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THE EFFECT OF TRAFFIC STRATEGIES ON EMISSIONS

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ABSTRACT

Air pollution and its relationship to the ecosystem and human life has always been the subject of a significant amount of study. The effect of highway air emissions on urban air quality has been studied for many years. This report contains a simulation of a single intersection in an urban area, using Arena®, a general purpose simulation program, and taking into account dynamic and stochastic considerations. The United States Environmental Protection Agency (USEPA) emission factors for idling situations were used to measure the emission of carbon monoxide (CO), volatile organic compounds (VOC) and nitrogen oxide (NOx) for the delay time. The simulation result predicts emission levels to be higher in a two-phase plan (unprotected left lane) with three different cycle times studied in this case (90, 120, and 140 seconds) compared to a three-phase plan (a protected left lane). However, the degree to which a two-phase plan is positively correlated with intersection cycle time suggests that a multi-faceted approach needs be taken in implementing modifications to reduce overall emissions.

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CHAPTER 1

INTRODUCTION

Air pollution has a substantial effect on both the ecosystem and the economy. It has been shown, for example, that even low concentrations of pollutants have pronounced negative effects on human health, and causes cancers, allergies, asthma and birth-related problems (Wilhelm, 2004). The resulting impact of pollution is now recognized as a significant factor, which threatens and endangers all forms of life on earth. The inevitable consequences of this phenomenon must be addressed now in order to protect the future of the planet. Luckily, local communities and governing bodies, and international organizations are beginning to take action.

Air pollution modeling is an important tool for the analysis of the relationships between various polluting factors, and the implications these relations have on overall emissions. For example, models made for a certain area or segment of a transport system - based on mathematical analysis and employing up-to-date computer programs can help solve traffic problems and their resulting increase in pollution. The facts that transportation systems and air pollution are related and the traffic patterns existing in transportation systems have dynamic, stochastic and discrete nature that can be easily modeled or simulated by various computer programs can justify the increasing efforts made for computer modeling of traffic problems and their consequential air pollution (Rodrigue et al., 2006).

The rapid growth of cities and increased commuting, either by public transportation or automobiles, are the main causes of traffic congestion problems. Rush hour traffic is responsible for about half of the traffic congestion in urban areas (Rodrigue et al., 2006). The other half is the result of random events such as severe weather or other natural phenomena in any given region or area. Generally, "congestion occurs when transport demand exceeds transport supply in a specific section of the transport system" (Rodrigue et al., 2006). The strategies to solve congestion problems have led to extensive construction of new transportation infrastructure which, in turn, has increased trip demands and caused more congestion in urban areas. The link between transportation and extensive environmental impact has been shown in various levels on geographical scales known as global warming to regional air emissions. By consuming energy in fossil fuels, there is not only a decrease in non-renewable energy sources but a related increase in air pollution. Carbon monoxide (CO), carbon dioxide (CO_2), NO_x (nitrogen oxides), and hydrocarbons (HC) are the main by-products of transportation systems which are damaging a number of ecological habitants. Air pollution is also the cause of decline of a number of species and by increasing the annual pollution, the number of species declined is getting larger (Van Dam et al., 1986). 70 to 90% of total CO emissions due to incomplete combustion of hydrocarbons is the result of transportation systems (Rodrigue et al., 2006). CO is one of the major contributors to greenhouse gases and 45 to 50% of total emissions of nitrogen oxides (NO_x) are made by transportation. High concentrations of these pollutants are toxic and affect the respiratory system. NO_x has increased at a rate of 0.2% annually over recent decades and 40% to 50% of total emissions of HC/VOC are caused by transportation as the result of incomplete combustion. They are components of acid rain and have effects on ozone depletion.

About 25% of total emissions of particulates are made by transportation (Rodrigue et al., 2006). They include smoke, soot, and dust which are the result of incomplete combustion of fossil fuels mostly from diesel engines. These contaminants have several effects on the human body, especially the lungs, if they are very small (Rodrigue et al., 2006). According to the Texas Transportation Institute (1999), drivers spend twice as much time in traffic as on vacation in one-third of cities in the US. Furthermore, time spent in traffic has increased by about 350 percent over the past 16 years in about half of the cities in the study. The annual cost of congestion was reported to be more than $72 billion in 1999 and various studies indicated that in half of the urban areas studied, the congestion cost was at least $500 million per year. This cost is related, of course to both extra travel time and consumed fuel, which in turn causes a decrease in fossil fuel sources world wide (Texas Transportation Institute, 1999). Thus, the temporal aspect of road pricing is as important as the space aspect of congestion cost analysis. This crucial point has to be taken into account in estimating the total cost of congestion (Yan and Lam, 1996).

An increase in greenhouse gas emissions during recent decades (one of the primary reasons for global warming) has become a focal point for policy making and strategic planning. Since transportation is one of the biggest contributors and the most controllable factor, transportation reforms have been the object of current challenges for decision makers. Indeed, according to Turchetta, the transportation segment is the cause of about one-quarter of the greenhouse gas emissions in the US and it is predicted that it is growing as the fastest source of emission in near future (Turchetta, 1999).

The results of Wilhelm's study show that a high concentration of pollutants caused by highway traffic air emissions effect birth outcomes such as low weight birth (LWB) and preterm birth. Furthermore, the results of epidemiological studies also conclude that there is a relationship between air pollution exposure and fetal development. As an example, an increase in CO and particulate matter and ozone concentration will increase the risk of LBW and preterm delivery. People who live near highways and major roads are more exposed to the pollution produced by vehicles, so they are more susceptible to respiratory diseases. The level of LWB among these people is higher than for those who live farther from highways (Wilhelm, 2004).

Greenhouse gases are emitted from natural (biogenic) sources and human activities (anthropogenic) sources. "Emissions of carbon dioxide are usually reported in terms of million metric tons of carbon equivalent (MMTCE) in US Department of Energy and EPA publications" (US department of transportation, 1998). Carbon units are defined as the weight of the carbon content of carbon dioxide (i.e., just the "C" in CO_2). Carbon units are the most common measure used by the scientific community since not all carbon from combustion is emitted in the form of carbon dioxide and carbon units are more convenient for comparisons with data on fuel consumption and carbon sequestration. "Emissions of other greenhouse gases are often reported in terms of the full molecular weight of the gas but may be converted into a carbon equivalent" (US department of transportation, 1998).

Table 1-1 depicts the annual green house gases increase for CO_2, CH_4, and N_2O in the atmosphere. It can be concluded that natural sources are the major emission sources of green house gases in comparison with anthropogenic sources. Although a large amount of these gases is absorbed by natural mechanisms, the total amount of green house gases is increased annually.

Table 1-1: green house gas sources and their annual increase in atmosphere

Gas	Biogenic sources	Anthropogenic sources	Absorption	Annual increase in gas in atmosphere
CO_2 (mmtCE)	150,000	7,100	154,000	3,100–3,500
CH_4 (mmt gas)	110-210	300-450	460-660	35–40
N_2O (mmt gas)	6-12	4-8	13-20	3–5

(US department of transportation, 1998)

The involvement in traffic assignments and environmental aspects of traffic design by transportation planners recently has got more attention and various studies have been conducted in this context. Based on Rilett and Benedek's report, an increase in consideration of environmental objectives and initiation of the Intelligent Vehicle-Highway System (IVHS) have had profound effects on traffic modeling assignments. Therefore, as the travel time is reduced by implementing the IVHS policies in signal-setting, the environmental pollution may decrease drastically (Rilett and Benedek's, 1994).

The issue of urban traffic planning and the future of public transportation has become a key topic of discussion among city officials and transportation engineers. The aim of a successful and extensive planning method is to ensure that its various segments and functionalities result in less traffic (queue and delay) and consequently, less pollution and negative environmental effects. In this study, it is proposed that this objective can be achieved through proper modeling and a network structure planned using an updated dynamic and interactive simulation model which considers the stochastic and probabilistic nature of traffic patterns (Rodrigue et al., 2006).

Research-supported decisions which aim to reduce the impact of traffic congestion are critical in urban and rural areas and may ultimately, at least, save a great deal of drivers' time and protect citizens' health. Several paradoxical phenomena have been identified in congested areas in various researches. According to Nagurney's study, making an improvement to the network may cause an increase in the total emissions within the area of the network. The emission increase can be a result of adding more roads to the network with no increase in travel demand. It is observed that in making any decision or policy for emission reduction by transportation authorities, network topology, travel demand and cost analysis must all be considered at the same time and with the same level of importance (Nagurney, 1999).

Computer simulation programs are effective tools for achieving detailed traffic modeling. They can be used to model various phenomena in traffic by calculating delay time, queue length and maximum capacity of a traffic segment (Fries et al., 2007). Isolated intersections and network-distributed intersections are two major categories of traffic signal design and each of these two approaches has its own advantages and disadvantages. A single junction is a node in a system which can be designed without considering other nodes and their connectivity with the system. It is easy to model and analyze and further analytical methods can be utilized to enhance its characteristics and functions in a rather short time. The results may be adequate for providing a solution to the congestion and traffic problems. There are a great number of articles that are devoted to the isolated intersections (Wong et al., 2001). However, other unpredictable phenomena of traffic may not be seen in the wider picture. Any change in circumstances will affect the node function and consequently can give a result far from that expected in the initial analysis. A traffic network has multiple complexities and there are many factors which must be considered. Therefore, the proposed model needs more complicated programs, and while the required time for analysis may be long and costly, the result is more accurate, reliable, and various aspects of the system can be predictable. Figure 1-1 and 1-2 on next page are schematic features of an isolated intersection and a simple network including several intersections that have to be modeled and analyzed.

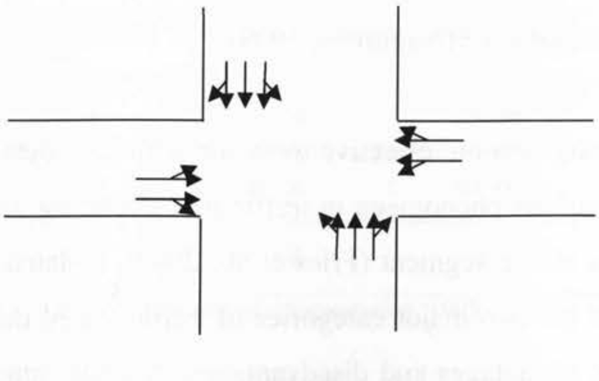


Figure 1-1: an isolated intersection

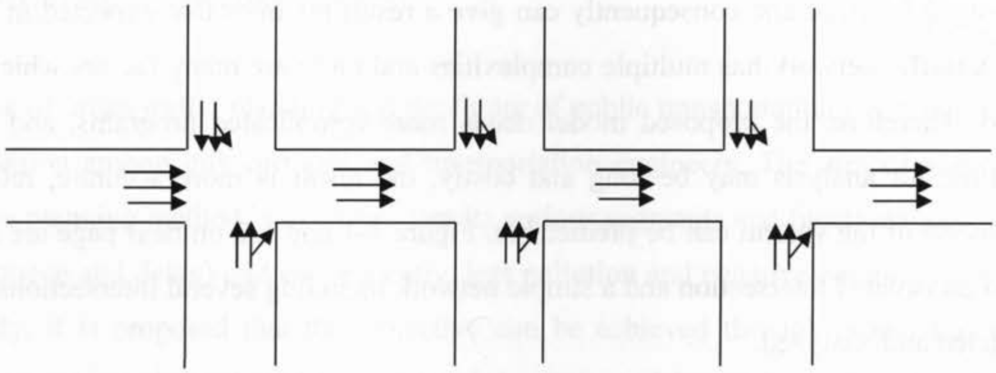


Figure 1-1: a simple network

CHAPTER 2

LITERATURE REVIEW

Generally, there are three types of operations in traffic signal-setting, (a) a pre-time operation that has a preset cycle time in which all intervals are being repeated, (b) a semi-actuated operation in which several detectors are installed on minor directions at an intersection, with the major street always getting the benefit of the green light until the detectors receive information about new arrivals on the minor approach, (c) a fully-actuated operation in which several detectors on each approach are active and the cycle time and green time can vary with respect to the arrival rate approaching from all directions (Sadoun, 2003). Isolated junctions (intersections) and junctions in a network as the main components of traffic and transportation planning especially in urban areas are further discussed. The review of what has been done and the efforts have been made to solve traffic problems can be a good start to eliminate or mitigate these issues.

ISOLATED INTERSECTIONS

A single junction basically is a case which has been studied since the phenomenon of traffic problems was first encountered and a great number of articles have been written to improve on the situation and suggest various solutions. Control variables for signalized intersections can be valued and used for problem solving by analytical methods. Computer programs for modeling intersection control can be divided into two groups: Class 1 and Class 2. In both methods, the stream flow and the saturation flow are known, but with Class 1 the green times must be calculated to optimize a certain objective while with Class 2 programs, green times can be evaluated from the compatibility of different approaches and the whole system in a cycle time will be optimized (Improta and Cantarella, 1984).

LANE-BASED OPTIMIZATION OF SIGNAL TIMINGS FOR ISOLATED JUNCTIONS

Although the group-based methods (a network including several intersections and their interactions), especially because of technological developments, have extensive applications, the stage-based method is still the method of choice of signal-setting and traffic control (Wong et al., 2001). Lane-based optimization of signal timing is a well known approach that applies the stage-based method. This optimization method is based on lane marking of an intersection regarding its traffic flow on each approach. First, the traffic lanes are grouped into traffic streams by traffic engineers, then, signal timing is determined using the stage-based method. Generally, there are three categories for signal timing optimization in this stage: capacity maximization, cycle time minimization, and delay minimization. Binary-Mix-Integer-Linear programs (BMILP) are used for capacity maximization and cycle time minimization and Binary-Mix-Integer-Non-Linear Programs (BMINLP) are used for delay minimization (Wong et al., 2001).

There are various formulations of these concepts that have gained favourable results and are being utilized in a broad range of instances when the distance between junctions is rather long. If the geometry of the junction is specified, the lane marking is the first step in the design and the signal-setting, using various methods of optimization, is the second step. However, there are some definitions and constraints that must be taken into account when calculating the signal timing by this method. The constraints being considered in this case to design a junction for minimum queuing and delay are as follows (Wong et al., 2001):

(i) Minimum permitted movement in a lane - there is at least a turning or through movement.

(ii) Permitted movements across adjacent lanes - for any two adjacent lanes k and k + 1, if turning to arm j is allowed for lane k + 1, for the safety reasons, turning movement to other arms is forbidden for lane k.

(iii) Cycle length - if $C_{\min}$ is the minimum cycle time, $C_{\max}$ is the maximum cycle time and C is the actual cycle time then $\frac{1}{C_{\min}} \geq \frac{1}{C} \geq \frac{1}{C_{\max}}$

(iv) Clearance time - if incompatible movements are permitted at the same time, a clearance time is required for safety purposes. Variables applied in this method are divided into binary variables; for example, permitted movements and continuous variables such as cycle length or the start and the duration of green light.

The name of the program (Binary-Mix-Integer-Linear programs) originated from the fact of including binary and continuous variables (Wong et al., 2001). Figure 2-3 depicts the lane-based optimization of signal timing for isolated junctions chart. The different stages of this method are illustrated in this chart.

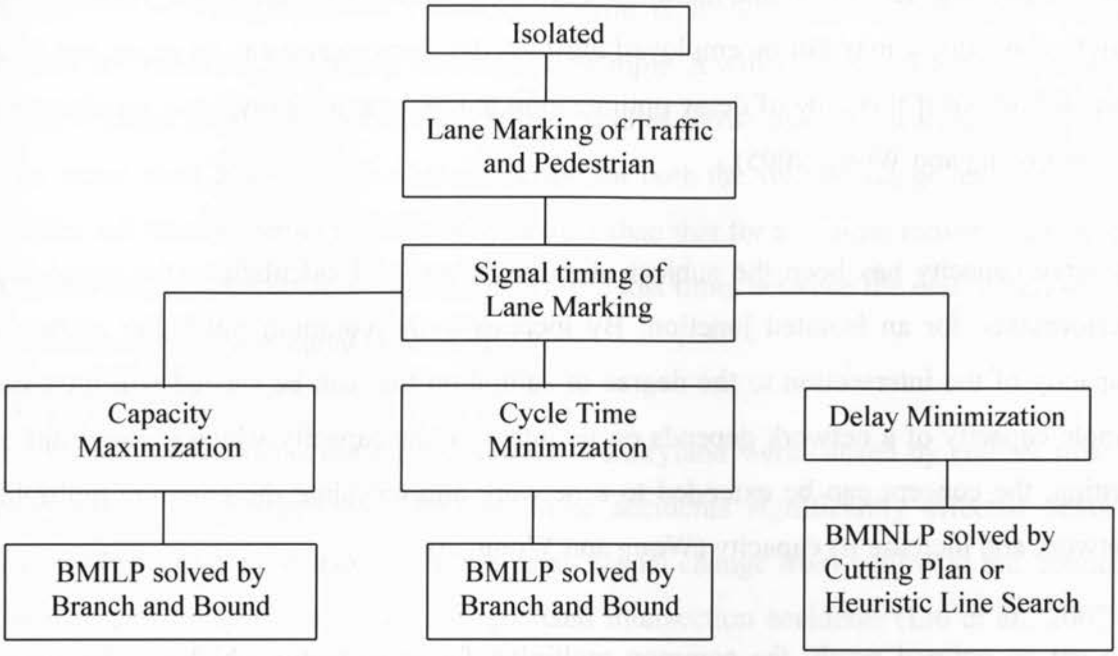


Figure 2-1: lane-based optimization of signal timing for isolated junctions chart

The capacity of an intersection which has a specific geometric layout can be maximized if appropriate and useful assumptions for turning movements are utilized and the turning traffic flows rather well. By increasing the junction capacity which requires a longer cycle length, an increased delay results and a longer queue is the inevitable outcome. Having determined the existing traffic flow and junction geometry, decreases in cycle length will reduce the reserve capacity of a junction (the increased allowable capacity exceeding the junction maximum degree of saturation) but delay time will be reduced which is preferred if a shorter queue is the objective of the design (Wong et al., 2001).

Traffic flows have different patterns at different times of the day. The signal design for the peak hours of one period may not be suitable for other periods in some cases. Multiple sets of lane markings could be a possible solution where safety is not a concern because frequent change of the lane marking within a day may result in undesirable accidents. To solve this problem, the original lane marking design will be refined, as well as the maximum and minimum cycle times. Additionally, the maximum degree of saturation and other variables of lane-based optimization methods have to be calculated or set, the maximum traffic demand in peak hours will be identified, and between the feasible limit of cycle length, the minimum delay will be obtained. In this method, the additional lane markings can be considered but because of the above mentioned safety concerns, it may not be employed during a day, however, it can be examined in any other day to find out if this type of delay optimization at this junction provides a satisfactory solution or not (Wong and Wong, 2005).

Reserve capacity has been the subject of formulations and calculations for signal-setting and performance for an isolated junction. By increasing the common multiplier of the flow, the capacity of the intersection to the degree of saturation that can be carried will increase. As the whole capacity of a network depends on its intersections capacity which is the result of signal-setting, the concept can be extended to a network and calculate the common multiplier of the network and increase its capacity (Wong and Wong, 2005).

To get an optimal result, the common multiplier factor (a factor which can be applied to an intersection's capacity) should apply to each link of an existing origin-destination matrix (this matrix is used to solve the traffic problems by analytical methods) in a way that the maximum demand does not exceed the maximum saturation of each intersection. In this case, there may be some changes to the routing of the system because the values may be getting close to the maximum capacity (Wong and Wong, 2005).

The Webster formula for optimal cycle length has been used for more than 40 years and its results are acceptable. It is very simple to use even without computer programs (Lan, 2004):

$$C_w = 5 + 1.5L / \left(1 - \sum Y_{ci}\right) \qquad C_{\min} \le C_w \le C_{\max}$$

C_w denotes the optimal cycle length in second, L is total lost time in second, $\sum Y_{ci}$, the intersection critical flow ratio, namely, sum of flow ratios (the approach volume divided by saturation flow rate) for critical movements or lane groups I, and $C_{\min}$ and $C_{\max}$ are practical minimum and maximum cycle lengths in second. Despite applicability of this formula in general conditions when the traffic volume is near a saturated or saturated condition, this formula results in a large cycle time which is not acceptable nor is it applicable when the critical flow ratio is equal to maximum saturated condition or exceeds it (Lan, 2004).

Setting the yellow change and red clearance interval is an important step in signal-setting in which there are various contributing factors. For example, it will affect a left turn movement and may cause a longer delay. The outcome of related studies shows that red clearance intervals (lost time) are rather short and suggests a longer period for both the yellow change and red clearance intervals for left turn movements, Preferably, longer than that for a straight movement (Liu et al., 2002). Three to five seconds is a value range for the lost time, between the end of yellow signal and beginning of the green signal (Branston, 1979).

About 30% of total accidents during last decade in Maryland were caused by crashes related to traffic signalling at intersections. Many of these accidents significantly effected delay and queuing problems, and were also sometimes fatal. Signal change from yellow to red, termed the dilemma zone, is a major contributor to signalized intersection accidents (Liu et al., 2007). As defined in the Institute of Traffic Engineers handbook, a "dilemma zone is the distance in which a vehicle approaching an intersection during the yellow phase can neither safely clear the intersection nor stop comfortably at the stop line" (Transportation and Traffic Engineering Handbook, 1985). Various factors like driver reaction time, vehicles arrival rate and speed, and vehicle acceleration and deceleration may affect the location and the length of a dilemma zone significantly. Different drivers have different approach and reaction times, as illustrated by the fact that young drivers are more aggressive and drive at higher speeds than would older and more conservative drivers. Different cars with different functionalities also have their effects on a dilemma zone. For example, a sedan, for the most part, moves slower than a sport car which again can be influenced by a particular driver's age and driving style. Indeed, a drivers' behaviour does not necessarily change with a longer yellow signal period and most drivers do not acknowledge the typical phase duration (Liu et al., 2007). Recent researchers learned that location and length of a dilemma zone has a dynamic nature and depends on drivers' behaviour, geometry of intersection, yellow light duration, and vehicle mechanical characteristics. By conducting a video-recording study at several critical intersections, all above factors were measured and the distribution of dilemma zone and its effect on the cycle time were found. The findings prove that theoretically calculated dilemma zones and their length are substantially different from the actual distribution of dilemma zones. Furthermore, while this zone for most drivers may be reduced or eliminated, there remain some driving groups who will continue to experience the dilemma zone even when with a yellow period of 6 seconds (Liu et al., 2007).

To set a signal on an isolated intersection, there are three important components which must be determined.

1) The phasing plan, which specifies the appropriate phases that should be assigned to an intersection. At this most important step of the design, observation and analysis must be combined with professional judgments;
2) Cycle time determination, which can be started with appropriate formulas and gets close to the optimal length by heuristic methods, using computer programming; and
3) Green light time allocation to the various phases (Sadoun, 2003).

