

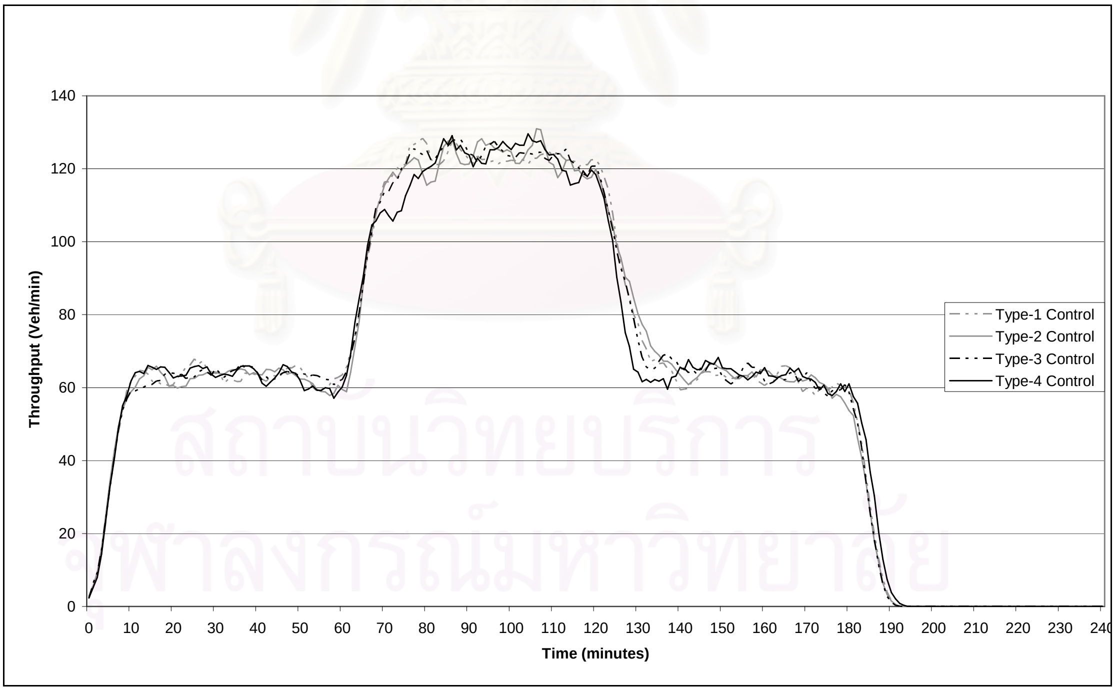


a) Demand-level 1 (unsaturated traffic condition)

Figure 4.5 Real-time throughput (pattern-1 demand)

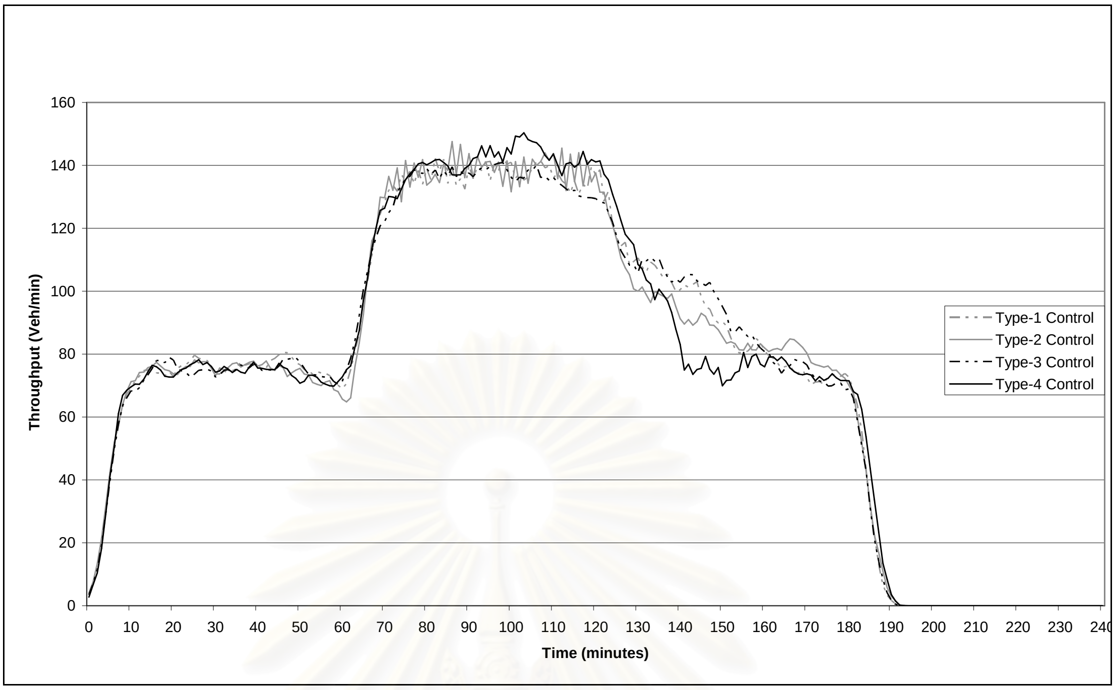


b) Demand-level 2 (unsaturated traffic condition)

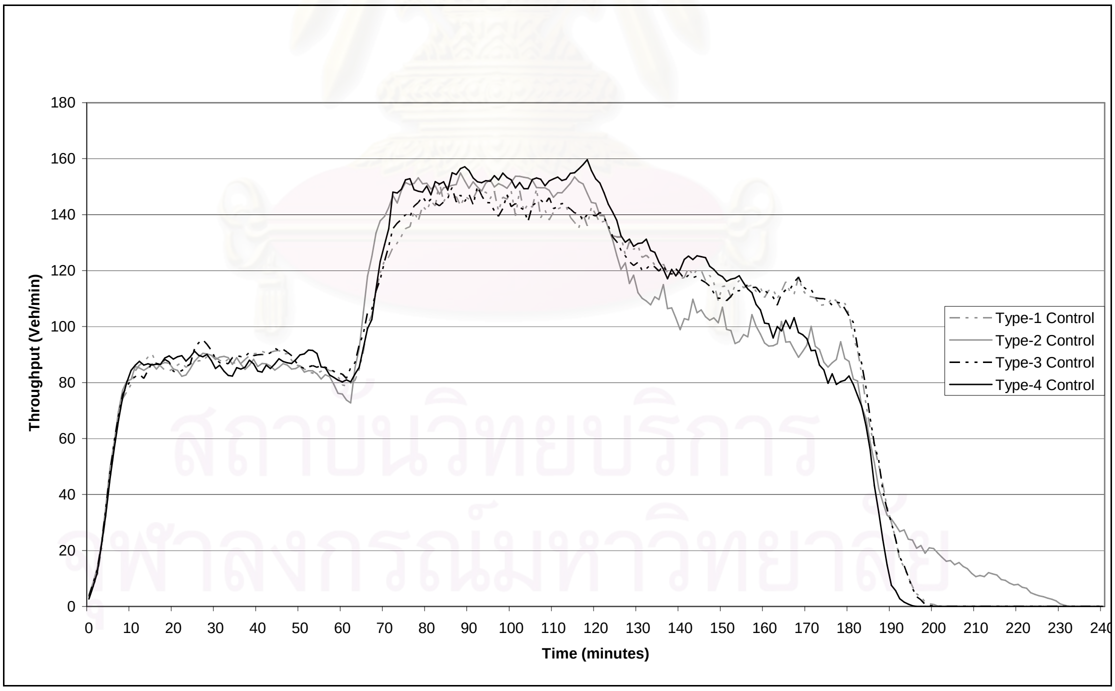


c) Demand-level 3 (saturated traffic condition)

Figure 4.5 Real-time throughput (pattern-1 demand) (continued)



d) Demand-level 4 (oversaturated traffic condition)



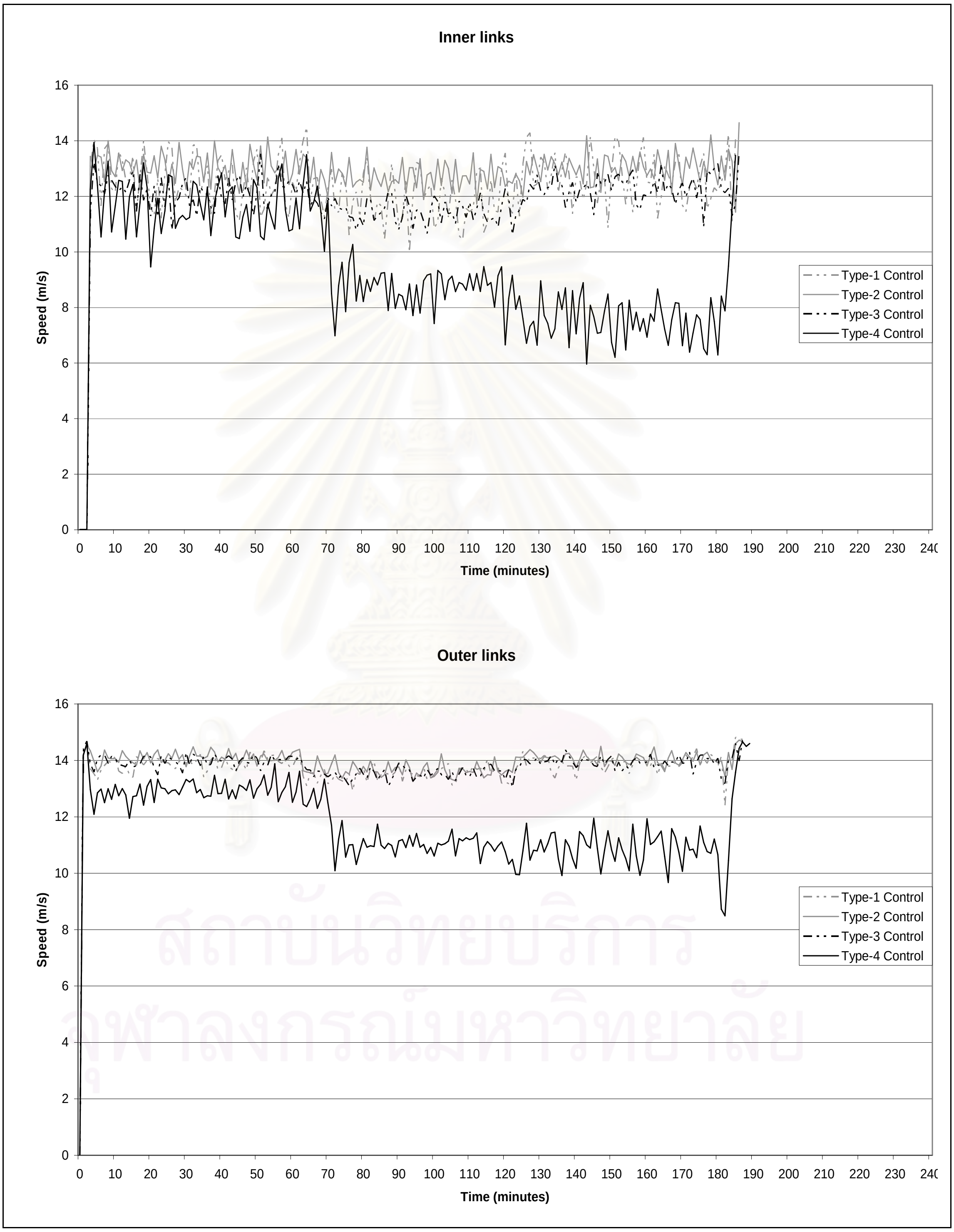
e) Demand-level 5 (oversaturated traffic condition)

Figure 4.5 Real-time throughput (pattern-1 demand) (continued)

The effectiveness of the traffic control methods could be measured by driver's perception indicators. Thus, the average speed of traffic vs time was presented for each traffic condition. The results were shown in Figure 4.6.

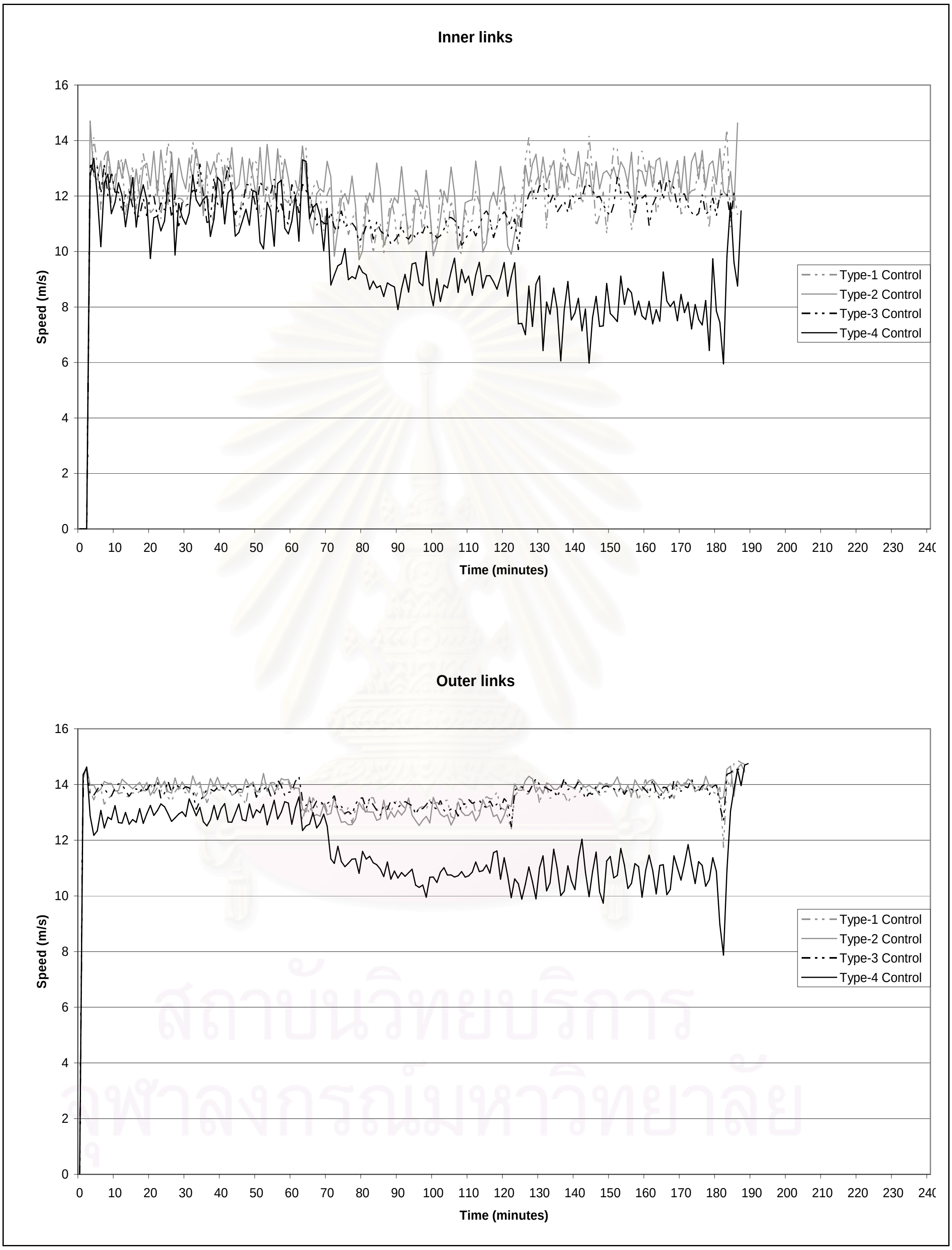
Figures 4.6 a) and 4.6 b) illustrated the average speed on the inner and outer links in unsaturated traffic conditions. This speed was averaged from real time detected speeds of all vehicles on each link. It had to be considered because it was possible to have low speed while there was high overall throughput. The type-4 control clearly reduced traffic speed on both inner and outer links. As the congestion level increased to saturation, as indicated in Figure 4.6 c), type-1 and type-3 controls gave the lowest speed on the inner links during the peak time, where type-2 control gave the highest. In contrast, type-1 and type-3 controls yielded the highest speed on the outer links and type-2 control gave the lowest. Type-2 control maintained low speed throughout the peak and dissipation periods. These figures concluded that the type-4 control brought the traffic speed lower throughout the congestion period during the unsaturated traffic conditions.

As the congestion increased from saturation to oversaturation, each control method performed differently. In general, the trend was similar, but the magnitude of impact was greater. Type-1 and type-3 produced the lowest speed during the peak time on the inner links, and the highest speed on the outer links. The outstanding characteristics were that type-1 and type-3 could not convert the inner links into a higher speed during the dissipation period. It took about 20 minutes to higher the inner link speed in the saturation condition, but 40 minutes in the oversaturation condition (Figure 4.6 e)). Type-2 control could higher the inner link speed back to average sooner than type-1 and type-3 controls, but performed the worst in improving the speed of the outer links. In fact, it could release traffic and higher speed after the vehicles stopped entering the network (toward the end of simulation). Type-4 control did not bring the speed down as low as the type-1 and type-3 controls on the inner links. However, it did not create as high speed as the two methods in the dissipation period. The similar tendency could be seen on the outer links. This implied that type-4 control maintained a more consistent speed than the other three.



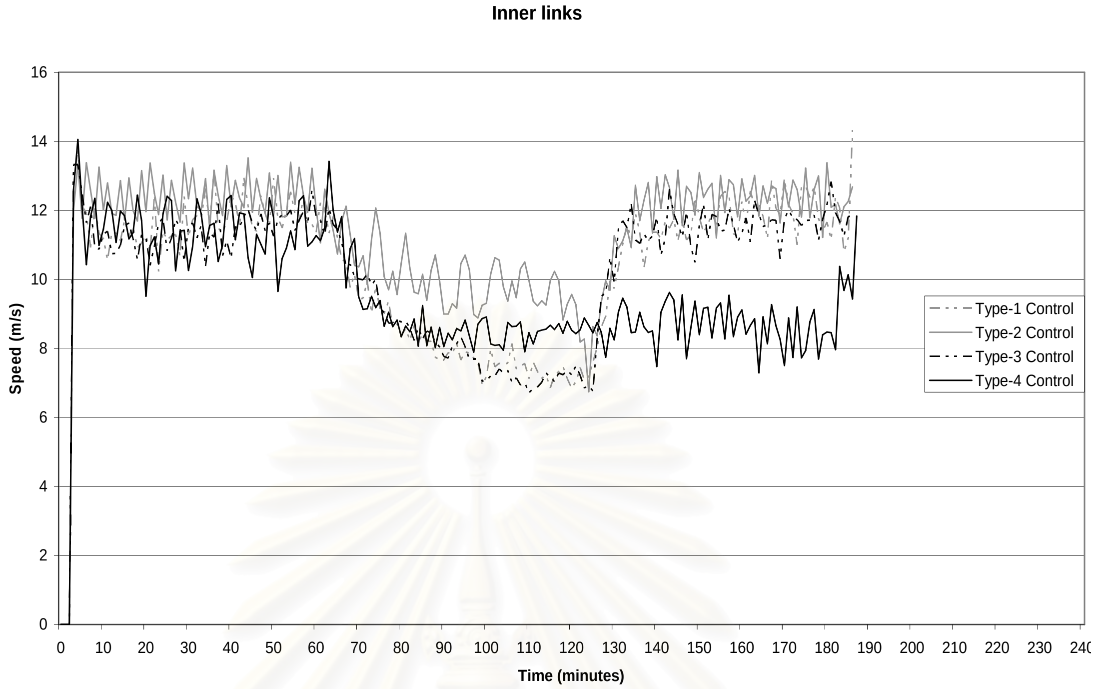
a) Demand-level 1 (unsaturated traffic condition)

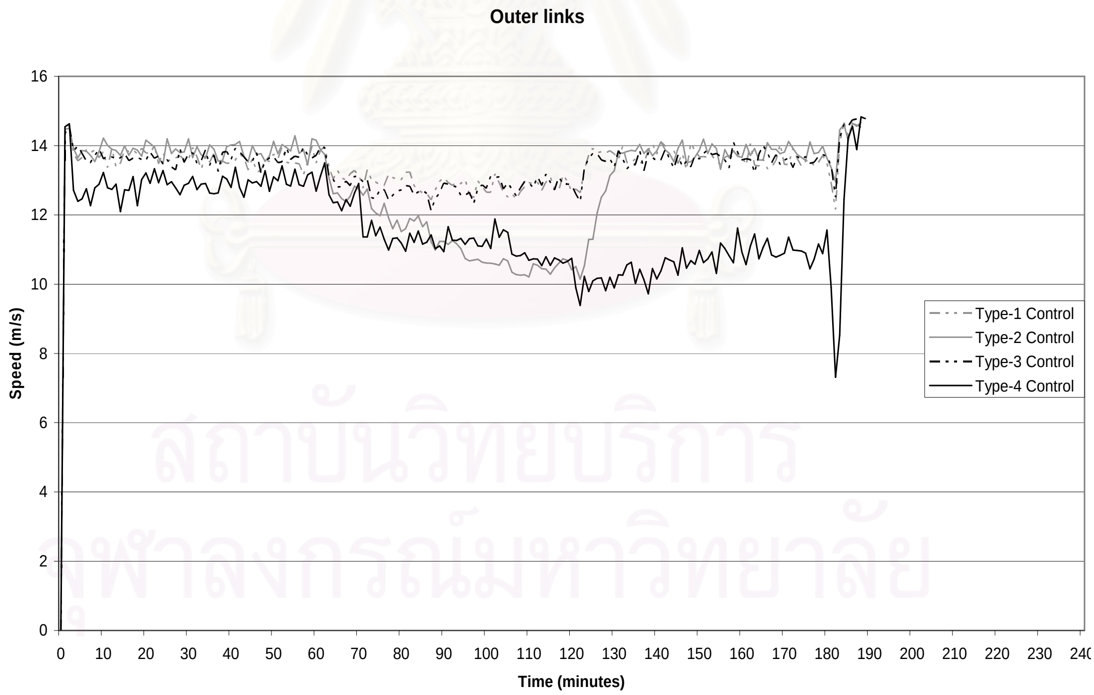
Figure 4.6 Real-time average speed in the system (pattern-1 demand)



b) Demand-level 2 (unsaturated traffic condition)

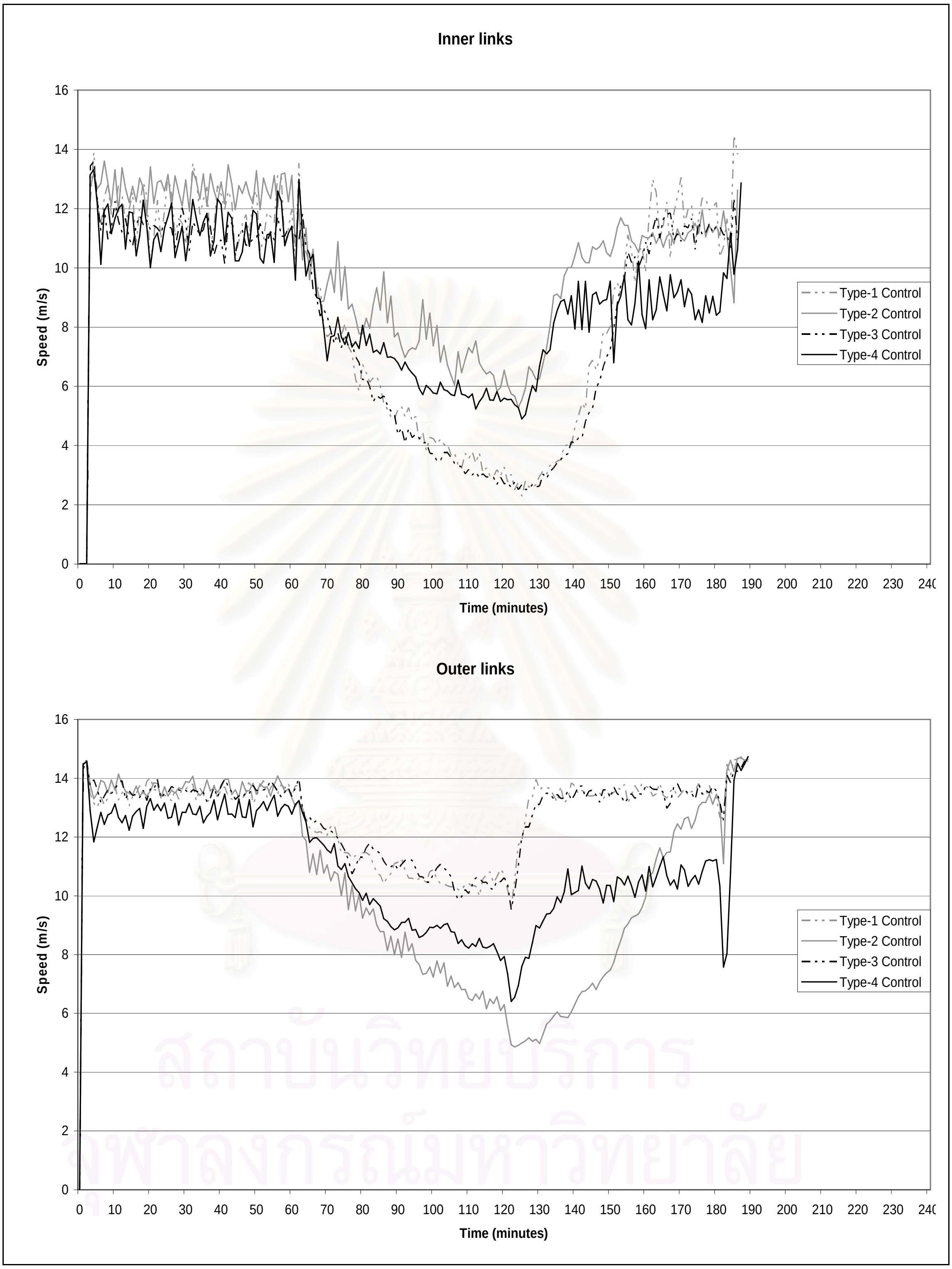
Figure 4.6 Real-time average speed in the system (pattern-1 demand) (continued)





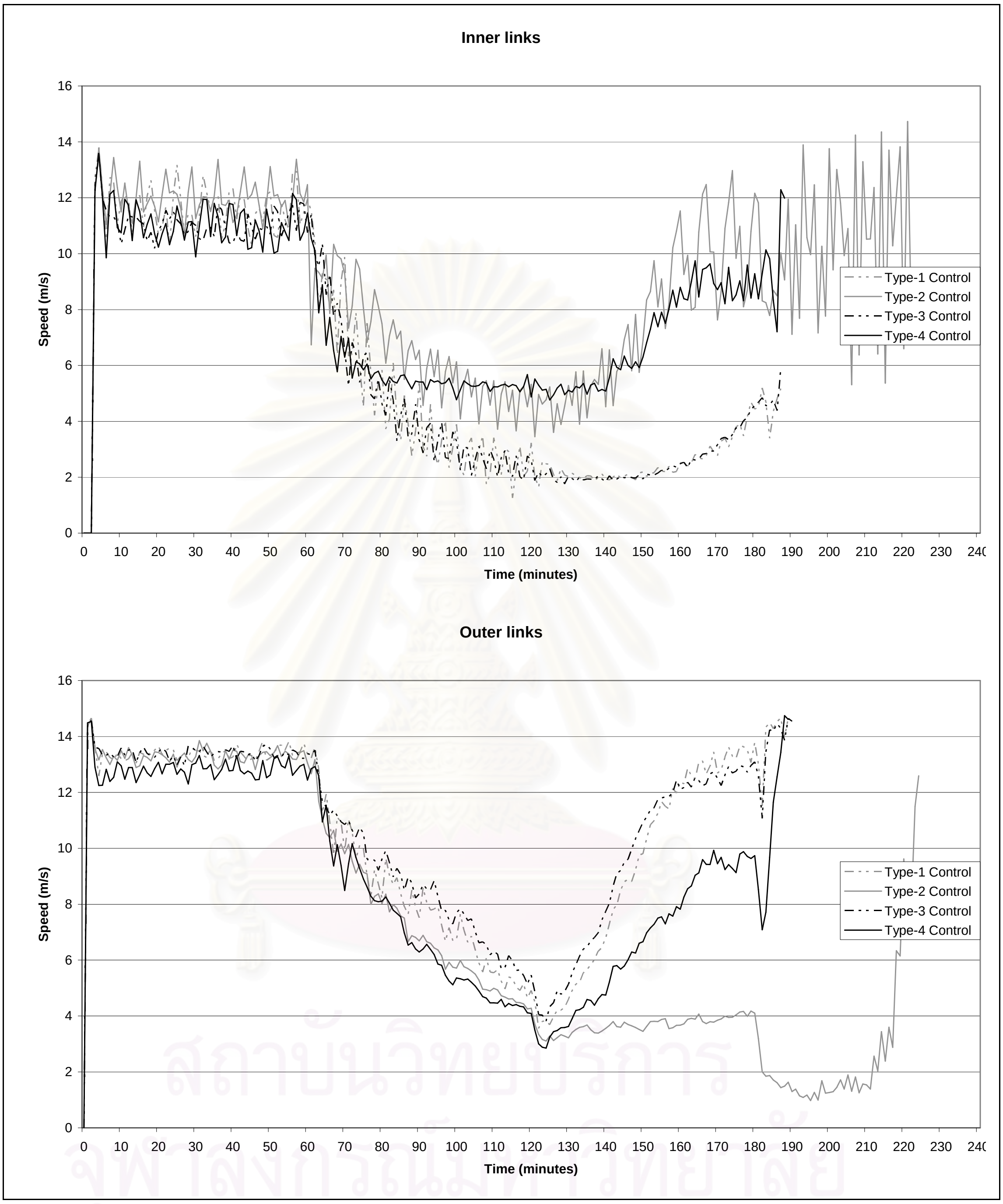
c) Demand-level 3 (saturated traffic condition)

Figure 4.6 Real-time average speed in the system (pattern-1 demand) (continued)



d) Demand-level 4 (oversaturated traffic condition)

Figure 4.6 Real-time average speed in the system (pattern-1 demand) (continued)



e) Demand-level 5 (oversaturated traffic condition)

Figure 4.6 Real-time average speed in the system (pattern-1 demand) (continued)

Figure 4.6 clearly illustrated the effect of traffic controls on speed during high oversaturation. Type-1 and type-3 controls produced the lowest speed on the inner links and the highest speed on the outer links, during the peak time. During the dissipation and end of simulation periods, the inner links had not reached a high speed anymore. Type-2 control gave the highest speed on the inner links, but could not manage the traffic on the outer links. It was seen that the traffic on the outer links could not reach a high speed during the dissipation and end of simulation periods. The effect on the outer links was so adverse as it delayed the last vehicles leaving the network, 30-40 minutes later than the other three control methods. For type-4 control, it was still seen that this method gave the most consistent speed.