CYCLE TIME AND PHASE PLAN DETERMINATION

Two-phase, three-phase, and four-phase plans are actual cases that are applied in current intersection situations. If there is no need to protect a left turn at any of the approaches, which is the simplest case, it is a two-phase plan. When left turns in two compatible approaches need to be protected, it is a three-phase plan, and when left turns in all compatible approaches have to be protected, it is a four-phase plan. To determine the left protection in any given approach, the following guidelines are given to simplify this situation. (a) If left turn volume is more than 250 vehicles per hour, left turn protection is required. (b) If left turn volume is less than 100 vehicles per hour there is no need for left turn protection. To calculate the cycle time the formula below is used (Sadoun, 2003):

$$C = [(N \cdot t_1)]/1 - V_c / [PHF \cdot v/c \cdot (3600/h)]$$

where N is the number of signal phases, t_1 is the lost time per phase in second, V_c is the sum of critical lane volumes in vehicle per hour, PHF is the peak-hour factor, v/c is the required volume-to-capacity ratio, and h is the saturation headway in second (h = 2.23 for a 12-foot lane).

To understand the components of this formula some concepts should be explained. The discharge headway is the time between two successive vehicles crossing the curb line as the rear wheel of the reference vehicle crosses the curb line. The first headway is relatively long because reaction time and the time to accelerate for the first are longer. The second headway is shorter; after four or five vehicles, the headway tends to level out to some level, called the saturation headway (*h*). Saturation flow rate (vehicles/hour) can be calculated using $S = 3600/h$, where h is the saturation headway (Sadoun, 2003).

To obtain an efficient cycle time which causes a minimum delay but does not reduce the demand capacity, a framework for modeling and analysis of a fluctuating demand and Monte Carlo simulation is utilized. The analysis result presented that a longer cycle time is not a reason for minimum delay, and also, a very short cycle time will decrease the intersection capacity and significantly increase delay (Han and Li, 2007).

NETWORKS OF INTERSECTIONS (ADAPTIVE SYSTEMS)

Different network analysis methods have different approaches to traffic problem solving processes. Signal control in a network traffic model affects the distribution of traffic on the equilibrium and elaborated mathematical framework which should be solved for signal-setting in an area network (Allsop, 1974). Traffic control strategies should employ comprehensive and systematic methods of design while taking into account appropriate optimization tools to offer an efficient and realistic signal design. Group-based optimization and parallel computation are two general approaches that can optimize a network or a set of networks signal design. Group-based optimization is a way to spot more than one junction (traffic light) and to control the whole system and apply a signal-setting system which can be efficient where queues are negligible or short as possible. Parallel computation for signal timing optimization is becoming popular because of its relatively low cost and easy applicability. This type of optimization for area traffic control considers heuristics including network-wide steps and junction-based steps which control variables for signal-setting at all junctions simultaneously while these variables change in turn in each junction. Using parallel computation will reduce computing time which in turn can lead to finding an optimal solution for each case (Wong, 1997).

Traffic signal design for diamond intersections, especially when two intersections are fairly close (122m-244m), is a challenge due to numerous interactive effects. The group based design as an adapted method is applied to solve the problem. The short distance between intersections decreases the capacity of vehicles in queues and without an appropriate cycle time and signal design, the downstream backlog will effect upstream approaches and block the traffic flow. Demand starvation is another effect that will significantly decrease the number of vehicles which cross the interchange. This phenomenon occurs when the duration of the downstream green light is not used efficiently because vehicles upstream are delayed by conditions which prevent them from reaching downstream. Optimization of the signal control for diamond interchanges is a crucial step that must be undertaken and adaptive methods are an effective and useful means to overcome this problem (Fang and Elefteriadou, 2006).

The TRANSYT traffic model provides an appropriate solution to the group-based traffic model for signal-setting and delay or queue calculation (Wong et al., 1999). The TRANSYT method is a simulation model for a network and its movements which assumes the effects of platoon dispersion. This assumption is that movement time is fixed but the delay for each link depends on the stream of other links in the network. This method has shown to be a good approach and provides the solution for complicated asymmetric traffic problems (Wong et al., 1999). TRANSYT is a powerful simulation tool which can solve traffic equilibriums made by assuming a common cycle time, based on a group of traffic models in a network. This heuristic method based on the equilibrium flow pattern will repeat the procedures to determine a better cycle and signal timing which can satisfy certain criteria and requirements (Wong and Yong, 1999). Sensitivity analysis on the network considering the signal timing variables is another reason for development of a network based calculation by TRANSYT modeling (Wong et al., 1999). An increase in the capacity of a network to the maximum level is a subject of study that can be solved by related modeling program like TRANSYT. If the total delay of a road network as equilibrium flows is minimized, the capacity maximization can then be formulated. To determine the reserve capacity of a network, a mathematical program with equilibrium constraints (MPEC) is used and the signal-setting with respect to a minimal value for the total delay is optimized. The projected gradient method (PGM) which is used to solve the problem has significant superiority over other methods in delay minimization and travel demand maximization and is calculated with shorter computation time and effort (Chiou, 2007).

The urban traffic network consists of above and below ground facilities which need to be maintained or expanded. The resulting closure of part of the traffic network for this purpose may cause problems such as queues and delays even in the areas far from the closure location in the network. The impact of network maintenance and expansion must be minimized or eliminated and to this end lane closures need to be planned, scheduled, coordinated, and simulated. Minimizing the impact of network road closures is a critical task to be undertaken in any given urban area. By using genetic algorithm as the search engine for the closure generation and its patterns, combined with distributed simulations, the related traffic assignments can be solved faster, by determining the total network travel time and optimizing signal-settings (Ma et al., 2004).

CYCLE TIME IN NETWORKS

Calculating or estimating of cycle times and allocation of green and red lights are important stages of signal-setting at intersections. In a network, a cycle time can be fixed or variable. The investigation of fixed-time cycle time strategies versus multiple cycle time strategies reveals that use of real-time data has more sensible and efficient outcomes than fixed-time data.

The main reasons for this preference are

(i) Traffic demands may differ within a given day and over different days, and these demands may change in the long term;

(ii) Change in turning movements (the demand) at the same way plus change in drivers reaction as a result of new signal-setting; and Incidents which may deteriorate the predictability of the system (Papageorgiou et al., 2003).

To apply an efficient method on an urban traffic network and use advanced computer programs, the whole network should be equipped with sensitive sensors and cameras to obtain detailed and enhanced input data. This information is then sent to a traffic control centre and the computers connected to this centre will compile the input data, and, finally, appropriate optimization methods for optimal signal design will be determined (Sadoun, 2003).

As mentioned above, adaptive methods, as fully actuated methods, have extensive applicability in traffic signal-setting. Split Cycle Offset Optimization Technique (SCOOT) is an adaptive traffic control system which, regarding embedded detectors signal at intersection(s), can respond automatically and change the cycle time and green light time allocation. This method has had a great impact on urban control traffic systems and is being used in the U.S., United Kingdom, and Canada (Sadoun, 2003). This system can reduce the delay time in urban area about 20% and has other applications such as incident detection, vehicle emissions estimates, bus priority, and on-line saturation occupancy measurement. Sydney Coordinated Adaptive Traffic System (SCATS) is another popular traffic control system which can adjust the signal timing in response to traffic change dynamically (Sadoun, 2003).

DELAY AND QUEUE LENGTH

To evaluate signalized intersections functionality, delay time and queue length are two quantitative measures which can be calculated when using various methods. In the past, as stated above, because of the lack of technological tools or their high cost the manual method were more attractive. With the advancement of technology and the demand for more accurate and precise methodologies, delay and queue calculations are now more feasible and popular.

There are two methods for vehicle delay and queue length calculation in real time - the Input-Output a manual technique and the Hybrid method - which takes advantage of further use of an advanced sensor (detector). These two techniques are similar in the use of advance detector actuations (for arrival measurement), phase change data, parametric data such as saturation headway, and storage capacity. The only difference is that the Hybrid technique uses stop bar detector, too. The stop bar detector measures the number of departures from the stop bar over time which gives a higher level of input data accuracy (departure profile for current signal cycle) and therefore, more accurate output. In both methods, the sensor (detector) location is important, and for spillback prevention caused by those queues that past the detector, it is best placed at, or more than, 130 m in advance of the intersection (Anuj et al., 2007).

While recurring congestion happens during the peak periods of morning or evening and cause delay, nonrecurring congestion occurs because of weather, car accidents and construction.

Recent data shows that about 57% of congestion situations in the United States are made by or are an effect of traffic incidents (Fries et al., 2007). Incident modeling in intersections and highways holds important applicability to traffic simulations and the resulting outcomes seem to be promising. It has been observed that if computational resources increase and the traffic network size decreases, the accuracy and precision of modeling dramatically improve (Fries et al., 2007). A fixed-time based traffic signal optimization for isolated junctions has had appropriate results that have been carried out for quite some time since its creation. Progress made in various techniques has given the opportunity to apply this method of signal time setting on coordinated junction in a network by the decomposition approach. Although adaptive methods like TRANSYT show good results, combining the isolated junction design method and decomposition approach has had a more efficient and useful optimization outcome. This method, which applies group-based techniques at isolated intersections, takes advantage of their flexibility over stage-based and inter-stage design techniques. The first step includes individual intersection analysis and optimization of minimum cycle length by using the group-based method techniques (three optimizations are done for each order of incompatible flows) followed by calculation of the reserve capacity and delay as based on the allocated cycle time. The second step includes stage and inter-stage sequencing taken from the cycle length optimization at the intersection which has the better result. The most important part of the third step is the common cycle optimization with respect to better performance in the network. And finally, in the fourth step, stage and inter-stage sequencing allocation, and general optimization of the whole network for stage duration will be prepared. If a junction situation in the network is critical, it is possible to do a more elaborate analysis and individualized analysis of this junction while still taking advantage of being in the network (Heydecker, 1996).

SPEED EFFECT

Speed is an important factor in both traffic congestion and air pollution (Ghassan, 2002). In U.S. about 70% of urban peak hour traffic is congested (Turner, 1999).

To manage a traffic system, which is mostly congested during peak-hours, more accurate methods are required. As science and technology develops, new techniques are available to solve traffic problems which have been presented for a long time. For example, the dynamic characteristics of intelligent transportation systems (ITS) are being applied to gain more enhanced and efficient results in integrated signal-setting in the area network (Ghassan, 2002). In integrated adaptive-signal dynamic-speed control method, the speed control has been considered as the core element of signal-setting and optimization and the optimized speed with respect to other signal control variants is identified. Drivers, by following this optimized speed when entering a link and in various traffic conditions, will get the maximum benefit of the system (Ghassan, 2002).

System performance will improve with applying variable speed as a result of this type of research. The difference between the first and the last cycle time as the speed varies is shown in the Figure 2-1. In the first cycle, the speed is relatively lower because the system is congested and all links have almost the same volume of traffic, but in the last cycle the flow is running smoothly and the speed is higher.

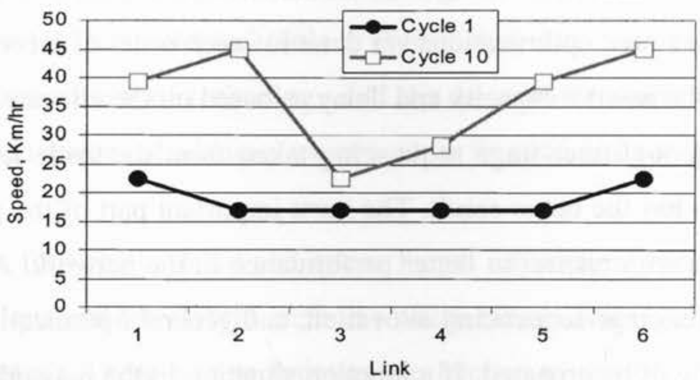


Figure 2-2: selected speeds along system for first and last cycles
(Ghassan, 2002, p. 449)

If a vehicle is not able to leave a link in the cycle time it arrived, this vehicle has a stop. The number of stops in a system network is an important indicator of the quality of a system because of its relationship to system improvement, air emission, and fuel consumption. Consequently, by decreasing the number of stops in a link, the queue length will be decreased and a more favourable link is being used. As Figure 2-2 depicts, the number of stops decreases by speed variance while in a constant speed it increases (Ghassan, 2002).

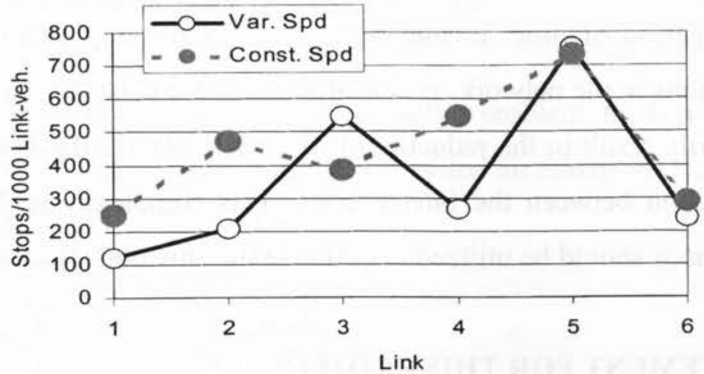


Figure 2-3: variation of number of stops on system links
(Ghassan, 2002, p. 450)

In another attempt to evaluate the effect of speed control on traffic delay and also to measure the emissions increased by this enforcement technique, data collected at a highway are utilized. In this model, cars with passenger and only in one lane are taken into account. The outcomes of this research show that the traffic speed pattern almost follows a normal distribution as experimental cases indicate. Gamma function gives a good estimate for the distribution of time between speed violators, and by conducting a Monte Carlo simulation the time intervals between violators is generated. The effective distances for measuring the initial and final speed are 200 m before an intersection in the approaching movement and 100 m after passing the intersection. For emission measurement and modeling, the time spent in different modes of vehicle operation is characterized; namely idling, accelerating, decelerating and cruising. It can be observed that if the mean and standard deviation of speed increase because of longer red times derived by speed violators, the delay will increase as well. Also, if all cars have to stop due to a high speed and signal-setting strategy, the HC emission increase is higher than the CO emission increase and generally, HC emission is more critical than others in an idling situation. On the other hand, an increase in the green time will increase the number of vehicles that pass the intersection and those that don't reduce their speed which consequently decreases their emission. The result of this experiment proves that in a long term application of this type of signal-setting, the drivers will change their attitude and, furthermore, the system will experience a balance which, by decreases in speed and passing the intersection, the emissions resulting from idling and queue length will significantly decrease (Coelho et al., 2005). Generally, regarding some other perspectives, there are two methods to solve the network signal design problem (NSDP); namely, the strategic method and the long-rage method. The former tries to set an optimal arrangement of signal-setting at a specific point in time while the latter strives to find an optimal form of signal-setting for a long period of time. In the long-range NSDP, emphasis is on nodes which are individual intersections in the network. It indicates that if the delay at one intersection is reduced it does not necessarily result in the reduction of the total delay in the system and that there is a complicated interaction between the intersections. This complexity in NSDP is so large that some heuristic methods should be utilized to optimize the situation (Horowitz and Patel, 2005).

PROBLEM STATEMENT FOR THIS PROJECT

Traffic signal design and its effects on the transportation network have been identified as the fundamental and critical parts of the traffic problem since the early days of studies on traffic problems. The early networks were considered a simple network consisting of two nodes which were connected by two links. More recent networks have been determined by linking the rather greater number of intersections as nodes. While there has been much research aimed at solving traffic problems and reducing or eliminating air emissions, there is still a lot of work to be done in order to arrive at a more practical and effective solution.

An intersection as a critical part of a traffic network, which is identified as an isolated or linked node, is the centre of this system. The study of an intersection which is in a systematic relationship with other junctions and the whole network is crucial. All techniques, strategies and constraints applied on an intersection can affect other parts of an integrated system. Thus, the objectives of this study are as follows:

- To present and discuss case studies that illustrate vehicle arrival patterns and their behavior when passing through an intersection;
- To consider various transportation strategies for signal-setting (two-phase and three-phase plans) and their effects on congestion and emissions;
- To perform sensitivity analyses on the traffic signal design, such as the effects of different cycle times on signal-setting for a shorter queue and less delay either in an intersection or on the network by modeling and simulating flows based on an up-to-date software such as Arena®;

- To combine Arena®'s ability to calculate queue length and delay time at intersections and their related emissions of VOC, CO and NOx.

In the next chapter, the methodology that is used for this problem, the formulas and factors taken from several references for calculation of queue times and air emissions are discussed.

CHAPTER 3

METHODOLOGY

Because of the probabilistic nature of traffic demands and the arrival rates, a trivial approach which assumes a uniform distribution of arrival to an intersection will not be an optimum method to calculate delay and level of service (Han and Li, 2007). If the arrival into a queuing system is independent, probabilistic and nondeterministic, a Poisson distribution is a good estimation for this system, and simulation utilizing a modeling process would be more realistic. When vehicles arrive in batches and their arrival rate is not independent, the Poisson distribution does not give a good estimation, and other forms of distributions should be considered (Burman and Smith, 1986). If the arrival of vehicles into intersections has a Poisson distribution with a specific mean, the vehicles inter-arrival rate is the reverse of this Poisson function mean. Thus, by calculating the arrival distribution mean, the inter-arrival rate that can be used in the simulation program will be found (Kelton et al., 2002). As mentioned before, the arrival patterns in queue systems should follow certain distributions to be able to fit into different design programs. Although having specific and clear data which present one or more probability distributions is preferred, sometimes the arrival patterns can be described by using general and ordinary terminology like many, less, fast or slow. According to fuzzy queuing theory, use of this type of wording can be realistic as much as more conventional terminology when appropriate accuracy and precision is not possible. The important point remains, however, that the description of the arrival rate has to be as precise as possible otherwise the outcomes will be imprecise. It is also important to employ performance measures consistent with input data to represent an accurate system (Chen, 2005).

To consider the discrete, dynamic, and stochastic nature of vehicles arrival rate, analytical methods may not be as effective and accurate as programming methods. Because most computer programs made for traffic design, take into account the actual characteristics of arrival rates, the results are more realistic and signal designing can be more efficient. Thus, using a computer program that simulates the actual case is the modeling choice and will be discussed later in this chapter.

THROUGH CAR EQUIVALENT

The analytical methods and model simulations use a number of vehicles in through approaches as the through vehicle arrival onto an intersection. Number of through car equivalent (TCU) on each approach has to be calculated by using the Table 3-1 which in fact addresses the impact of these movements on cycle lengths (National Institute for Advanced Transportation Technology at the University of Idaho). Thus, left-turns and right-turns movement must be converted to correspondent through movements. To convert turning movements to through approaches, time, length, number of cars on the opposite through direction and number of pedestrians passing the intersection must be considered as the factors that are applied to each turning movement.

Table 3-1: through car equivalents

Through Car Equivalents for Left-Turning Vehicles, E_{LT}

Opposing Flow	Number of Opposing Lanes, N_o	
V_o (vph)	1	3
0	1.1	1.1
200	2.5	1.8
400	5	2.5
600	10	4
800	13	6
1000	15	10
> 1200	15	15

E_{LT} (protected left turns) =1.05

Number of conflicting pedestrian (peds/hr)	Equivalent
None (0)	1.18
Low (50)	1.21
Moderate (200)	1.32
High (400)	1.52
Extreme (800)	2.13

(National Institute for Advanced Transportation Technology at the University of Idaho)

SIMULATION

Simulation, as a type of modeling, refers to a broad collection of methods and applications to mimic the behavior of real systems, usually on a computer with appropriate software. In fact simulation can be an extremely general term since the idea applies across many fields, industries, and applications. More specifically, computer simulation deals with models of systems, where a system is a facility or process (either actual or planned), such as:

- A manufacturing plant with machines, people, transport devices, conveyor belts, and storage space.
- An emergency facility in a hospital, including personnel, rooms, equipment, supplies, and patient transport.
- A freeway system of road segments, interchanges, controls, and traffic.
- An intersection with its traffic system
- A fast-food restaurant with workers of different types, customers, equipment, and supplies (Kelton et al., 2002).

A system has to be studied by specialists who measure its performance, improve its operation, or design it if it does not exist. Computer simulation refers to methods for studying a wide variety of models of real world systems by numerical evaluation using software designed to imitate the system's operations or characteristics, often overtime. By simulating a transportation network and observing its elements interaction and also considering the traffic applications and the implemented strategies, it can be seen that if the determined strategies are beneficial and efficient or not (Pritsker et al., 1989).

While most research in last 50 years studied the static aspect of traffic assignments, new studies tend to consider the dynamic nature of traffic patterns especially in peak periods. According to a study based on some simplifying assumptions, the total time spent by a vehicle on a link consists of fixed travel time and a waiting time in a link queue when the vehicle passes the link. This method expresses the dynamic traffic assignment model against the static form of this assignment and shows the space-time relation of the total travel time (Drissi-Kaitouni and Hameda-Benchekroun, 1992).

Regarding various aspects and characteristics of traffic problems and their arrival and inter-arrival patterns which are described in the literature review, utilization of software that includes the following qualities and characteristics seems to be necessary:

(a) capability of considering the discrete nature of the subjects since the arrival of a vehicle into the system is totally independent from other vehicles arrival,

(b) ability to model cases which are dynamic by nature,

(c) ability to present the stochastic approach to traffic patterns which are stochastic by nature,

(d) has the latest progress and advanced programming ability in modeling and solving the traffic problems.

ARENA®

Arena®, a program that possesses the advanced modeling ability via simulation of complicated cases, and which has all the required characteristics mentioned above, is a suitable alternative for traffic assignment modeling. Although there are several programs such as Cincro and TRANSYT for traffic problem simulation in real time which yield acceptable results, these programs are expensive and not easily accessible. Additionally, the availability of Arena® and the exploration of its ability to solve signal-setting problems create a strong motive to use this program for this project.

There are few studies on the calibration of an iterative simulation-based dynamic traffic assignment (DTA). To obtain an acceptable result of the calibration of a simulation model, its output should be compared with the corresponding empirical data. It has been observed that calibration attempts improve the path flows and the turning movement streams at the same time (Mahut et al., 2004). The outputs resulted from Arena® simulation software and the relevant graphs are analyzed and discussed in Chapter 4.

Because of two important reasons, traffic modeling for an isolated intersection (single junction) is the case study for this project: (1) the significant effects of isolated intersections on traffic patterns as stated in the review and (2) the unknown level functionality of Arena® in traffic case simulation. The outcomes of the simulation after analysis will be a clear indication of the ability and effectiveness of Arena® in simulation traffic problems and for their further investigations, such as network modeling and urban traffic planning.

CASE STUDY

Car arrival measured at one of the major intersections in the rural-urban area (the part that has been urbanized in a fast pace in recent years but is still different from downtown and its complexities) of the North-East in Toronto have been used as the input data. These arrival rates have been measured by a transportation consulting company. The intersection includes four arms - each arm has three lanes and East-West approaches have higher arrival rates. To calculate the cycle time utilizing one of the analytical methods formula (Sadoun, 2003) discussed in literature review, through car equivalents of right and left turn movements (table 2-1) taken from National Institute for Advanced Transportation Technology at the University of Idaho are used to calculate the total arrivals on all directions in the rush hour period (7:45 to 8:45 am in this study). Figure 3-1 in the next page shows the case study intersection with different arrival rates in three peak hours and the total numbers of vehicles in eight hours including the three mentioned peak hours (in the next chapter more details about peak hour periods and selection of the most critical hour are discussed). The percentage shown in the figure is the percentage of the number of trucks of total vehicle arrival for each peak hour measurement. The number of pedestrians crossing the intersection is also shown in this figure. The single arrow in each direction (arm) is the total number of vehicles from turning and through movements that enter the intersection through the same arm and eventually cross the intersection.

It is important to carefully consider economical and ecological factors when solving traffic problems and determining appropriate analytical methods. The next chapter discusses the traffic problems that are the subject of this project. It will also discuss different cycle times (which are important variables that effect signal design) and the results of two different strategies: two-phase and three-phase plans. The emission factors are also discussed in context of calculating the total emissions produced by traffic flow.

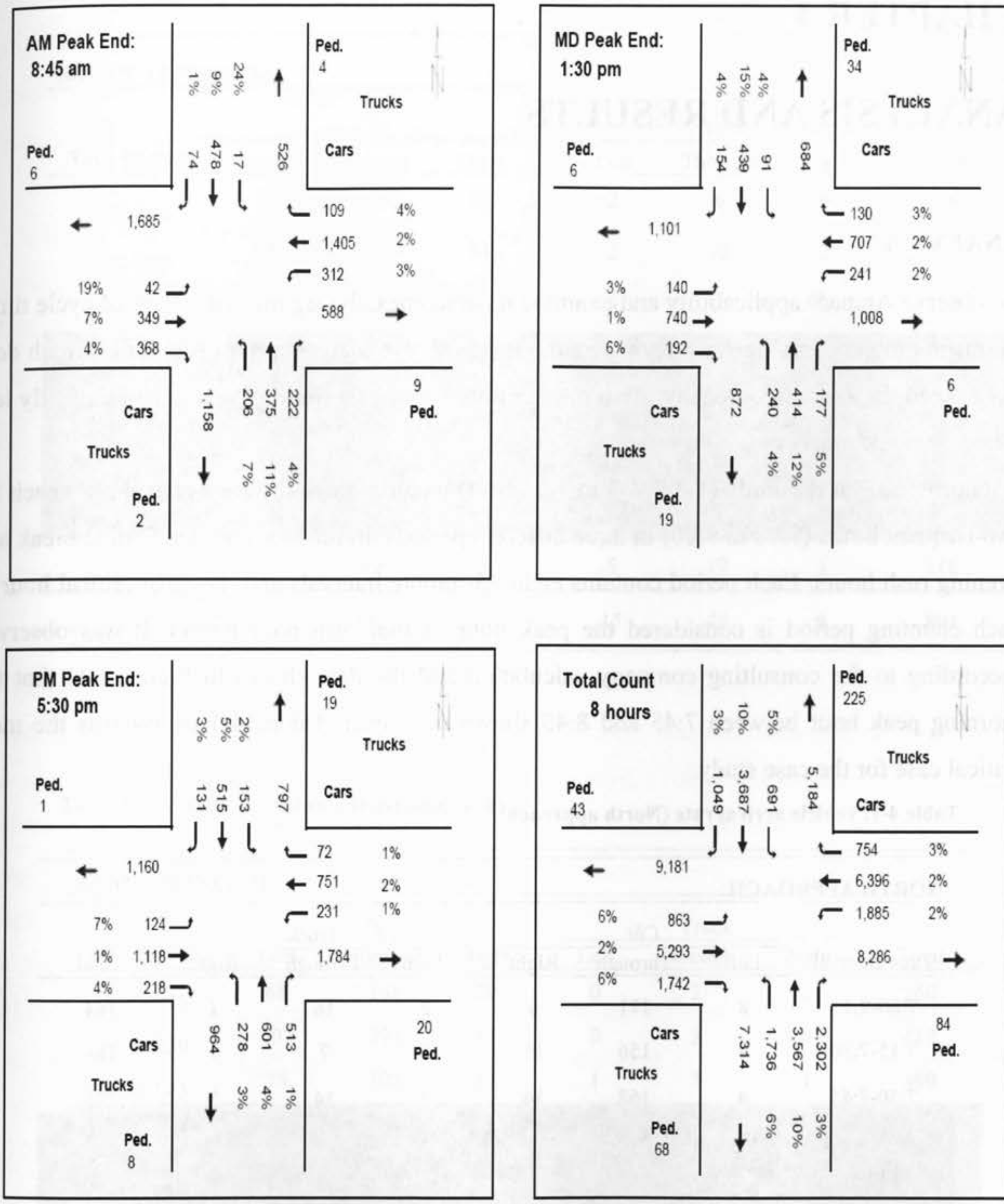


Figure 3-1: vehicle arrival at the case study intersection in different peak hours and the total numbers within 8 hour

CHAPTER 4

ANALYSIS AND RESULTS

ANALYSIS

To observe Arena® applicability and examine its outcomes, the significant effects of cycle times in traffic problem solving were reviewed and it was shown that a shorter cycle time, which does not exceed the saturated capacity of an intersection, creates shorter delays and consequently less idling time and emissions.

Data provided for the study (Table 4-1 to Table 4-4) includes three different arrival rates each for two frequent hours (7:00 to 9:00) in three different periods: in the morning, lunch time break and evening rush hours. Each period contains eight 15-minute intervals and the most critical hour of each counting period is considered the peak hour of that rush hour period. It was observed (according to the consulting company calculation and the data shown in Figure 3-1) that the morning peak hour between 7:45 and 8:45 shown in gray in Tables 4-1 to 4-4 was the most critical case for the case study.

Table 4-1: vehicle arrival rate (North approach)

NORTH APPROACH:							
	Car			Truck			
Time Interval	Left	Through	Right	Left	Through	Right	Total
7:00-7:15	8	111	6	2	16	1	144
7:15-7:30	4	156	16	0	7	1	184
7:30-7:45	4	168	14	3	14	3	206
7:45-8:00	3	116	15	1	9	0	144
8:00-8:15	1	125	28	1	11	0	166
8:15-8:30	3	108	17	1	9	1	139
8:30-8:45	6	87	13	1	13	0	120
8:45-9:00	5	125	18	0	14	3	165
∑Peak hours	13	436	73	4	42	1	569

Table 4-2: vehicle arrival rate (South approach)

SOUTH APPROACH:

	Car			Truck			
Time Interval	Left	Through	Right	Left	Through	Right	Total
7:00-7:15	31	67	30	2	6	0	136
7:15-7:30	35	85	34	2	12	2	170
7:30-7:45	46	82	30	3	14	2	177
7:45-8:00	39	88	75	3	11	2	218
8:00-8:15	56	85	50	2	15	3	211
8:15-8:30	43	78	39	3	8	2	173
8:30-8:45	53	82	50	7	8	1	201
8:45-9:00	45	79	69	5	19	1	218
∑Peak hours	191	333	214	15	42	8	803

Table 4-3: vehicle arrival rate (East approach)

EAST APPROACH:

	Car			Truck			
Time Interval	Left	Through	Right	Left	Through	Right	Total
7:00-7:15	68	168	20	0	2	2	260
7:15-7:30	107	291	11	0	3	0	412
7:30-7:45	75	302	13	1	7	1	399
7:45-8:00	86	338	13	1	2	0	440
8:00-8:15	70	410	46	1	10	2	539
8:15-8:30	70	365	12	3	10	1	461
8:30-8:45	78	262	34	3	8	1	386
8:45-9:00	69	249	27	1	7	1	354
∑Peak hours	304	1375	105	8	30	4	1826

Table 4-4: **vehicle arrival rate (West approach)**

WEST APPROACH:

Time Interval	Car			Truck			Total
	Left	Through	Right	Left	Through	Right	
7:00-7:15	2	48	16	1	2	2	71
7:15-7:30	2	40	43	0	4	1	90
7:30-7:45	6	47	54	2	5	3	117
7:45-8:00	6	66	61	0	8	1	142
8:00-8:15	11	95	92	4	6	3	211
8:15-8:30	6	74	75	2	11	5	173
8:30-8:45	11	88	124	2	1	7	233
8:45-9:00	14	96	60	2	8	4	184
∑Peak hours	34	323	352	8	26	16	759

Number of left-turn vehicles has an impact on the characteristics of an intersection and can affect the cycle time, saturation rate and smooth movement of cars in an intersection. If the number of left-turn cars is greater than 200, a left-protected lane is recommended (Sadoun, 2003). As it is shown in car arrival rates, the number of left-turn cars on East approach exceeds 200 and a left-protected lane seems to be a reasonable solution to decrease the waiting time and its consequent delay. Therefore, to determine the different aspects and characteristics of two-phase and three-phase plans for the traffic problem solution in urban area, both of these plans are studied and their results are shown. To calculate the critical lane volume on each compatible direction (discussed in literature review), first, the through car equivalents for each direction should be found. The factors shown in Table 2-1 are used to calculate through car equivalents for all directions. Number of pedestrians in this case on all approaches is low (between zero and 9), so the pedestrian factors shown in Table 2-1 are not considered.

Inter-arrival times are one of the characteristic of arrival process when the number of arrival is infinite. Car arrivals in an intersection are not scheduled and have random nature. If probability distribution of arrival times for each arrival rate is specified, the inter-arrival times and their distribution can be specified too. The Poisson distribution is the most important and practical model for random arrival cases. If an arrival process with Poisson distribution with mean λ, which is the number of customers per time unit or number of cars per time unit, the inter-arrival rate has an exponential distribution with a mean$1/\lambda$ time unit (Banks et al., 1995).

Table 4-5 and Table 4-6 in the next page depict car and truck arrivals, total through vehicle equivalent, and inter-arrival distributions in peak hour for the two-phase plan. Table 4-7 and Table 4-8 depict car and truck arrivals, total through vehicle equivalent, and inter-arrival distributions in peak hour for the three-phase plan.

Table 4-5: car and truck arrival and through vehicle equivalent for two-phase plan

Time	Approach	Movement	No. of cars per lane	No. of trucks per lane	Total volume (vph)	Equivalent factor (E_{LT})	Car equivalent	Truck equivalent	Car shared volume (TCU)
		Left	304	8	312	5.17	1572	41	1572
Peak hour end:8:45	East (Arm 1)	Through	1375	30	1405	1	1375	30	751
		Right	105	4	109	1.21	127	5	751
		Left	34	8	42	15	510	120	510
Peak hour end:8:45	West (Arm 3)	Through	323	26	349	1	323	26	374
		Right	352	16	368	1.21	426	19	374
		Left	13	4	17	4	52	16	52
Peak hour end:8:45	North (Arm 2)	Through	436	42	478	1	436	42	262
		Right	73	1	74	1.21	88	1	262
		Left	191	15	206	3.64	695	55	695
Peak hour end:8:45	South (Arm 4)	Through	333	42	375	1	333	42	296
		Right	214	8	222	1.21	259	10	296

Table 4-6: car and truck arrival, total through vehicle equivalent and inter-arrival distribution for two-phase plan

Time	Approach	Movement	Truck shared volume (TCU)	Total through car volume per arm TCU /arm	Total through truck volume per arm TCU /arm	Car arrival rate (Poisson mean = truck equivalent /60)	Car Inter-arrival distribution (Exponential mean = 1/poisson mean)	Truck arrival rate (Poisson mean = truck equivalent /3600)	Truck Inter-arrival distribution (Exponential mean = 1/poisson mean)
		Left	41						
Peak hour end:8:45	East (Arm 1)	Through	17	3074	76	51.23	0.02	1.27	0.79
		Right	17						
		Left	120						
Peak hour end:8:45	West (Arm 3)	Through	23	1259	165	20.98	0.05	2.76	0.36
		Right	23						
		Left	16						
Peak hour end:8:45	North (Arm 2)	Through	22	576	59	9.61	0.1	0.99	1.01
		Right	22						
		Left	55						
Peak hour end:8:45	South (Arm 4)	Through	26	1287	106	21.45	0.05	1.77	0.56
		Right	26						

Table 4-7: car and truck arrival and through vehicle equivalent for three-phase plan

Time	Approach	Movement	No. of cars per lane	No. of trucks per lane	Total volume (vph)	Equivalent factor	Car equivalent	Truck equivalent	Car shared volume (TCU)
		Left	304	8	312	1.05	319	8	319
		Through	1375	30	1405	1	1375	30	751
Peak hour end:8:45	East (Arm 1)	Right	105	4	109	1.21	127	5	751
		Left	34	8	42	1.05	36	8	36
		Through	323	26	349	1	323	26	374
Peak hour end:8:45	West (Arm 3)	Right	352	16	368	1.21	426	19	374
		Left	13	4	17	4	52	16	52
		Through	436	42	478	1	436	42	262
Peak hour end:8:45	North (Arm 2)	Right	73	1	74	1.21	88	1	262
		Left	191	15	206	3.64	695	55	695
		Through	333	42	375	1	333	42	296
Peak hour end:8:45	South (Arm 4)	Right	214	8	222	1.21	259	10	296

Table 4-8: car and truck arrival, total through vehicle equivalent and inter-arrival distribution for three-phase plan

Time	Approach	Movement	Truck shared volume (TCU)	Total through car volume per arm TCU /arm	Total through truck volume per arm TCU /arm	Car arrival rate (Poisson mean = truck equivalent /60)	Car Inter-arrival distribution (Exponential mean = 1/poisson mean)	Truck arrival rate (Poisson mean = truck equivalent /3600)	Truck Inter-arrival distribution (Exponential mean = 1/poisson mean)
		Left	8	319	8	5.32	0.19	0.14	7.14
		Through	17	1502	35	25.03	0.04	0.58	1.72
Peak hour end:8:45	East (Arm 1)	Right	17						
		Left	8	36	8	0.6	1.68	0.14	7.14
		Through	23	749	45	12.48	0.08	0.76	1.32
Peak hour end:8:45	West (Arm 3)	Right	23						
		Left	16						
		Through	22	576	59	9.61	0.1	0.99	1.01
Peak hour end:8:45	North (Arm 2)	Right	22						
		Left	55						
		Through	26	1287	106	21.45	0.05	1.77	0.56
Peak hour end:8:45	South (Arm 4)	Right	26						

CYCLE TIME AND PHASE DETERMINATION

To observe the effects of different cycle times on queue length and delay, three different cycle times - 90, 120, and 140 seconds have been examined. As discussed before, an optimal cycle time and allocation of green, yellow and red times have a significant effect on signal design and can provide a smooth movement that the capacity of the intersection is not reduced. The cycle times are chosen arbitrarily and can be different, but they should not be too long or too short, otherwise they increase the capacity to a level that will increase the delay or they reduce the capacity which in turn will increase the delay.

The number of left-turn movements specifies the number of phases required. If number of left-turn vehicles is between 200/hr and 250/hr, a protected left-turn is required and combined with appropriate cycle time selection will provide traffic flow that turning movements do not reduced the capacity of the intersection. Because on East approach there are 312/hr vehicles that turn left, a left-protected lane has to be considered but for the North and South approaches 206 vehicles turn left, so there is no need for any protected left lane. To examine the effects of different strategies, two-phase and three-phase plans are modeled.

By using analytical methods, cycle times for two strategies are calculated:

For the two-phase plan, there is no need to provide any left turn protection (Sadoun, 2003).

To find a cycle time that can be an estimation for the first run in the simulation (within the accepted range), the formulas mentioned in the literature review (Sadoun, 2003) were used:

$S = 3600/h$, h=2.11 to 5.66 veh/s

S=3600/2.11=1707 for this case the maximum number of left turn vehicle is less than 250 so the plan includes two phases $N = 2$

$V_c = 4543$ taken from Table 4-5

$t_1 = 3$ s lost time for each phase

$C = [(N \cdot t_1)]/1 - V_c / [PHF \cdot v/c \cdot (3600/h)]$

PHF =0.92 $v/c = 0.90$ so the cycle length is $C = -3$

For the three-phase plan, there are left-protected lanes on East and West directions:

$N = 3$ and with the assumption that all other factors are similar,, the cycle time $C = -10$

These cycle times for two-phase and three-phase could be considered as the starting points and first estimations of the simulation models but the negative sign implies that because the number of vehicles arriving onto intersection is too large and especially much larger than the saturated flow, the formula will not be useful and the calculated cycle times are not reasonable and practical. It seems that direct simulation by Arena® or any other computer programs is the best way to design the signal-setting of the intersection in this case.

EMISSION VALUES

To develop emission control strategies and to determine the applicability of the control programs, emissions have to be quantified in a specified framework. As the fundamental requirement in any effort to control various forms of pollution these values have an important role to identify an activity and its corresponding emission type and quantity which has been released to the atmosphere. These values are usually expressed as the weight of pollutant per unit weight, volume, or distance. In most cases, these values are simply averages of all available data and are generally assumed to be representative of long-term averages. Emission values depend on characteristics of travel activities such as vehicle type, age, vehicle speed, trip length distribution, operating mode, and ambient temperature may vary (Environmental Protection Agency, 2005).

The United States Environmental Agency (USEPA) has done extensive studies of emission modeling and various programs are available to calculate the emissions made by highway car movements. According to USEPA (Procedures for Emission Inventory Preparation, Volume IV: Mobile Sources) sources of pollutant emission are divided into two major categories of point sources and area sources. The point source category is described as large manufacturing or production plants which are stationary sources of pollutants released into the atmosphere. Stacks, vent, and other individual emission points and discrete fugitive emission source are examples of point sources.

The area source category consists in individual sources that are smaller than point sources and emit a small amount of pollution, including those that effect wider areas (because they can be widespread). Auto-body painting, dry-cleaners, residential wood heating, and consumer solvent use are the examples of area sources of emission (Introduction to stationary point source, USEPA, 2001). Mobile sources of emission are part of area source category and because mobile sources of emission are responsible for a high portion of total emissions of volatile organic compounds (VOC), Carbon monoxide (CO) and nitrogen oxides, their measurement and calculation are crucial. Mobile sources of emission include highway vehicles, aircrafts, locomotives and non-road mobile sources such as recreational marine equipment and commercial marine vessels.

USEPA has provided detailed procedures to describe how to calculate exhaust emissions and emissions from the fuel carried in vehicles (evaporative VOC emissions) for mobile source categories.

HIGHWAY VEHICLES, THE INDIVIDUAL MOBILE SOURCE CATEGORIES

This category includes all registered vehicles that use the public roadways. Automobiles, trucks and buses are vehicles that are included in this category, although automobiles are the most significant producers of highway related emissions.

To calculate the highway vehicles emission, there are several characteristics of each vehicle that have to be specified. Model year, the age distribution of vehicles within the class, annual mileage by vehicle age and average speed are those characteristics which affect any methods of emission calculation.

The term vehicle miles traveled (VMT), which indicates vehicle activity, is a standard practice in the calculation of highway vehicle emissions. It can be expressed as the emission factor in units of grams per mile of travel. Vehicles can produce emissions even when they do not move and are in stationary situations - this can happen when cars go into a drive-through line up or are waiting in queues at intersections or any kind of traffic jams. The term “equivalent per mile emissions” , which can be used for non-moving vehicles, is an estimate of miles traveled by vehicles of a particular age.

To measure emission rates, two fundamental and important processes have been used: the baseline emission rate and the deterioration that is related to the vehicle age that occurs over each 10,000 mile interval. Temperature, humidity, vehicle load, and the distribution of starting conditions are characteristics that need to be considered when measuring the emissions. Because all vehicles are different, a series of correction factors which have been developed will cover the difference. Therefore, emission factors count for all vehicles that have been categorized in EPA emission inventory system.

The total highway vehicle population can be characterized by eight individual vehicle type categories (Table 4-5).

Table 4-9: definition of vehicle types

Vehicle Type	Definition
LDGV	Light-duty gasoline-fueled vehicles, up to 6000 lb Gross Vehicle Weight GVW (gasoline-fueled passenger cars)
LDGT1 & LDGT2	Light-duty gasoline-fueled trucks, up to 8500 lb GVW (includes pick-up trucks, minivans, passenger vans, sport-utility vehicles, etc.)
HDGV	Heavy-duty gasoline-fueled vehicles, 8501+ lb GVW (gas heavy-duty trucks)
LDDV	Light-duty diesel vehicles, up to 6000 lb GVW (passenger cars with diesel engines)
LDDT	Light-duty diesel trucks, up to 8500 lb GVW (light trucks with diesel engines)
HDDV	Heavy-duty diesel vehicles, 8501+ lb GVW (diesel heavy-duty trucks)
MC	Motorcycles (only those certified for highway use; all gasoline-fueled)

Procedures for Emission Inventory Preparation, Volume IV: Mobile Sources, 1992, USEPA

EMISSION FACTOR FOR IDLING VEHICLE EMISSIONS

To calculate the emissions produced by delays which occur at intersections, waiting time and the number of cars waiting should be determined. This situation is also one in which idling time and the estimates of emission from idling cars should be considered. Idling emissions such as driving emissions are affected by various parameters. Emission factors differ from warm to cold weather for VOC, CO and NOx. When a detailed specific emission estimate provided for local conditions are not needed, the emission factors in below tables are adequate and can be used as first-order approximations of emissions under idle conditions. These idle emission factors are from the

MOBILE-5b highway vehicle emission factor model. "These emission factors are national averages for all vehicles in the in-use fleet as of January 1, 1998 (winter) or July 1, 1998 (summer)" (Emission fact, USEPA, 1998). Table 4-10 depicts the emission factors for different categories of vehicles that is provided by USEPA.

Table 4-10: emission factor according to USEPA idling vehicle emissions

Winter Conditions (30°F, 13.0 psi RVP gasoline)

Pollutant	Unit	LDGV	LDGT	HDGV	LDDV	LDDT	HDDV	MC
VOC	g/hr	21.1	30.7	44.6	3.63	4.79	12.6	20.1
CO	g/hr	371	487	682	10.1	11.5	94.6	388
NOx	g/hr	6.16	7.47	11.8	6.66	6.89	56.7	2.51

Summer Conditions (75°F, 9.0 psi RVP Gasoline)

Pollutant	Unit	LDGV	LDGT	HDGV	LDDV	LDDT	HDDV	MC
VOC	g/hr	16.1	24.1	35.8	3.53	4.63	12.5	19.4
CO	g/hr	229	339	738	9.97	11.2	94.0	435
NOx	g/hr	4.72	5.71	10.2	6.50	6.67	55.0	1.69

Emission fact, USEPA, 1998

Because winter emission factors are larger than summer factors, the larger factors have been selected for simplification. Among eight different types of EPA vehicle categorizations, the light-duty gasoline vehicle (LDGV) and the heavy-duty diesel Truck (HDDT) are chosen to represent the vehicle that produced idling emissions in traffic jams. Table 4-11 depicts the idling emission produced by each of these two patterns taken from Table 4-10.