Table 4.2 Average speed, meters per second (pattern-1 demand)

Demand Levels	Control Types	Average Speed on Inner Links					Average Speed on Outer Links					Average Speed in the Network				
		The 1st hour	The 2nd hour	The 3rd hour	The 4th hour	Average	The 1st hour	The 2nd hour	The 3rd hour	The 4th hour	Average	The 1st hour	The 2nd hour	The 3rd hour	The 4th hour	Average
1	1	12.6	11.9	12.6	12.9	12.3	13.9	13.5	13.9	13.8	13.7	13.5	13.0	13.4	13.4	13.2
	2	13.1	12.7	13.0	13.2	12.9	14.1	13.7	14.0	14.0	13.9	13.8	13.4	13.7	13.6	13.5
	3	12.2	11.5	12.2	12.1	11.8	14.0	13.5	13.9	13.6	13.7	13.4	12.8	13.3	12.8	13.1
	4	11.6	9.0	7.6	9.3	9.0	12.9	11.2	10.8	10.6	11.5	12.5	10.5	9.5	10.0	10.6
2	1	12.4	11.2	12.2	12.7	11.7	13.8	13.2	13.7	13.6	13.5	13.3	12.5	13.2	13.2	12.9
	2	12.7	11.6	12.5	12.6	12.1	14.0	13.0	13.9	13.9	13.4	13.6	12.5	13.4	13.3	13.0
	3	11.9	10.9	11.7	11.9	11.3	13.8	13.3	13.8	13.7	13.5	13.2	12.4	13.1	12.8	12.8
	4	11.5	9.2	8.0	8.6	9.3	12.9	11.2	10.7	10.5	11.4	12.5	10.5	9.7	9.6	10.7
3	1	11.7	8.2	10.5	11.7	9.4	13.6	12.9	13.6	13.4	13.3	13.0	11.0	12.4	12.6	11.8
	2	12.4	10.0	11.2	12.0	10.9	13.9	11.3	13.3	13.7	12.3	13.4	10.9	12.6	12.9	11.9
	3	11.4	8.1	10.5	12.0	9.4	13.7	12.8	13.6	13.7	13.2	12.9	10.9	12.4	12.8	11.7
	4	11.4	8.8	8.6	9.2	9.3	12.9	11.3	10.6	10.4	11.4	12.4	10.4	9.9	9.8	10.7
4	1	11.9	4.8	5.1	11.1	5.8	13.5	11.0	13.2	13.4	12.1	13.0	8.1	8.6	12.3	9.1
	2	12.7	7.8	9.0	10.9	9.0	13.7	8.3	7.5	13.3	8.9	13.4	8.1	7.9	12.0	8.9
	3	11.3	4.5	4.7	11.2	5.4	13.6	11.1	13.1	13.5	12.2	12.8	7.8	8.2	12.3	8.8
	4	11.3	6.7	7.6	9.7	7.7	12.9	9.5	9.7	10.3	10.2	12.3	8.4	8.9	10.0	9.3
5	1	11.6	3.6	2.3	4.4	3.5	13.4	7.4	7.5	13.0	8.4	12.8	5.8	4.2	6.5	5.9
	2	12.0	6.2	6.5	9.8	7.3	13.3	6.6	3.6	1.7	5.7	12.9	6.5	4.3	3.0	6.1
	3	11.0	3.6	2.3	4.6	3.5	13.4	8.1	8.2	12.9	9.0	12.6	6.0	4.3	6.8	6.1
	4	11.0	5.6	6.4	9.2	6.7	12.8	6.3	5.6	9.2	6.9	12.2	6.1	5.8	9.2	6.8

Table 4.3 Summary of graphical analysis for pattern-1 demand

Scenarios	Peak-Hour Throughput				Inner-Link Speed				Outer-Link Speed			
	Type-1 Control	Type-2 Comtrol	Type-3 Comtrol	Type-4 Control	Type-1 Control	Type-2 Comtrol	Type-3 Comtrol	Type-4 Control	Type-1 Control	Type-2 Comtrol	Type-3 Comtrol	Type-4 Control
1	low	high	low		high			low	high			low
2	high			low	high			low	high			low
3	equal				low	high	low		high	medium	high	low
4	low	medium	low	high	low	high	low	high	high	low	high	medium
5	low	medium	low	high	low	high	low	high	high	low	high	medium

Table 4.3 provided a qualitative assessment of the effectiveness of the traffic control methods, in terms of throughput and speed. These values came

from the graphical analysis. This summarized the conclusion that type-4 control gave the low (lowest) throughput in the unsaturated conditions, but the highest throughput in the oversaturated conditions. More distinction could be seen at the high oversaturation condition. Type-4 provided the most consistent speed throughout the entire period.

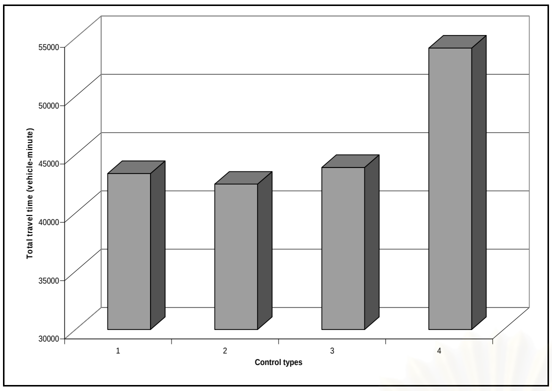
4.2.3 Numerical Analysis on Total Travel Time and Peak Throughput

The traffic performance could be summarized into numerical values. Total travel time and peak throughput were selected to be the measures of effectiveness of the system performance (yielded by operators). These values were calculated from numerical determination from simulation data. The results presented in Figures 4.7 and 4.8.

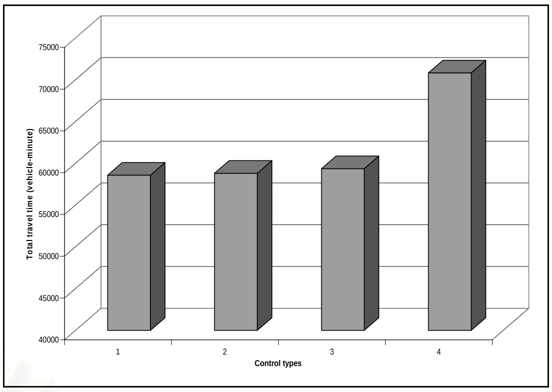
From the figures in the unsaturated condition, there was no significant difference between the results from type-4 traffic control method and each of other control methods in the view of peak throughput. For total travel time, type-4 control method cause around 19.3 to 27.5 percent more total travel time than each of other three methods.

For the saturated condition, there was no significant difference between the results from type-4 traffic control method and each of other control methods in the view of peak throughput. For travel time, type-4 control method caused around 8.9 to 10.4 percent more travel time than each of other three methods.

Finally, for the oversaturated condition, type-4 control gave the best result in both the view of total travel time and also the view of peak throughput. The improvement of total travel time was significant only in the cases of comparing to type-2 and type-3 control methods with level-5 demand. The improvement of peak throughput was significant only in the case of comparing to type-1 and type-3 control methods. The improvement were around 10.8 to 11.9 percent and 4.5 to 6.9 percent for total travel time and peak throughput respectively.

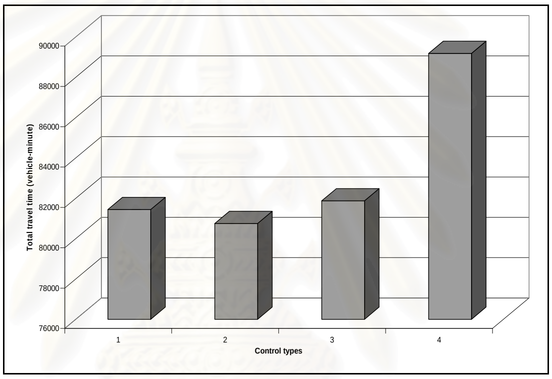


a) Demand level 1



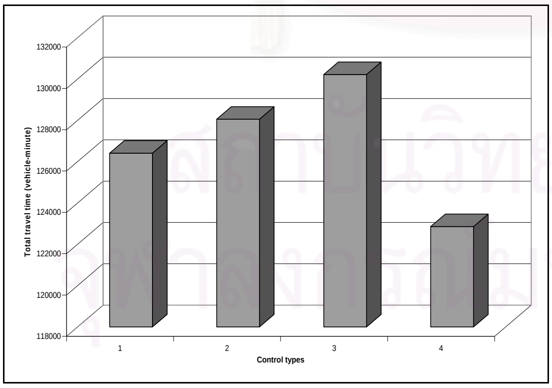
b) Demand level 2

(Unsaturated traffic condition)

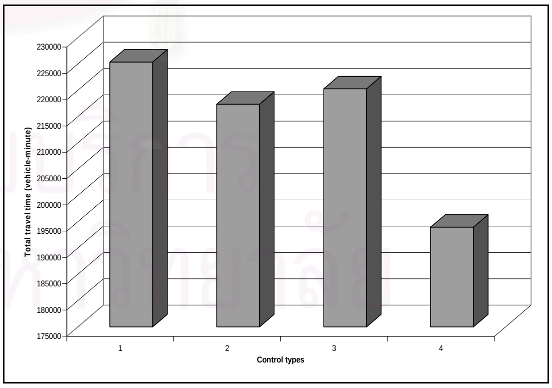


c) Demand level 3

(Saturated traffic condition)



d) Demand level 4



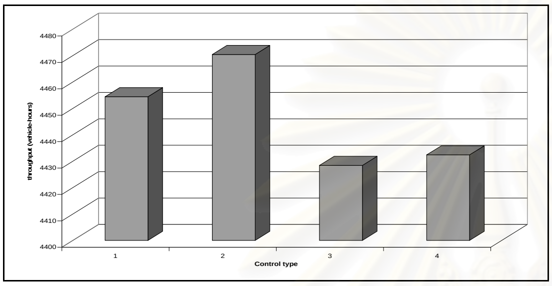
e) Demand level 5

(Oversaturated traffic condition)

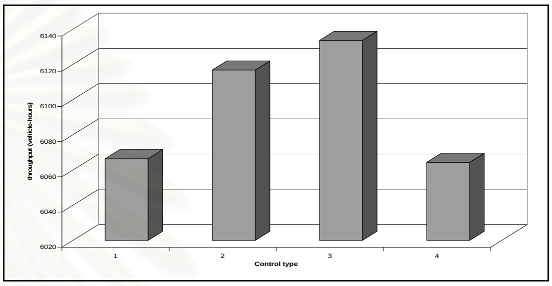
Figure 4.7 Total travel time (pattern-1 demand)

Table 4.4 Total travel time, vehicle-minute (pattern-1 demand)

Demand levels	Control type 1		Control type 2		Control type 3		Control type 4	
	Average	SD	Average	SD	Average	SD	Average	SD
1	43,379	718	42,467	661	43,900	438	54,133	896
2	58,571	337	58,801	433	59,348	940	70,811	1,978
3	81,445	2,984	80,752	1,407	81,876	1,846	89,185	1,148
4	126,414	4,485	128,059	5,206	130,221	5,844	122,860	3,652
5	225,387	14,341	217,362	3,560	220,298	5,317	193,983	4,970

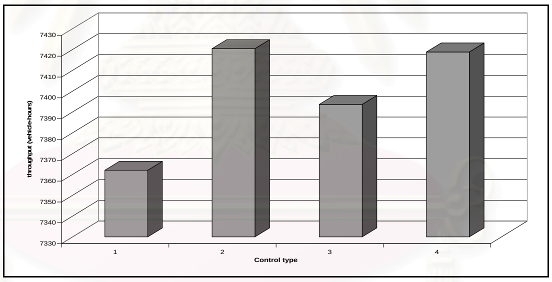


a) Demand level 1



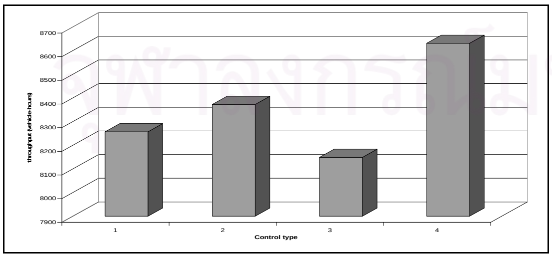
b) Demand level 2

(Unsaturated traffic condition)

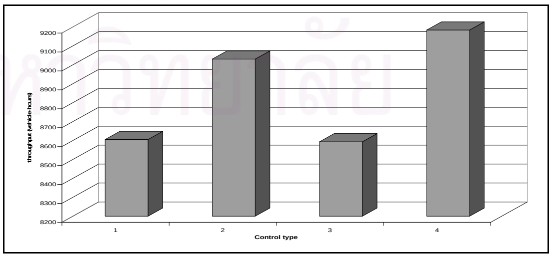


c) Demand level 3

(Saturated traffic condition)



d) Demand level 4



e) Demand level 5

(Oversaturated traffic condition)

Figure 4.8 Peak 30-minute throughput (pattern-1 demand)

Table 4.5 Peak 30-minute throughput, vehicle/hour (pattern-1 demand)

Demand levels	Control type 1		Control type 2		Control type 3		Control type 4	
	Average	SD	Average	SD	Average	SD	Average	SD
1	4,454	107	4,470	105	4,428	67	4,432	72
2	6,066	142	6,117	95	6,134	90	6,064	101
3	7,362	149	7,420	65	7,394	155	7,419	151
4	8,257	72	8,372	109	8,149	135	8,631	157
5	8,605	186	9,030	103	8,593	135	9,184	63

The results of the numerical analysis during the peak period were summarized in Table 4.6. Positive values in Table 4.6 meant that type-4 traffic control method gave higher value of that MOE than the base method while the negative values meant that type-4 traffic control method gave lower value of that MOE than the base method. Therefore, type-4 traffic control method was expected to work more effectively if the value of difference in total travel time was negative while the value of difference in peak 30-minute throughput was positive. The value with a star was a significant difference.

Table 4.6 Summary of numerical analysis for Pattern-1 demand

Scenarios	Difference in Total Travel Time (percent)			Difference in Peak 30-Minute Throughput (percent)		
	Type-4 from Type-1	Type-4 from Type-2	Type-4 from Type-3	Type-4 from Type-1	Type-4 from Type-2	Type-4 from Type-3
1	24.8*	27.5*	23.3*	-0.5	-0.9	0.1
2	20.9*	20.4*	19.3*	0.0	-0.9	-1.1
3	9.5*	10.4*	8.9*	0.8	0.0	0.3
4	-2.8	-4.1	-5.7	4.5*	3.1	5.9*
5	-13.9	-10.8*	-11.9*	6.7*	1.7	6.9*

* significant at 99%

The results from the Table 4.6 confirmed the findings from the previous sections that type-4 control did not give much benefits over other methods during the peak period. As stated earlier, the benefits of the type-4 control came from the ability to not only manage traffic during the peak hour, but also to manage traffic during the dissipation period. By providing a good preparation and maintaining a manageable level of link congestion during the peak period, the type-4 control brought about the highest overall performance in the oversaturated traffic conditions.

4.3 Results for Scenarios with Cross Demand (Patern-2 Demand)

This section presented the results from the case of having cross demand as described in Chapter III, the second pattern of demand for the test.

4.3.1 Network Condition

Firstly, for the unsaturated level, the road systems filled with each same amount of demand were graphically illustrated in each of Figure 4.9 a) and 4.9 b).

Level-1 and level-2 demand were planned at 60 percents and 80 percents of the saturated one in the peak hour, respectively. There were some very short queues occurring in both of them. Note that these scenarios reflected the condition that the demand was less than the capacity, an unsaturated condition.

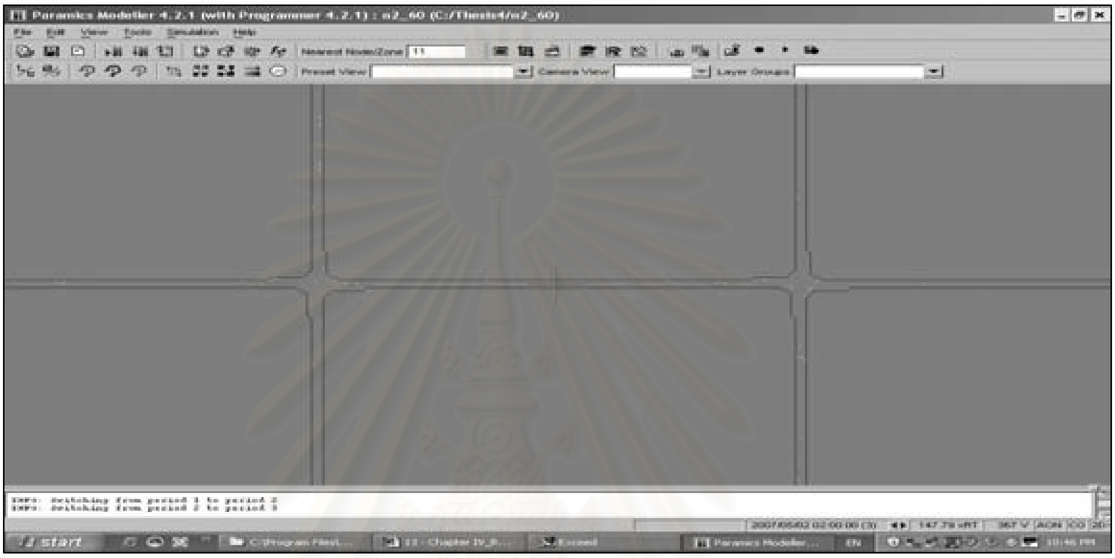
Then, the system filled with each saturated level of demand was graphically shown in Figure 4.9 c).

Level-3 demand was the saturated one. Some queues waiting for the next green time could be seen. This condition reflected the condition that the demand was almost equal to the capacity, a saturated condition.

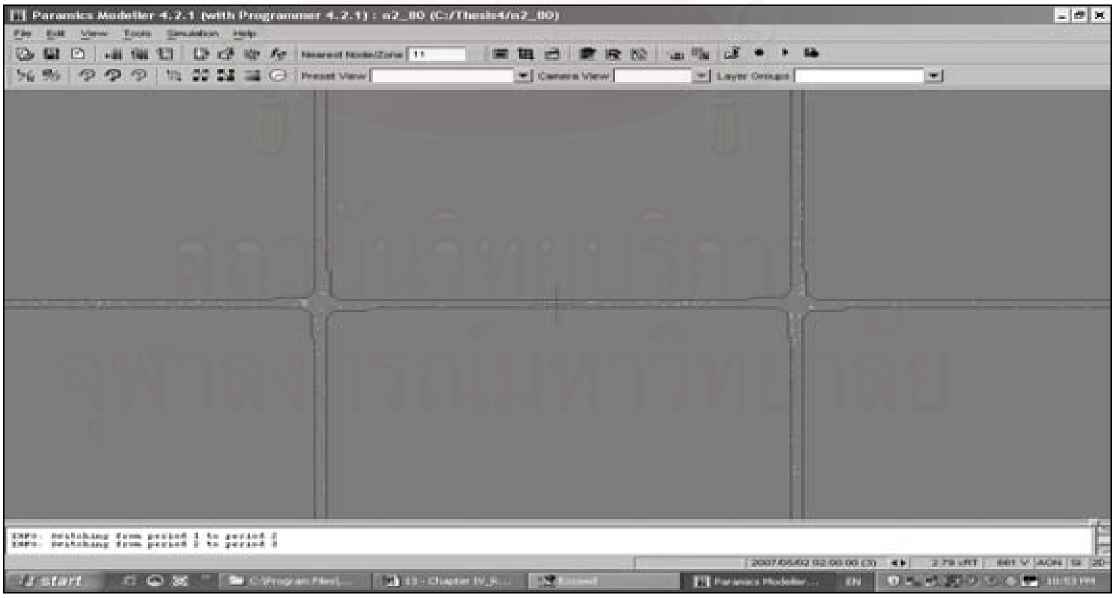
The systems filled with oversaturated levels of demand were graphically shown in each of Figures 4.9 d) and 4.9 e).

Level-4 and level-5 demands were 120 percents and 140 percents of the saturated one in the peak hour, respectively. Some long queues were experienced both cases. The longer queues were seen in the latter case. Note that the queue grew and reached the highest length approximately at the end of the peak periods. The figures illustrated the condition that the demand exceeded the capacity, an oversaturated condition.

Figure 4.1 was captured at the end of the peak hour when it is expected to have the highest average length of queues.

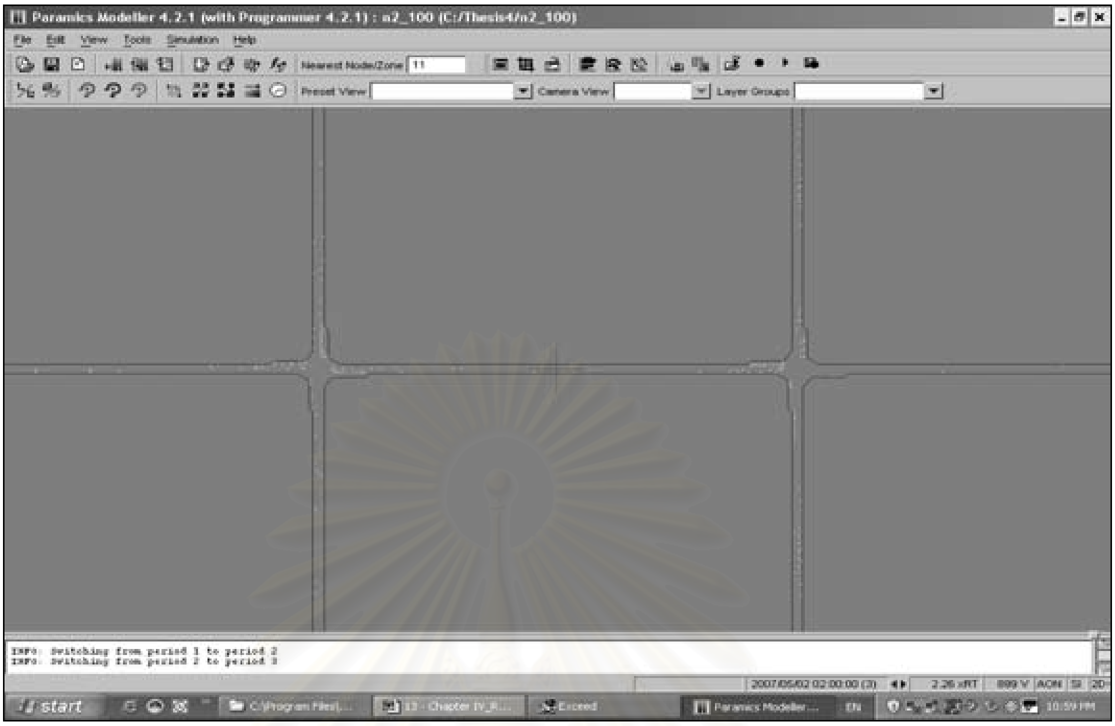


a) Demand-level 1 (unsaturated traffic condition)

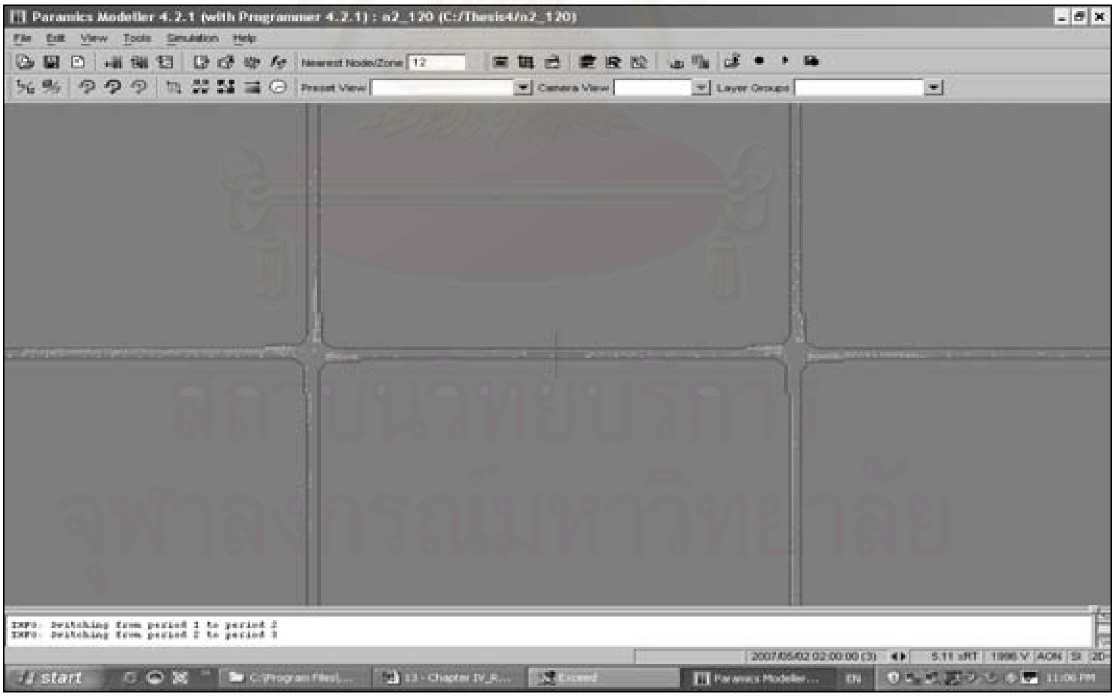


b) Demand-level 2 (unsaturated traffic condition)

Figure 4.9 Network conditions (pattern-2 demand)

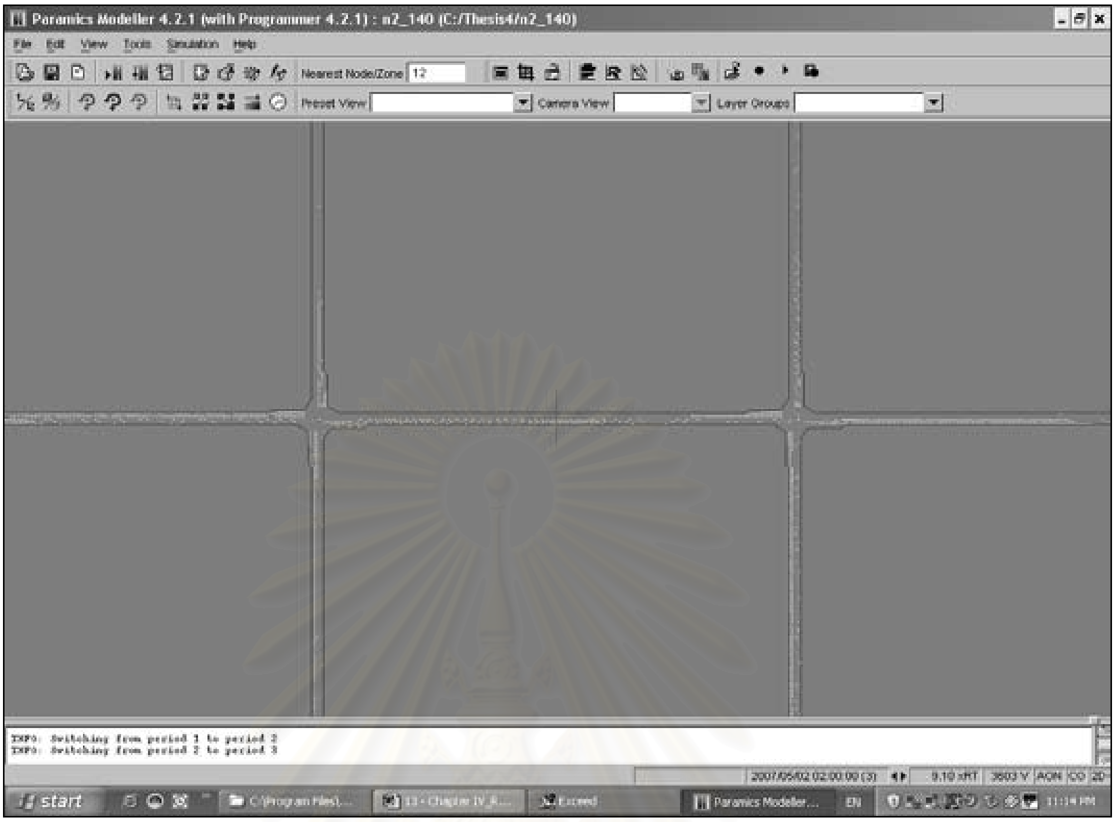


c) Demand-level 3 (saturated traffic condition)



d) Demand-level 4 (oversaturated traffic condition)

Figure 4.9 Network conditions (pattern-2 demand) (continued)



e) Demand-level 5 (oversaturated traffic condition)

Figure 4.9 Network conditions (pattern-2 demand) (continued)

Figure 4.10 displayed the overall pictures of the performances of each control method, expressed by the ability to release (hold) the vehicles out of the road system. Figures 4.10 a) and 4.10 b) illustrated the number of vehicles in the road network, which was one of principle measures of effectiveness. The figures showed that type-4 control made the whole system with these unsaturated levels of demand carried the highest amount vehicles, while the other three controls perform similarly. The figures showed that with low demand, the dynamic control (type-4) did not work so effectively, as the control held vehicles on the road longer and yielded a large number of vehicles on the network, not only in the peak period but also after the peak period when the queue accumulated during the peak time dissipated. Generally, the effectiveness was lower when the demand was lower (Figure 4.10 a) compared to Figure 4.10 b)).