Table 4-11: emission factor for two types of vehicles in winter condition

Pollutant	Units	LDGV	HDDV
VOC	g/min	0.352	0.211
CO	g/min	6.19	1.58
NOx	g/min	0.103	0.945

The inter-arrival rates for cars and trucks in each approach taken from Tables 4-6 and 4-8 are the inputs for Arena® simulation.

Because the critical period is an hour in the early morning, the replication time in Arena® modeling is an hour too. In this period of time, the system can reach a steady state situation (time independent) and a replication time of several hours is not necessary. This fact can be checked by increasing the replication time to 24 or more hours and observe that the result is proportional with the replication time and by increasing this time the output will change respectively. Arena® is run for three cycle times and two strategies and the results of simulation are discussed in the next section.

RESULT

After running the program for three cycle times: 90, 120, and 140 seconds and for two different phased plans (two and three-phase plans), queue time and consequently the emission produced in each case are calculated and shown in Table 4-12 to Table 4-17. The first row shows the average waiting time for each type of vehicles in different arms (appendix B), the second row represents the number of vehicles waiting in each arm in an hour (appendix B), the other rows (3 to 5) represent the emissions produced in an hour (gram per hour), and the last column shows the total amounts of emission for each pollutant which are converted to kilogram per hour.

Table 4-12: queue and related emissions for 2-phase model (cycle time = 90 second)

	Car arm 1	Truck arm 1	Car arm 2	Truck arm 2	Car arm 3	Truck arm 3	Car arm 4	Truck arm 4	
Time waiting (minute)	25	1	17	30	19	2	23	30	
Number of vehicle in Queue	1297	1	163	29	366	6	473	50	Total (kg/hr)
VOC Emission (gr/min)	11578	0	986	184	2440	3	3841	317	19
CO Emission (gr/min)	203601	1	17344	1375	42909	19	67546	2370	335
NOx Emission (gr/min)	3388	1	289	822	714	11	1124	1418	8

Table 4-13: queue and related emissions for 2-phase model (cycle time = 120 second)

	Car arm 1	Truck arm 1	Car arm 2	Truck arm 2	Car arm 3	Truck arm 3	Car arm 4	Truck arm 4	
Time waiting (minute)	25	0	16	1	18	13	23	30	
Number of vehicle in Queue	1266	1	153	28	336	39	461	50	Total (kg/hr)
VOC Emission (gr/min)	11252	0	882	8	2102	106	3703	317	18
CO Emission (gr/min)	197873	0	15513	61	36959	794	65119	2370	319
NOx Emission (gr/min)	3293	0	258	37	615	475	1084	1418	7

Table 4-14: queue and related emissions for 2-phase model (cycle time = 140 second)

	Car arm 1	Truck arm 1	Car arm 2	Truck arm 2	Car arm 3	Truck arm 3	Car arm 4	Truck arm 4	
Time waiting (minute)	25	1	17	30	18	7	24	30	
Number of vehicle in Queue	1280	2	161	29	350	23	470	50	Total (kg/hr)
VOC Emission (gr/min)	11232	1	982	184	2225	36	3894	317	19
CO Emission (gr/min)	197525	4	17271	1375	39127	267	68485	2370	326
NOx Emission (gr/min)	3287	2	287	822	651	160	1140	1418	8

Table 4-15: queue and related emissions for 3-phase model (cycle time = 90 second)

	Car arm 1, left trun	Truck arm 1, left trun	Car arm 3, left trun	Truck arm 3, left trun	Car arm 1	Truck arm 1	Car arm 2	Truck arm 2	Car arm 3	Truck arm 3	Car arm 4	Truck arm 4	
Time waiting (minute)	21	0	1	2	23	1	17	1	16	1	23	2	
Number of vehicle in Queue	104	6	1	0	591	0	178	1	203	1	457	3	Total (kg/hr)
VOC Emission (gr/min)	757	0	0	0	4731	0	1093	0	1140	0	3664	1	11
CO Emission (gr/min)	13313	1	4	1	83190	1	19227	2	20042	1	64441	9	200
NOx Emission (gr/min)	222	0	0	1	1384	0	320	1	333	0	1072	5	3

Table 4-16: queue and related emissions for 3-phase model (cycle time = 120 second)

	Car arm 1, left trun	Truck arm 1, left trun	Car arm 3, left trun	Truck arm 3, left trun	Car arm 1	Truck arm 1	Car arm 2	Truck arm 2	Car arm 3	Truck arm 3	Car arm 4	Truck arm 4	
Time waiting (minute)	20	0	1	1	24	1	21	1	19	1	24	1	
Number of vehicle in Queue	99	6	1	0	635	1	212	1	247	1	491	2	Total (kg/hr)
VOC Emission (gr/min)	682	0	0	0	5423	0	1531	0	1678	0	4205	0	14
CO Emission (gr/min)	11988	1	3	1	95358	1	26928	2	29508	1	73946	3	238
NOx Emission (gr/min)	199	0	0	0	1587	1	448	1	491	1	1230	2	4

Table 4-17: queue and related emissions for 3-phase model (cycle time = 140 second)

	Car arm 1, left trun	Truck arm 1, left trun	Car arm 3, left trun	Truck arm 3, left trun	Car arm 1	Truck arm 1	Car arm 2	Truck arm 2	Car arm 3	Truck arm 3	Car arm 4	Truck arm 4	
Time waiting (minute)	19	0	1	1	25	1	22	1	21	1	25	1	
Number of vehicle in Queue	93	6	1	0	654	1	227	1	265	1	506	2	Total (kg/hr)
VOC Emission (gr/min)	608	0	0	0	5755	0	1767	0	1942	0	4520	0	15
CO Emission (gr/min)	10691	1	4	0	101207	1	31081	3	34152	2	79494	4	257
NOx Emission (gr/min)	178	0	0	0	1684	1	517	2	568	1	1323	2	4

Figures 4-1 and 4-2 in the next page show the relation between the cycle time and emission levels for three different pollutants in two and three-phase plans according to the total emission resulted from Table 4-12 to Table 4-17.

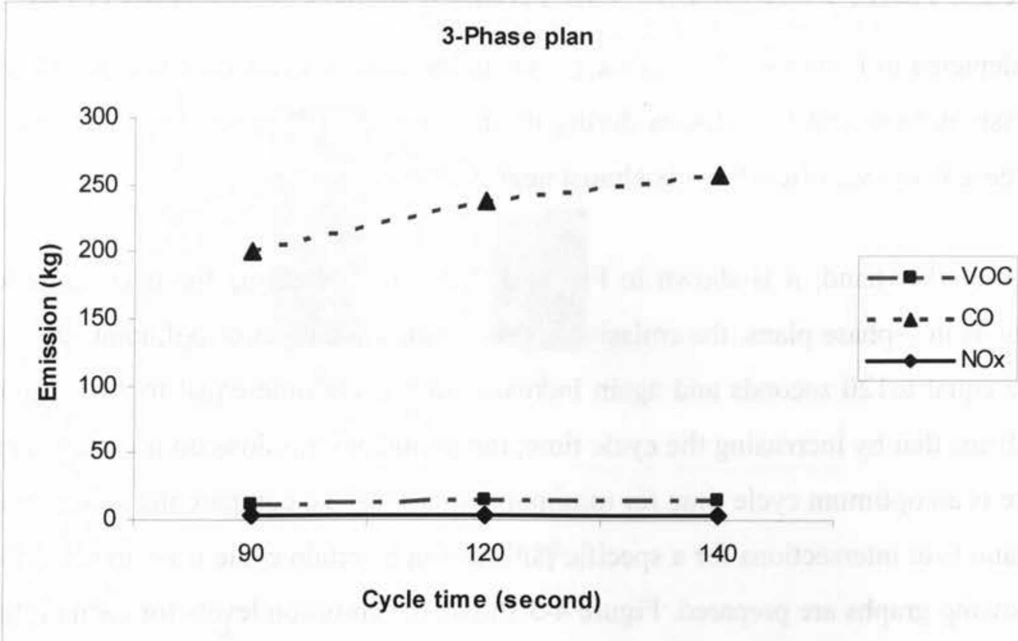


Figure 4-1: emission against cycle time length in a 3-phase plan

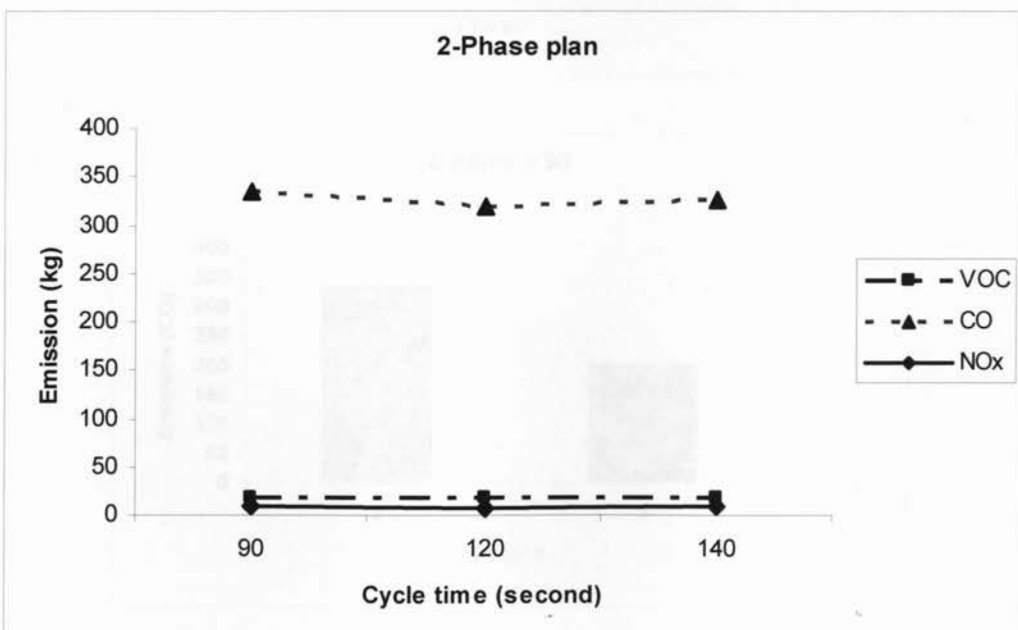


Figure 4-2: emission against cycle time length in a 2-phase plan

CYCLE TIME, TWO-PHASE, AND THREE-PHASE PLANS STRATEGIES

As depicted in Figure 4-1 in 3-phase plans, an increase in cycle time length will increase the CO emissions produced by vehicles during idling times, but the trend for VOC and NOx emission increase is not significant and is almost negligible.

On the other hand, it is shown in Figure 4-2 that by increasing the cycle time length from 90 seconds in 2-phase plans, the emission levels for all three types of pollutants decreases in a cycle time equal to120 seconds and again increases in a cycle time equal to 140 seconds. This trend confirms that by increasing the cycle time, the pollutant emissions do not necessary decrease and there is an optimum cycle time for minimum emissions. To compare the emission levels in idling situations at intersections for a specific pollutant in a certain cycle time and in different plans the following graphs are prepared. Figure 4-3 shows the emission levels for each pollutant in a cycle time equal to 90 seconds for two-phase and three-phase plans. Figure 4-4 shows the emission levels for each pollutant in a cycle time equal to 120 seconds for two-phase and three-phase plans. Figure 4-5 shows the emission levels for each pollutant in a cycle time equal to 140 seconds for two-phase and three-phase plans.

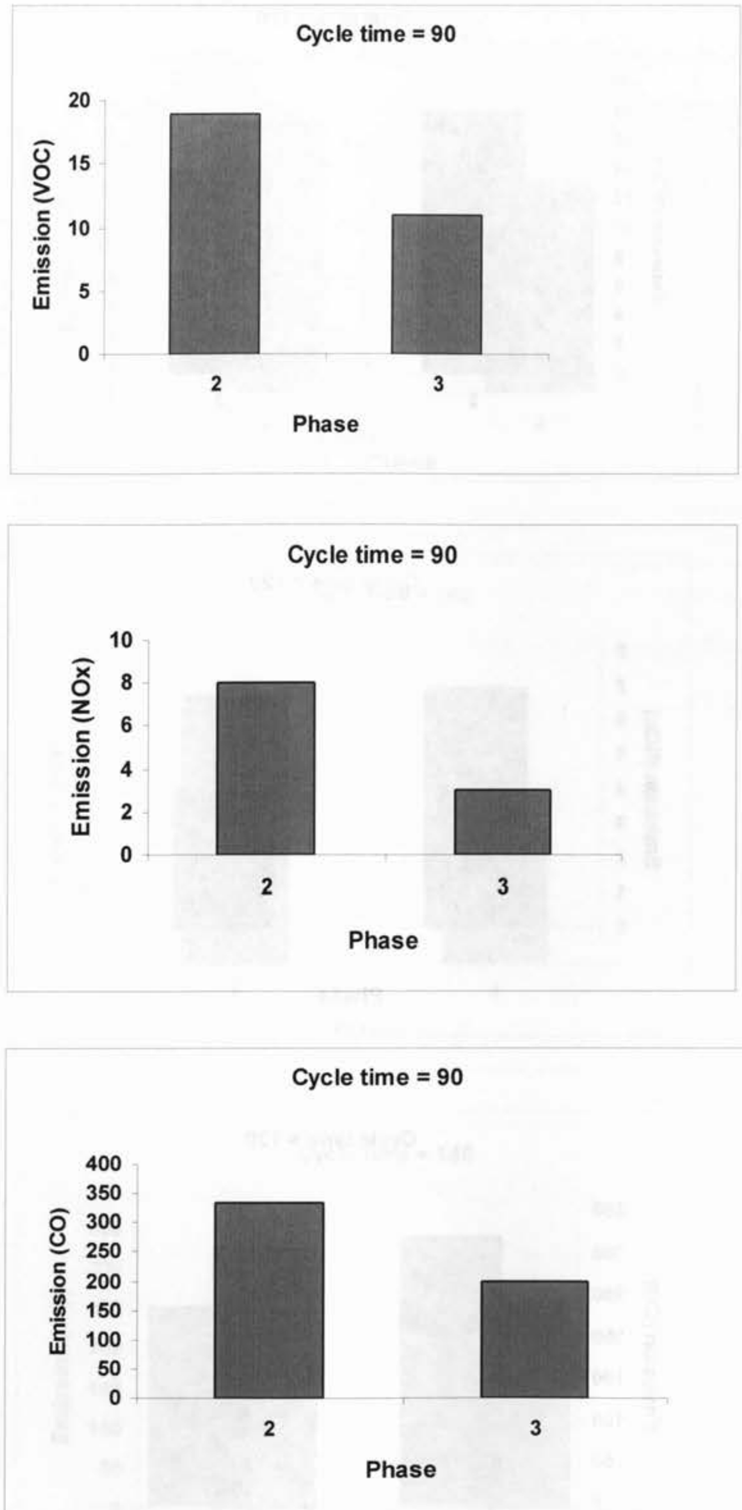


Figure 4-3: emission levels in different traffic plans, cycle time = 90 seconds.

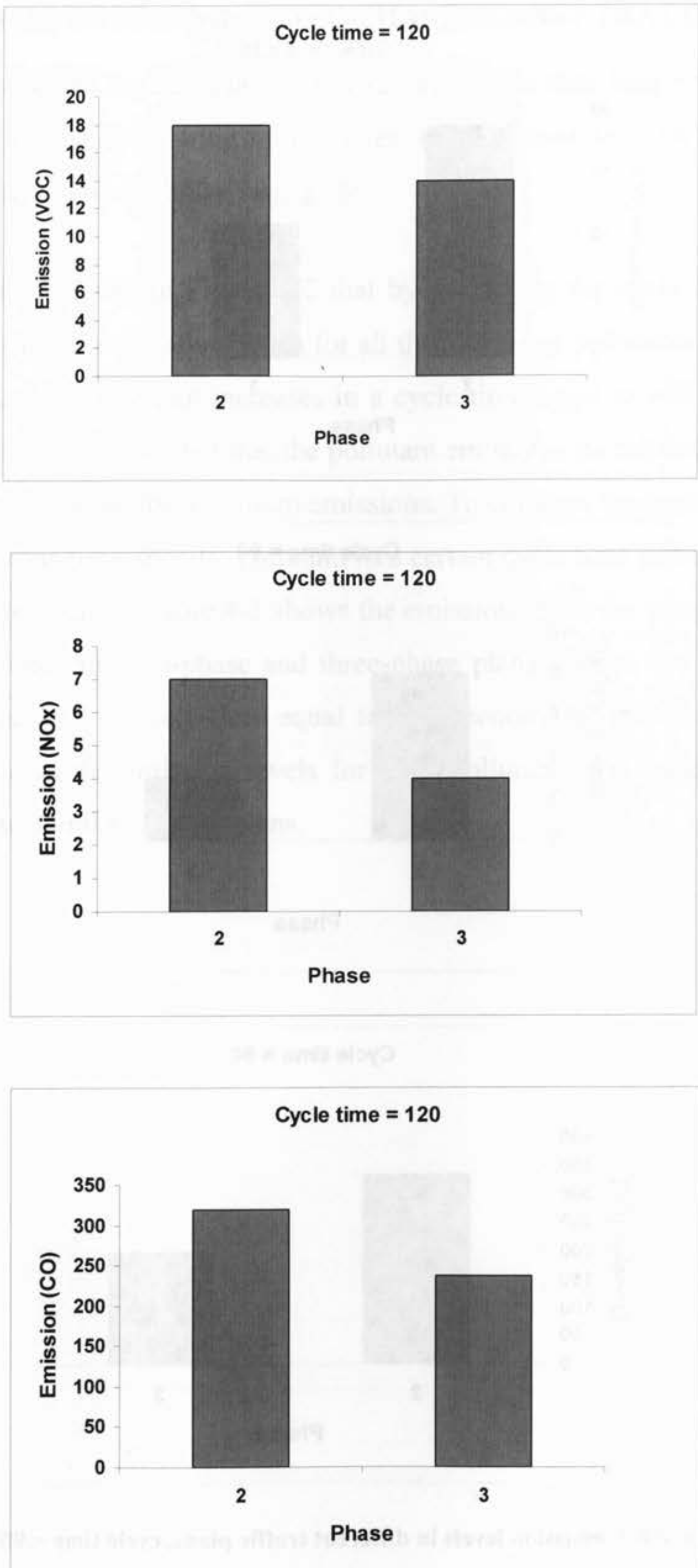


Figure 4-4: emission levels in different traffic plans, cycle time = 120 seconds

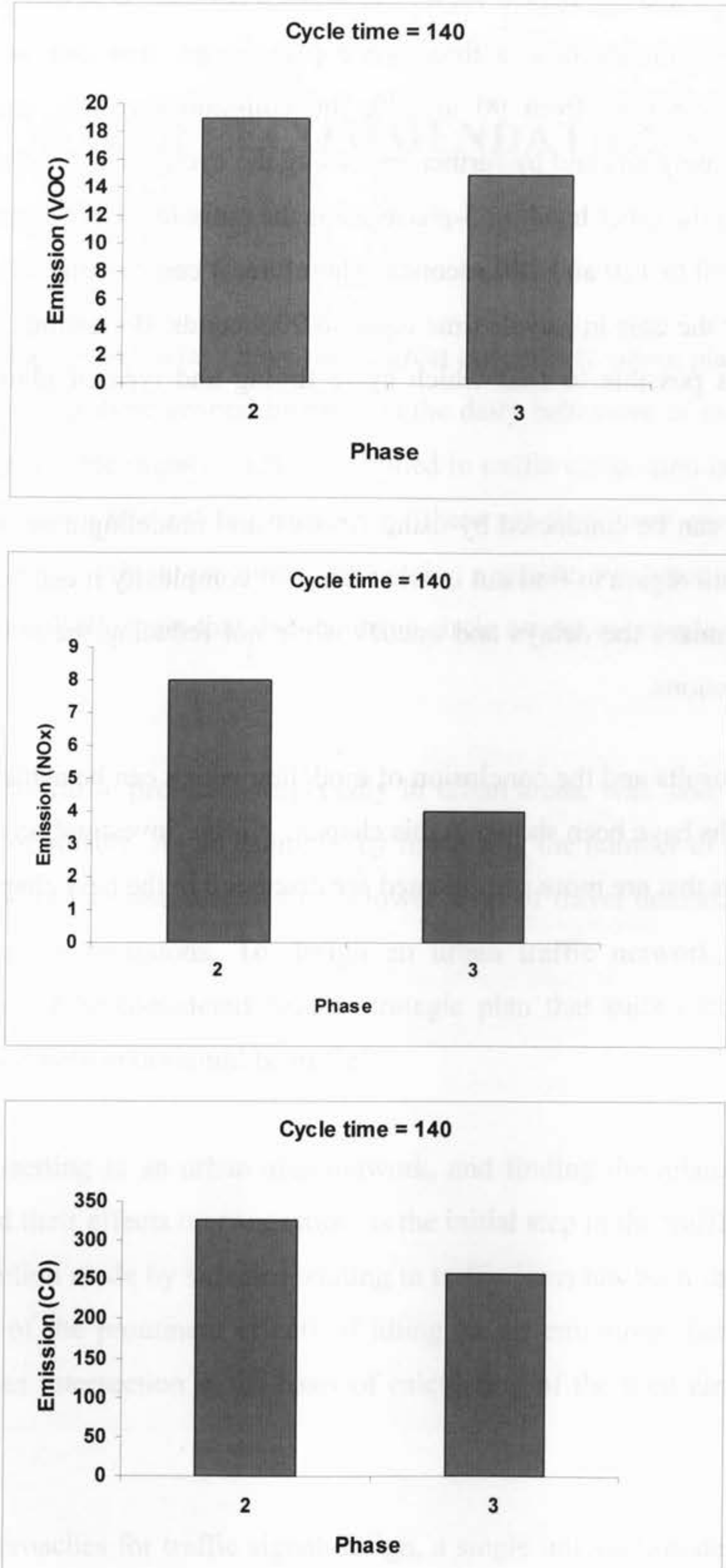


Figure 4-5: emission levels in different traffic plans, cycle time = 140 seconds.

As Figures 4-3, 4-4 and 4-5 depict, for all cycle times considered in this study (90, 120, and 140 seconds), the emission levels in a three-phase plan is less than that in a two-phase plan. By increasing the cycle time from 90 to 120, the emission level for each of three pollutants decreases in 2-phase plans and by further increasing the cycle time to 140 seconds, the emissions elevate again. On the other hand, in 3-phase plans the emission levels decrease by increasing the cycle time from 90 to 120 and 140 seconds. Therefore, it can be conclude that a 3-phase plan is more suitable for the case in a cycle time equal to 90 seconds. By testing the model for a range of cycle times, it is possible to find which cycle timing and type of plan is most efficient and suitable.