Figure 4.10 c) showed the traffic control performance when the road was loaded at approximately capacity. The test controls performed differently from the unsaturated scenarios, in terms of where to hold the traffic and overall performances. It was seen from the Figure that the type-2 control allowed the smallest number of vehicles to store on the inner links during the peak hour. On the outer links, the type-2 and type-4 controls let more vehicles stay on the links than the other two methods during the peak hour. During the queue dissipation period, controls type-1 and type-3 performed similarly. For type-2 control, the queue on outer links rapidly disappeared until it averagely reached the same length as type-1 and type-3 controls while control type-4 held many vehicles on both inner and outer links. Note that the control type-4 had a poor ability to release the traffic out of the links during the queue dissipation period, as observed on both inner and outer links.

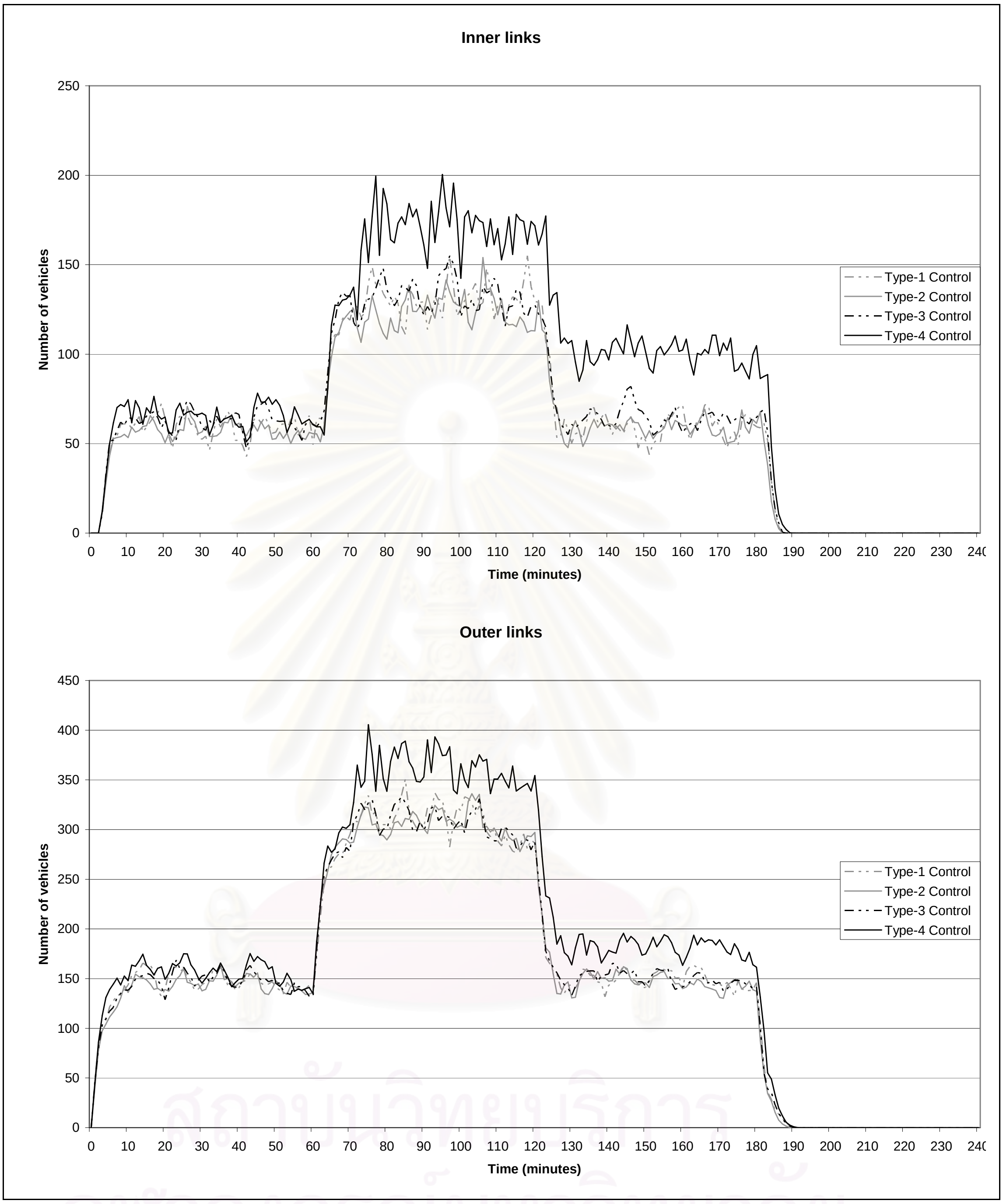
Figures 4.10 d) and 4.10 e) showed the effectiveness of each control method in releasing traffic out of the road network in the oversaturated conditions. During these conditions, the effect of traffic control and effect of oversaturation (queue spillback) made different traffic patterns from the unsaturated and saturated traffic conditions, as indicated by different shapes of traffic congestion formulation on both inner and outer links. Compared to Figures 4.10 a) to 4.10 c), the number of vehicles on the links were increasing from the beginning until the end of peak time period.

In these oversaturated traffic conditions, the differences in the effectiveness of traffic control were clearly seen. From the Figures, the type-1 and type-3 controls held a larger number of vehicles on the inner links, while the type-2 and type-4 controls let more vehicles stay on the outer links than the other two methods proportionally. The orders of the control methods holding the vehicles on the inner links (from highest to lowest) were type-1, type-3, type-4, and type-2, in the 120 percent oversaturation case, and type-3, type-1, type-2, and type-4 in the 140 percent oversaturation case, while the order of the control methods holding the vehicles on the outer links were type-2, type-4, type-3, and type-1 in the 120 percent oversaturation case and type-4, type-2, type-1, and type-3 in the 140 percent oversaturation case. It implied from the figures that the types of traffic control placed significant effects on levels of congestion on each link. The type-3 and type-1 controls held a large number of vehicles on the inner links, making these links very congested during the peak period. The number of vehicles holding on the inner links was approximately as twice of those from type-4 and type-2 control at the end of the peak period. Type-2 and type-4 controls held a larger number of vehicles on the outer links proportionally. Note that Type-2 control held a larger number of vehicles on the out links than type-4 control in the 120 percent oversaturation, but less number of vehicles in the 140 percent oversaturation. The amount of the vehicles that were held by the type-2 control was approximately 50 percent greater than those by type-1 and type-3 control at the end of the peak period.

It was observed from the Figures 4.10 d) and 4.10 e) that, at the end of peak time (120 minutes after the beginning of the simulation), the accumulation of vehicles on the links still continued, especially the inner links. Type-1 and type-3 controls still created the vehicle accumulation until reaching the highest number of vehicles some times later (approximately 10 minutes after the peak time in the 120 percent oversaturation and 20 minutes after the peak time in the 140 oversaturation case). This was one important consideration since, the more accumulation and the later in time, the more difficult on congestion dissipation.

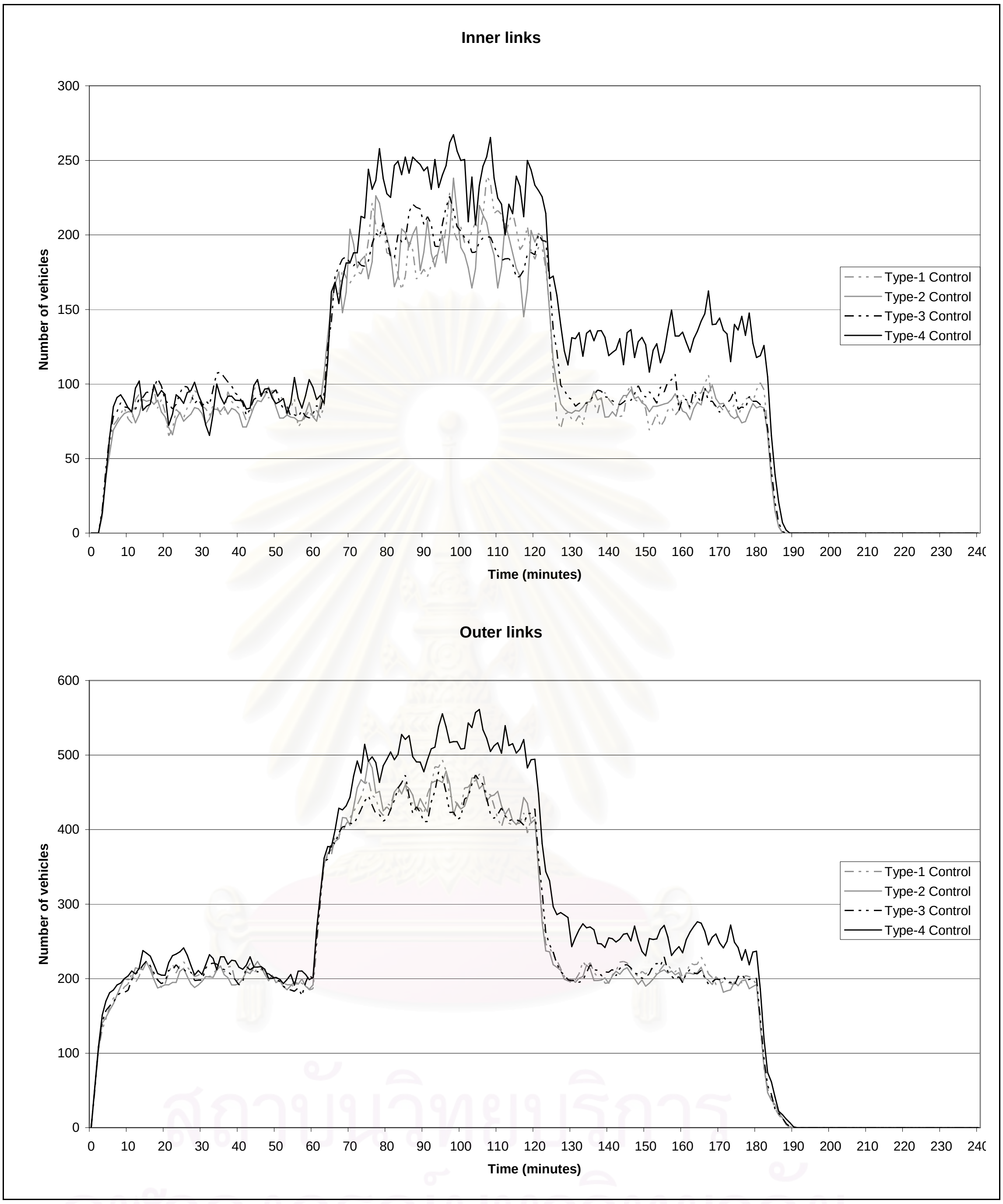
After the end of peak time, the number of vehicles stored in each part of the system was decreasing until it reaches stability. From Figures 4.10 d) and 4.10 e), the rates of reduction in the number of vehicles stored on each link type were various, depending on the amount and time (the highest number of vehicles stored) and the effectiveness of the traffic control to dissipate the queue. From Figure 4.10 d), type-2 and type-4 controls had an ability to dissipate traffic on the inner links and reach the stability level sooner than type-1 and type-3 controls, although the rates of reduction were less than those of type-1 and type-3 controls. This was clearly seen from Figure 4.10 e) that type-2 and type-4 controls made traffic reach stability approximately 20 minutes sooner than type-1 and type-3 controls (approximately 10 minutes sooner in the 120 percent oversaturation case in Figure 4.10 d)). For the case of 140% saturation, the period of stability after queue dissipation was very short because it started only a few minutes before the third hour ends. The slow rate of reduction in the number of vehicles on the inner links meant the type-2 and type-4 controls managed the traffic on the congested links under the existence of waiting demand from the outer links.

It could be viewed that during the dissipation period (the time period after the peak time when the vehicles were released from the links and the traffic conditions turned to normal congestion level (stability)), the outer links where the excessive demand were stored during the peak period released the traffic onto the inner links. The rate of the decrease in the vehicle accumulation was dependent to the ability of the inner links to accept these amounts of traffic. Figure 4.10 d) illustrated that each control method took different time period to get back to normal (stability). The type-2 control, which developed the highest congestion on the outer links, took longer time to be back to stability. The same trend could be observed in Figure 4.10 e). Note that type-4 control could perform better than type-2 control in bringing the level of congestion (number of vehicles on links) down to stability. However, the type-2 control was not as good as type-1 and type-3 controls, in terms of the time to reach stability and the level of traffic at stability (a larger number of vehicles was maintained even during the stability period). It was remarked that the type-4 control maintained the level of traffic at stability until the end of demand period (the 180th minute in the simulation). Type-2 control created the longest dissipation period, as clearly observed in Figure 4.10 e) on both inner and outer links. It took more than 40 minutes longer to clear the entire traffic out of the network at the end of simulation study.



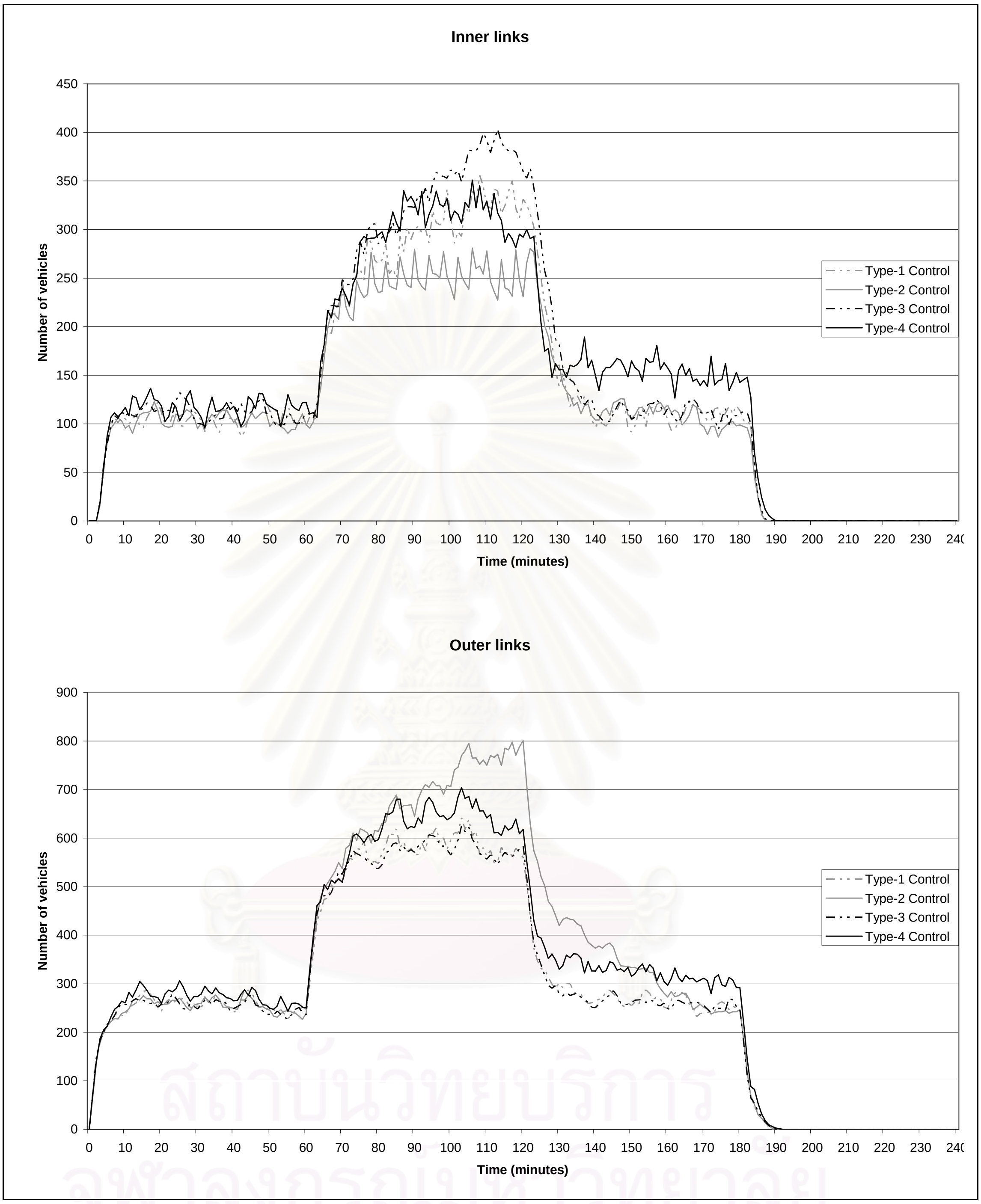
a) Demand level 1 (unsaturated traffic condition)

Figure 4.10 Number of vehicles containing in the system (pattern-2 demand)



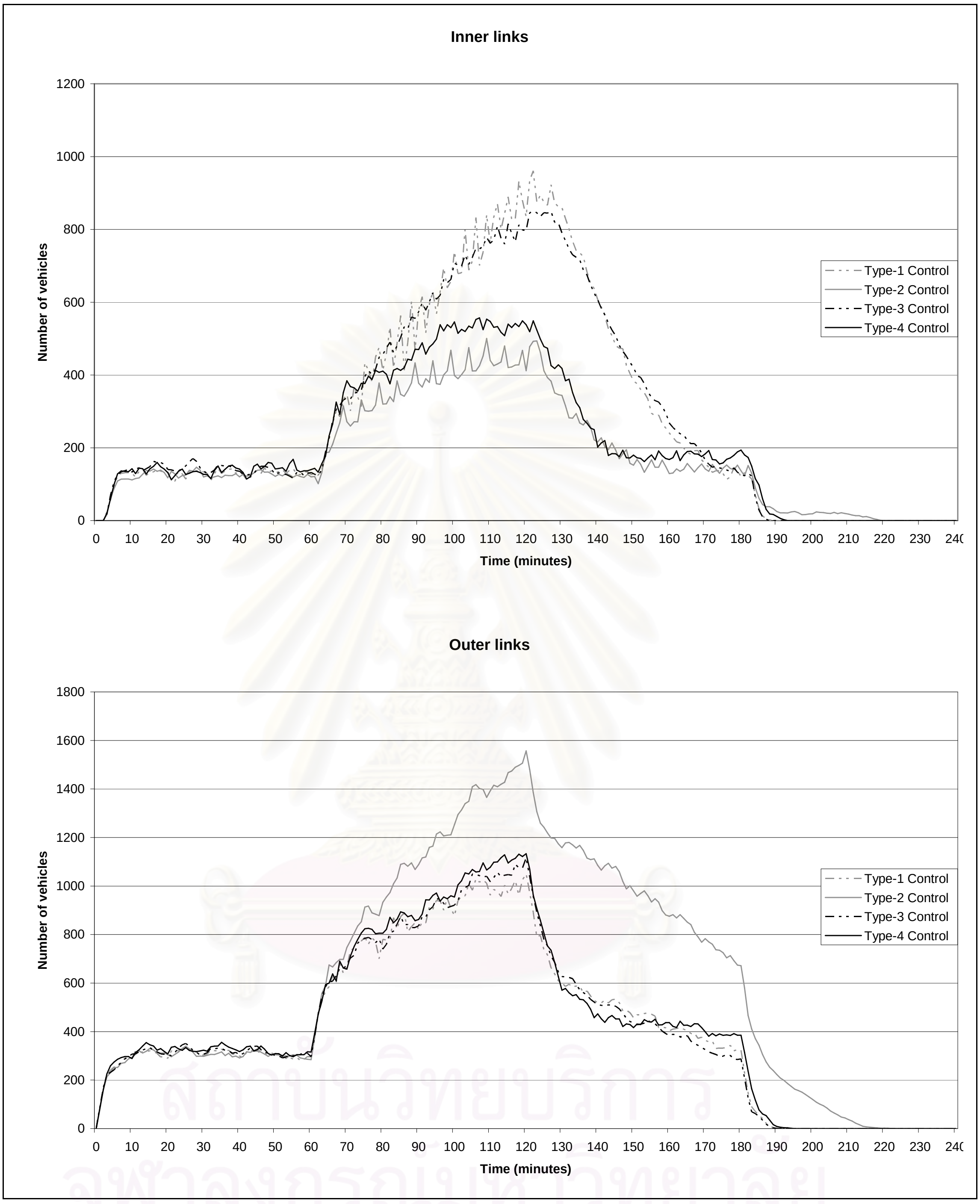
b) Demand level 2 (unsaturated traffic condition)

Figure 4.10 Number of vehicles containing in the system (pattern-2 demand) (continued)



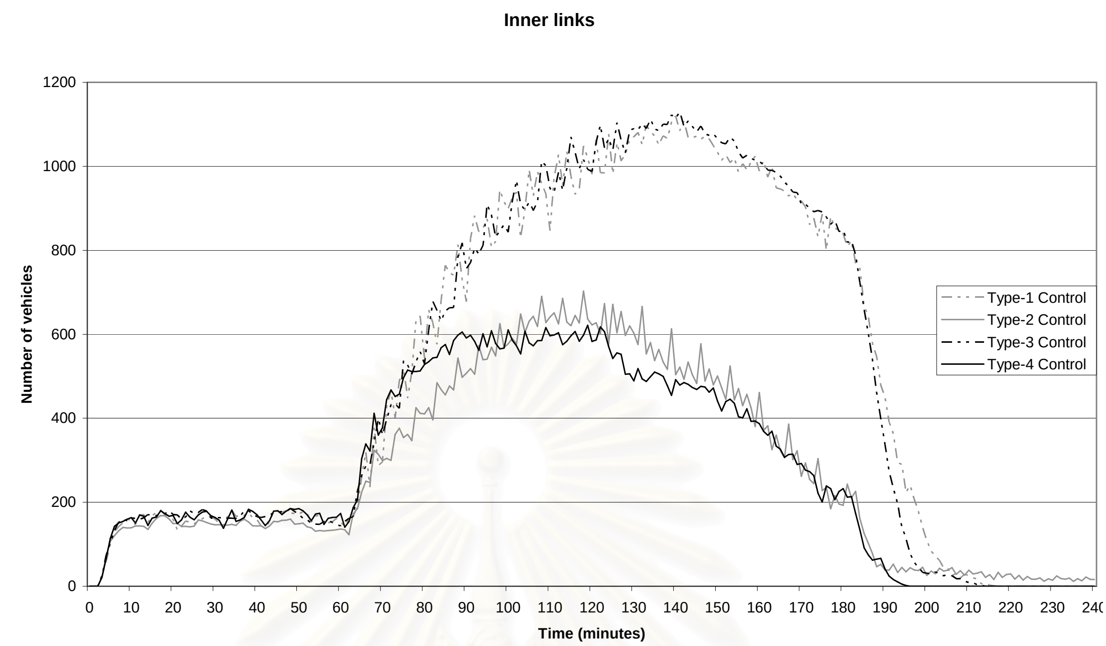
c) Demand level 3 (saturated traffic condition)

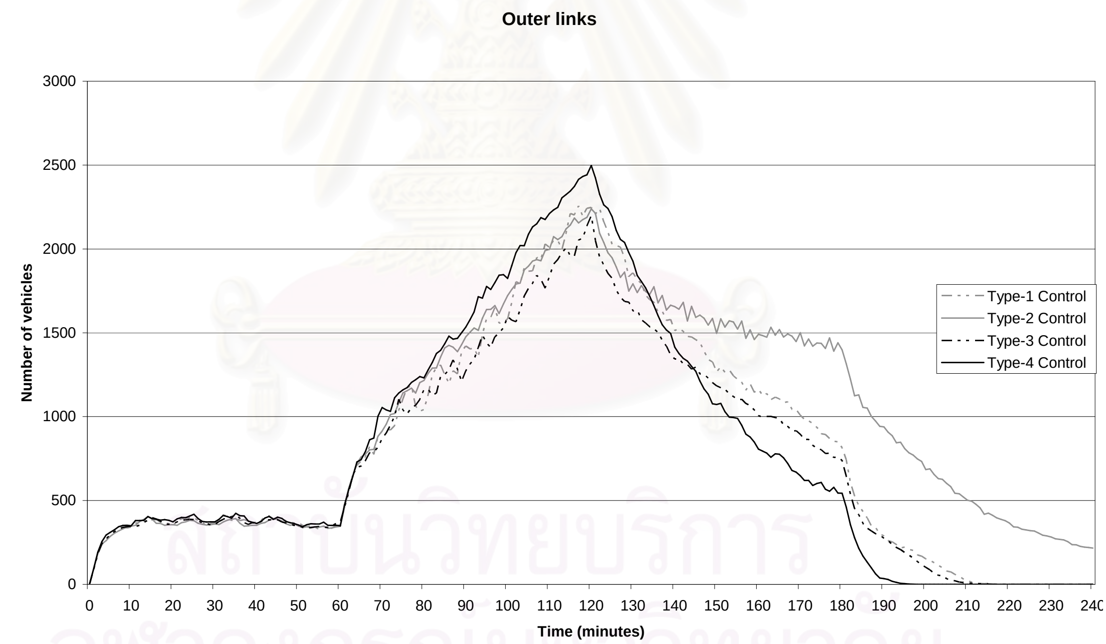
Figure 4.10 Number of vehicles containing in the system (pattern-2 demand) (continued)



d) Demand level 4 (oversaturated traffic condition)

Figure 4.10 Number of vehicles containing in the system (pattern-2 demand) (continued)

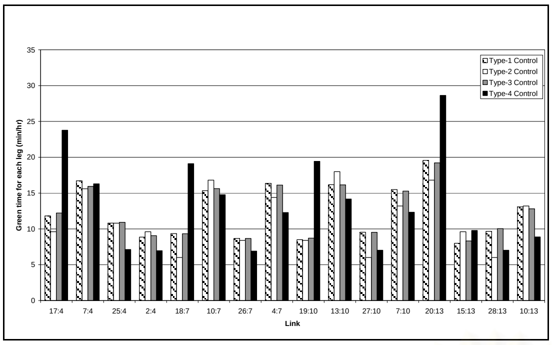




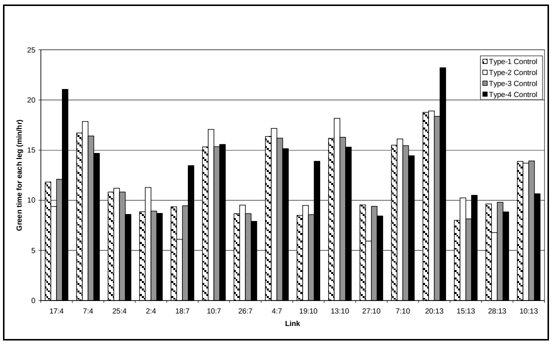
e) Demand level 5 (oversaturated traffic condition)

Figure 4.10 Number of vehicles containing in the system (pattern-2 demand) (continued)

The effectiveness of the traffic control came from the ability to allocate proper green time, both in terms of duration and timing. In principle, considering an average green time period given to each link during a peak hour, if the first

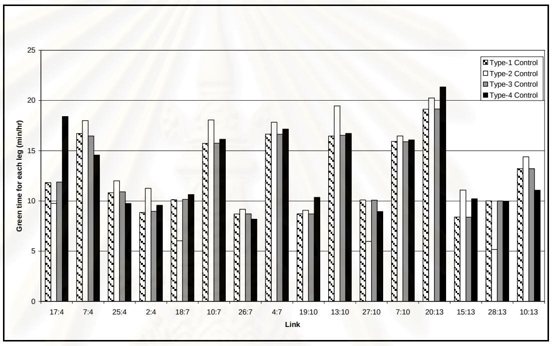


a) Demand level 1



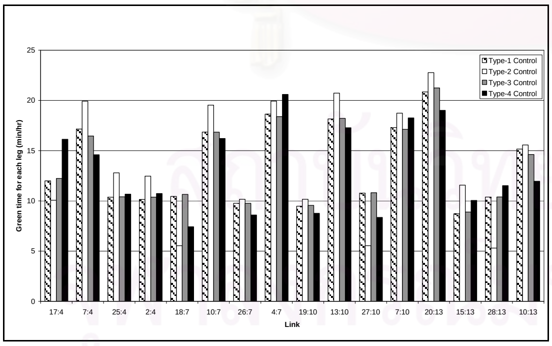
b) Demand level 2

(Unsaturated traffic condition)

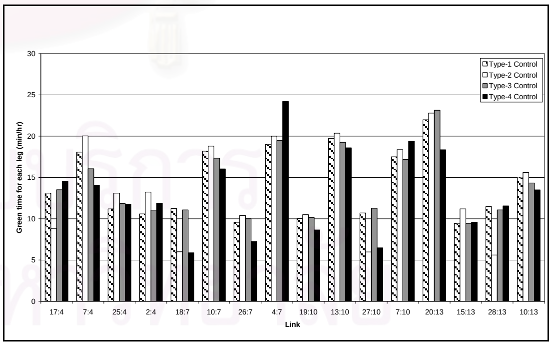


c) Demand level 3

(Saturated traffic condition)



d) Demand level 4



e) Demand level 5

(Oversaturated traffic condition)

Figure 4.11 Average green-time given to each link during a peak hour

(pattern-2 demand)

control type was said to equally give the green time to each leg due to the v/c ratio on the leg, type-4 control aimed to give more priority only to the legs that seemed to produce the maximum throughput. The type-2 and type-3 control could be inferred to the methods described in Chapter III. Thus, it was important to learn about the green time allocation and this consideration could be viewed as the total green time period allocated to each link, the examples of green time allocation on selected links were shown in Figure 4.11 for pattern-2 demand. The figure showed only the legs of the intersections on the main corridors.

As seen from Figure 4.11, the green time allocation (averaged from several simulation runs for the same scenarios) under different control methods varied. However, since this figure presented the total green time period, it could not determine the proper timing of the allocation. Table 4.7 summarized the range of green time during the peak period for each leg. It indicated that each control method gave different green time period. As different performances of the control were seen from the Figure 4.7, it implied that the allocation of green time was attributable to the effectiveness of the overall control.

Table 4.7 Summary of network conditions for pattern-2 demand

Scenarios	Maximum Number of Vehicles Containing on inner links				% Difference			Maximum Number of Vehicles Containing on Outer links				% Difference			Average Green Time Range Among Legs of An Intersection During A Peak hour (Minutes)			
	Type-1 Control	Type-2 Control	Type-3 Control	Type-4 Control	Type-4 from Type-1	Type-4 from Type-2	Type-4 from Type-3	Type-1 Control	Type-2 Control	Type-3 Control	Type-4 Control	Type-4 from Type-1	Type-4 from Type-2	Type-4 from Type-3	Type-1 Control	Type-2 Control	Type-3 Control	Type-4 Control
6	155	154	155	200	29	30	29	351	336	333	406	16	21	22	8.71	9.90	8.17	15.77
7	239	238	226	267	12	12	18	493	493	477	561	14	14	18	8.51	10.98	8.04	10.35
8	356	281	403	351	-1	25	-13	642	801	625	704	10	-12	13	8.58	12.22	8.50	9.24
9	961	500	849	557	-42	11	-34	1038	1557	1116	1133	9	-27	2	9.17	14.22	8.93	9.38
10	1115	703	1129	621	-44	-12	-45	2256	2239	2196	2497	11	12	14	9.78	14.20	9.32	10.69

Table 4.7 also summarized the maximum number of vehicles containing on the inner links and outer links in each scenario due to traffic control types. It showed that when comparing the devised control method type-4 with others, the type-4 control maintained a higher maximum number of vehicles on the inner and outer links when the traffic condition was unsaturated. During oversaturation, however, the type-4 control held the vehicles at the outer links,

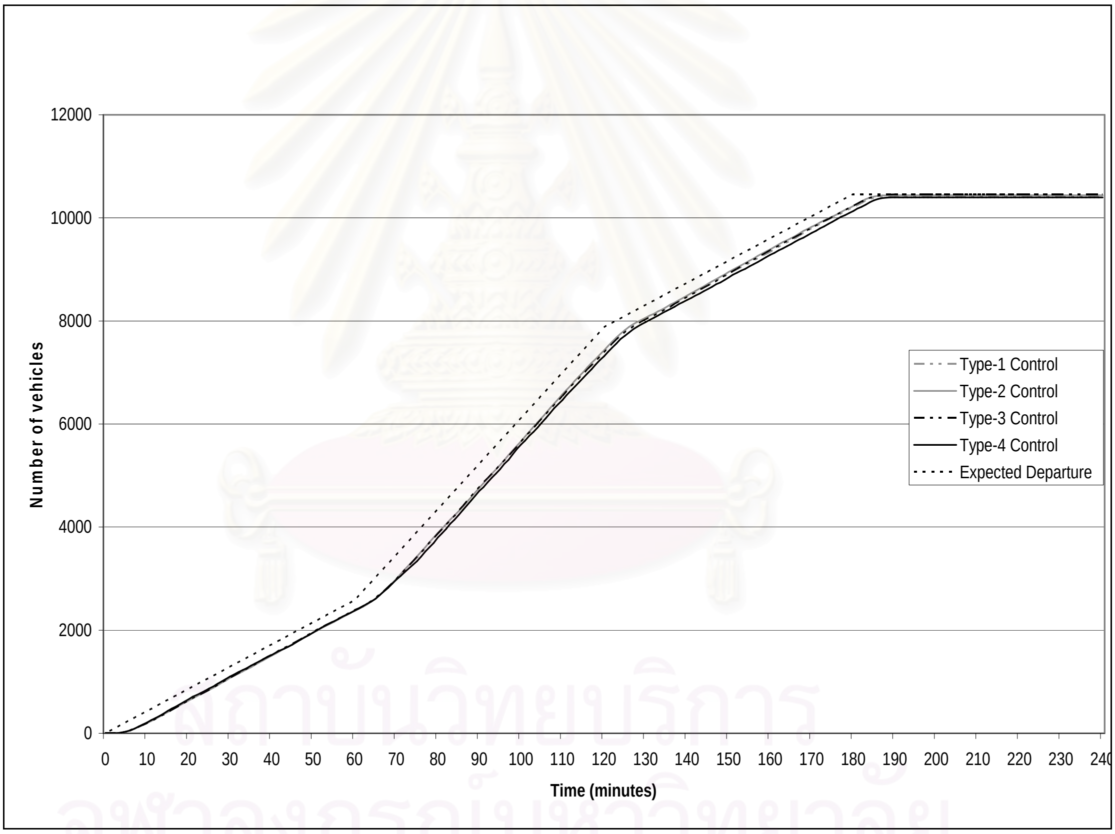
creating a higher maximum number of vehicles at these outer links (except when compared to type-2 control). Under the 140 percent oversaturation, the type-4 control maintained the minimum amount of vehicles on the inner links.

4.3.2 Graphical Analysis on Throughput and Speed

Cumulative plots of the number of vehicles arriving their destination over time were presented in Figure 4.12. The plots showed the ability of the control to let road users accomplish their goals and reach destinations. The higher number of vehicles implies a better effectiveness. For an unsaturated condition, Figures 4.12 a) and 4.12 b) showed that the order of effectiveness during the peak and dissipation periods was type-2, type-1, type-3, and type-4. Note that the type-4 control method had not produced much different effectiveness in handling the peak demand compared to other three methods. Actually, because the demand was less than the capacity, the result should be like this and the difference should mainly depend on the way that each control managed some short queues, lining up at each intersection.

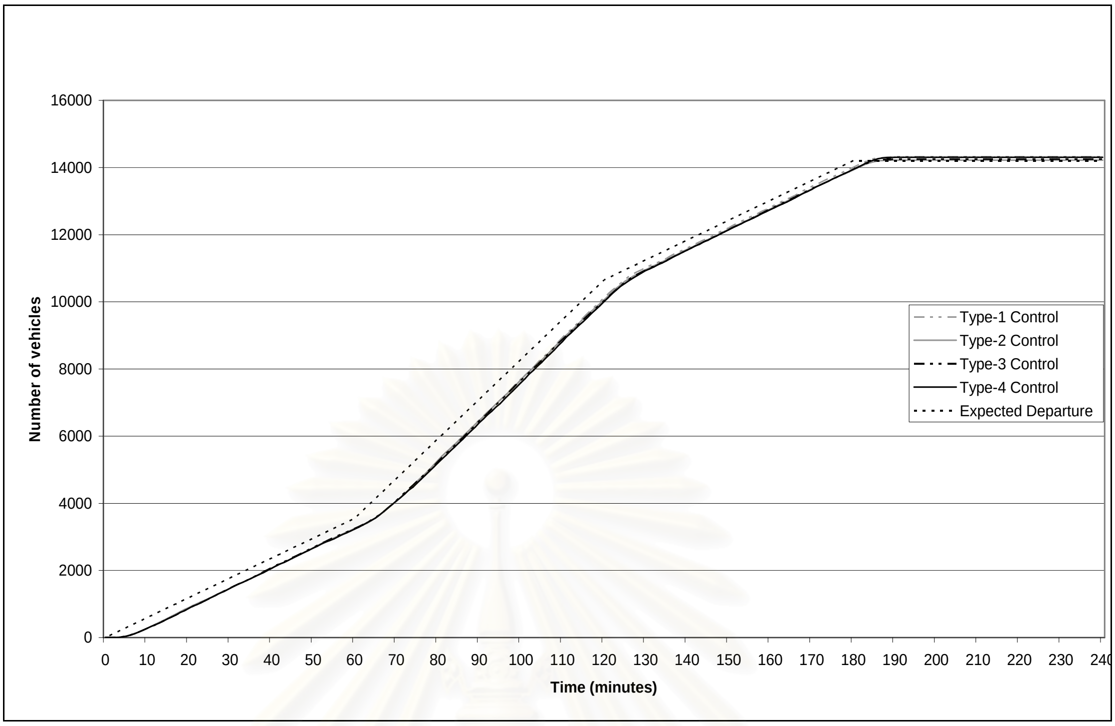
For a saturated condition, the curves in Figure 4.12 c) showed that all control methods performed very similarly during the peak period. However, during the dissipation period, the order of effectiveness (high to low) was as follow: type-1, type-2, type-3, and type-4, respectively. Again note that type-4 control method had almost equal efficiency in handling the peak demand compared to other three methods.

For an oversaturated condition, the curves in Figures 4.12 d) and 4.12 e) showed that all control methods performed similarly during the peak time, except the type-1 in the cases of 120 and 140 percent oversaturation and type-3 control in the 140 percent oversaturation. However, during the queue dissipation, the control methods gave various effectiveness. Under 120 percent oversaturation, type-4 and type-2 controls performed better at the beginning of the dissipation period but performed worse at the end of the period. Under 140 percent oversaturation, type-4 control method performed the best during the dissipation period. Type-2 control performed well at the beginning of the period, but it became the worst control at the end of the period. It could also be seen that the type-2 control took a longer time to clear the entire vehicles out of the system at the end of simulation period. From Figure 4.12 e), the order of effectiveness (high to low) of the control methods in clearing the vehicles out of the network (seen at the end of the simulation period) was type-4, type-3, type-1, and type-2, respectively. The findings confirmed the conclusion on the control effectiveness from the consideration of the vehicles containing on the links in the previous section.

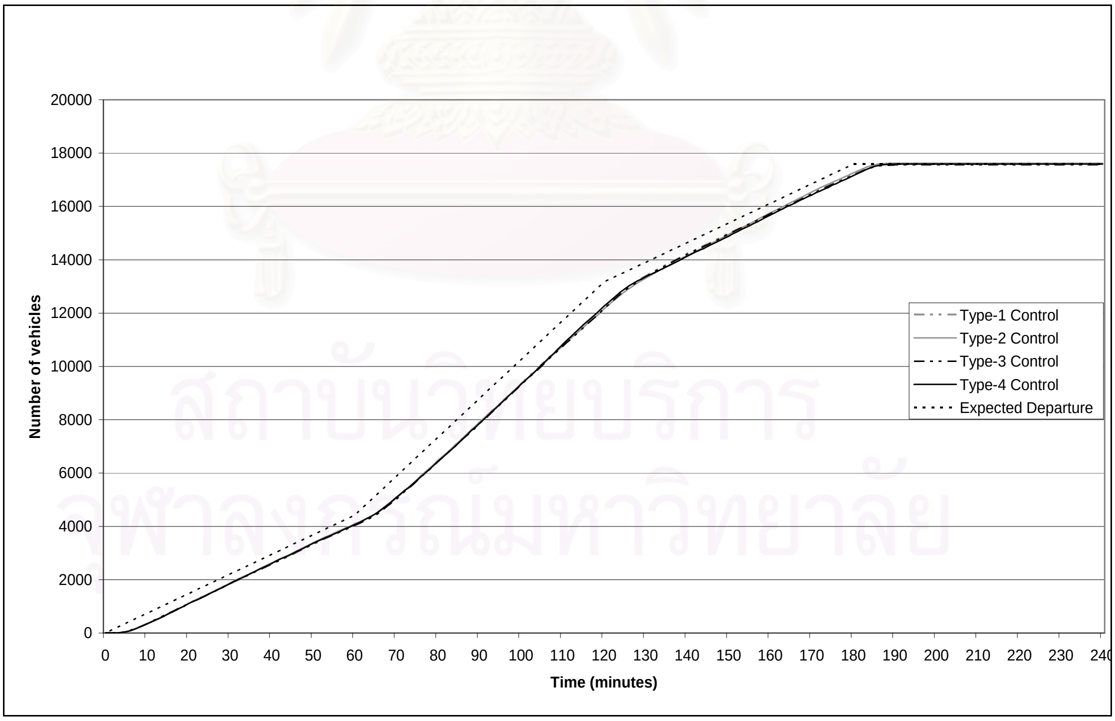


a) Demand-level 1 (unsaturated traffic condition)

Figure 4.12 Arrival at all destinations (pattern-2 demand)

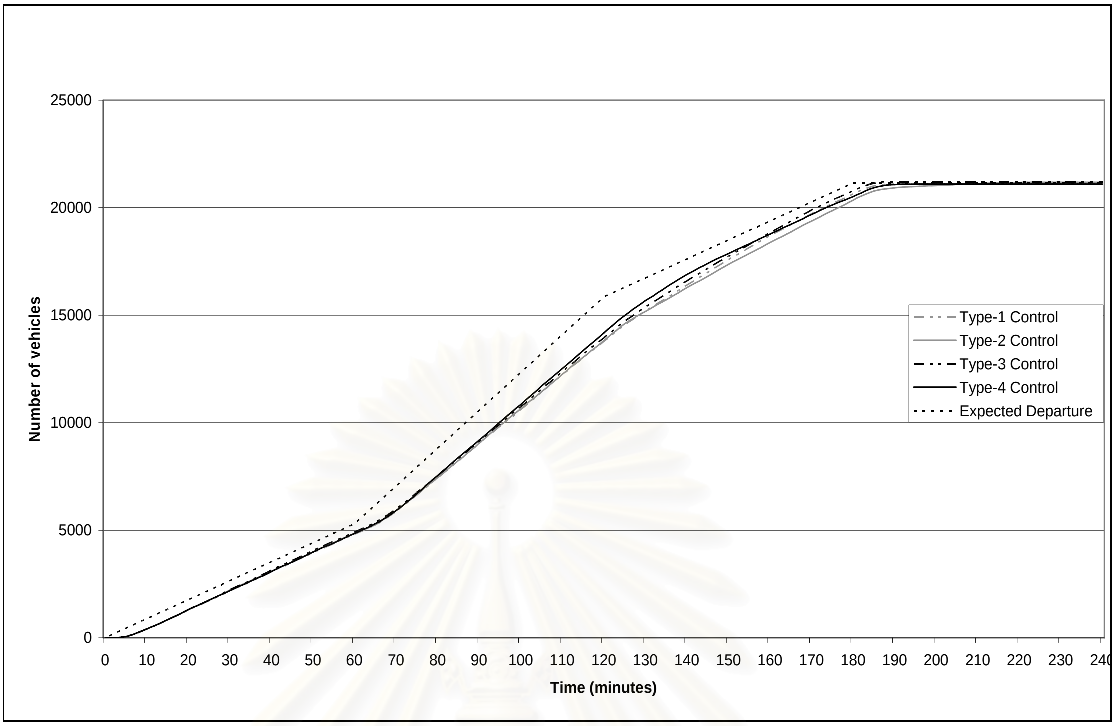


b) Demand-level 2 (unsaturated traffic condition)

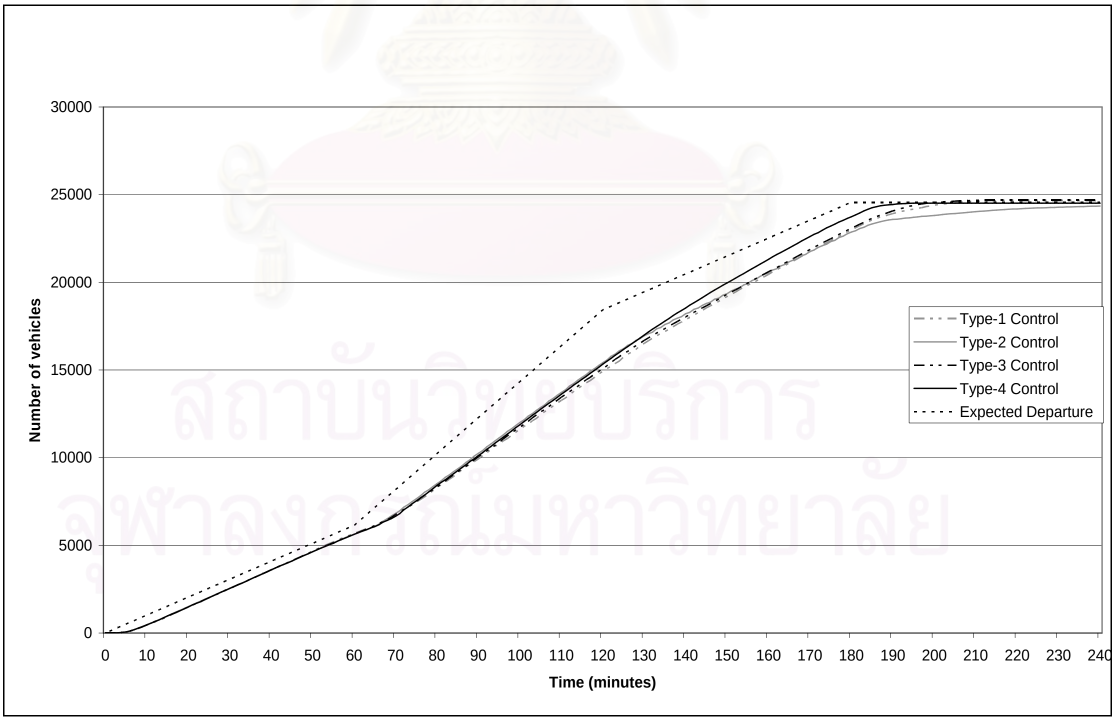


c) Demand-level 3 (saturated traffic condition)

Figure 4.12 Arrival at all destinations (pattern-2 demand) (continued)



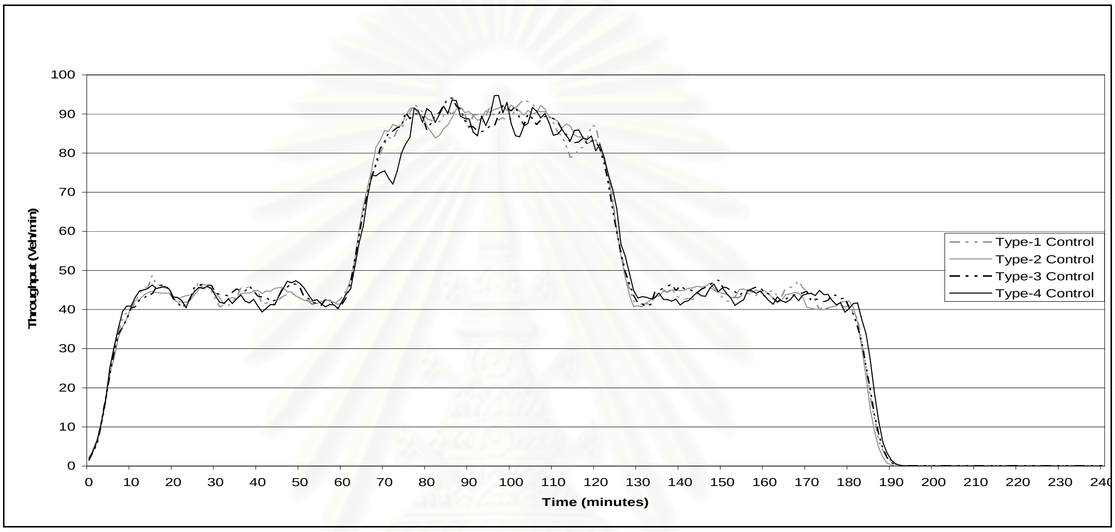
d) Demand-level 4 (oversaturated traffic condition)



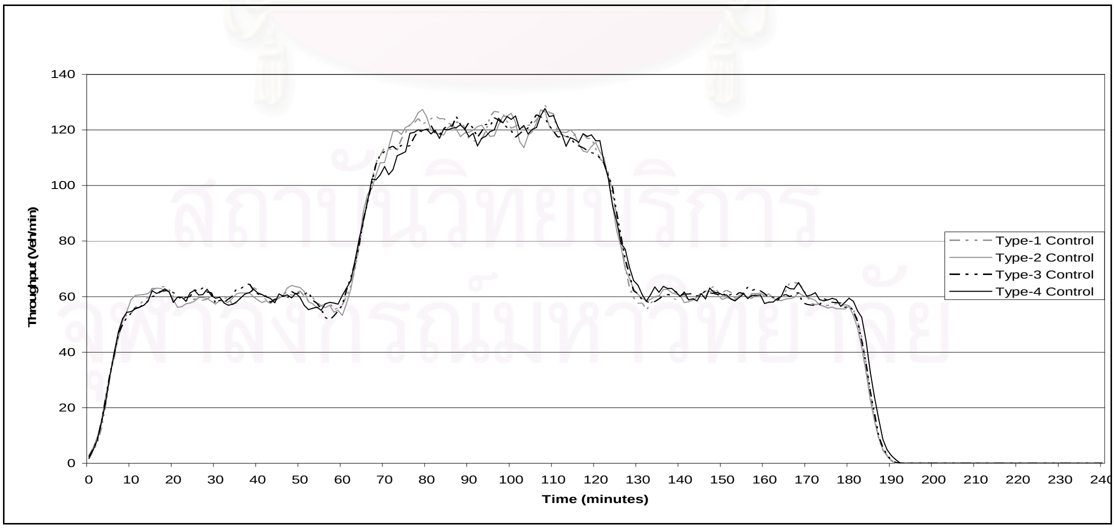
e) Demand level 5 (oversaturated traffic condition)

Figure 4.12 Arrival at all destinations (pattern-2 demand) (continued)

Apart from the cumulative plots, the throughput vs time could be presented to obtain the effectiveness of each control method at a particular time. Figure 4.13 presented the throughput or number of vehicles arriving their destination over time under various traffic conditions. This consideration was not the same as throughput of the intersection where the number of vehicles departing the stop-line was used as a measure of throughput.

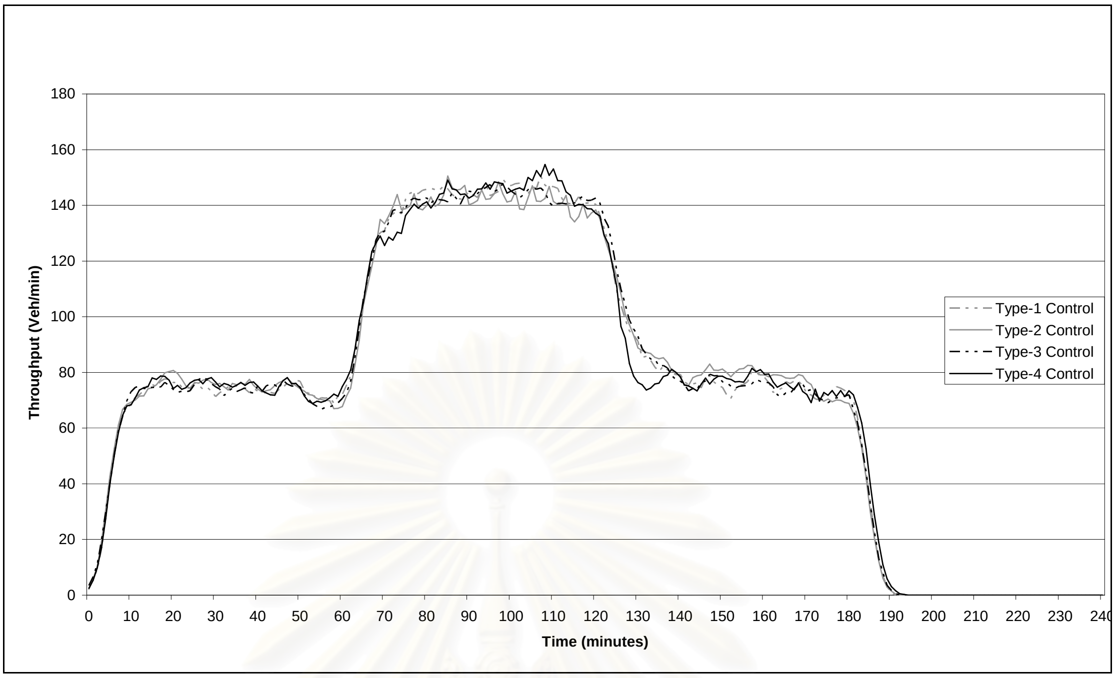


a) Demand-level 1 (unsaturated traffic condition)

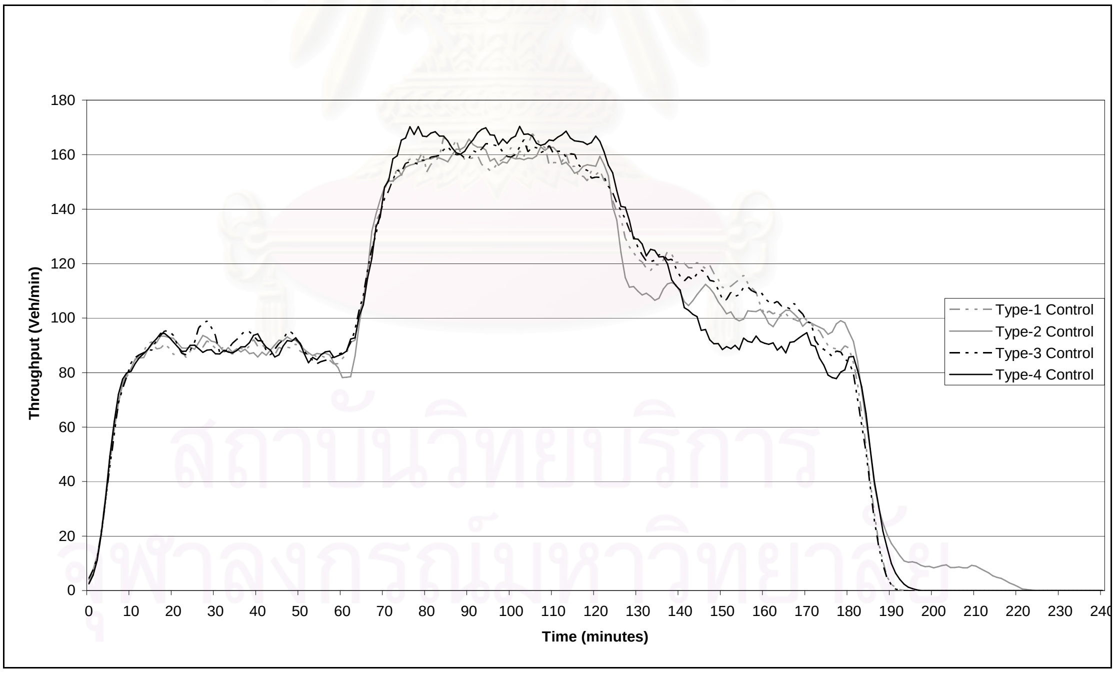


b) Demand-level 2 (unsaturated traffic condition)

Figure 4.13 Real-time throughput (pattern-2 demand)

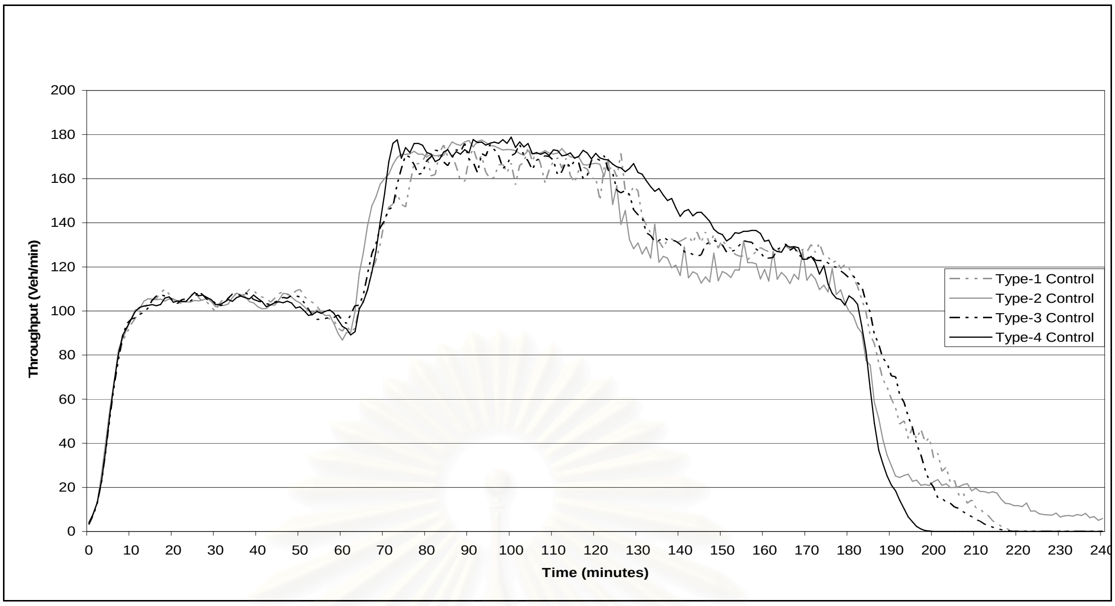


c) Demand-level 3 (saturated traffic condition)



d) Demand-level 4 (oversaturated traffic condition)

Figure 4.13 Real-time throughput (pattern-2 demand) (continued)



e) Demand-level 5 (oversaturated traffic condition)

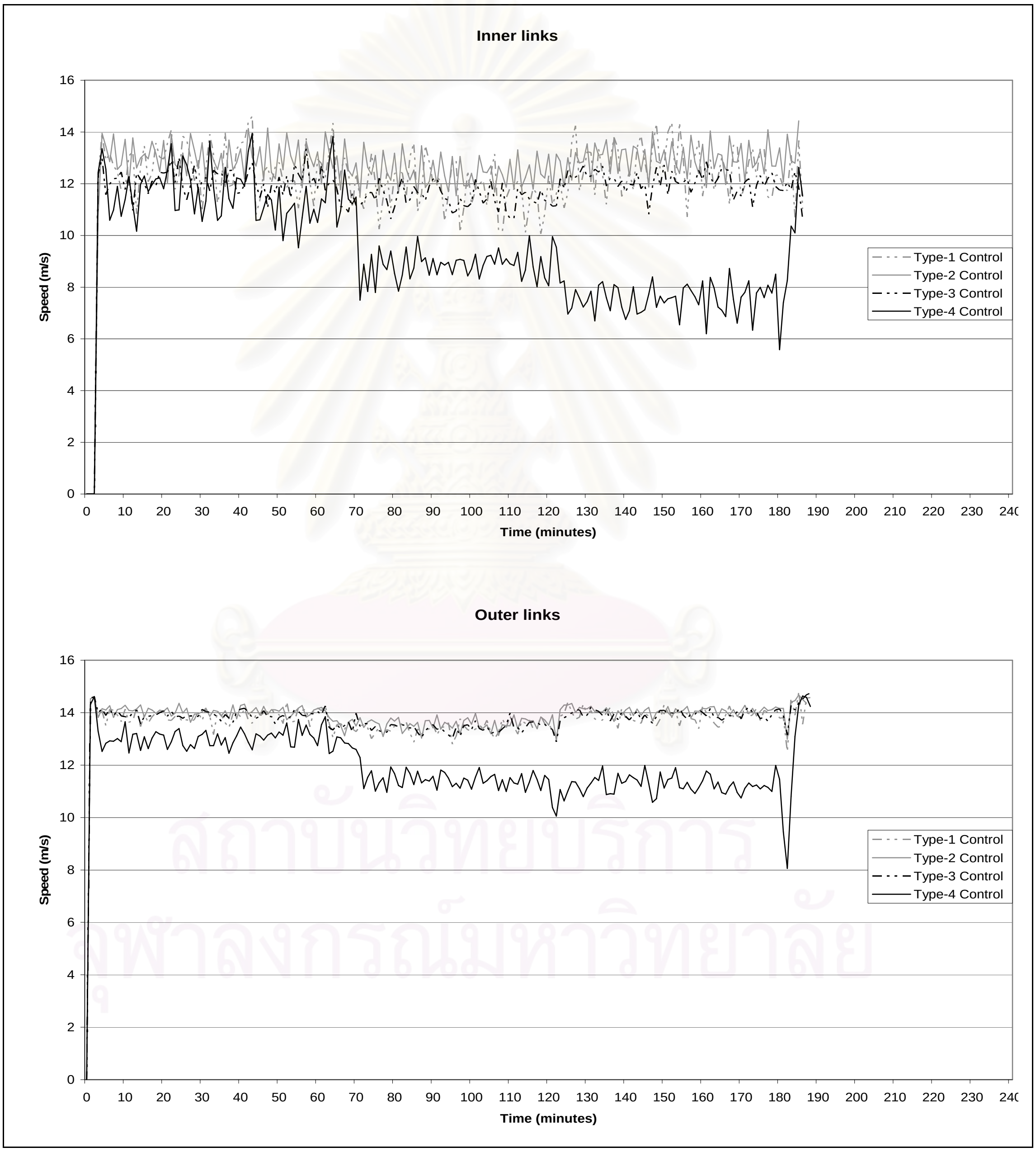
Figure 4.13 Real-time throughput (pattern-2 demand) (continued)

Figure 4.13 yielded the same conclusion as those from Figure 4.12. However, the differences in control effectiveness during time periods were more discernable. In general, all control methods performed similarly during the peak period. The exception was at the high oversaturation (Figure 4.13 e)) where type-2 and type-4 controls produced high throughput during the peak period. The outstanding differences in control effectiveness could be seen from the ability to maintain the rate of throughput during the dissipation period and the time to completely release traffic out of the network. Focusing on type-4 control, the control performed similarly with unsaturated traffic condition, but when the traffic became saturated, the control had the least ability to maintain high throughput (120^{th} to 180^{th} minute). This resulted in the slowest method to release traffic out of the network. However, during the high oversaturated condition (Figure 4.13 e)), the type-4 control could maintain high rate of throughput during the end of the peak period and the beginning of the dissipation period. The outstanding effectiveness could be seen from the ability to release traffic out of the system sooner than any other control methods (200th minute). The slowest control method was the type-2 method (after 240th minute).