Further research can be conducted by using Arena® and modeling a network of traffic lights in an area or a certain region to find out in what range of complexity it can be used to design a good plan which minimizes the delays and queues while not reducing the intersection capacity, and causing less emissions.

The significant results and the conclusion of modeling which can be achieved by the comparison of resulting graphs have been shown in this chapter. Further investigation and direction for future research for cases that are more complicated are discussed in the next chapter.

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

CONCLUSION

Traffic congestion, queues and delays have raised great concerns in urban planning and decision-making. This problem has three general impacts on the daily behaviour of every population, and can cause further irreversible impacts. Time consumed in traffic congestion is as important as the space required for building new infrastructures, and there are significant environmental impacts and health-related effects, which are direct, and major negative outcomes of traffic problems. Further investigation will illustrate that this recurring circle causes economic problems which are hard to solve.

Decisions pertinent to traffic problems, especially in urban areas, will have significant impacts on different aspects of society. As an example, by increasing the number of roads in a network, the emission levels may increase, and having a lower level of travel demand in a network may not necessarily decrease emissions. To design an urban traffic network, all of the above-mentioned items should be considered, and a strategic plan that suits each specific case and provides efficient road networks should be made.

The study of signal-setting in an urban area network, and finding the related characteristics of each intersection and their effects on congestion, is the initial step in the traffic planning process. The analysis of pollution made by vehicles waiting in traffic jams has been the main objective of this study. Because of the prominent effects of idling on air emissions, the additional waiting time of vehicles in an intersection is the basis of calculating of the total air emission for these vehicles.

Among different approaches for traffic signal design, a single intersection design was chosen to be studied. This decision was made based on two reasons: the importance of a single intersection and its effect on the whole system, and Arena® applicability in traffic design.

A major intersection vehicle arrival rate was prepared and by using Arena® (a simulation software), the existing situation was simulated. Because of the significant effects of cycle time on traffic signal-setting, the model in this simulation was run by allocating different cycle times and assuming two and three-phase plans for an intersection. The delay and waiting time caused by congestion in peak periods at rush hour have been determined, and by using the United States Environmental Agency (USEPA) idling emission factors, the total emissions made by vehicles waiting in queues have been calculated. VOC, CO and NOx were the pollutants which have been considered in this study.

It can be concluded that by increasing the cycle time from 90 seconds to 120 and 140 seconds, CO emissions have the higher levels and are minimized in a 120 second cycle time in two-phase plans, whereas 90 second cycle times are more efficient for three-phase plans. Generally, a longer cycle time is not the best solution for signal-setting problems and usually a longer cycle time increases the total delay, while by shortening a cycle length (to a certain point that has to be found by modeling and various methods of optimization), the intersection capacity increases to its maximum level and consequently decreases vehicle delay. Because of the stochastic and dynamic nature of the vehicle arrival rate, each traffic case should be investigated individually and a specific setting for signals should be determined. Furthermore, it has been seen that in a cycle time equal to 120 seconds in two-phase plans, all three pollutants are reduced to minimum emission levels, but for cycle times equal to 90 seconds in three-phase plans, these pollutants are reduced to minimum levels of emission.

RECOMMENDATIONS

Further investigations and modeling of a network of intersections can be the subject of future studies by utilizing Arena® simulation modeling. Through comparisons between Arena® modeling outputs and other programs, the accuracy and precision of Arena® can be examined and it can be applied to more complicated traffic networks. Traffic accidents and road closures for maintenance reasons are cases that can be modeled and their results may be used in traffic control systems. In a more ideal situation, computers could be linked to reduce the process time and Arena® could be used to model a series of networks that interact with each other and provide an efficient traffic plan.

APPENDIX A

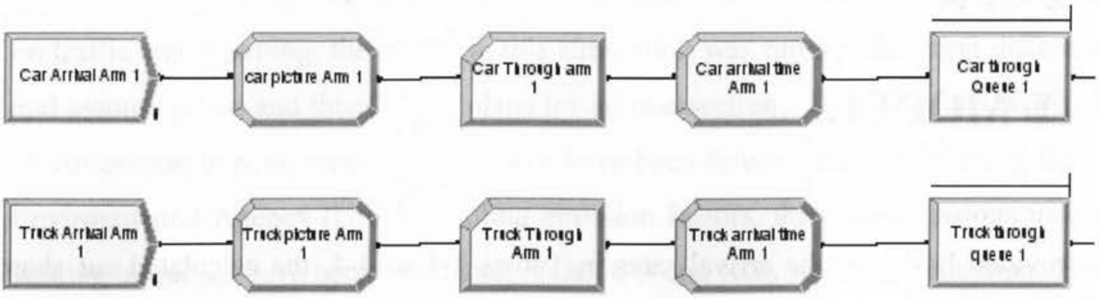
ARENA® MODEL

Input data provided based on the arrival rates in Tables 4-1 to 4-4, the calculated car shared volume after applying through car equivalent factors. Car and truck inter-arrival rates taken from Tables 4-6 and 4-8 are used as data input in Arena® simulation process. Two different strategies, a two-phase plan (without left protection) and a three-phase plan (with left protection) were considered and cycle times equal 90, 120, and 140 seconds were assumed for each strategy. Each cycle time was divided into different green, yellow, and red light times. The time allocation was based on the weighted average of total number of vehicles arrived onto intersection from two incompatible approaches within the specific peak hour (between 7:45 and 8:45). Table A-1 shows the general inputs for the case study. To clarify the program algorithm, in page 56 one arm (arm 1) is selected as the representative of all arms and the program algorithm is shown in the boxes below each line. The algorithm of the common loop that belongs to all arms also is shown in a box below the common loop. This procedure is the same for all other cases and strategies (different cycle times and two-phase or three-phase plans).

Table A-1: Inputs for simulation in Arena®

	Arena® Inputs
1	Cars and trucks Inter-arrival rates taken from Tables 4-6 and 4-8.
2	Identifying the phasing plan (2 or 3-phase) that can be found from left-turn movements (if more than 200 vehicles per hour turn left, a left lane protection is needed which means an extra phase is added to the plan).
3	Time allocation of different phases and their green, yellow, and red lights.

Cycle time =90 second in 2-phase plan

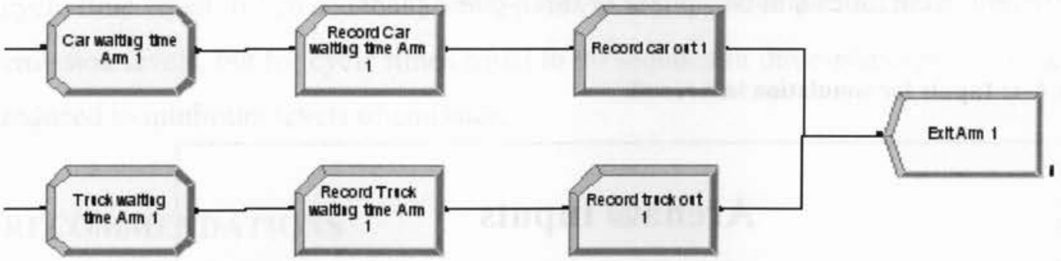


Cars and trucks inter-arrival rates are entered and the related distribution is assigned.

Cars and trucks pictures are assigned to each type of vehicle (this part is needed for animation when the program is running so the simulated cars and trucks can be seen.

Cars and trucks that arrive to an arm are counted separately and the total number of each vehicle is calculated at the end.

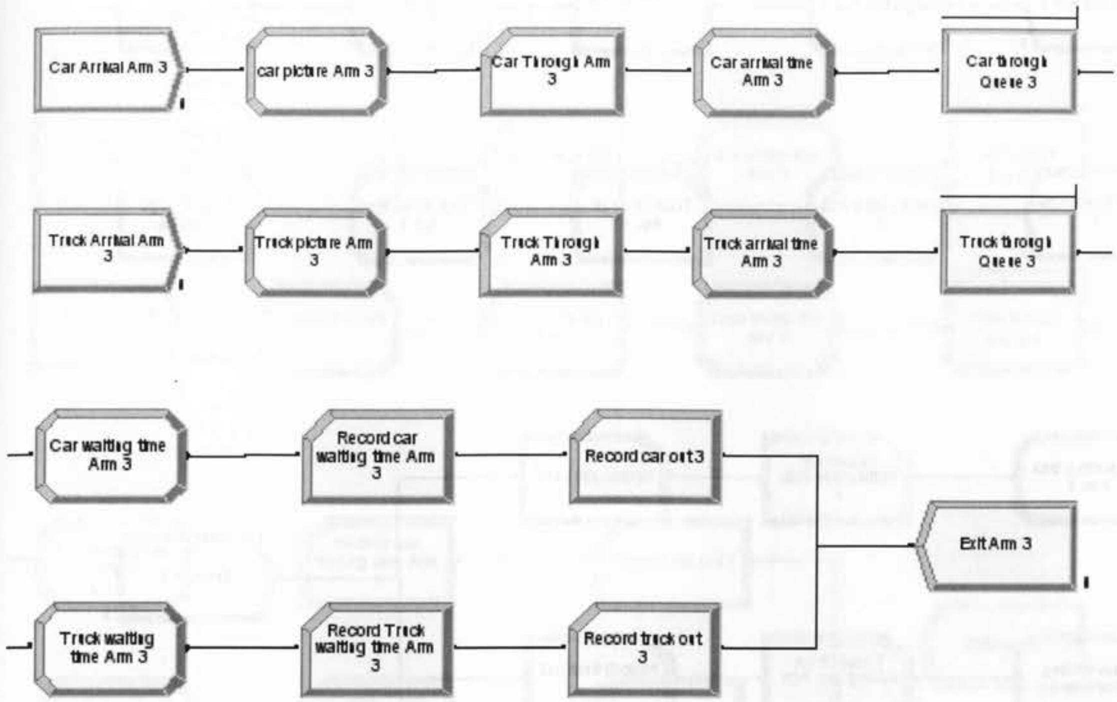
Cars and trucks are waiting in queues and the waiting times are calculated.

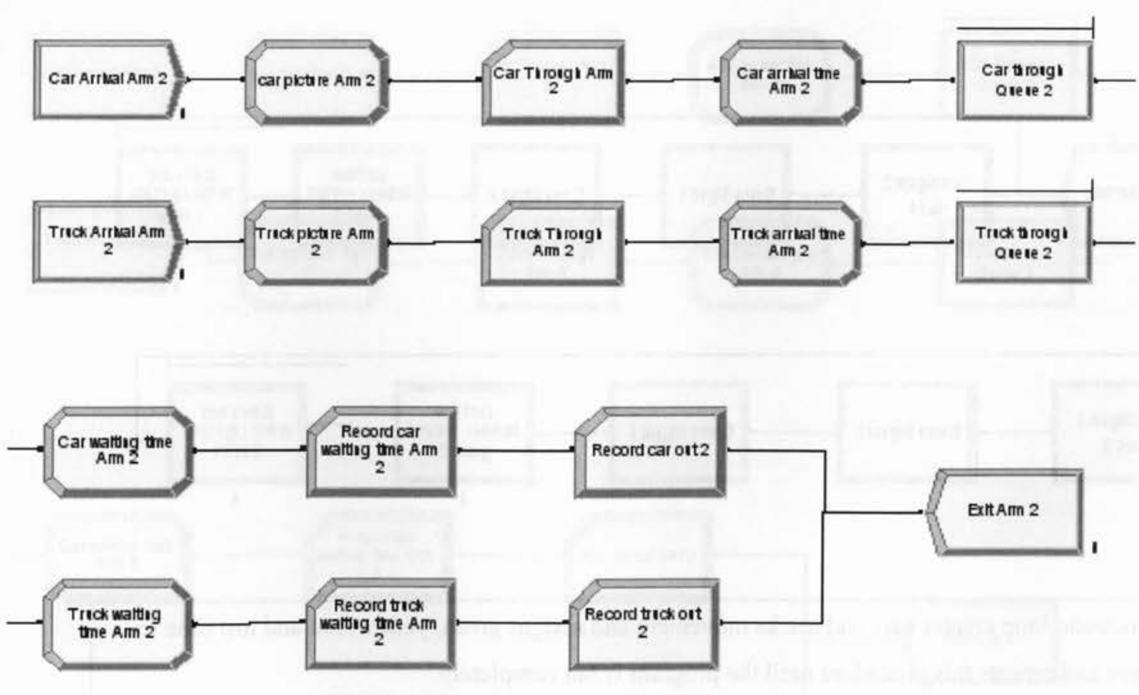


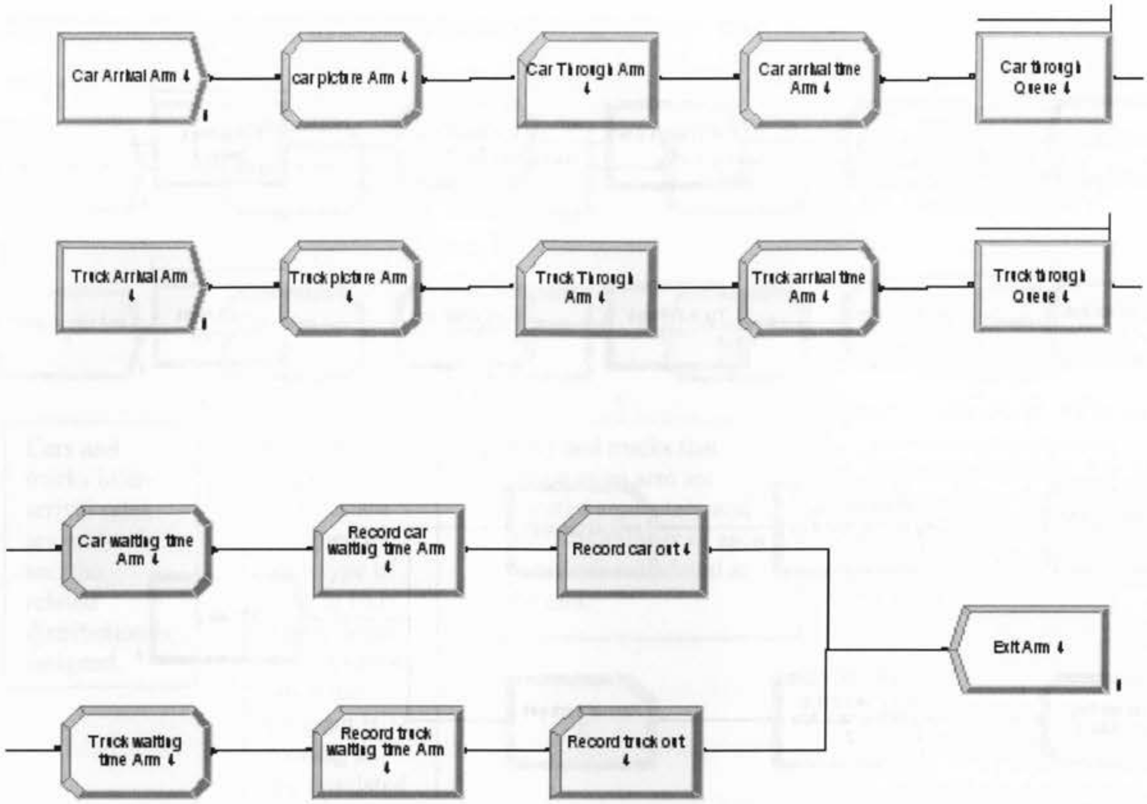
Cars and truck waiting times that are calculated in each cycle time by the common loop below are added and total waiting time for each vehicle is calculated.

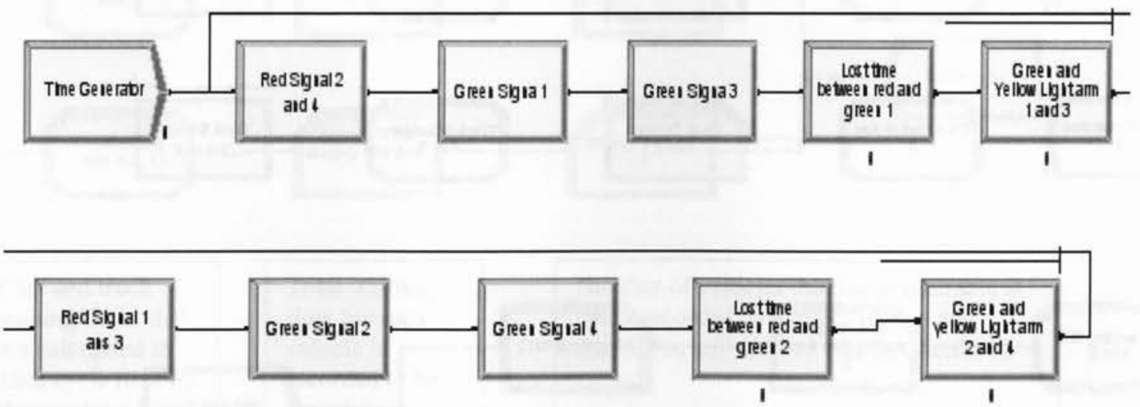
Total waiting time for each vehicle is recorded to be shown in outputs.

Number of vehicles that leave each arm is being recorded so number of cars and trucks that still waiting can be calculated.


Car Arrival Arm 3
car picture Arm 3
Car Through Arm 3
Car arrival time Arm 3
Car through Queue 3
Truck Arrival Arm 3
Truck picture Arm 3
Truck Through Arm 3
Truck arrival time Arm 3
Truck through Queue 3
Car waiting time Arm 3
Record car waiting time Arm 3
Record car out 3
Exit Arm 3
Truck waiting time Arm 3
Record Truck waiting time Arm 3
Record truck out 3

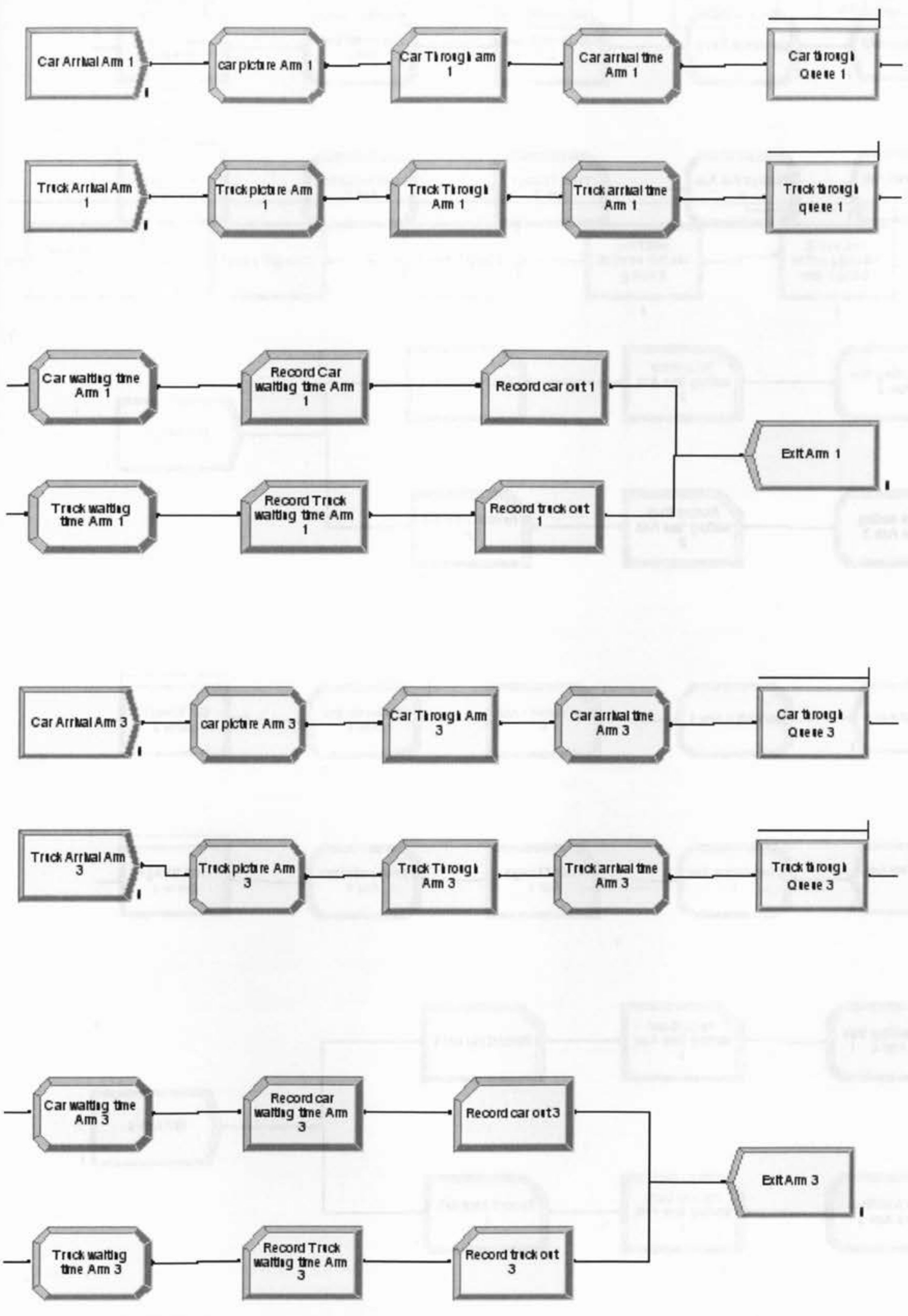

Car Arrival Arm 2
car picture Arm 2
Car Through Arm 2
Car arrival time Arm 2
Car through Queue 2
Truck Arrival Arm 2
Truck picture Arm 2
Truck Through Arm 2
Truck arrival time Arm 2
Truck through Queue 2
Car waiting time Arm 2
Record car waiting time Arm 2
Record car out 2
Exit Arm 2
Truck waiting time Arm 2
Record truck waiting time Arm 2
Record truck out 2

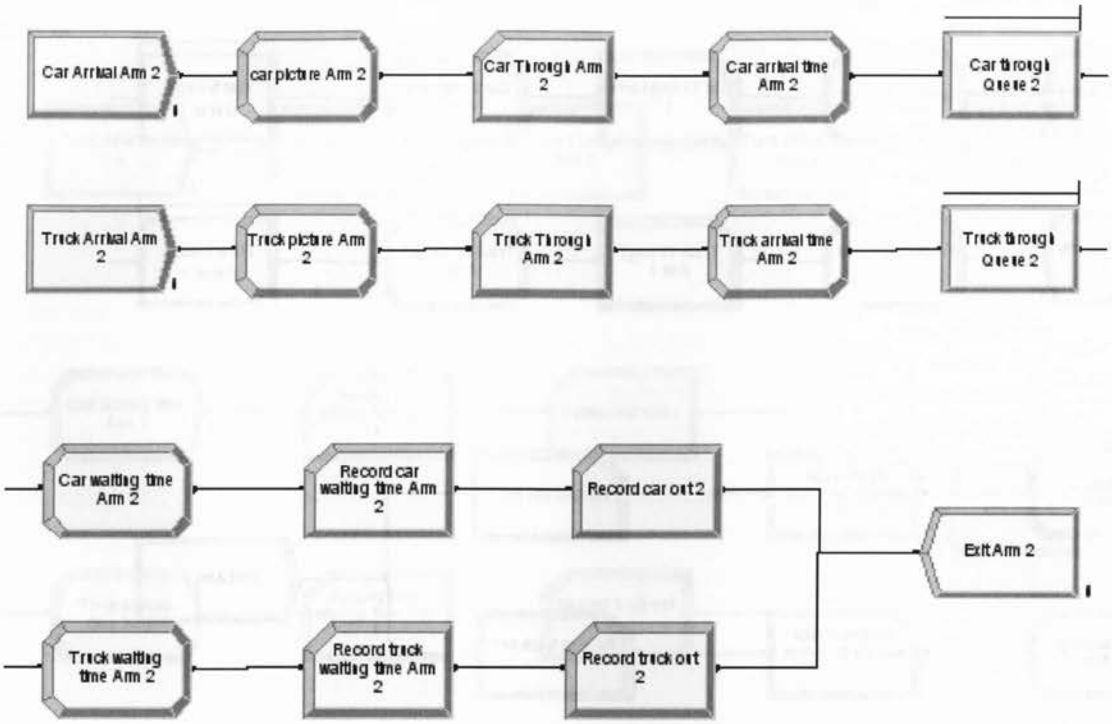


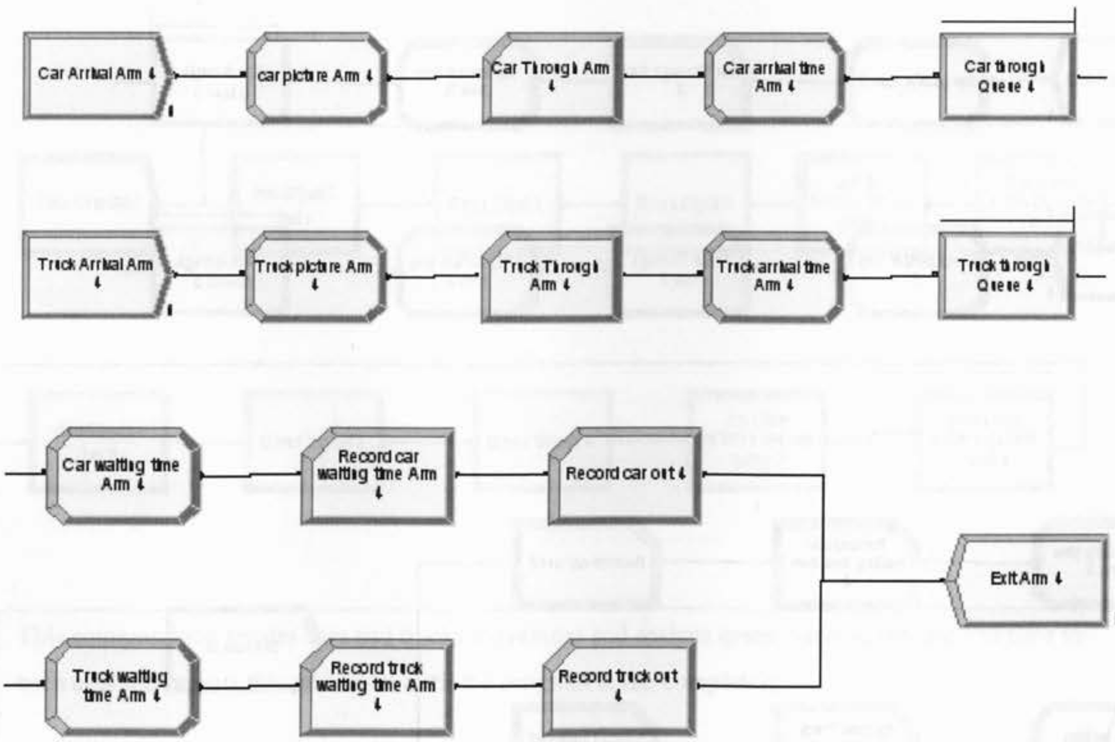


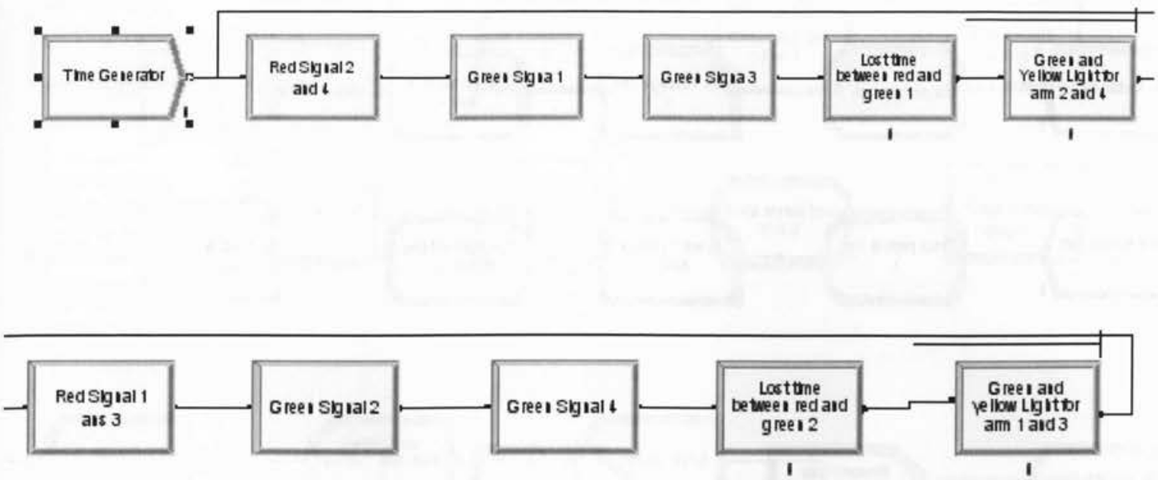
This common loop creates cars and trucks movement and assigns green, yellow, red and lost time to each arm and repeats this procedure until the program is ran completely

Cycle time =120 second in 2-phase plan



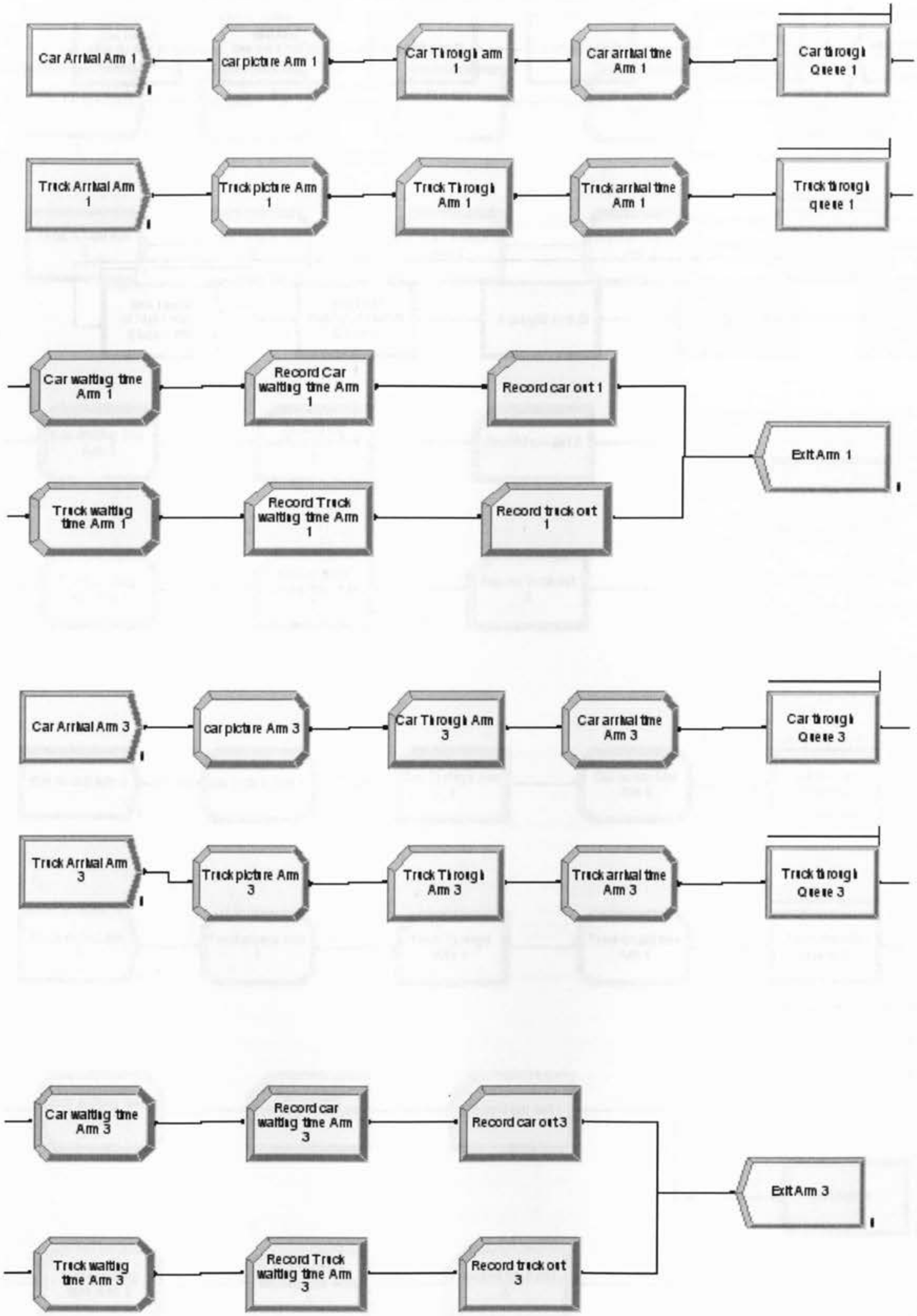

Car Arrival Arm 2
car picture Arm 2
Car Through Arm 2
Car arrival time Arm 2
Car through Queue 2
Truck Arrival Arm 2
Truck picture Arm 2
Truck Through Arm 2
Truck arrival time Arm 2
Truck through Queue 2
Car waiting time Arm 2
Record car waiting time Arm 2
Record car out 2
Exit Arm 2
Truck waiting time Arm 2
Record truck waiting time Arm 2
Record truck out 2

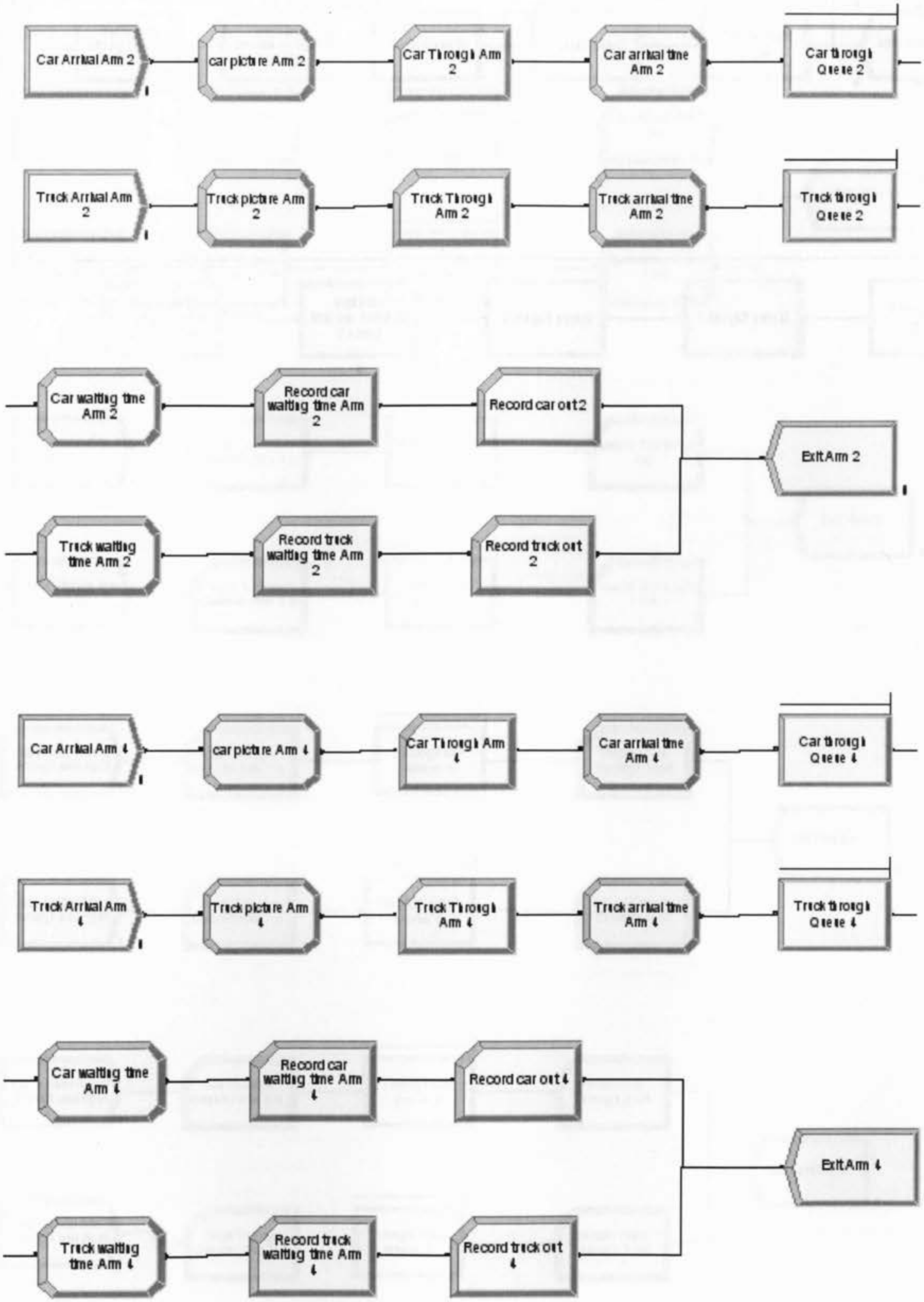

Car Arrival Arm 4
car picture Arm 4
Car Through Arm 4
Car arrival time Arm 4
Car through Queue 4
Truck Arrival Arm 4
Truck picture Arm 4
Truck Through Arm 4
Truck arrival time Arm 4
Truck through Queue 4
Car waiting time Arm 4
Record car waiting time Arm 4
Record car out 4
Exit Arm 4
Truck waiting time Arm 4
Record truck waiting time Arm 4
Record truck out 4



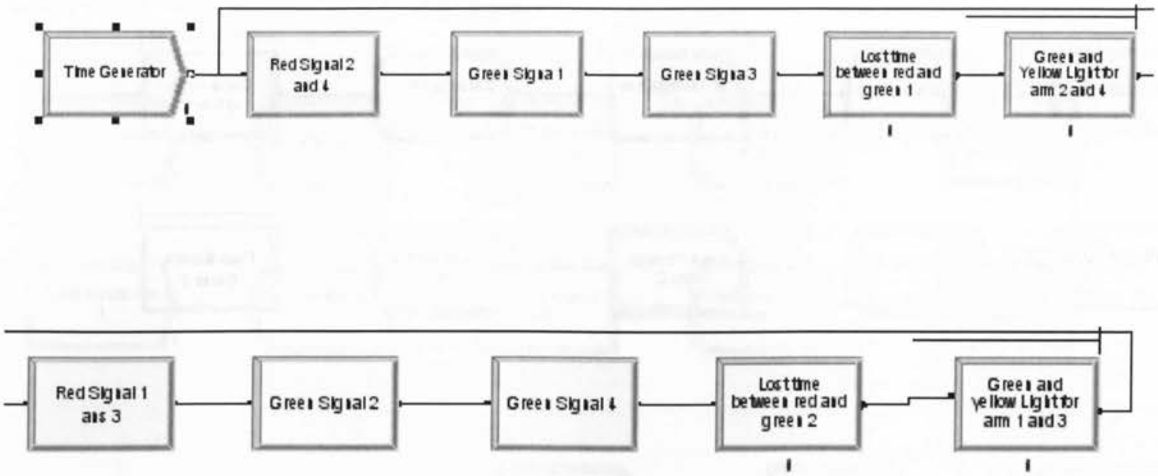
Time Generator
Red Signal 2 and 4
Green Signal 1
Green Signal 3
Lost time between red and green 1
Green and Yellow Light for arm 2 and 4
Red Signal 1 ans 3
Green Signal 2
Green Signal 4
Lost time between red and green 2
Green and yellow Light for arm 1 and 3