The effectiveness of the traffic control methods could be measured by driver's perception indicators. Thus, the average speed of traffic vs time was presented for each traffic condition. The results were shown in Figure 4.14.

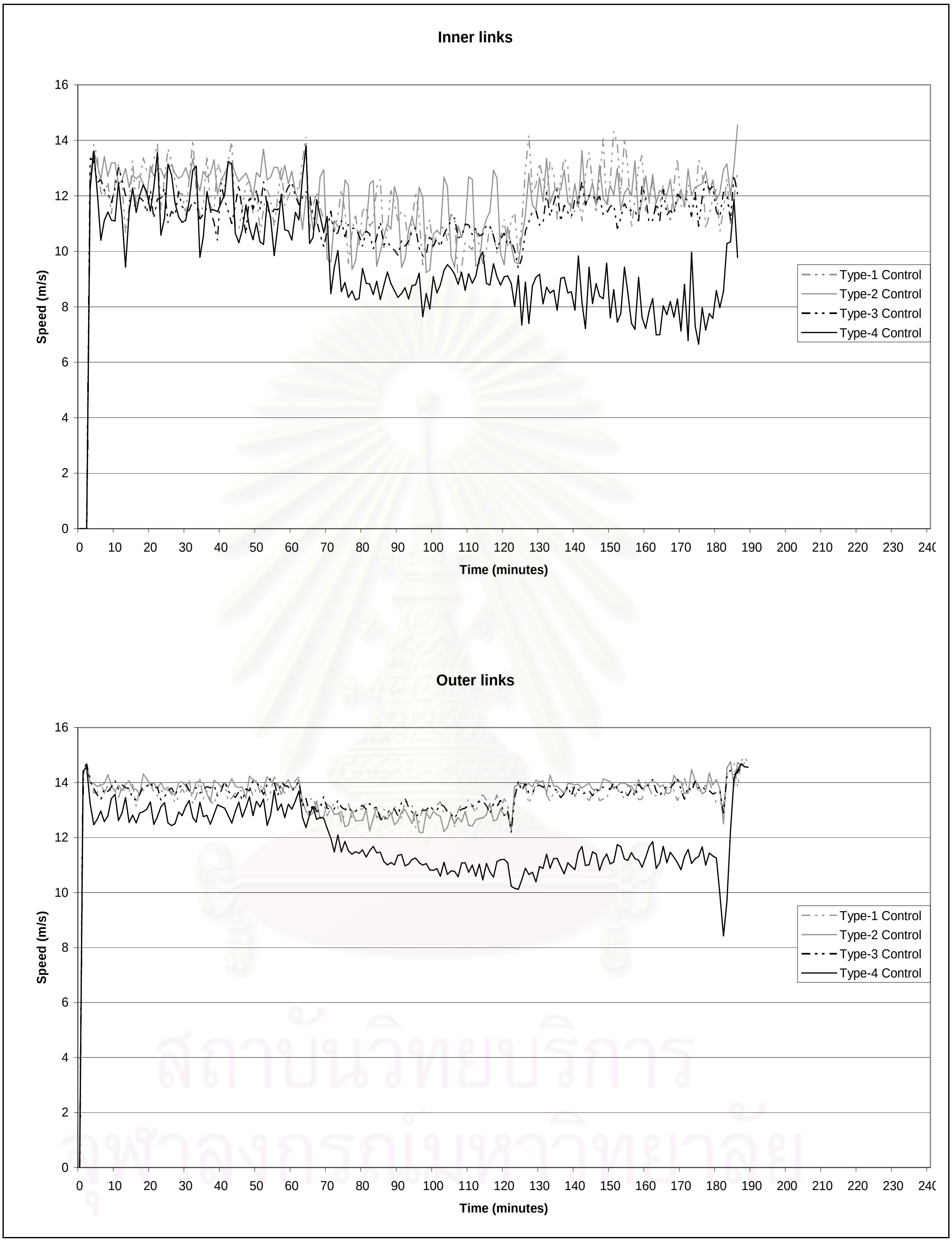
Figures 4.14 a) and 4.14 b) illustrated the average speed on the inner and outer links in unsaturated traffic conditions. This speed was averaged from real time detected speeds of all vehicles on each link. It had to be considered because was possible to have low speed while there was high overall throughput. The type-4 control clearly reduced traffic speed on both inner and outer links. As the congestion level increased to saturation, as indicated in Figure 4.14 c), type-1 and type-3 controls gave the lowest speed on the inner links during the peak time, where type-2 control gave the highest. In contrast, type-1 and type-3 controls yielded the highest speed on the outer links and type-2 control gave the lowest. Type-2 control maintained low speed throughout the peak and dissipation periods. These figures concluded that the type-4 control brought the traffic speed lower throughout the congestion period during the unsaturated traffic conditions.

As the congestion increased from saturation to oversaturation, each control method performed differently. In general, the trend was similar, but the magnitude of impact was greater. Type-1 and type-3 produced the lowest speed during the peak time on the inner links, and the highest speed on the outer links. The outstanding characteristics were that type-1 and type-3 could not convert the inner links into a higher speed during the dissipation period. It took about 20 minutes to higher the inner link speed in the saturation condition, but 60 minutes in the oversaturation condition (Figure 4.14 e)). Type-2 control could higher the inner link speed back to average sooner than type-1 and type-3 controls, but it performed the worst in improving the speed of the outer links. In fact, it could release traffic and higher speed after the vehicles stopped entering the network (toward the end of simulation). Type-4 control did not bring the speed down as low as the type-1 and type-3 controls on the inner links. However, it did not create as high speed as the two methods in the dissipation period. The similar tendency could be seen on the outer links. This implied that type-4 control maintained a more consistent speed than the other three.



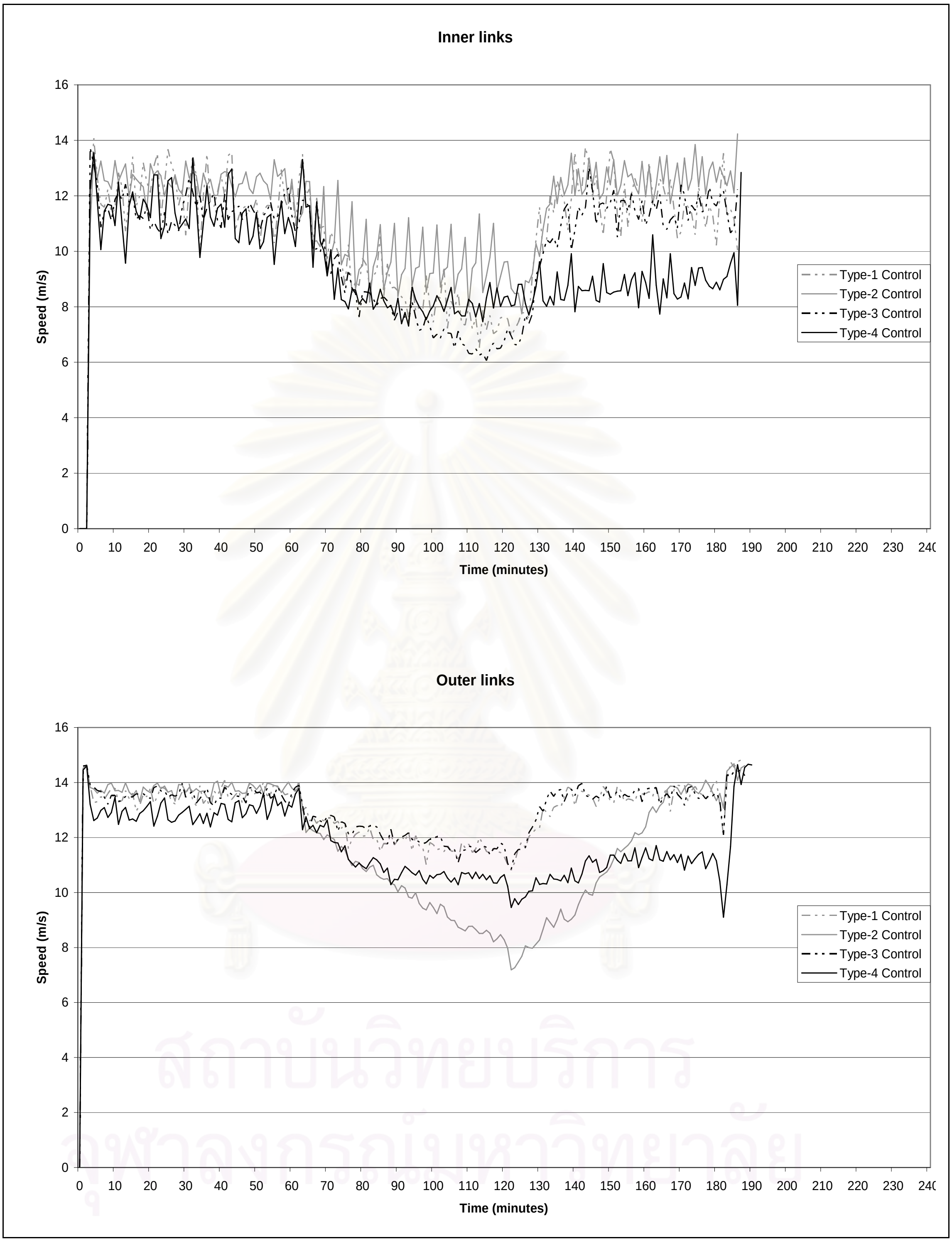
a) Demand-level 1 (unsaturated traffic condition)

Figure 4.14 Real-time average speed in the system (pattern-2 demand)



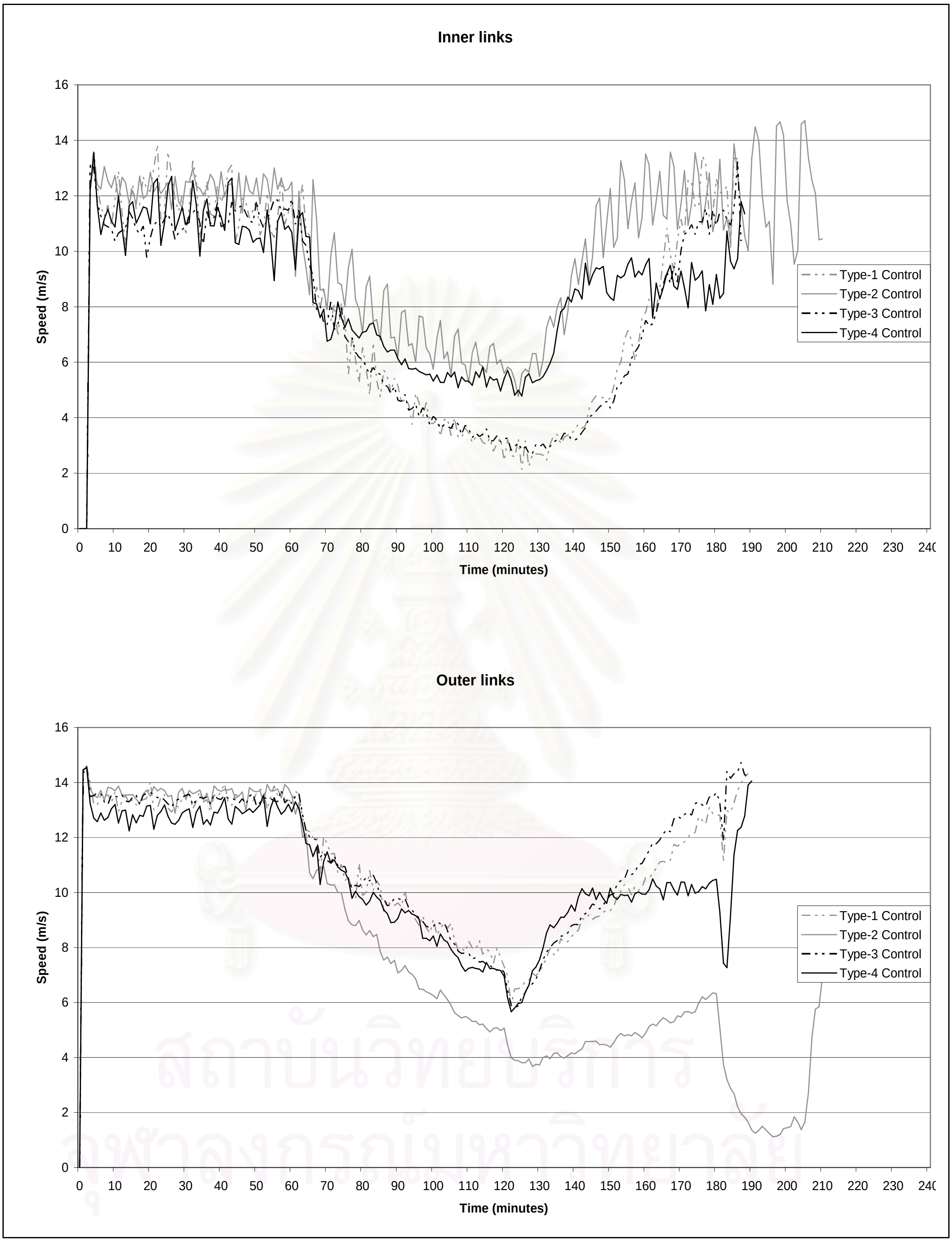
b) Demand-level 2 (unsaturated traffic condition)

Figure 4.14 Real-time average speed in the system (pattern-2 demand) (continued)



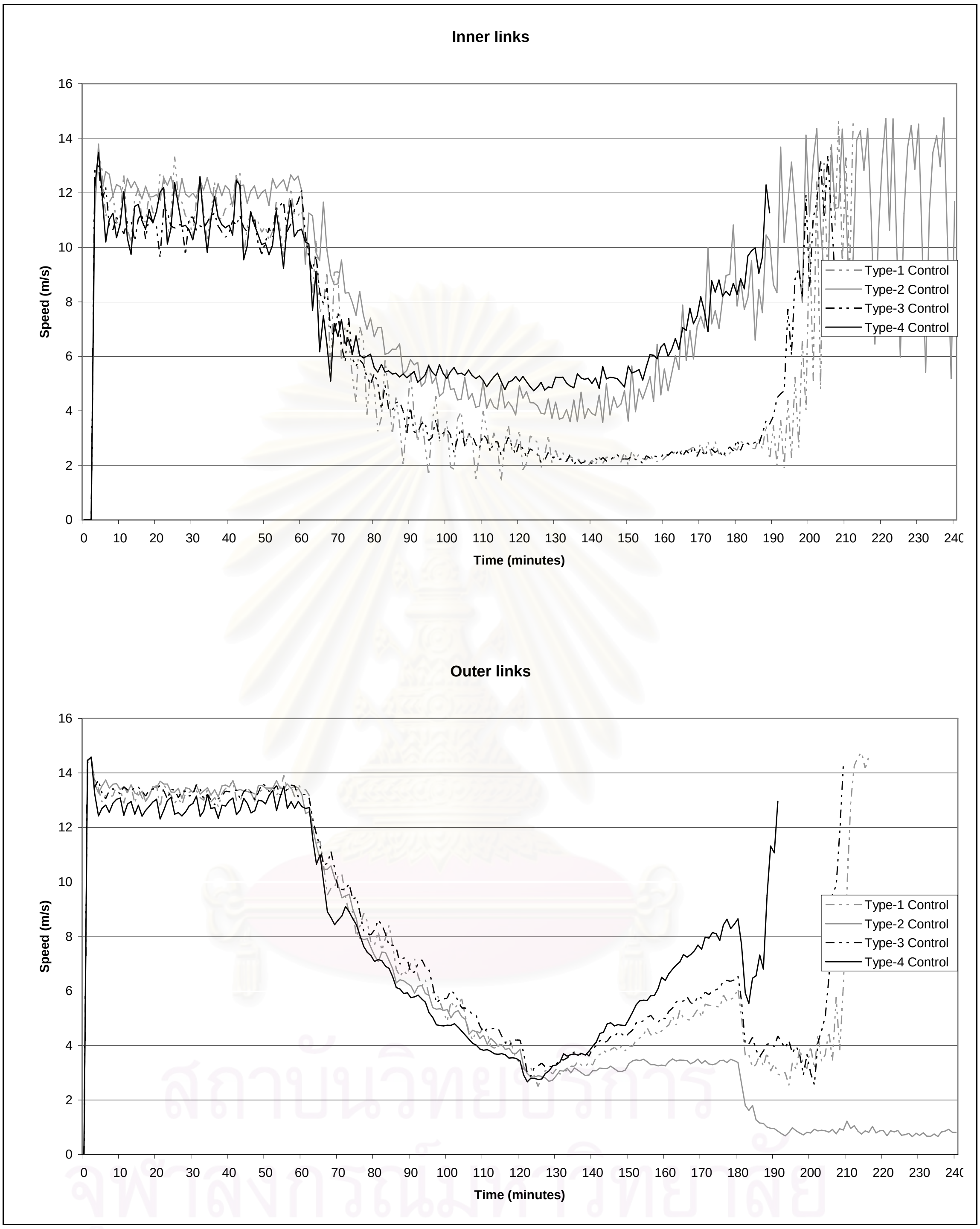
c) Demand-level 3 (saturated traffic condition)

Figure 4.14 Real-time average speed in the system (pattern-2 demand) (continued)



d) Demand-level 4 (oversaturated traffic condition)

Figure 4.14 Real-time average speed in the system (pattern-2 demand) (continued)



e) Demand-level 5 (oversaturated traffic condition)

Figure 4.14 Real-time average speed in the system (pattern-2 demand) (continued)

Figure 4.14 clearly illustrated the effect of traffic controls on speed during high oversaturation. Type-1 and type-3 controls produced the lowest speed on the inner links and the highest speed on the outer links, during the peak time. During the dissipation and end of simulation periods, the inner links had not reached a high speed anymore. Type-2 control was one of the two controls that gave the highest speed on the inner links, but it could not manage the traffic on the outer links. It was seen that the traffic on the outer links could not reach a high speed during the dissipation and end of simulation periods. The effect on the outer links was so adverse as it delayed the last vehicles leaving the network, more than 20 minutes later than the other three control methods. For type-4 control, it was still seen that this method gave the most consistent speed.

Table 4.8 Average speed, meters per second (pattern-2 demand)

Demand Levels	Control Types	Average Speed on Inner Links					Average Speed on Outer Links					Average Speed in the Network				
		The 1st hour	The 2nd hour	The 3rd hour	The 4th hour	Average	The 1st hour	The 2nd hour	The 3rd hour	The 4th hour	Average	The 1st hour	The 2nd hour	The 3rd hour	The 4th hour	Average
1	1	12.6	11.8	12.5	12.0	12.2	13.9	13.4	13.8	13.7	13.6	13.5	12.9	13.4	13.0	13.2
	2	13.0	12.5	12.9	12.9	12.7	14.1	13.6	14.0	14.0	13.8	13.8	13.3	13.7	13.5	13.5
	3	12.1	11.5	12.0	11.9	11.8	13.9	13.4	13.9	13.9	13.7	13.4	12.8	13.3	13.0	13.1
	4	11.5	9.1	7.7	9.3	9.1	13.0	11.6	11.2	10.6	11.8	12.6	10.8	9.9	10.0	10.9
2	1	12.2	10.9	12.1	11.4	11.5	13.7	13.1	13.6	13.7	13.4	13.3	12.4	13.2	12.6	12.8
	2	12.8	10.9	11.9	12.7	11.6	13.9	12.8	13.8	13.8	13.3	13.6	12.2	13.2	13.3	12.8
	3	11.8	10.6	11.3	11.7	11.1	13.8	13.1	13.7	13.8	13.4	13.2	12.3	12.9	12.9	12.7
	4	11.4	9.1	8.2	9.3	9.3	13.0	11.3	11.1	10.7	11.6	12.5	10.7	10.1	10.0	10.9
3	1	12.0	8.6	10.6	12.1	9.9	13.6	11.9	13.1	13.4	12.6	13.1	10.9	12.3	12.8	11.7
	2	12.5	9.8	11.5	12.7	10.9	13.8	10.1	10.4	13.8	10.9	13.4	10.0	10.7	13.3	10.9
	3	11.5	7.7	10.1	11.6	9.1	13.6	12.1	13.2	13.4	12.7	12.9	10.5	12.1	12.6	11.5
	4	11.3	8.4	8.6	9.1	9.0	12.9	11.0	10.8	11.0	11.4	12.5	10.2	10.1	10.1	10.6
4	1	11.7	4.6	4.5	11.8	5.4	13.4	9.4	9.1	12.6	10.1	12.9	7.5	6.9	12.3	8.1
	2	12.3	7.3	8.8	11.1	8.7	13.6	7.1	4.6	2.5	6.7	13.2	7.2	5.4	4.0	7.2
	3	11.1	4.7	4.4	11.3	5.4	13.4	9.3	9.3	13.4	10.1	12.7	7.5	6.9	12.5	8.2
	4	11.1	6.3	7.4	9.0	7.4	12.9	8.9	8.9	9.1	9.6	12.4	8.1	8.4	9.1	8.9
5	1	11.3	3.7	2.4	3.4	3.6	13.3	6.2	3.9	3.7	5.9	12.7	5.4	3.3	3.5	5.0
	2	12.2	5.7	4.9	10.0	6.3	13.4	6.0	3.2	1.0	4.8	13.1	5.9	3.6	1.7	5.1
	3	10.9	3.8	2.3	3.6	3.6	13.3	6.6	4.4	4.2	6.4	12.6	5.6	3.4	3.9	5.3
	4	10.9	5.6	5.7	8.7	6.5	12.8	5.5	4.6	6.8	6.0	12.2	5.6	4.9	7.6	6.1

Table 4.9 provided a qualitative assessment of the effectiveness of the traffic control methods, in terms of throughput and speed. These values came from the graphical analysis. This summarized the conclusion that type-4 control gave the low (lowest) throughput in the unsaturated conditions, but the highest throughput in the oversaturated conditions. More distinction could be seen at the high oversaturation condition. Type-4 control provided the most consistent speed throughout the entire period.

Table 4.9 Summary of graphical analysis for pattern-2 demand

<table>
<tr><th rowspan="2">Scenarios</th><th colspan="4">Peak-Hour Throughput</th><th colspan="4">Inner-Link Speed</th><th colspan="4">Outer-Link Speed</th></tr>
<tr><th>Type-1 Control</th><th>Type-2 Comtrol</th><th>Type-3 Comtrol</th><th>Type-4 Control</th><th>Type-1 Control</th><th>Type-2 Comtrol</th><th>Type-3 Comtrol</th><th>Type-4 Control</th><th>Type-1 Control</th><th>Type-2 Comtrol</th><th>Type-3 Comtrol</th><th>Type-4 Control</th></tr>
<tr><td>6</td><td colspan="4">equal</td><td colspan="3">high</td><td>low</td><td colspan="3">high</td><td>low</td></tr>
<tr><td>7</td><td colspan="4">equal</td><td colspan="3">high</td><td>low</td><td colspan="3">high</td><td>low</td></tr>
<tr><td>8</td><td colspan="3">low</td><td>high</td><td>medium</td><td>high</td><td colspan="2">low</td><td>high</td><td>low</td><td>high</td><td>low</td></tr>
<tr><td>9</td><td colspan="3">low</td><td>high</td><td>low</td><td>high</td><td>low</td><td>high</td><td>high</td><td>low</td><td>high</td><td>high</td></tr>
<tr><td>10</td><td>low</td><td>medium</td><td>low</td><td>high</td><td>low</td><td>high</td><td>low</td><td>high</td><td>high</td><td>low</td><td>high</td><td>high</td></tr>
</table>

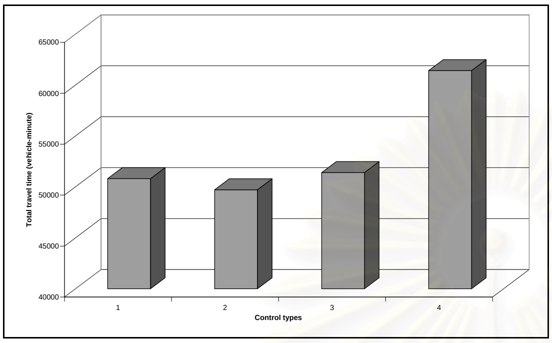
4.3.3 Numerical Analysis on Total Travel Time and Peak Throughput

The traffic performance could be summarized into numerical values. Total travel time and peak throughput were selected to be the measures of effectiveness of the system performance (yielded by operators). These values were calculated from numerical determination from simulation data. The results presented in Figures 4.15 and 4.16.

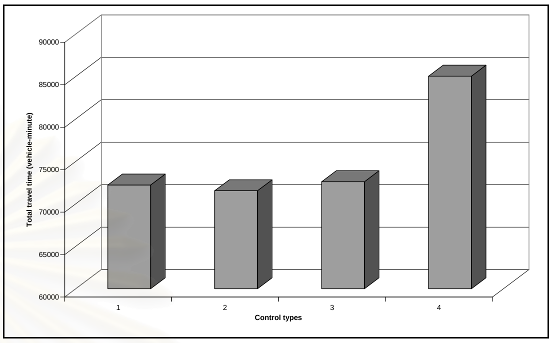
From the figures in the unsaturated condition, there was no significant difference between the results from type-4 traffic control method and each of other control methods in the view of peak throughput. For travel time, type-4 control method caused around 17.1 to 23.5 percent more travel time than each of other three methods.

For the saturated condition, there was no significant difference between the results from type-4 traffic control method and each of other control methods in the view of peak throughput. For travel time, type-4 control method caused around 8.1 to 10.7 percent more travel time than each of type-1 and type-3 methods respectively but the different was not significant in the case of comparing to type-2 method.

Finally, for the oversaturated condition, type-4 control gave the best result in both the view of total travel time and also the view of peak throughput. The improvement of total travel time was significant with both oversaturated levels of demand. The improvement of peak throughput was significant only in the case of comparing to type-2 method with demand level 5. The improvement were around 8.5 to 19.8 percent and 0.9 percent for total travel time and peak throughput, respectively.