Cycle time =140 second in 2-phase plan

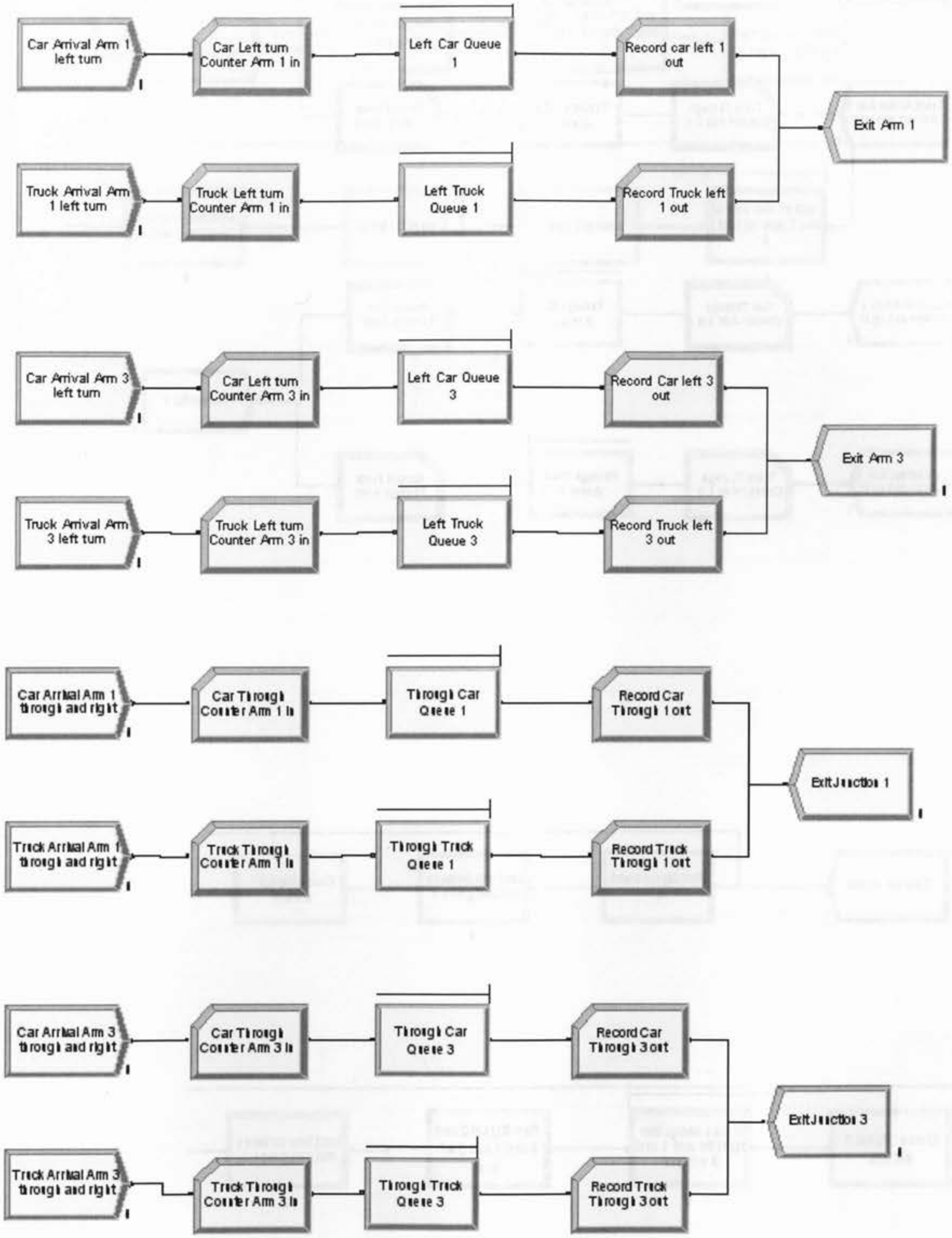


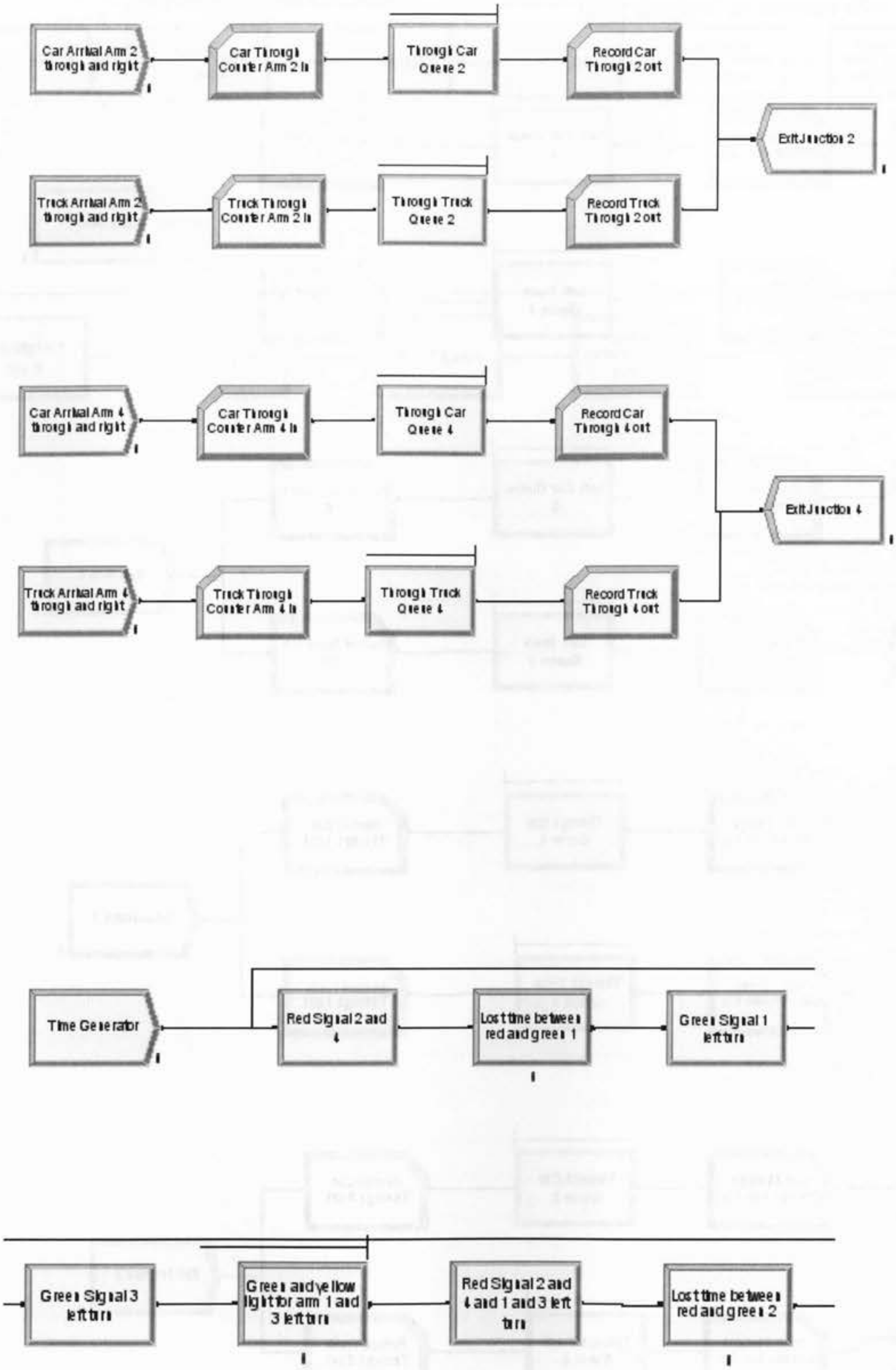


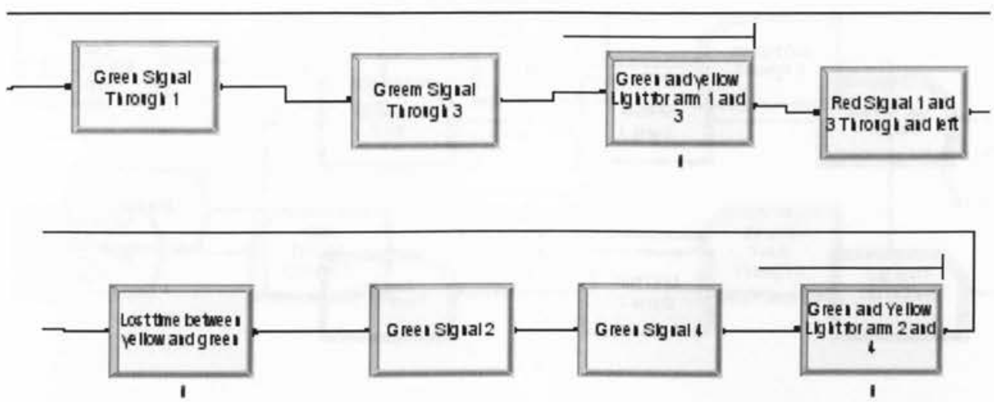
Car Arrival Arm 2
car picture Arm 2
Car Through Arm 2
Car arrival time Arm 2
Car through Queue 2
Truck Arrival Arm 2
Truck picture Arm 2
Truck Through Arm 2
Truck arrival time Arm 2
Truck through Queue 2
Car waiting time Arm 2
Record car waiting time Arm 2
Record car out 2
Exit Arm 2
Truck waiting time Arm 2
Record truck waiting time Arm 2
Record truck out 2
Car Arrival Arm 4
car picture Arm 4
Car Through Arm 4
Car arrival time Arm 4
Car through Queue 4
Truck Arrival Arm 4
Truck picture Arm 4
Truck Through Arm 4
Truck arrival time Arm 4
Truck through Queue 4
Car waiting time Arm 4
Record car waiting time Arm 4
Record car out 4
Exit Arm 4
Truck waiting time Arm 4
Record truck waiting time Arm 4
Record truck out 4


Time Generator
Red Signal 2 and 4
Green Signal 1
Green Signal 3
Lost time between red and green 1
Green and Yellow Light for arm 2 and 4
Red Signal 1 and 3
Green Signal 2
Green Signal 4
Lost time between red and green 2
Green and yellow Light for arm 1 and 3

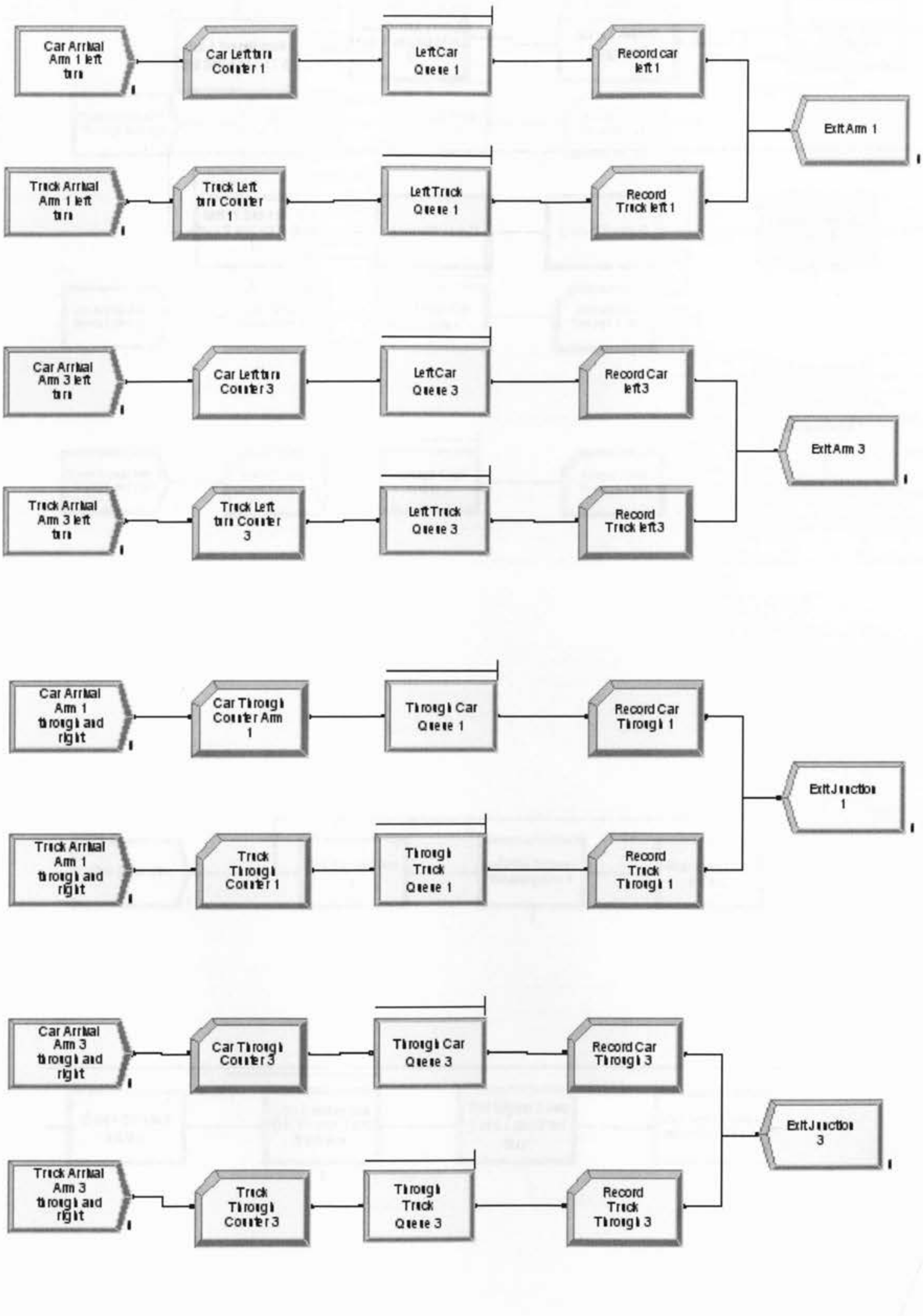
Cycle time =90 second in 3-phase plan

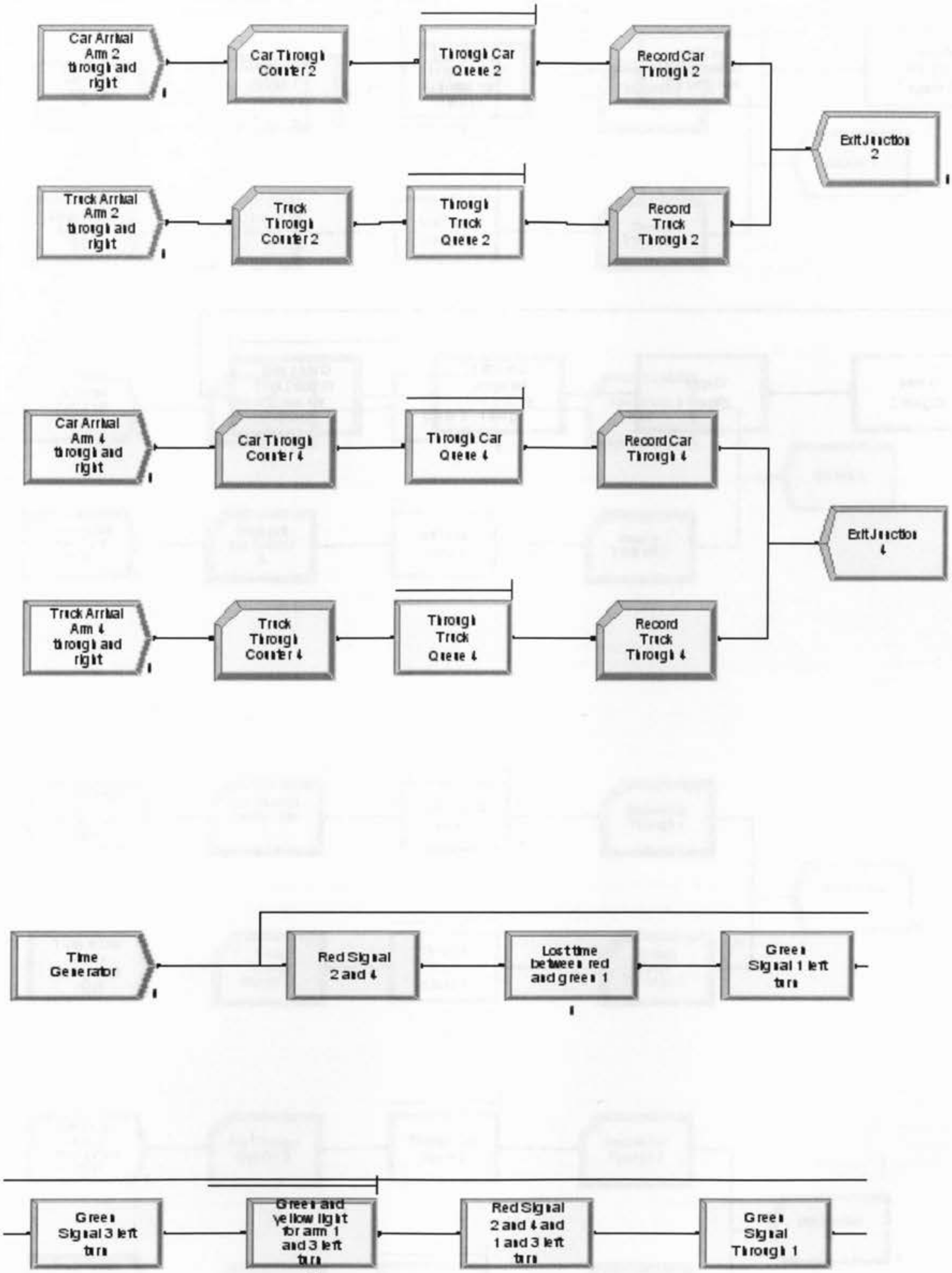



Car Arrival Arm 2 through and right
Car Through Counter Arm 2 In
Through Car Queue 2
Record Car Through 2 out
Exit Junction 2
Truck Arrival Arm 2 through and right
Truck Through Counter Arm 2 In
Through Truck Queue 2
Record Truck Through 2 out
Car Arrival Arm 4 through and right
Car Through Counter Arm 4 In
Through Car Queue 4
Record Car Through 4 out
Exit Junction 4
Truck Arrival Arm 4 through and right
Truck Through Counter Arm 4 In
Through Truck Queue 4
Record Truck Through 4 out
Time Generator
Red Signal 2 and 4
Lost time between red and green 1
Green Signal 1 left turn
Green Signal 3 left turn
Green and yellow light for arm 1 and 3 left turn
Red Signal 2 and 4 and 1 and 3 left turn
Lost time between red and green 2

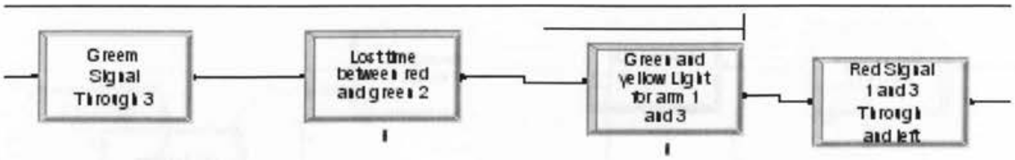

Green Signal Through 1
Greem Signal Through 3
Green and yellow Light for arm 1 and 3
Red Signal 1 and 3 Through and left
Lost time between yellow and green
Green Signal 2
Green Signal 4
Green and Yellow Light for arm 2 and 4

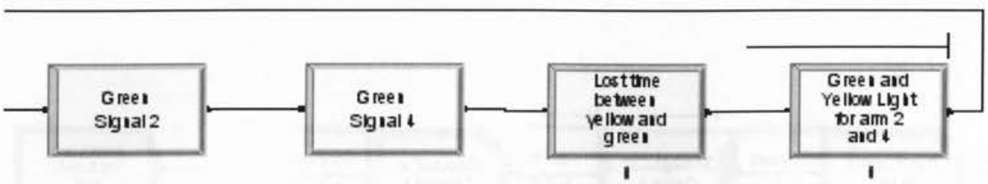
Cycle time =120 second in 3-phase plan



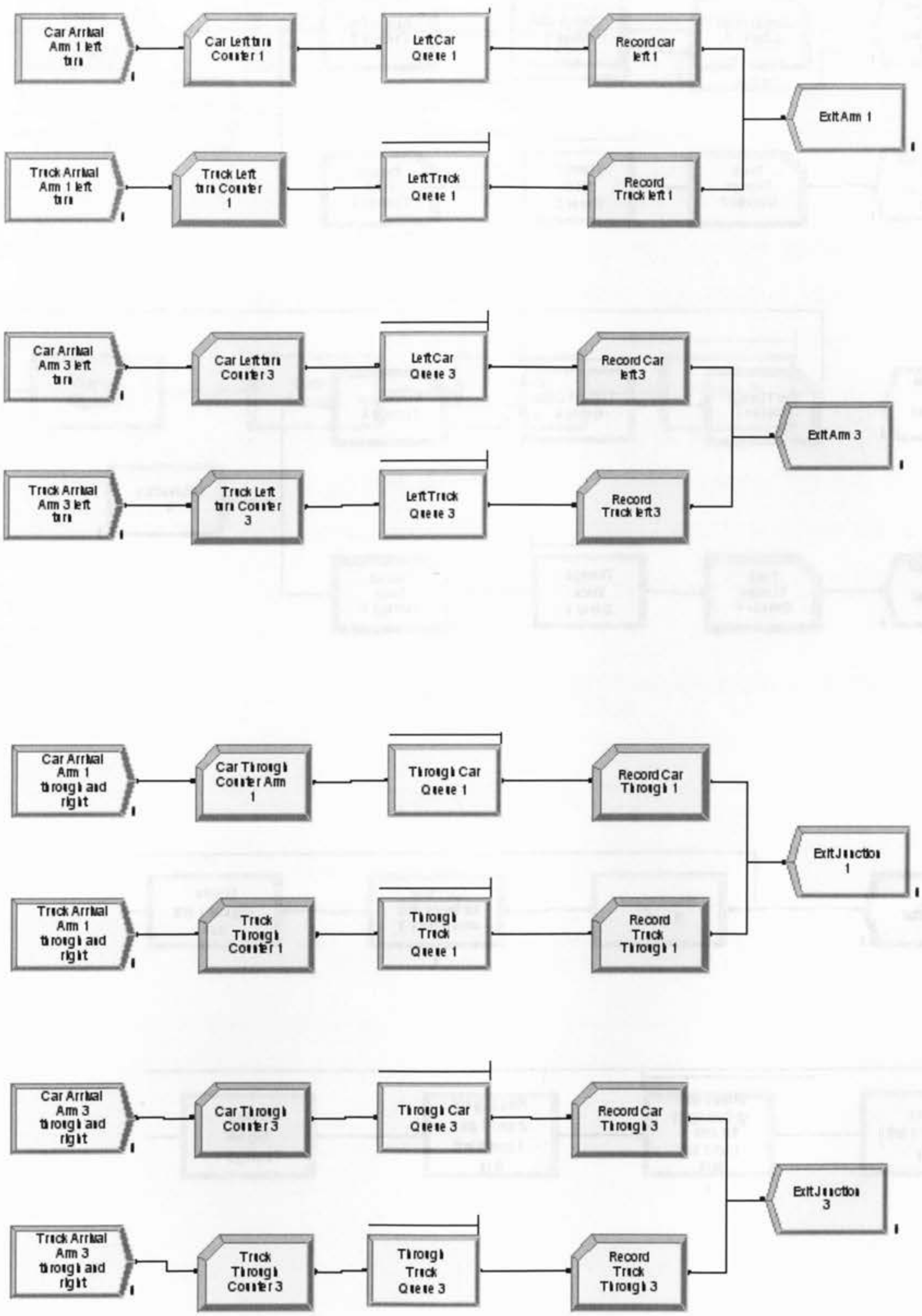


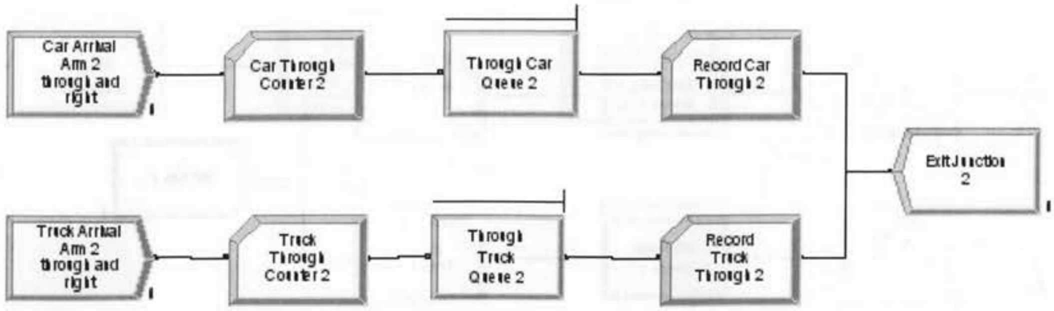
Car Arrival Arm 2 through and right
Car Through Counter 2
Through Car Queue 2
Record Car Through 2
Exit Junction 2
Truck Arrival Arm 2 through and right
Truck Through Counter 2
Through Truck Queue 2
Record Truck Through 2
Car Arrival Arm 4 through and right
Car Through Counter 4
Through Car Queue 4
Record Car Through 4
Exit Junction 4
Truck Arrival Arm 4 through and right
Truck Through Counter 4
Through Truck Queue 4
Record Truck Through 4
Time Generator
Red Signal 2 and 4
Lost time between red and green 1
Green Signal 1 left turn
Green Signal 3 left turn
Green and yellow light for arm 1 and 3 left turn
Red Signal 2 and 4 and 1 and 3 left turn
Green Signal Through 1

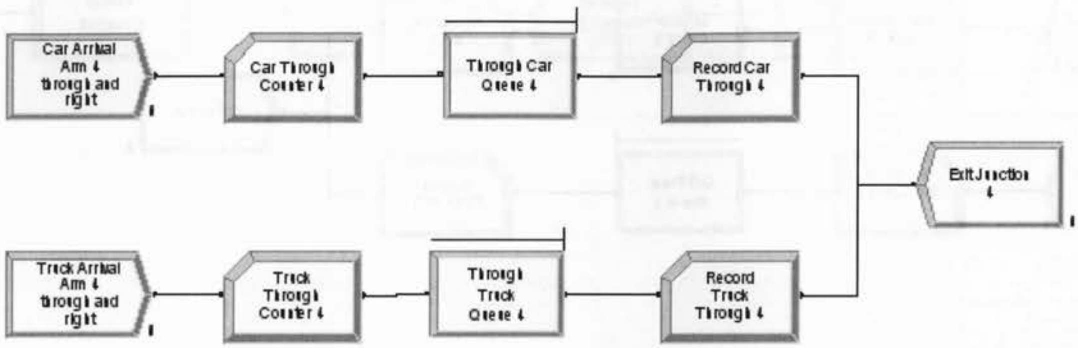

Greem Signal Through 3
Lost time between red and green 2
Green and yellow Light for arm 1 and 3
Red Signal 1 and 3 Through and left

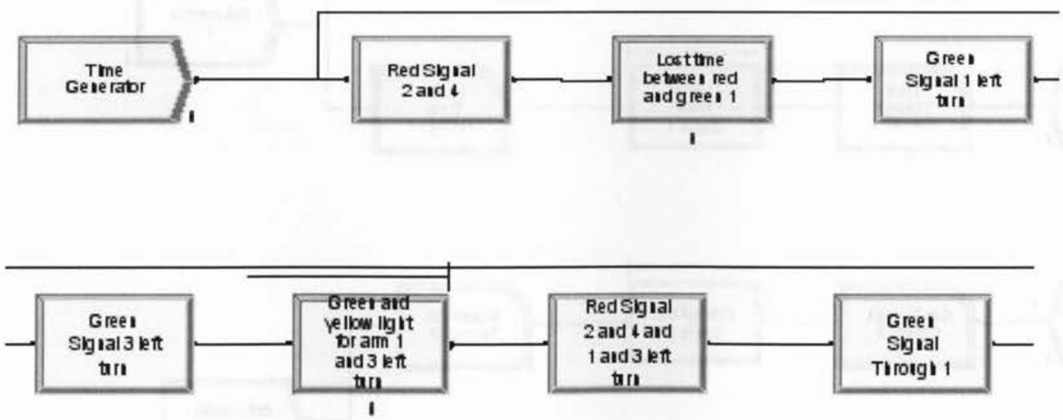

Green Signal 2
Green Signal 4
Lost time between yellow and green
Green and Yellow Light for arm 2 and 4

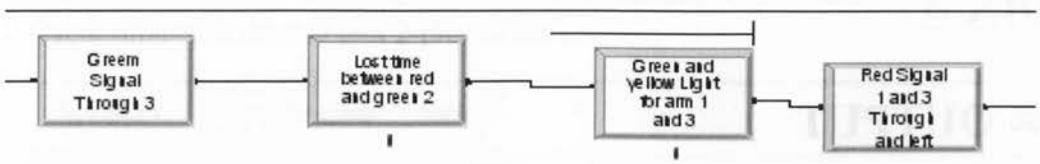
Cycle time =140 second in 3-phase plan

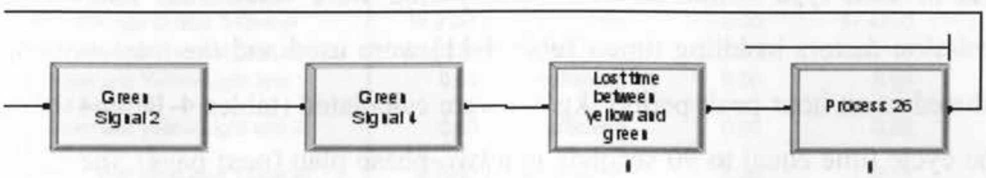



Car Arrival Arm 2 through and right
Car Through Counter 2
Through Car Queue 2
Record Car Through 2
Exit Junction 2
Truck Arrival Arm 2 through and right
Truck Through Counter 2
Through Truck Queue 2
Record Truck Through 2


Car Arrival Arm 4 through and right
Car Through Counter 4
Through Car Queue 4
Record Car Through 4
Exit Junction 4
Truck Arrival Arm 4 through and right
Truck Through Counter 4
Through Truck Queue 4
Record Truck Through 4


Time Generator
Red Signal 2 and 4
Lost time between red and green 1
Green Signal 1 left turn
Green Signal 3 left turn
Green and yellow light for arm 1 and 3 left turn
Red Signal 2 and 4 and 1 and 3 left turn
Green Signal Through 1


Greem Signal Through 3
Lost time between red and green 2
Green and yellow Light for arm 1 and 3
Red Signal 1 and 3 Through and left


Green Signal 2
Green Signal 4
Lost time between yellow and green
Process 25

APPENDIX B

ARENA® OUTPUT

Next pages include the Arena® outputs after running for a replication time equal to 60 minutes (1 hour). The average waiting time for each type of vehicles (cars and trucks) and the average number of vehicles in each type within an hour peak period were calculated. These results considering the emission factors in idling time (Table 4-11) were used and the total amount of the emissions produced in an hour peak period (kg/hr) were calculated (tables 4-12 to 4-17). As an example, for the cycle time equal to 90 seconds in a two-phase plan (next page), the average waiting time and the average number of each vehicle waiting in the queue used for emission calculation, are shown in boxes to point out the required outputs. Table B-1 shows the general types of outputs used for emissions calculation.