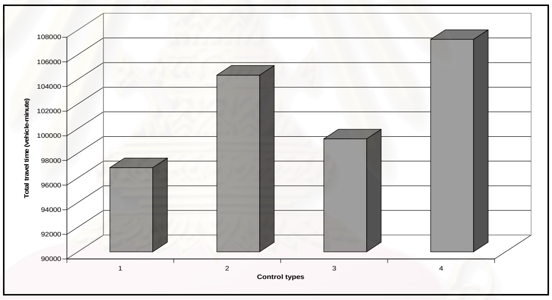


a) Demand level 1



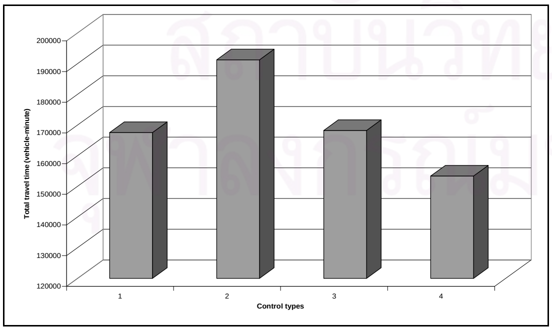
b) Demand level 2

(Unsaturated traffic condition)

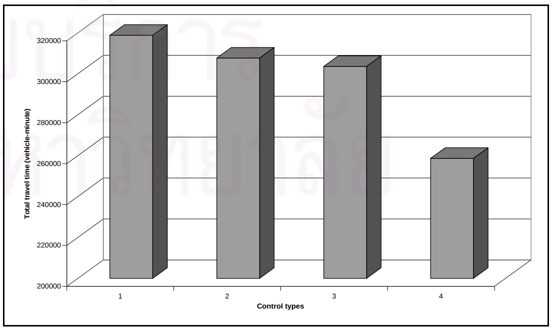


c) Demand level 3

(Saturated traffic condition)



d) Demand level 4



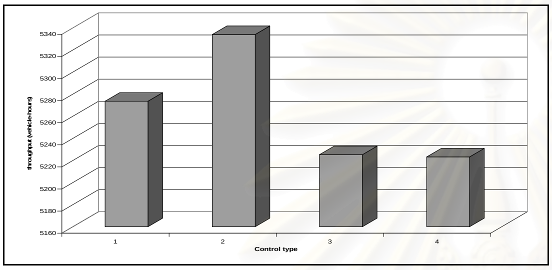
e) Demand level 5

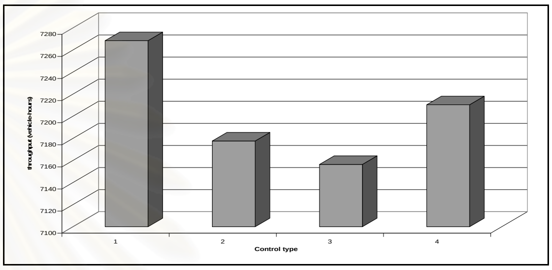
(Oversaturated traffic condition)

Figure 4.15 Total travel time (pattern-2 demand)

Table 4.10 Total travel time, vehicle-minute (pattern-2 demand)

Demand levels	Control type 1		Control type 2		Control type 3		Control type 4	
	Average	SD	Average	SD	Average	SD	Average	SD
1	50,807	399	49,714	632	51,405	909	61,420	687
2	72,226	649	71,549	720	72,617	603	85,043	1,330
3	96,834	1,374	104,339	2,166	99,167	2,913	107,243	2,244
4	167,554	5,977	191,253	5,689	168,224	4,574	153,381	2,118
5	318,863	9,494	307,723	9,908	303,641	9,064	258,712	6,484

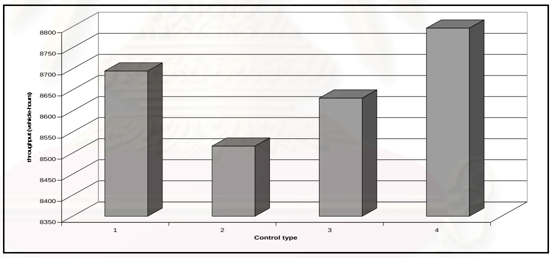




a) Demand level 1

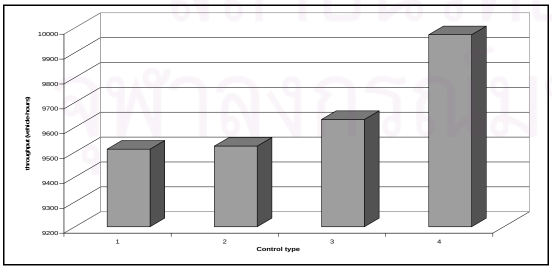
b) Demand level 2

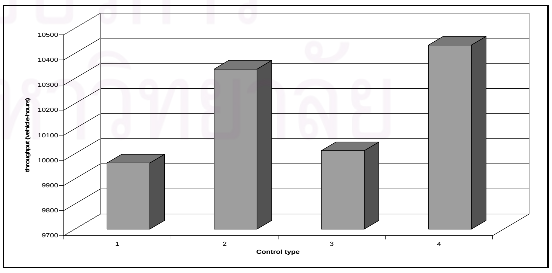
(Unsaturated traffic condition)



c) Demand level 3

(Saturated traffic condition)





d) Demand level 4

e) Demand level 5

(Oversaturated traffic condition)

Figure 4.16 Peak 30-minute throughput (pattern-2 demand)

Table 4.11 Peak 30-minute throughput, vehicle/hour (pattern-2 demand)

Demand levels	Control type 1		Control type 2		Control type 3		Control type 4	
	Average	SD	Average	SD	Average	SD	Average	SD
1	5,274	89	5,334	85	5,225	37	5,223	31
2	7,268	126	7,178	60	7,156	117	7,210	202
3	8,695	124	8,516	62	8,630	99	8,796	164
4	9,512	162	9,524	174	9,632	36	9,972	122
5	9,964	135	10,338	85	10,013	108	10,433	174

The results of the numerical analysis during the peak period were summarized in Table 4.12. Positive values in Table 4.12 meant that type-4 traffic control method gave higher value of that MOE than the base method while the negative values means that type-4 traffic control method gave lower value of that MOE than the base method. Therefore, type-4 traffic control method was expected to work more effectively if the value of difference in total travel time was negative while the value of difference in peak 30-minute throughput was positive. The value with a star was a significant difference.

Table 4.12 Summary of numerical analysis for Pattern-2 demand

Scenarios	Difference in Total Travel Time (percent)			Difference in Peak 30-Minute Throughput (percent)		
	Type-4 from Type-1	Type-4 from Type-2	Type-4 from Type-3	Type-4 from Type-1	Type-4 from Type-2	Type-4 from Type-3
6	20.9*	23.5*	19.5*	-1.0	-2.1	0.0
7	17.7*	18.9*	17.1*	-0.8	0.4	0.8
8	10.7*	2.8	8.1*	1.2	3.3	1.9
9	-8.5*	-19.8*	-8.8*	4.8	4.7	3.5
10	-18.9*	-15.9*	-14.8*	4.7	0.9*	4.2

*significant at 99%

The results from the Table 4.12 confirmed the findings from the previous sections that type-4 control did not give much benefits over other methods during the peak period. As stated earlier, the benefits of the type-4 control came from the ability to not only manage traffic during the peak hour, but also to manage traffic during the dissipation period. By providing a good preparation and maintaining a manageable level of link congestion during the peak period, the type-4 control brought about the highest overall performance in the oversaturated traffic conditions.

CHAPTER V

CONCLUSIONS

5.1 Conclusions

Traffic congestion, especially occurring in a large crowded city, causes so many problems that worsen welfare of the population. The problem can be solved or reduced by two main ways which are managing travel demand and improving network's capacity. The second way is also separated into two main practices which are to improve infrastructures and to improve the way to control the system. The second practice seems to cost less and also normally take less time to implement. In fact, a good combination of solutions is the most efficient way to handle the problem.

There are many different methods of traffic-signal control developed for effectively controlling many different situations. This project aims to develop a method of control that is suitable for the managing peak-period traffic in a large congested road network, especially for morning peak period of a weekday and test its efficiency in handling some different levels of traffic demand such as unsaturated level, saturated level, and oversaturated level. Though the method is designed for an oversaturated case, it is necessary to know the limitation of its usage over different levels of demand.

Some literature review is provided in the second chapter in order to prepare some information required for running the project. Some key knowledge from each reference papers is combined together in an appropriate order to cover all necessary details of each of four main topics: traffic signal control, travel-demand management, road-user behavior, and use of computer simulation in road-traffic engineering.

Chapter 3 describes test environment and also the method used in the test.

A microscopic computer-simulation program called Paramics is the tool selected to simulate situations of managing each level of demand with each of the two main patterns by using the method and three other methods selected to be compared with it. The network is a simplified corridor with four signalized intersections. Each intersection has four legs with three main lanes and two additional turning pocket at the end close to an intersection. Three base controls, which are selected for comparing efficiency with the one developed for maximizing total throughput, are unsynchronized fixed-time control based on Webster's method, synchronized fixed-time control based on Synchro 5 computer program, and the fixed-time method with prevention of green-time loss as described in the chapter. Most methods do not consider the variation of situation because they are set to be only the base method for comparison. Giving better result than these base control methods does not mean that the method has very high performance.

Some assumptions to be used are also listed in order to simplify the test. It is mainly related to the method of control and the limitation of the simulation program.

The performances of traffic control method are captured through 5 key indicators: 1) number of vehicles containing in the system, 2) throughput, 3) average speed, 4) total travel time, and 5) peak time throughput. The first and second indicators present the overall effectiveness of the control over the entire time period. The third indicator, speed, reflects the achievement by various drivers. The fourth indicator presents total benefit of using each method. The fifth indicator focuses on the peak period, which is considered closely by traffic operators. The results explain the details on the effect of control over two kinds of links: inner links and outer links. The performances of the control over the entire time period can be examined into three main time periods: during the peak time, after the peak time when it is used for releasing the congestion (dissipation time), and the end time of simulation when no traffic is entering the network and the vehicles in the network are cleared to empty the network.

The traffic control methods mentioned in Chapter III are tested under two traffic demand patterns with five levels of congestion each. These five levels are 60%, 80%, 100%, 120%, and 140% of the saturated level. The results of analysis on demand-pattern 1 indicate that traffic control with dynamic adaptation (type-4) control performs the best in the oversaturated traffic condition, as it efficiently compromise the level of congestion in both time periods (during peak time and dissipation period) and throughout the network (both inner and outer links). However, the type-4 control is not quite effective during unsaturated traffic conditions, as indicated by high number of vehicles containing in the system and the lower average speed. Compared to type-1 control, a network-optimized fixed time control, type-4 control efficiently manages the throughput throughout the entire time period, thus giving the better performances during the congestion period, and relieving the overall traffic faster than in the heavy oversaturated condition.

In details, the followings are the summary of MOEs for the case of demand pattern 1. With unsaturated level of demand, type-4 control makes the system carry the highest amount of vehicles while other controls perform similarly. For saturated condition, type-1 and type-3 controls allow the greatest number of vehicles to store on the inner links during the peak hour. On the outer links, type-2 and type-4 controls let more vehicles stay on the links than the other two methods during the peak hour. During the queue dissipation period, controls type-1, type-2, and type-3 perform similarly while control type-4 holds many vehicles on both inner links and outer links. For the oversaturated condition, type-1 and type-3 controls hold a large number of vehicles on the inner links, while type-2 and type-4 controls let more vehicles stay on the outer links. The order of the control methods holding the vehicles on the inner links (from the highest to the lowest) is type-3, type-1, type-4, and type-2, respectively. The order of the control methods holding the vehicles on the outer links are type-2, type-4, type-1, and type-3 in the 120 percent oversaturated case and type-4, type-2, type-1, and type-3 in the 140 percent oversaturation case. After the end of peak time, the number of vehicles stored in each part of the system is decreasing until it reaches stability. Type-2 and type-4 controls make traffic reach stability approximately 20 minutes sooner than type-1 and type-3 controls in the 140 percent oversaturation case (approximately 10 minutes sooner in the 120 percent oversaturation case). Type-2 control creates the longest dissipation period. Both cumulative number of vehicles that already arrive their destinations and real-time throughput yield the same conclusion that all control methods perform similarly during the peak period except in the high oversaturation case with 140% of saturated demand where type-2 and type-4 controls produce high throughput during the peak period. The total throughput during the dissipation period is less important and it is not included into the main consideration because it is possible that the lack of vehicles occurs and limits the throughput in the case of using effective methods of control that well manage the traffic during the previous period. For average inner-link speed, type-1 and type-3 controls produce the high speed only in the unsaturated condition while type-2 method gives it in all cases and type-4 gives it only in the oversaturated condition. For average outer link speed, the high speed is produce only in the case of applying type-1 or type-3 control to all conditions and the case of applying type-2 control to the unsaturated condition. In the unsaturated conditions, there is no significant difference in peak throughput but, for total travel time, type-4 control method significantly causes more total travel time than other three methods. For the saturated condition, there is no significant difference in peak throughput but, for total travel time, type-4 control method significantly causes more total travel time than other three methods. For the over saturated condition, type-4 control gives the best result in both the view of total travel time and also the view of through put. The improvement of travel time is significant only in the case of comparing to type-2 and type-3 control methods with 140% oversaturated demand. The improvement of peak throughput is significant only in the case of comparing to type-1 and type-3 control methods.

For demand pattern 2, most results seem to have the same trend as those from the test with demand pattern 1. The results of analysis indicate that traffic control with dynamic adaptation (type-4) control performs the best in the oversaturated traffic condition, as it efficiently compromises the level of congestion in both time periods (during peak time and dissipation period) and throughout the network (both inner and outer links). However, the type-4 control is not quite effective during unsaturated traffic conditions, as indicated by high number of vehicles containing in the system and the lower average speed. Compared to type-1 control, a network-optimized fixed time control, type-4 control efficiently manages the throughput throughout the entire time period, thus giving the better performances during the congestion period, and relieving the overall traffic faster in the heavy oversaturated condition. The main difference between the results from these two patterns is that type-2 control, the synchronized one, completely fails in dissipating queue after the demand period in high congested condition with demand pattern 2. The reason of this failure is expected to be the policy that the control method uses. It tries to improve serviceability of only the main street. For the second pattern, there is another main flow path cut through the network. This makes type-2 control works inefficiently while type-4 control still works with high efficiency.

In details, the followings are the summary of MOEs for the case of demand pattern 2. With unsaturated level of demand type-4 control makes the system carry the highest amount of vehicles while other controls perform similarly. For saturated condition, type-2 control allows the smallest number of vehicles to store on the inner links during the peak hour. On the outer links, type-2 and type-4 controls let more vehicles stay on the links than the others during the peak hour. During the queue dissipation period, controls type-1 and type-3 perform similarly. For type-2 control, the queue on outer links rapidly disappears until it averagely reaches the same length as type-1 and type-3 controls while type-4 holds many vehicles on both inner and outer links. In the oversaturated condition, type-1 and type-3 controls hold a larger number of vehicles on the inner links, while type-2 and type-4 controls let more vehicles stay on the outer links than other two methods proportionally. The order of the control methods holding the vehicles on the inner links (from highest to lowest) are type-1, type-3, type-4, and type-2 in the 120 percent oversaturation case, and type-3, type-1, type-2, and type-4 in the 140 percent oversaturation case, while the order of the control methods holding the vehicles on the outer links are type-2, type-4, type-3, and type-1 in the 120 percent oversaturation case and type-4, type-2, type-1, and type-3 in the 140 percent oversaturation case. After the end of peak-time, the number of vehicles stored in each part of the system is decreased until it reaches stability. Type-2 and type-4 controls make traffic reach stability approximately 20 minutes sooner than type-1 and type-3 controls in the 140 percent oversaturation case (approximately 10 minutes sooner in the 120 percent oversaturation case). Type-2 control creates the longest dissipation period. Both cumulative number of vehicles that already arrive their destinations and real-time throughput yields the same conclusion that all control methods perform similarly during the peak period except in the high oversaturation case with 140% of saturated demand where type-2 and type-4 controls produce high throughput during the peak period. The total throughput during the dissipation period is less important and it is not include into the main consideration because it is possible that the lack of vehicles occurs and limits the throughput in the case of using effective methods of control that well manage the traffic during the previous period. For average inner-link speed, type-1 and type-3 control produce the high speed only in the unsaturated condition while type-2 method gives it in all cases and type-4 method gives it only in the oversaturated condition. For average outer-link speed, the high speed is produced in the case of applying type-1 or type-3 method of control to all cases, the case of using type-2 control with the unsaturated condition, and the case of applying type-4 control to the oversaturated levels of demand. In the unsaturated conditions, there is no significant difference in peak throughput but, for total travel time, type-4 control method significantly causes more total travel time than other three methods. For the saturated condition, there is no significant difference in peak throughput but for total travel time, type-4 control method significantly causes more total travel time than each of type-1 and type-3 methods but the difference is not significant in the case of comparing to type-2 method of control. For oversaturated condition, type-4 control gives the best result in both view of total travel time and also the view of peak throughput. The improvement of total travel time is significant with both oversaturated levels of demand. The improvement of peak throughput is significant only in the case of comparing to type-2 method with 140% oversaturated demand.

It is concluded that the throughput-maximizing method, developed for controlling a congested network during morning-peak period, is proved to work the best on a congested network with oversaturated demand.

For demand-pattern 1, it seems to give higher level of throughput rather than type-1 and type-3 control methods do in the oversaturated condition. Compared to type-2 control, the improvement during the peak hour is not significant. But after the hour, throughput maximizing method works more effectively.

For demand-pattern 2, it seems to significantly give higher level of throughput only rather than type-2 control method do. But in long term, throughput maximizing method works more effectively than all others.

For saturated demand, the method still works quite well and eliminates vehicles as fast as the others do but the throughput-maximizing method is much more complicated and requires some more resources. It is not so practical to apply the method at this level of demand or lower.

The control does not perform as well as others when it faces an unsaturated condition. Changing phases too often makes the throughput-maximizing method of control waste a significant start-up lost time at the beginning of each phase and also some loss at the end. Another reason is that there are too short queues stored on major legs to use green time effectively.

Though it works well for some levels of demand but the method still has a major disadvantage that is it keeps giving green signal only to some major legs that are expected to produce a greater value of throughput. Other minor legs have to wait until the considerable amount of vehicles, choosing each direction at the end of each leg, accumulates as a queue on that approach or all major legs of an intersection have too short queues.

The method seems to optimize each signalized intersection separately but it also includes some predictions of future effect to the whole system as the estimated throughput for each combination of green time allocation. It can be said that the method optimizes not only an individual intersection but also overall system without fixing either phase or length of cycle. Actually, there is no common cycle for this kind of control.

It can be said that type-2 traffic control method can be considered as one kind of the throughput-improving method that aims to improve serviceability of overall network by giving priority to some selected links. This makes the method works well in the peak hour and also reduces the equity. Unlike type-4 control, type-2 control focuses on only the main continuous path to be synchronized.

Finally, Throughput-maximizing traffic control method is proved to be another choice of controlling a highly congested network. Some adaptation should be applied to the method before the practical usage in order to improve its serviceability and to make the method suitable for each local situation (and limitations). And if the situation keeps changing, the methods of control also should be switched due to the prevailing condition.

5.2 Difficulties and Problems

Firstly, like many other computer-simulation programs, Paramics also has some weaknesses and limitations. Each vehicle arrives at its destination as soon as it reaches boundary of a destination zone. This causes a conflict with the real practice that vehicle should be able to leave the system anywhere along each link. Because of this limitation, the throughput function has to be adapted.

The interface to the Paramics program is still not fully developed. Some additional control codes cannot be directly added to the program. It requires another application program to build a plug-in file. As a concequence, the program does not have an ability to directly debug any plug-in and also causes difficulty in transferring plug-in files.

For links, nodes, and some other objects, their names are not in an integer form. Some difficulties occur when programmers try to identify the objects.

For the graphic mode of Paramics, the Modeller, it takes a lot of simulation time even for a small network with high demand of road users.

For coding plug-ins, functions provided by Paramics are not grouped into objects, hence this produces some difficulties to access each function.

The problems mentioned above are related to program usage but there are some problems related to other dimensions. For example, working with statistical data from an extremely congested network causes difficulty to graphical analysis and comparison.

Only a small amount of studies in the past focuses on controlling the network which is extremely congested because it is hardly frequent to find a very serious oversaturated traffic situation in the urban street network which happens in a wide area and occurs regularly. And it is also because there has been no sufficient capacity of tools and efforts to investigate this kind of congestion.

5.3 Recommendations

To prevent the situation that keeps giving green signal to only some major legs and let others wait too long, this throughput-maximizing control method should be adapted by limiting maximum red time to be not too long or considering the value of waiting time to increase by the increment of approaches waiting-time length.

Using the method in a real-world, a set of non-major-conflict combination of giving green time for each intersection should be listed in order to minimize calculating time.

If there is any information of the distribution of destination point along each link, it should be better to include that information to the consideration.

It is not important to use just one method of control for the whole system because this throughput-maximizing method optimizes each intersection separately by considering the expected effect from each choice to the system throughput. It is practical to use this controlling method in just some intersections that are considered to be appropriate and let others controlled by other methods. This may not only improve the serviceability but also reduces cost of instrument used in the system.

It is neither easy for ordinary users to track all hidden algorithms in Paramics program and other computer-simulation programs nor to understand every assumption in it. Bear in mind that although a micro-simulation program is devised for testing the impact of a change on a simulated network, the investigation on simulation approach may reduce ability to reflect some specific (complex) characteristics of the real network. Using the program for proving some theories, as performed in this study, is acceptable because the simulated network is used as a test ground for comparing the performance of various control methods, under the same controlled environments. However, if the case is to attempt to predict some outcomes for other real networks, such as a bigger network, another method of testing and test ground may be considered. For example, for a large real network, a macro-simulation program based on statistics that uses less resource or traditional statistical calculation seems to give sufficient reliability when it appropriately includes some hidden characteristics of each network in some statistical functions.

Without real-time traffic signal control, the throughput can also be improved by giving “more priority” to the approaches that head to the main destination zones and the legs that come from these main destination zones. It can increase the rate that vehicles pass the zones and then the rate that the vehicles leave the system.

Finally, for a more complicated network that an effect from letting an approach go or stop it can repeat loop transported back to each link so many times, the throughput function should be adapted. The first part of the function which considers only the time that the released vehicle is expected to use for getting out of the system may have limited time of consideration. It can be limited by using the whole consideration period left or the time that only a small proportion of vehicle is still expected to be trapped in the system. It also can be limited by assuming that vehicles will not use the route that passes the same road section or the same intersection twice. The second solution is more theoretical and reasonable because it reflects the real behavior of users that usually do not drive in circle. For the second part of the function, the queue filling up, it focuses only on expected amount of vehicles leaving while the queue is being filled up and the time that it occurs. In this case, it is reasonable to have some effects transfer back to the same link more than one time. Limit of considering time can also be use for the estimation.

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APPENDICES

APPENDIX A

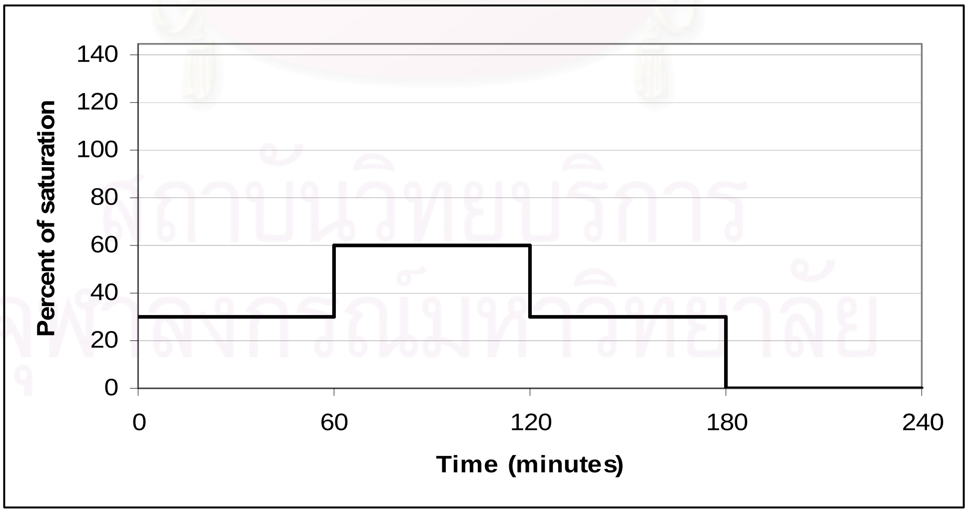
Travel Demand

Table A-1 The first demand pattern

Origins	Destinations (unit/hour)																					
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	0	1	1	1	1	1	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
2	1	0	1	1	1	1	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
3	1	1	0	1	1	1	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
4	1	1	1	0	1	1	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
5	1	1	1	1	0	1	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
6	1	1	1	1	1	0	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
7	1	1	1	1	1	1	0	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
8	1	1	1	1	1	1	1	0	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
9	1	1	1	1	1	1	1	1	0	1	1	4	4	4	1	1	1	1	1	1	1	1	1
10	1	1	1	1	1	1	1	1	1	0	1	4	4	4	1	1	1	1	1	1	1	1	1
Total	9	9	9	9	9	9	9	9	9	9	10	40	40	40	10	10	10	10	10	10	10	10	10

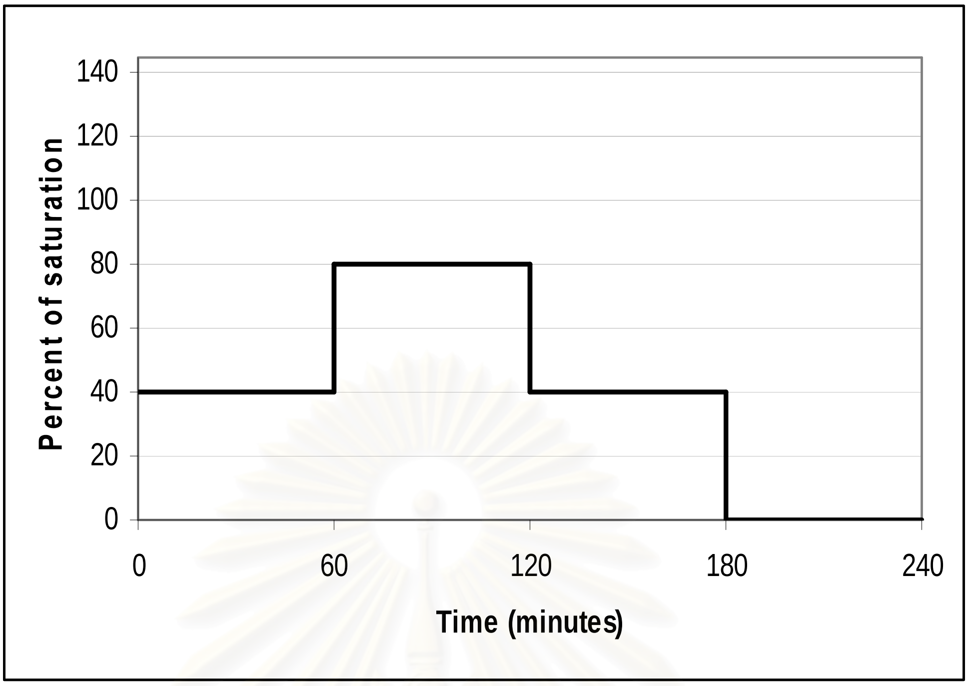
Table A-2 The second demand pattern

Origins	Destinations (unit/hour)																					
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	0	1	1	1	1	1	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
2	1	0	1	1	1	1	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
3	1	1	0	1	1	1	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
4	1	1	1	0	1	1	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
5	1	1	1	1	0	1	1	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
6	1	1	1	1	1	0	1	1	1	41	1	4	4	4	1	1	1	1	1	1	1	1	1
7	1	1	1	1	1	1	0	1	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
8	1	1	1	1	1	1	1	0	1	1	1	4	4	4	1	1	1	1	1	1	1	1	1
9	1	1	1	1	1	1	1	1	0	1	1	4	4	4	1	1	1	1	1	1	1	1	1
10	1	1	1	1	1	1	1	1	1	0	1	4	4	4	1	1	1	1	1	1	1	1	1
Total	9	9	9	9	9	9	9	9	9	49	10	40	40	40	10	10	10	10	10	10	10	10	10

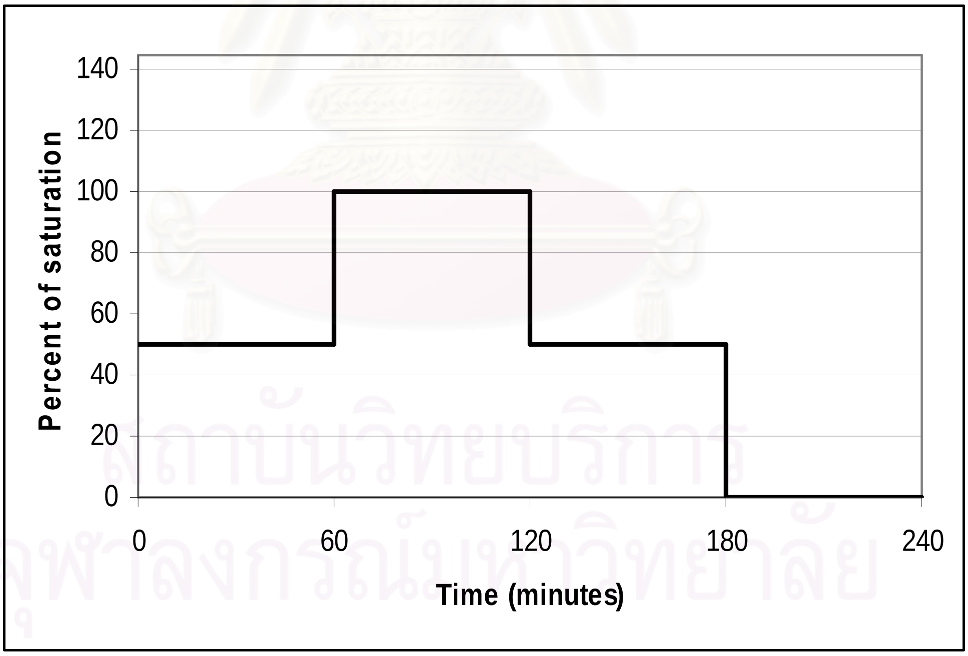


a) Demand-level 1 (unsaturated condition)

Figure A-1 Travel demand

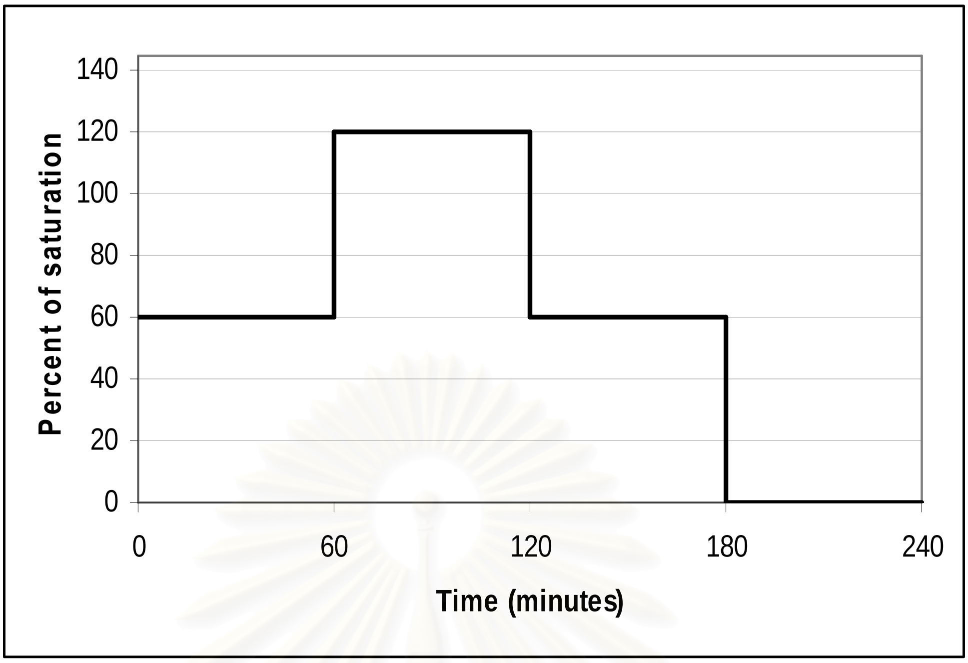


b) Demand-level 2 (unsaturated condition)

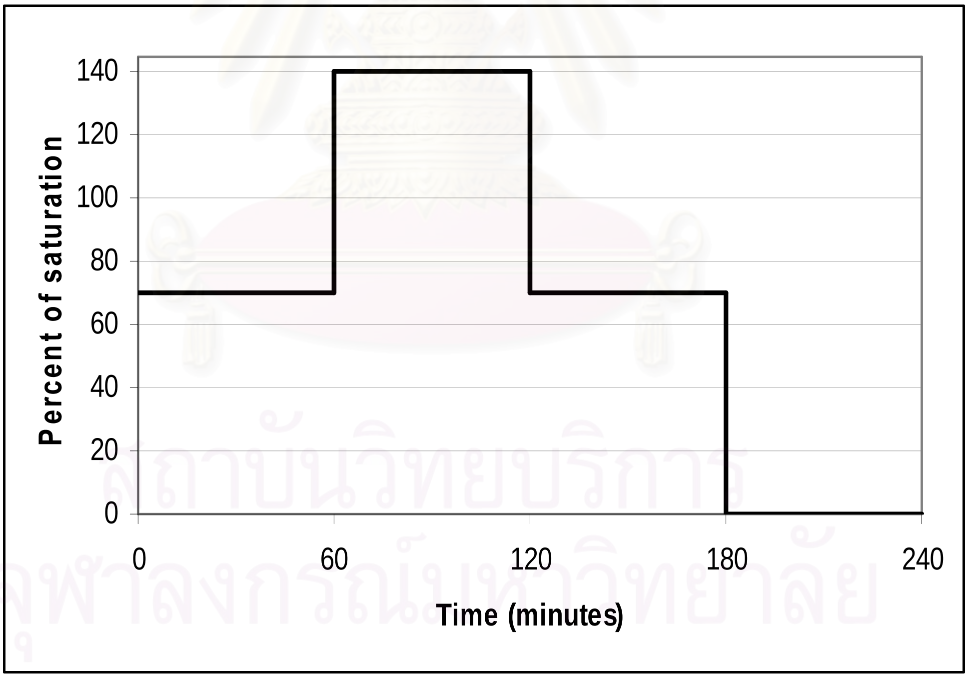


c) Demand-level 3 (saturated condition)

Figure A-1 Travel demand (continued)



d) Demand-level 4 (oversaturated condition)



e) Demand-level 5 (oversaturated condition)

Figure A-1 Travel demand (continued)

APPENDIX B

Level of Service for Type-1 Control

Table B-1 Expected levels of service for type-1 control (60% saturation)

Intersections	Upstream nodes	Upstream-node locations	Levels of service (demand pattern 1)			Levels of service (demand pattern 2)		
			Left	Through	Right	Left	Through	Right
4	17	North of an intersection	A	C	C	A	C	C
	7	East of an intersection	A	B	B	A	B	B
	25	South of an intersection	A	C	C	A	C	C
	2	West of an intersection	B	C	C	A	C	C
7	18	North of an intersection	A	C	C	A	C	C
	10	East of an intersection	A	A	A	A	B	B
	26	South of an intersection	A	C	C	A	C	C
	4	West of an intersection	A	C	C	A	C	C
10	19	North of an intersection	A	C	C	A	C	C
	13	East of an intersection	A	C	C	A	C	C
	27	South of an intersection	A	C	C	A	C	C
	7	West of an intersection	A	B	A	A	B	B
13	20	North of an intersection	A	C	C	A	C	C
	15	East of an intersection	A	C	C	B	C	C
	28	South of an intersection	A	C	C	A	C	C
	10	West of an intersection	A	C	B	A	C	C

Table B-2 Expected levels of service for type-1 control (80% saturation)

Intersections	Upstream nodes	Upstream-node locations	Levels of service (demand pattern 1)			Levels of service (demand pattern 2)		
			Left	Through	Right	Left	Through	Right
4	17	North of an intersection	A	C	C	A	C	C
	7	East of an intersection	A	C	B	A	C	B
	25	South of an intersection	A	C	C	A	C	C
	2	West of an intersection	A	C	C	A	C	C
7	18	North of an intersection	A	C	C	A	C	C
	10	East of an intersection	A	B	B	A	C	C
	26	South of an intersection	A	C	D	A	C	D
	4	West of an intersection	A	C	C	A	C	C
10	19	North of an intersection	A	C	D	A	C	D
	13	East of an intersection	A	C	C	A	C	C
	27	South of an intersection	A	C	C	A	C	C
	7	West of an intersection	A	B	A	A	B	B
13	20	North of an intersection	A	C	C	A	C	C
	15	East of an intersection	A	C	C	B	D	C
	28	South of an intersection	A	C	C	A	C	C
	10	West of an intersection	A	C	B	A	C	C

Table B-3 Expected levels of service for type-1 control (100% saturation)

Intersections	Upstream nodes	Upstream-node locations	Levels of service (demand pattern 1)			Levels of service (demand pattern 2)		
			Left	Through	Right	Left	Through	Right
4	17	North of an intersection	A	C	C	A	C	C
	7	East of an intersection	A	C	B	A	C	B
	25	South of an intersection	A	C	C	A	C	C
	2	West of an intersection	A	E	C	A	E	C
7	18	North of an intersection	A	C	C	A	C	C
	10	East of an intersection	A	D	B	A	D	C
	26	South of an intersection	A	C	E	A	C	F
	4	West of an intersection	A	E	C	A	E	C
10	19	North of an intersection	A	C	E	A	C	F
	13	East of an intersection	A	E	C	A	E	C
	27	South of an intersection	A	C	C	A	C	C
	7	West of an intersection	A	D	B	B	D	C
13	20	North of an intersection	A	C	C	A	E	C
	15	East of an intersection	A	D	C	A	F	C
	28	South of an intersection	A	C	C	A	C	C
	10	West of an intersection	A	C	B	A	E	C

Table B-4 Expected levels of service for type-1 control (120% saturation)

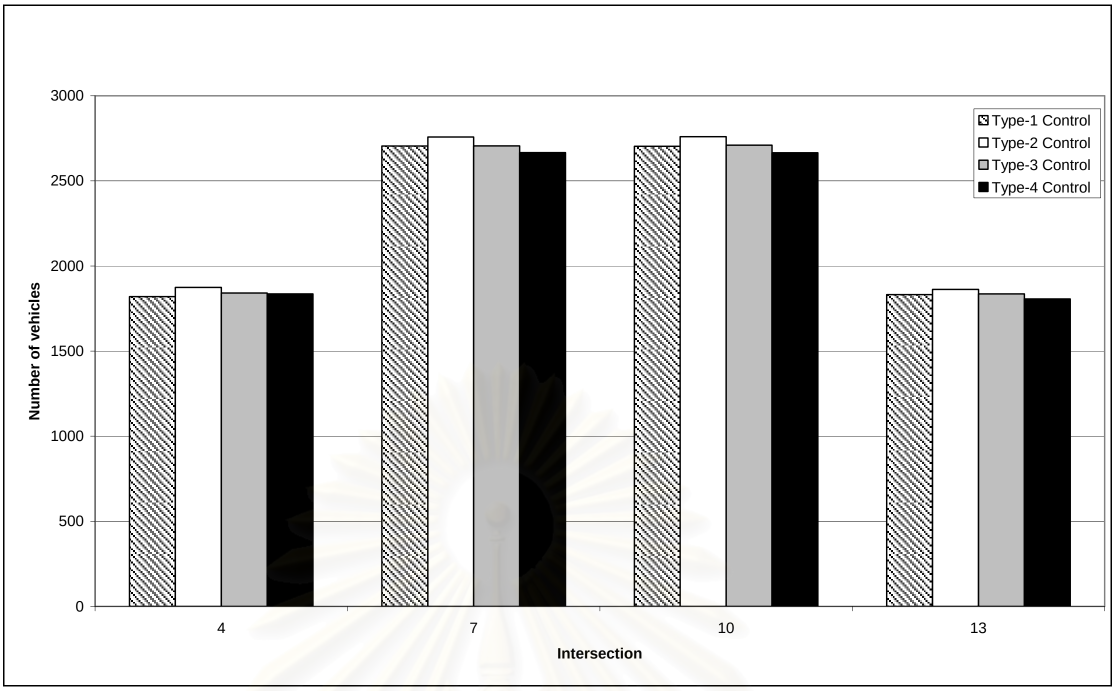
Intersections	Upstream nodes	Upstream-node locations	Levels of service (demand pattern 1)			Levels of service (demand pattern 2)		
			Left	Through	Right	Left	Through	Right
4	17	North of an intersection	A	C	C	A	C	C
	7	East of an intersection	A	D	C	A	D	C
	25	South of an intersection	A	C	E	A	C	F
	2	West of an intersection	A	F	C	A	E	C
7	18	North of an intersection	A	D	D	A	E	E
	10	East of an intersection	C	F	E	A	F	D
	26	South of an intersection	A	D	F	A	E	F
	4	West of an intersection	B	F	D	C	F	D
10	19	North of an intersection	A	D	F	A	E	F
	13	East of an intersection	B	F	D	C	F	D
	27	South of an intersection	A	D	D	A	E	E
	7	West of an intersection	C	F	E	C	F	D
13	20	North of an intersection	A	C	E	B	F	D
	15	East of an intersection	A	E	C	B	F	E
	28	South of an intersection	A	C	C	A	E	E
	10	West of an intersection	A	D	C	B	F	D

Table B-5 Expected levels of service for type-1 control (140% saturation)

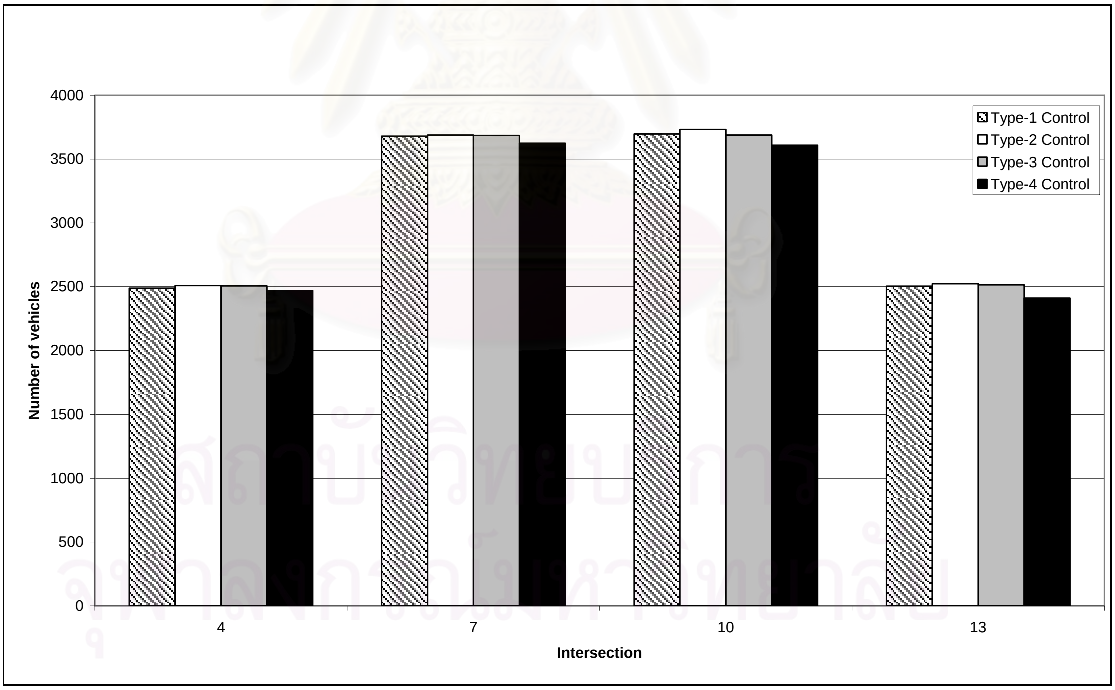
Intersections	Upstream nodes	Upstream-node locations	Levels of service (demand pattern 1)			Levels of service (demand pattern 2)		
			Left	Through	Right	Left	Through	Right
4	17	North of an intersection	A	C	C	A	D	D
	7	East of an intersection	A	E	C	A	E	C
	25	South of an intersection	A	C	E	A	D	F
	2	West of an intersection	A	F	C	A	F	D
7	18	North of an intersection	A	F	F	A	F	F
	10	East of an intersection	F	F	F	D	F	E
	26	South of an intersection	A	F	F	A	F	F
	4	West of an intersection	F	F	F	F	F	F
10	19	North of an intersection	A	F	F	A	F	F
	13	East of an intersection	F	F	F	E	F	F
	27	South of an intersection	A	F	F	A	F	F
	7	West of an intersection	F	F	F	F	F	F
13	20	North of an intersection	A	C	E	E	F	F
	15	East of an intersection	A	F	C	F	F	F
	28	South of an intersection	A	C	C	C	F	F
	10	West of an intersection	A	E	C	F	F	F

APPENDIX C

Vehicle Transfer

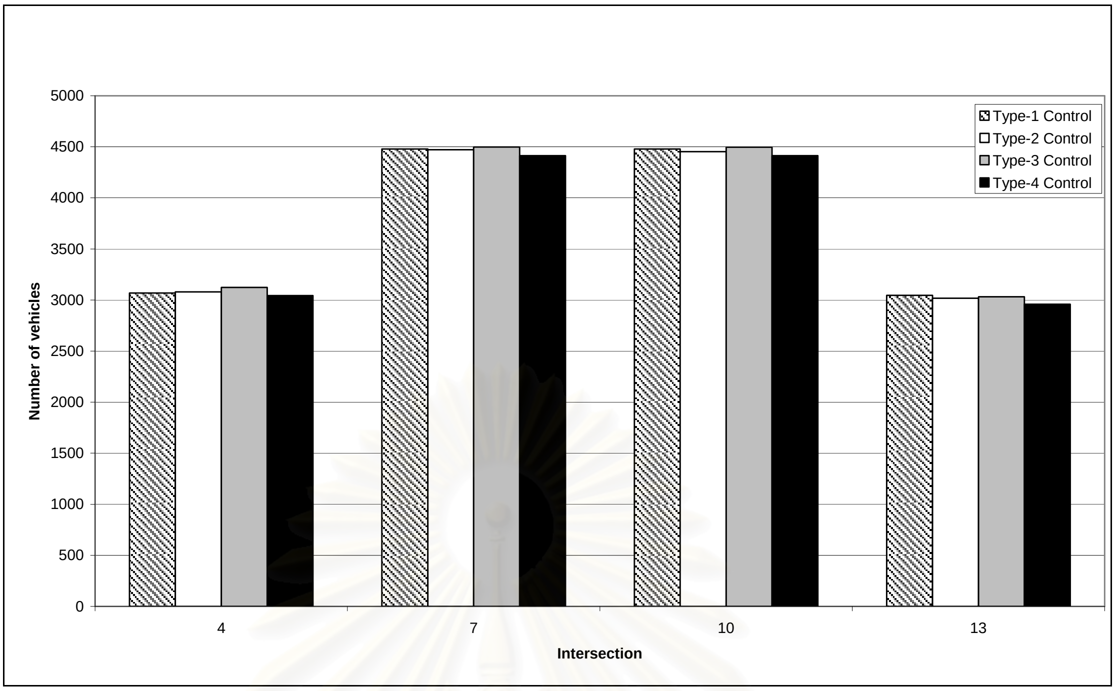


a) Demand-level 1 (unsaturated condition)

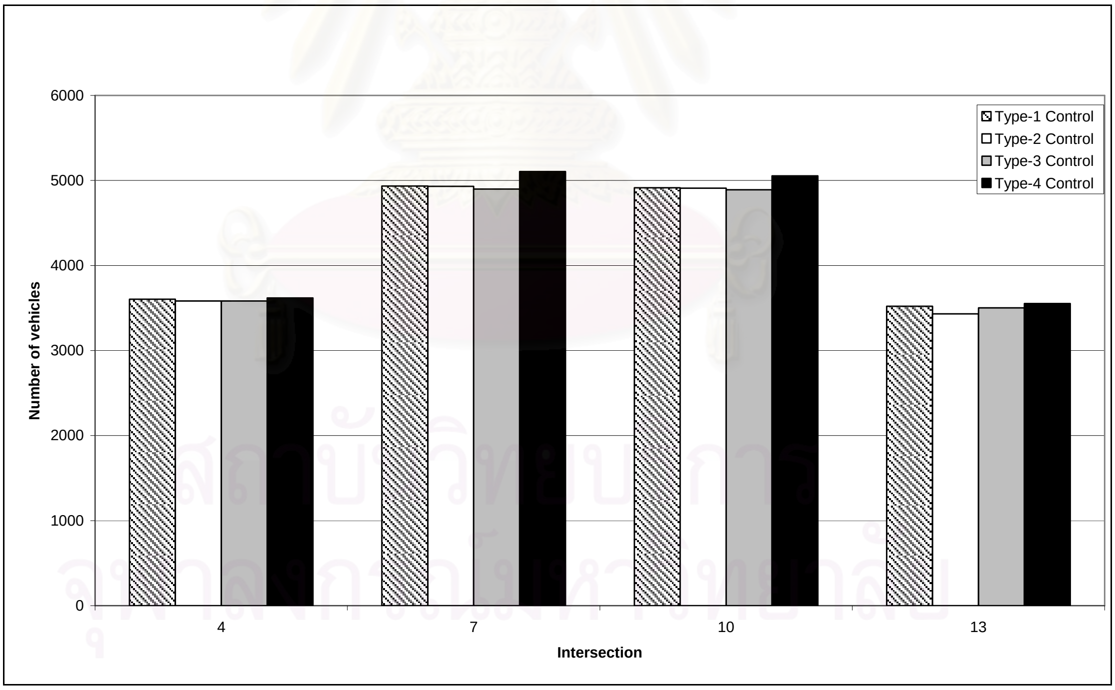


b) Demand-level 2 (unsaturated condition)

Figure C-1 Peak-hour number of vehicles passing each intersection (pattern-1 demand)

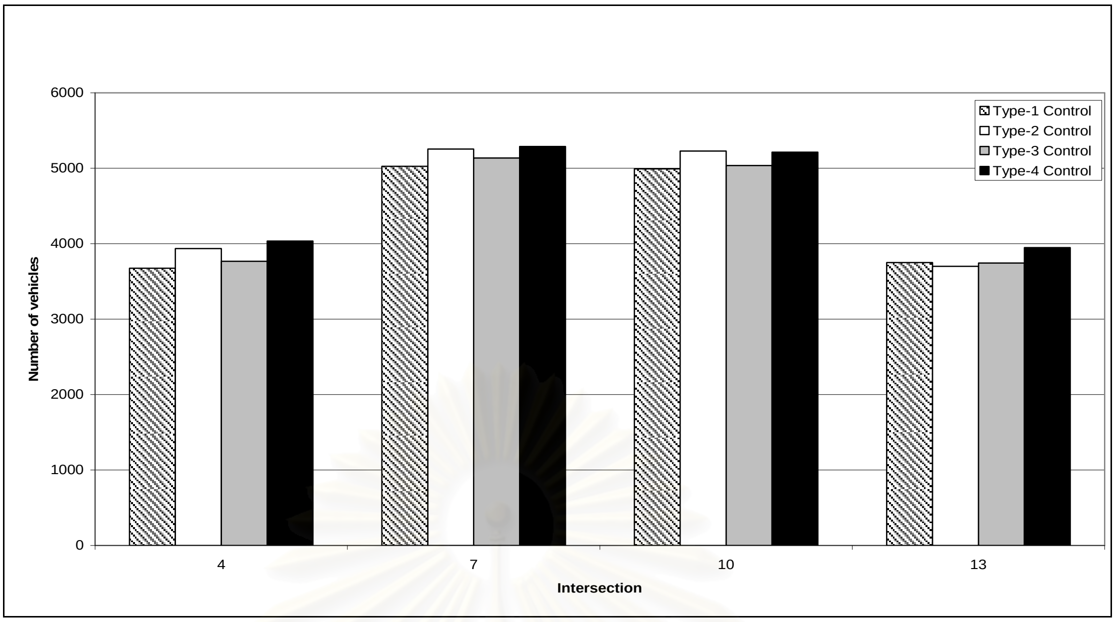


c) Demand-level 3 (saturated condition)



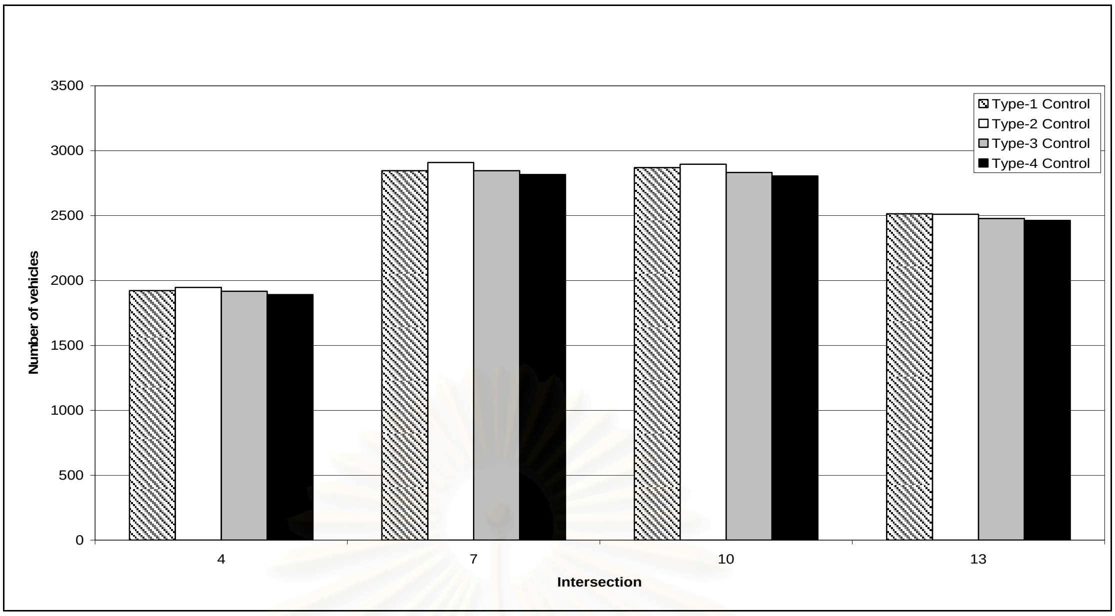
d) Demand-level 4 (oversaturated condition)

Figure C-1 Peak-hour number of vehicles passing each intersection (pattern-1 demand) (continued)

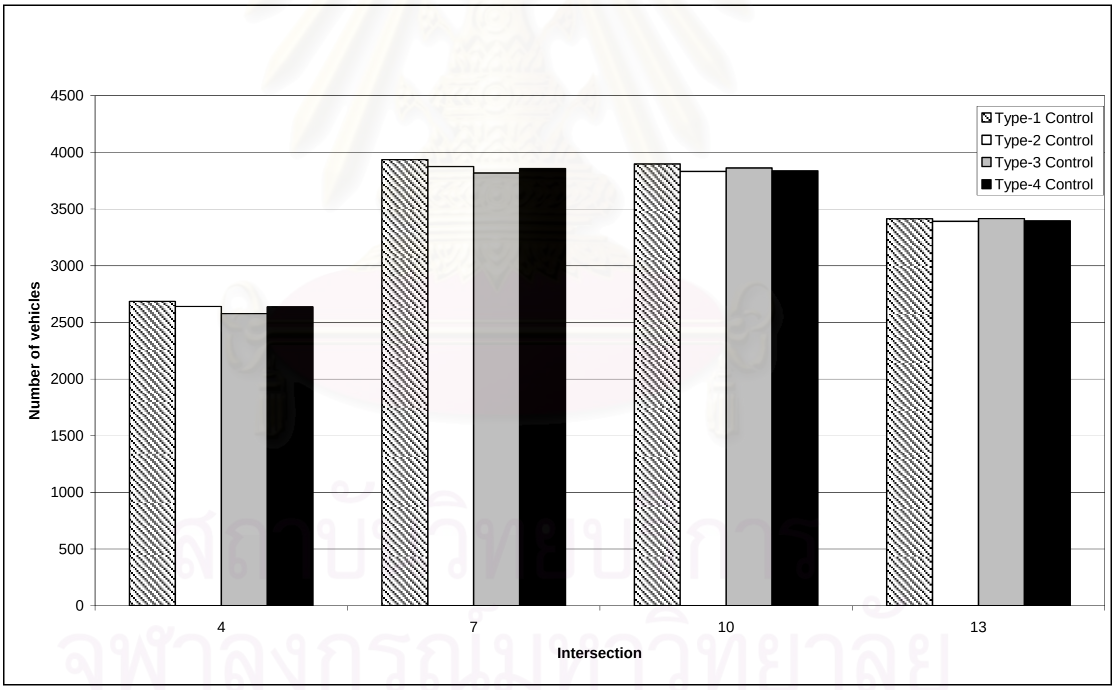


e) Demand-level 5 (oversaturated condition)

Figure C-1 Peak-hour number of vehicles passing each intersection (pattern-1 demand) (continued)

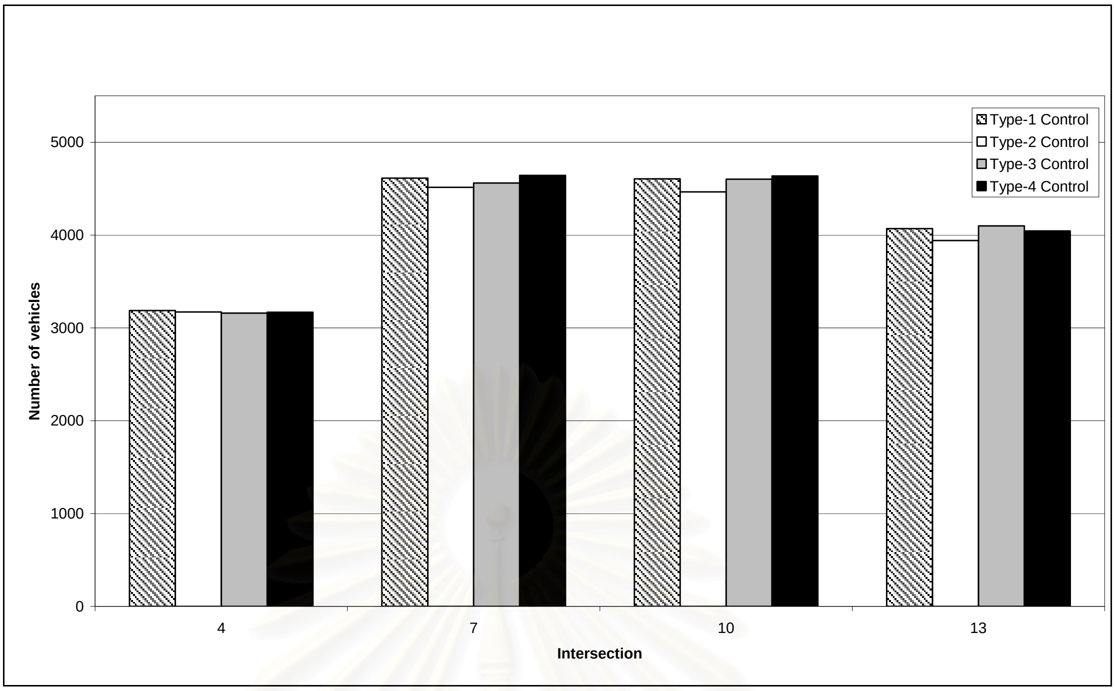


a) Demand-level 1 (unsaturated condition)

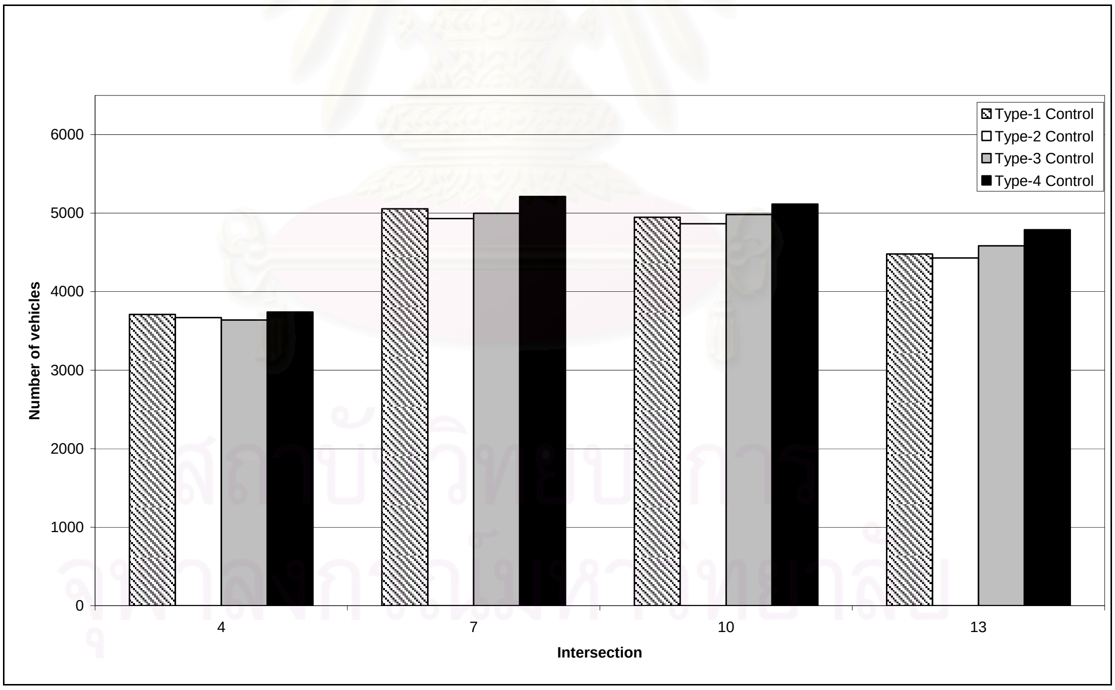


b) Demand-level 2 (unsaturated condition)

Figure C-2 Peak-hour number of vehicles passing each intersection (pattern-2 demand)

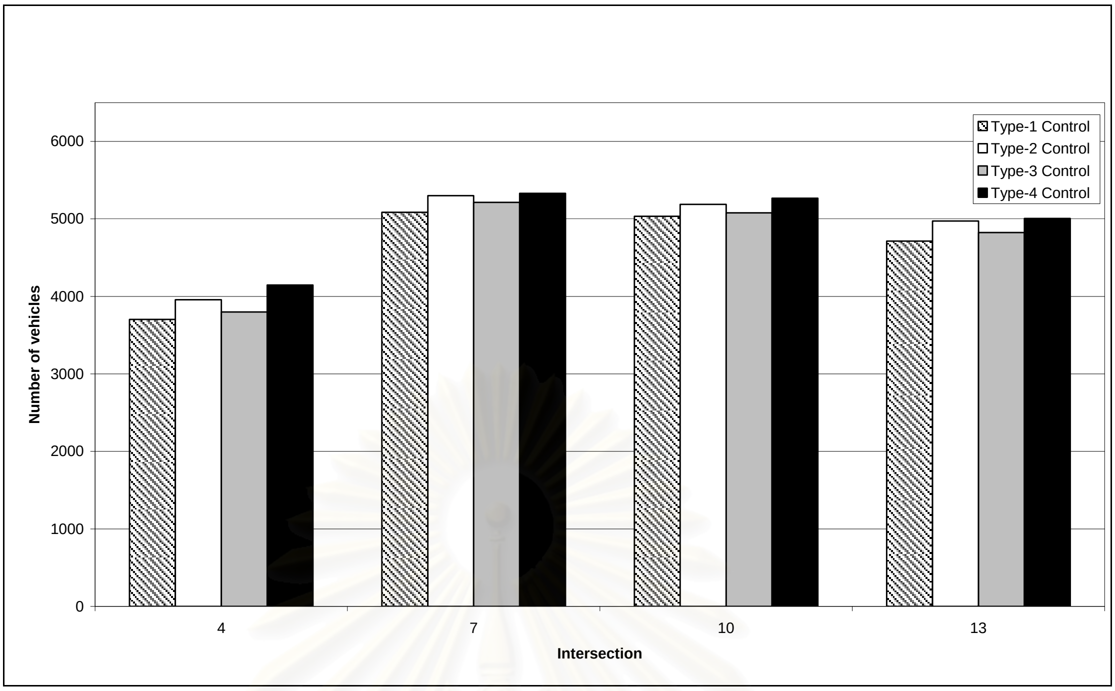


c) Demand-level 3 (saturated condition)



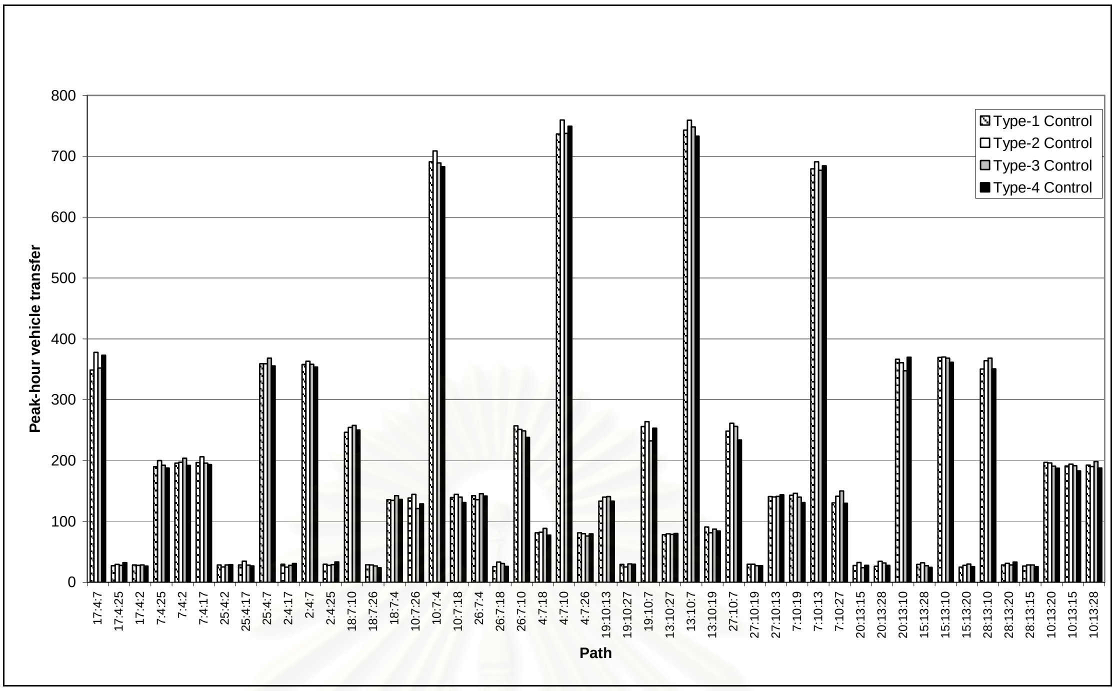
d) Demand-level 4 (oversaturated condition)

Figure C-2 Peak-hour number of vehicles passing each intersection (pattern-2 demand) (continued)

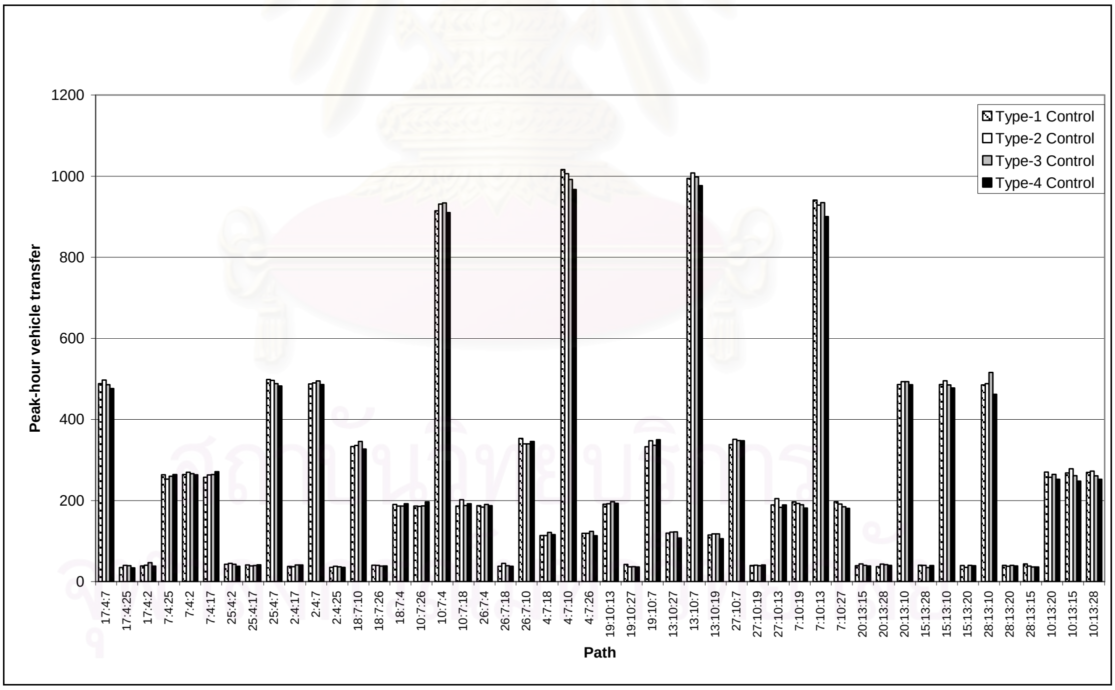


e) Demand-level 5 (oversaturated condition)

Figure C-2 Peak-hour number of vehicles passing each intersection (pattern-2 demand) (continued)

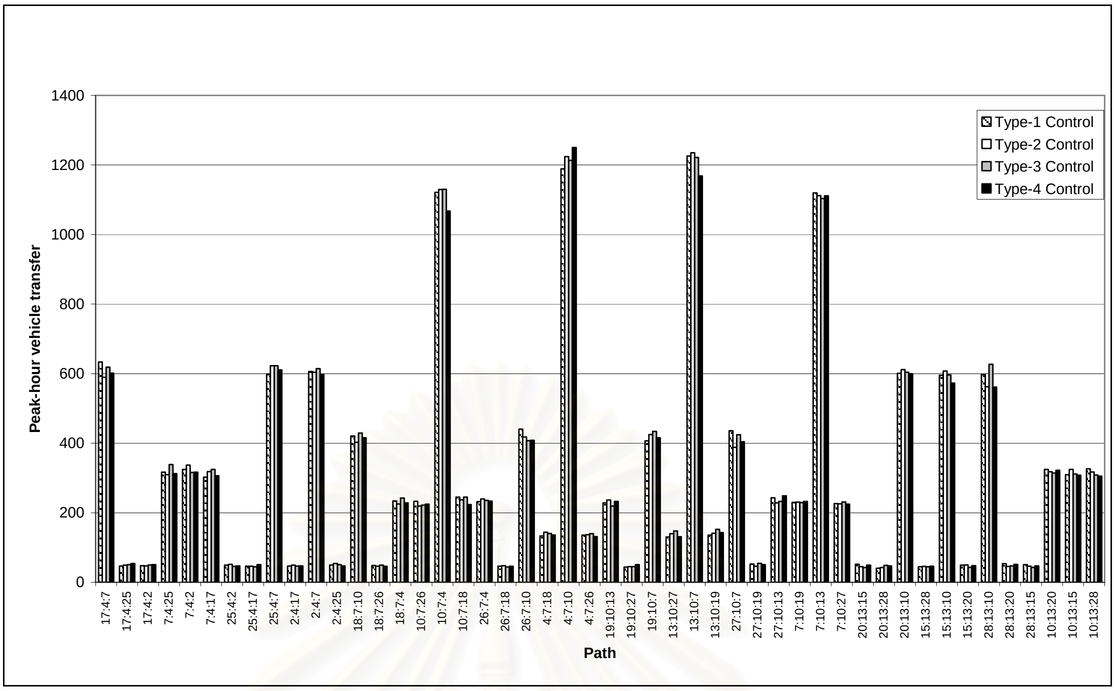


a) Demand-level 1 (unsaturated condition)

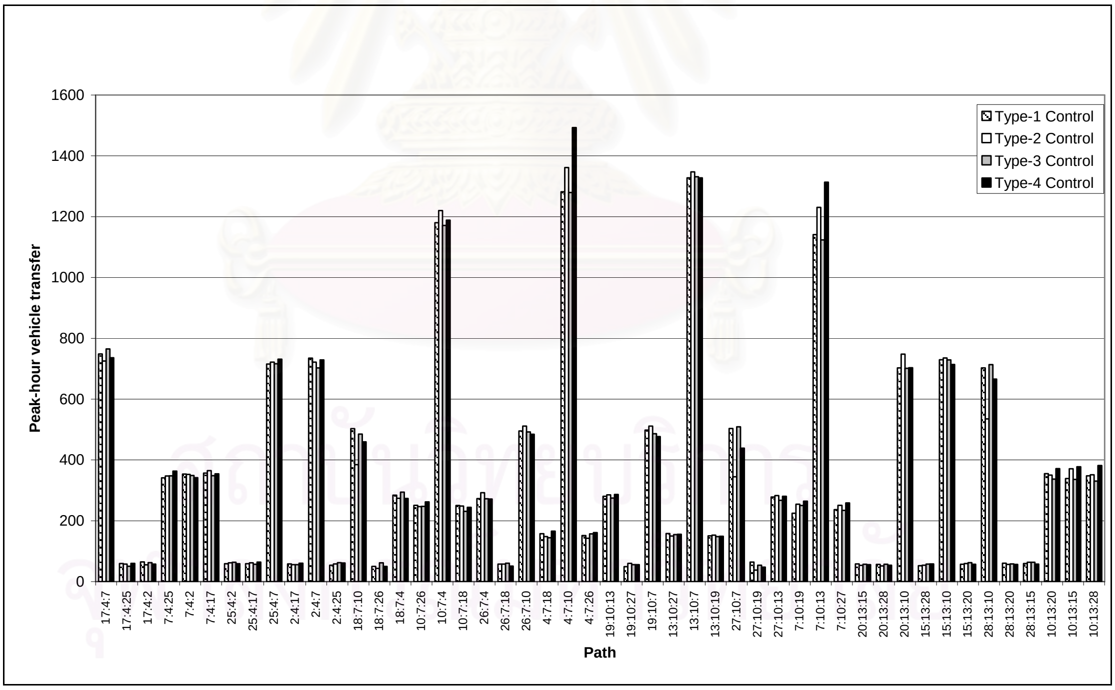


b) Demand-level 2 (unsaturated condition)

Figure C-3 Peak-hour vehicle transfer (pattern-1 demand)

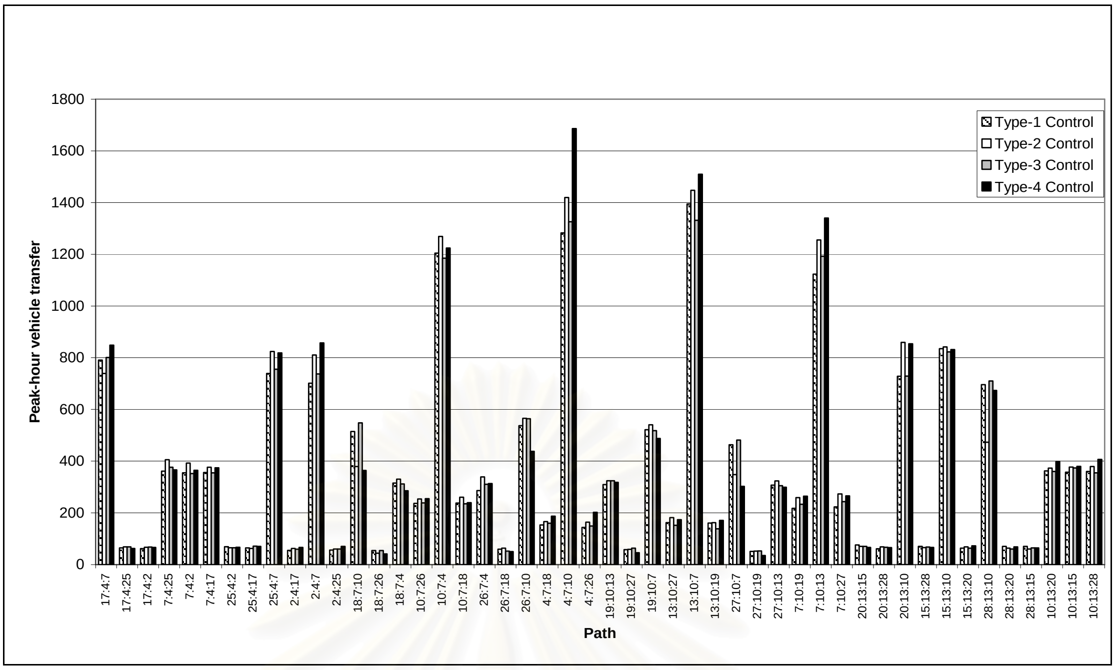


c) Demand-level 3 (saturated condition)



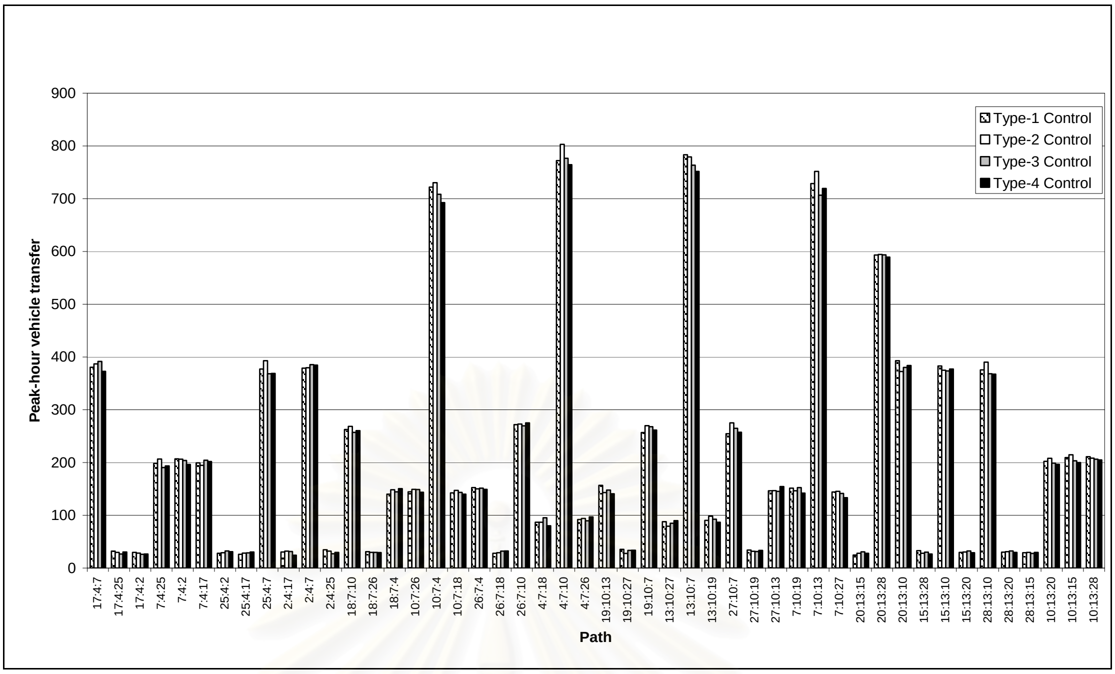
d) Demand-level 4 (oversaturated condition)

Figure C-3 Peak-hour vehicle transfer (pattern-1 demand) (continued)

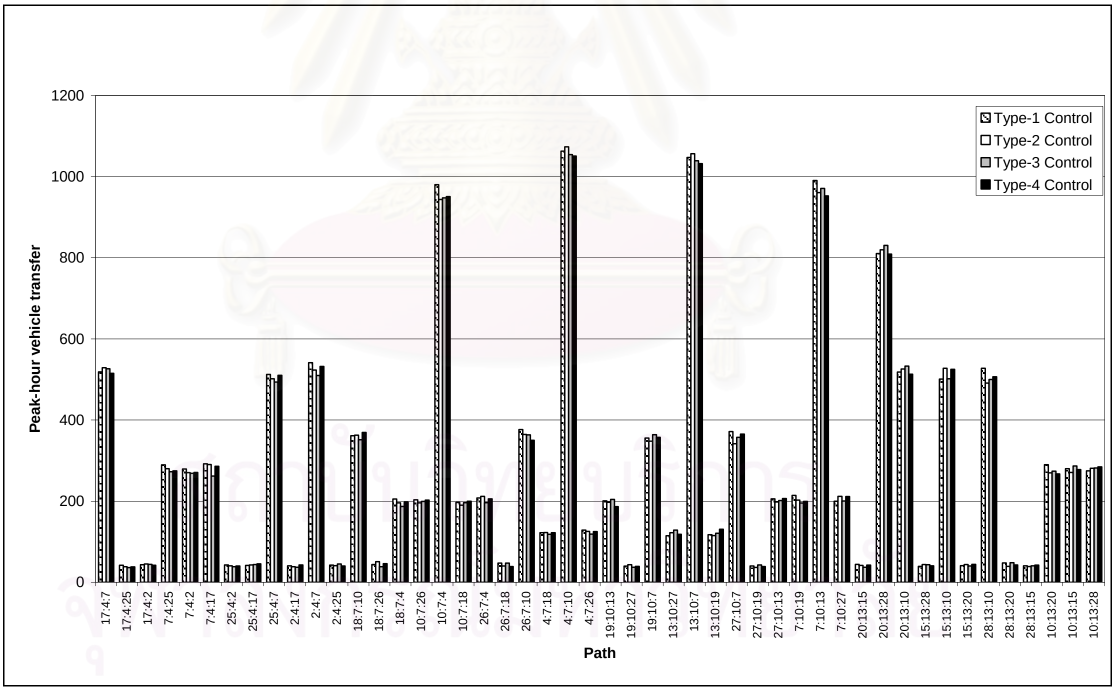


e) Demand-level 5 (oversaturated condition)

Figure C-3 Peak-hour vehicle transfer (pattern-1 demand) (continued)

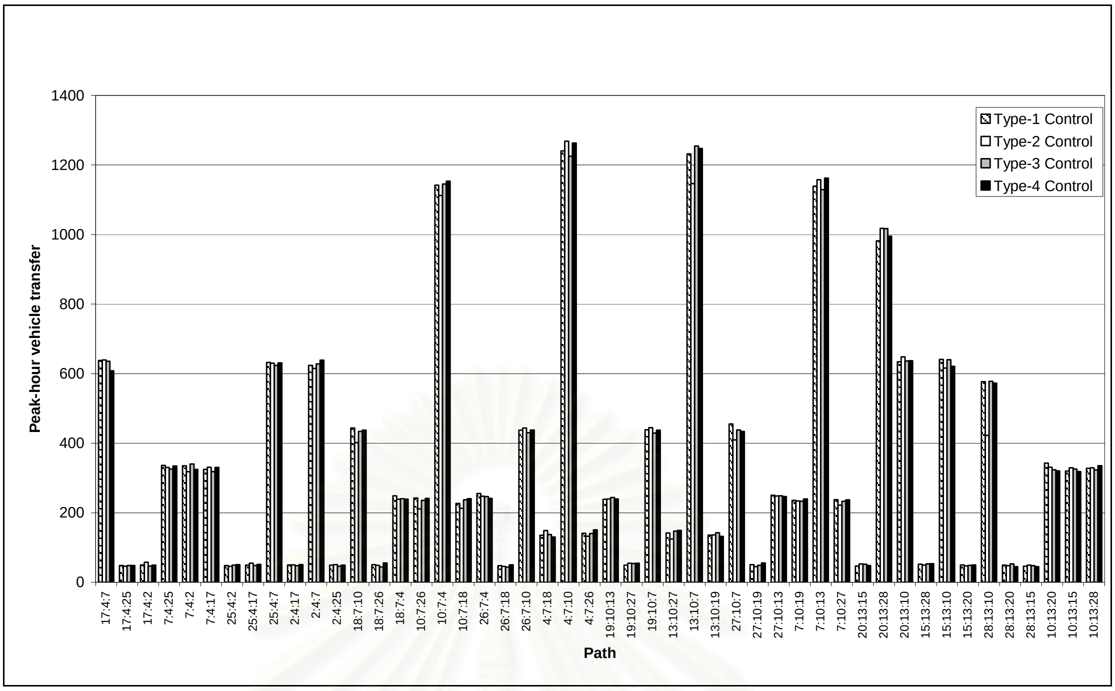


a) Demand-level 1 (unsaturated condition)

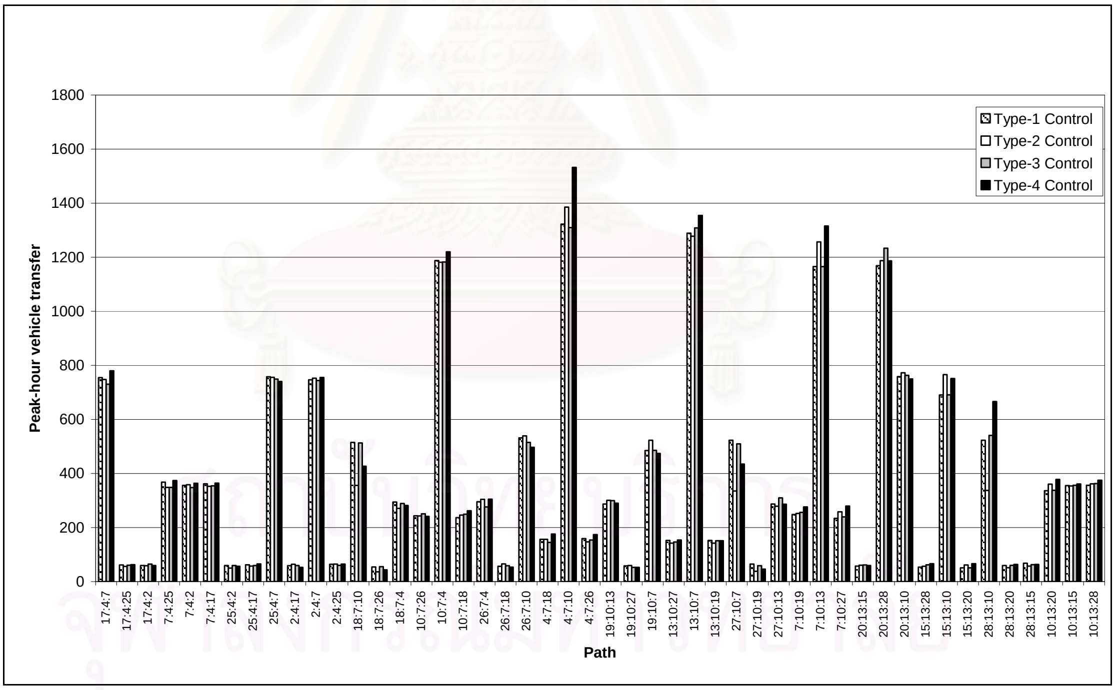


b) Demand-level 2 (unsaturated condition)

Figure C-4 Peak-hour vehicle transfer (pattern-2 demand)

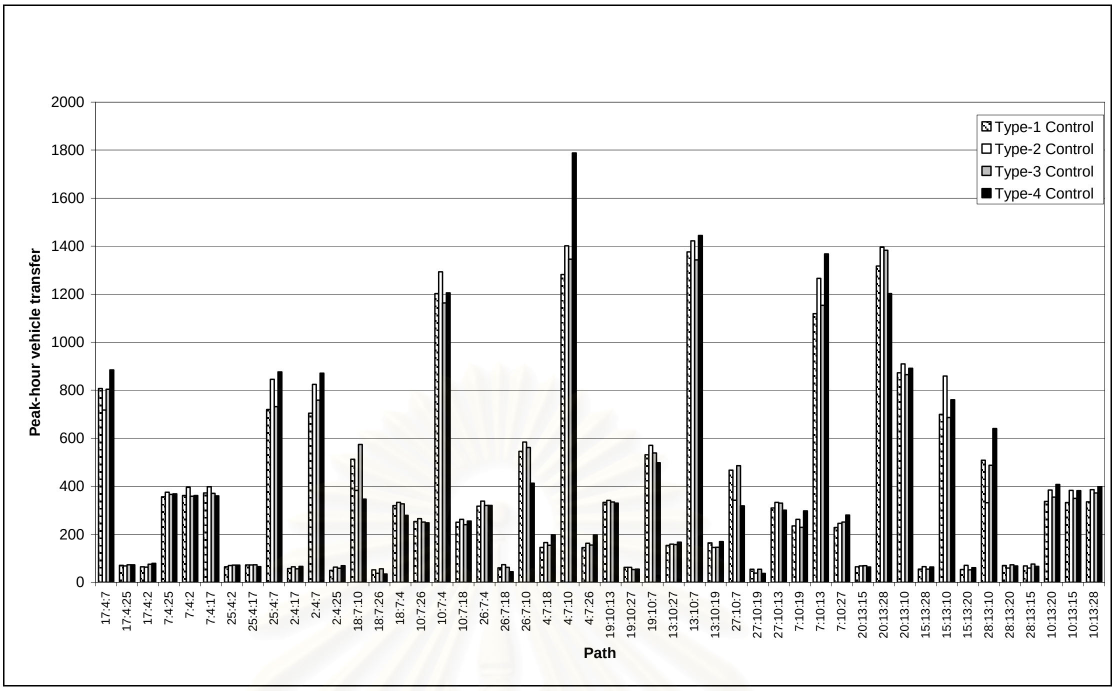


c) Demand-level 3 (saturated condition)



d) Demand-level 4 (oversaturated condition)

Figure C-4 Peak-hour vehicle transfer (pattern-2 demand) (continued)



e) Demand-level 5 (oversaturated condition)

Figure C-4 Peak-hour vehicle transfer (pattern-2 demand) (continued)

VITAE

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