Table B-1: Outputs for simulation in Arena®

	Arena® outputs
1	Queuing average time for each type of vehicles in an hour peak period.
2	Number of vehicles in the queue in an hour peak period.

Cycle time = 90 second in a 2-phase plan

Replications: 1 Time Units: Minutes

Queue

Time

Waiting Time	Average	Half Width	Minimum Value	Maximum Value
Car through Queue 1.Queue	25.3588	(Correlated)	0.00	49.7434
Car through Queue 2.Queue	17.1903	(Insufficient)	0.4118	32.6033
Car through Queue 3.Queue	18.9393	(Correlated)	0.00	37.4990
Car through Queue 4.Queue	23.0738	(Insufficient)	0.6953	44.8838
Green and Yellow Light arm 1 and 3.Queue	0.00	(Insufficient)	0.00	0.00
Green and yellow Light arm 2 and 4.Queue	0.00	(Insufficient)	0.00	0.00
Truck through queue 1.Queue	0.7032	(Insufficient)	0.00	1.4688
Truck through Queue 3.Queue	2.0052	(Insufficient)	0.00	5.1685

Other

Number Waiting	Average	Half Width	Minimum Value	Maximum Value
Car through Queue 1.Queue	1297.02	(Correlated)	0.00	2563.00
Car through Queue 2.Queue	163.18	(Correlated)	0.00	313.00
Car through Queue 3.Queue	366.30	(Correlated)	0.00	739.00
Car through Queue 4.Queue	472.66	(Correlated)	0.00	942.00
Green and Yellow Light arm 1 and 3.Queue	0.00	(Insufficient)	0.00	0.00
Green and yellow Light arm 2 and 4.Queue	0.00	(Insufficient)	0.00	0.00
Truck through queue 1.Queue	0.8673	(Insufficient)	0.00	4.0000
Truck through Queue 2.Queue	28.8204	(Insufficient)	0.00	60.0000
Truck through Queue 3.Queue	6.1965	(Insufficient)	0.00	18.0000
Truck through Queue 4.Queue	49.5622	(Insufficient)	0.00	94.0000

Cycle time =120 second in a 2-phase plan

Replications: 1 Time Units: Minutes

Queue

Time

Waiting Time	Average	Half Width	Minimum Value	Maximum Value
Car through Queue 1.Queue	25.2506	(Correlated)	0.00	49.3951
Car through Queue 2.Queue	16.3799	(Insufficient)	0.0905	30.2793
Car through Queue 3.Queue	17.7701	(Correlated)	0.00	34.1802
Car through Queue 4.Queue	22.8164	(Insufficient)	0.9220	44.3266
Green and yellow Light for arm 1 and 3.Queue	0.00	(Insufficient)	0.00	0.00
Green and Yellow Light for arm 2 and 4.Queue	0.00	(Insufficient)	0.00	0.00
Truck through queue 1.Queue	1.0330	(Insufficient)	0.00	2.0465
Truck through Queue 2.Queue	1.3800	(Insufficient)	1.3800	1.3800
Truck through Queue 3.Queue	12.8790	(Insufficient)	0.00	25.2047

Other

Number Waiting	Average	Half Width	Minimum Value	Maximum Value
Car through Queue 1.Queue	1266.90	(Correlated)	0.00	2509.00
Car through Queue 2.Queue	152.88	(Correlated)	0.00	291.00
Car through Queue 3.Queue	336.18	(Correlated)	0.00	685.00
Car through Queue 4.Queue	461.38	(Correlated)	0.00	915.00
Green and yellow Light for arm 1 and 3.Queue	0.00	(Insufficient)	0.00	0.00
Green and Yellow Light for arm 2 and 4.Queue	0.00	(Insufficient)	0.00	0.00
Truck through queue 1.Queue	1.2740	(Insufficient)	0.00	5.0000
Truck through Queue 2.Queue	27.8434	(Insufficient)	0.00	59.0000
Truck through Queue 3.Queue	39.1218	(Insufficient)	0.00	72.0000
Truck through Queue 4.Queue	49.5622	(Insufficient)	0.00	94.0000

Cycle time =140 second in a 2-phase plan

Replications: 1 Time Units: Minutes

Queue

Time

Waiting Time	Average	Half Width	Minimum Value	Maximum Value
Car through Queue 1.Queue	24.9274	(Correlated)	0.00	48.2437
Car through Queue 2.Queue	17.3334	(Insufficient)	0.1088	33.2560
Car through Queue 3.Queue	18.0593	(Correlated)	0.00	35.0037
Car through Queue 4.Queue	23.5377	(Insufficient)	1.0364	45.6206
Green and Yellow Light for 1 and 3.Queue	0.00	(Insufficient)	0.00	0.00
Green and yellow Light for 2 and 4.Queue	0.00	(Insufficient)	0.00	0.00
Truck through queue 1.Queue	1.2424	(Insufficient)	0.00	2.3191
Truck through Queue 3.Queue	7.3600	(Insufficient)	0.00	13.2512

Other

Number Waiting	Average	Half Width	Minimum Value	Maximum Value
Car through Queue 1.Queue	1280.32	(Correlated)	0.00	2531.00
Car through Queue 2.Queue	160.50	(Correlated)	0.00	310.00
Car through Queue 3.Queue	349.59	(Correlated)	0.00	707.00
Car through Queue 4.Queue	469.99	(Correlated)	0.00	937.00
Green and Yellow Light for 1 and 3.Queue	0.00	(Insufficient)	0.00	0.00
Green and yellow Light for 2 and 4.Queue	0.00	(Insufficient)	0.00	0.00
Truck through queue 1.Queue	1.5323	(Insufficient)	0.00	5.0000
Truck through Queue 2.Queue	28.8204	(Insufficient)	0.00	60.0000
Truck through Queue 3.Queue	22.8468	(Insufficient)	0.00	41.0000
Truck through Queue 4.Queue	49.5622	(Insufficient)	0.00	94.0000

Cycle time =90 second in a 3-phase plan

Replications: 1 Time Units: Minutes

Queue

Time

Waiting Time	Average	Half Width	Minimum Value	Maximum Value
Green and yellow light for arm 1 and 3 left turn.Queue	0.00	(Insufficient)	0.00	0.00
Green and yellow Light for arm 1 and 3.Queue	0.00	(Insufficient)	0.00	0.00
Green and Yellow Light for arm 2 and 4.Queue	0.00	(Insufficient)	0.00	0.00
Left Car Queue 1.Queue	20.6795	(Insufficient)	0.05000000	42.0587
Left Car Queue 3.Queue	0.9803	(Insufficient)	0.02237475	2.3689
Left Truck Queue 1.Queue	0.05000000	(Insufficient)	0.05000000	0.05000000
Left Truck Queue 3.Queue	1.8736	(Insufficient)	0.05000000	3.6459
Through Car Queue 1.Queue	22.7379	(Correlated)	0.01362211	45.2255
Through Car Queue 2.Queue	17.4540	(Insufficient)	0.2917	33.4809
Through Car Queue 3.Queue	15.9468	(Correlated)	0.1562	31.7695
Through Car Queue 4.Queue	22.7763	(Insufficient)	0.4299	45.4638
Through Truck Queue 1.Queue	0.7880	(Insufficient)	0.05772303	1.4880
Through Truck Queue 2.Queue	0.9874	(Insufficient)	0.03091398	3.1172
Through Truck Queue 3.Queue	0.7758	(Insufficient)	0.02936243	2.0228
Through Truck Queue 4.Queue	1.7480	(Insufficient)	0.02335545	3.8597

Other

Number Waiting	Average	Half Width	Minimum Value	Maximum Value
Green and yellow light for arm 1 and 3 left turn.Queue	0.00	(Insufficient)	0.00	0.00
Green and yellow Light for arm 1 and 3.Queue	0.00	(Insufficient)	0.00	0.00
Green and Yellow Light for arm 2 and 4.Queue	0.00	(Insufficient)	0.00	0.00
Left Car Queue 1.Queue	103.62	(Correlated)	0.00	201.00
Left Car Queue 3.Queue	0.6631	(Insufficient)	0.00	3.0000
Left Truck Queue 1.Queue	6.3323	(Insufficient)	0.00	11.0000
Left Truck Queue 3.Queue	0.3747	(Insufficient)	0.00	2.0000
Through Car Queue 1.Queue	591.02	(Correlated)	0.00	1160.00
Through Car Queue 2.Queue	178.37	(Correlated)	0.00	362.00
Through Car Queue 3.Queue	203.44	(Correlated)	0.00	412.00
Through Car Queue 4.Queue	456.97	(Correlated)	0.00	905.00
Through Truck Queue 1.Queue	0.4860	(Insufficient)	0.00	2.0000
Through Truck Queue 2.Queue	1.1026	(Insufficient)	0.00	7.0000
Through Truck Queue 3.Queue	0.6724	(Insufficient)	0.00	4.0000
Through Truck Queue 4.Queue	3.2337	(Insufficient)	0.00	8.0000

Cycle time =120 second in a 3-phase plan

Replications: 1 Time Units: Minutes

Queue

Time

Waiting Time	Average	Half Width	Minimum Value	Maximum Value
Green and yellow light for arm 1 and 3 left turn.Queue	0.00	(Insufficient)	0.00	0.00
Green and yellow Light for arm 1 and 3.Queue	0.00	(Insufficient)	0.00	0.00
Green and Yellow Light for arm 2 and 4.Queue	0.00	(Insufficient)	0.00	0.00
Left Car Queue 1.Queue	19.5932	(Insufficient)	0.05000000	39.3819
Left Car Queue 3.Queue	0.8960	(Insufficient)	0.05000000	1.9108
Left Truck Queue 1.Queue	0.05000000	(Insufficient)	0.05000000	0.05000000
Left Truck Queue 3.Queue	1.3844	(Insufficient)	0.05000000	2.7020
Through Car Queue 1.Queue	24.2585	(Insufficient)	0.1397	48.0422
Through Car Queue 2.Queue	20.5156	(Insufficient)	0.5317	40.6598
Through Car Queue 3.Queue	19.2897	(Insufficient)	0.1362	37.4012
Through Car Queue 4.Queue	24.3326	(Insufficient)	0.6699	48.2852
Through Truck Queue 1.Queue	0.9294	(Insufficient)	0.1065	1.9580
Through Truck Queue 2.Queue	1.0505	(Insufficient)	0.03581602	2.5872
Through Truck Queue 3.Queue	0.8564	(Insufficient)	0.00793649	1.8393
Through Truck Queue 4.Queue	0.9581	(Insufficient)	0.02166250	2.1781

Other

Number Waiting	Average	Half Width	Minimum Value	Maximum Value
Green and yellow light for arm 1 and 3 left turn.Queue	0.00	(Insufficient)	0.00	0.00
Green and yellow Light for arm 1 and 3.Queue	0.00	(Insufficient)	0.00	0.00
Green and Yellow Light for arm 2 and 4.Queue	0.00	(Insufficient)	0.00	0.00
Left Car Queue 1.Queue	98.8632	(Insufficient)	0.00	192.00
Left Car Queue 3.Queue	0.6151	(Insufficient)	0.00	3.0000
Left Truck Queue 1.Queue	6.3323	(Insufficient)	0.00	11.0000
Left Truck Queue 3.Queue	0.2769	(Insufficient)	0.00	2.0000
Through Car Queue 1.Queue	635.42	(Correlated)	0.00	1251.00
Through Car Queue 2.Queue	212.20	(Correlated)	0.00	426.00
Through Car Queue 3.Queue	246.86	(Correlated)	0.00	502.00
Through Car Queue 4.Queue	490.80	(Correlated)	0.00	970.00
Through Truck Queue 1.Queue	0.5731	(Insufficient)	0.00	3.0000
Through Truck Queue 2.Queue	1.1589	(Insufficient)	0.00	7.0000
Through Truck Queue 3.Queue	0.7422	(Insufficient)	0.00	5.0000
Through Truck Queue 4.Queue	1.7647	(Insufficient)	0.00	7.0000

Model Filename: C:\Documents and Settings\Owner\Desktop\Arena files and examples\New Fo Page 7 of 10

Cycle time =140 second in a 3-phase plan

Replication 1 Start Time: 0.00 Stop Time: 60.00 Time Units: Minutes

Queue Detail Summary

Time

	Waiting Time
Green and yellow light for arm 1 and 3 left turn.Queue	0.00
Green and yellow Light for arm 1 and 3.Queue	0.00
Left Car Queue 1.Queue	18.60
Left Car Queue 3.Queue	0.99
Left Truck Queue 1.Queue	0.05
Left Truck Queue 3.Queue	0.79
Process 26.Queue	0.00
Through Car Queue 1.Queue	25.01
Through Car Queue 2.Queue	22.12
Through Car Queue 3.Queue	20.82
Through Car Queue 4.Queue	25.38
Through Truck Queue 1.Queue	1.23
Through Truck Queue 2.Queue	1.20
Through Truck Queue 3.Queue	1.07
Through Truck Queue 4.Queue	1.10

Other

	Number Waiting
Green and yellow light for arm 1 and 3 left turn.Queue	0.00
Green and yellow Light for arm 1 and 3.Queue	0.00
Left Car Queue 1.Queue	92.86
Left Car Queue 3.Queue	0.67
Left Truck Queue 1.Queue	6.33
Left Truck Queue 3.Queue	0.16
Process 26.Queue	0.00
Through Car Queue 1.Queue	653.51
Through Car Queue 2.Queue	227.10
Through Car Queue 3.Queue	264.94
Through Car Queue 4.Queue	505.70
Through Truck Queue 1.Queue	0.76
Through Truck Queue 2.Queue	1.34
Through Truck Queue 3.Queue	0.93
Through Truck Queue 4.Queue	2.04

Model Filename: C:\Documents and Settings\Owner\Desktop\Arena files and examples\New Fo Page 1 of 5

REFERENCES

Allsop, R. E., 1974, "Some possibility for using traffic control to influence trip distribution and route choice.", Proceedings of the Sixth International Symposium on Transportation and Traffic Theory, Elsevier, Amsterdam, pp. 345-374.

Banks, J., Carson, J. S., Nelson, B. L., and Nicol, D. M., 1995, "Discrete-Event System Simulation", Pearson-Prentice Hall, pp. 187-204.

Branston, D., 1979, "Some factors affecting the capacity of signalized intersection", Traffic Engineering Control, Vol. 20, pp. 390-396.

Burman, D. Y., and Smith, D. R., 1986 "An asymptotic analysis of a queueing system with markov-modulated arrivals", Operations Research, Vol. 34, No. 1, pp. 105-119.

Chen, S. P., 2005, "Parametric nonlinear programming approach to fuzzy queues with bulk service", European Journal of Operational Research, Vol.163, No. 2, pp. 434–444.

Chiou, S. W., 2007, "Reserve capacity of signal-controlled road network", Applied Mathematics and Computation, Vol. 190, pp. 1602-1611.

Coelho, M. C., Farias, T. L., and Rouphail, N. M., 2005, "Impact of speed control traffic signals on pollutant emissions", Transportation Research part D, Vol. 10, No. 4, pp. 323-340.

Drissi-Kaitouni, O. and Hameda-Benchekroun, A., 1992, "A dynamic traffic assignment model and a solution algorithm", Transportation Science, Vol. 26, No. 2, pp. 119-128.

Emission facts: Idling vehicle emissions, 1998, USEPA.

http://www.epa.gov/otaq/consumer/f98014.pdf (Accessed on January 25, 2008)

Fang, C. F., and Elefteriadou, L., 2006, "Development of an Optimization Methodology for Adaptive Traffic Signal Control at Diamond Interchanges", Journal of Transportation Engineering, Vol. 132, No. 8, pp. 629-637.

Fries, R., Inamdar, I., Chowdhury, M. A., Taaffe, K., and Ozbay, K., 2007, "Feasibility of Traffic Simulation for Decision Support in Real-Time Regional Traffic Management", Transportation Research Record: Journal of the Transportation Research Board, Vol. 2035 pp.169-176.

Ghassan, A. L., 2002, "Integrated Adaptive-Signal Dynamic-Speed Control of Signalized Arterials", Journal of Transportation Engineering, Vol. 128, No. 5, pp. 447-451.

Han Lee, D., and Li, J. M., 2007, "Short Or Long--Which Is Better? A Probabilistic Approach to Cycle Length Optimization", Transportation Research Record: Journal of the Transportation Research Board, Vol. 2035,
pp. 150-157.

Heydecker, B. G., 1996, "A decomposition approach for signal optimization in road networks", Transportation Research, Part B, Vol. 30, No. 2, pp. 99-114.

Horowitz, A. J., and Patel, M. H., 2005, "The Network Signal Design Problem for Long-Range Travel Forecasting", Journal of Transportation Engineering, Vol. 131, No. 3, pp. 183–192.

Improta, G., and Cantarella, G. E., 1984, "Control System Design for an Individual Signalized Junction", Transportation Research, Part B, Vol. 18, No. 2, pp. 147-167.

Introduction to stationary point source emission inventory development, 2001, USEPA. http://www.epa.gov/ttn/chief/eiip/techreport/volume02/ii01_may2001.pdf (Accessed on February 05, 2008)

Kelton, W., Sadowski, R., and Sadowski, D., 2002, "Simulation with Arena", McGraw-Hill, pp. 5-120.

Lan, C. J., 2004, "New Optimal Cycle Length Formulation for Pretimed Signals at Isolated Intersections", Journal of Transportation Engineering, Vol. 130, No. 5, September 1, pp. 637–647.

Liu, C., Yu, L., Khajohnsak, S., and Hong, S. Oey, 2002, "Determination of Left-Turn Yellow Change and Red Clearance Interval", Journal of Transportation Engineering, Vol. 128, No. 5, pp. 452-457.

Liu, Y., Chang, G. L., Tao, R., Hicks, T., and Tabacek, E., 2007, "Empirical Observations of Dynamic Dilemma Zones at Signalized Intersections", Transportation Research Record: Journal of the Transportation Research Board, Vol. 2035, pp. 122-133.

Mahut, M., Florian, M., Tremblay, N., and Campbell, M., 2004, "Calibration and application of a simulation-based dynamic traffic assignment model", Transportation Research Record: Journal of the Transportation Research Board, Vol. 1876, pp. 101-111.

Ma, W., Cheu, R. L., and Lee, D. H., 2004, "Scheduling of Lane Closures Using Genetic Algorithms with Traffic Assignments and Distributed Simulations", Journal of Transportation Engineering, Vol. 130, No. 3, pp. 322-329.

Nagurney, A. (1999) "Congested urban transportation networks and emission paradoxes", Department of Finance and Operations Management, Isenberg School of Management, University of Massachusetts Press.

National Institute for Advanced Transportation Technology at the University of Idaho. http://www.webs1.uidaho.edu/niattproject/TCUS_Tables.htm (Accessed on November 20, 2007)

Papageorgiou, M., Diakaki, C., Dinopoulou, V., Kotsialos, A., and Wang, Y., 2003, "Review of Road Traffic Control Strategies", Proceedings of The IEEE, Transactions on Intelligent Transportation Systems, Vol. 91, No. 12, pp. 2043-2067.

Procedures for Emission Inventory Preparation, Volume IV: Mobile Sources, 1992, USEPA. http://www.epa.gov/otaq/invntory/r92009.pdf (Accessed on November 30, 2007)

Rilett, L. R., and Benedek, C. M., 1994, "Traffic assignment under environmental and equity objectives", Transportation Research Record: Journal of the Transportation Research Board , Vol. 1443, pp. 92-99.

Rodrigue, J. P., Comtois, C., and Slack, B., 2006, "The Geography of Transport Systems", New York: Routledge, pp. 187-211.
http://people.hofstra.edu/geotrans/eng/ch6en/conc6en/ch6c4en.html

Sadoun, B., 2003, "An Efficient Simulation Methodology for the Design of Traffic Lights at Intersections in Urban Areas", Simulation, Vol. 79, No. 4, pp. 243-251.

Texas Transportation Institute, (1999), "Study shows traffic worsening in a variety of ways and places", Texas Transportation Researcher, Vol. 35, No. 4, pp. 4.

Transportation and Global Climate Change: a Review and Analysis of the Literature, 1998, US department of transportation, Federal highway administration office of environmental and planning, chapter 2, pp. 4-5.
http://www.fhwa.dot.gov/environment/glob_c2.pdf (Accessed on October 14, 2007)

Transportation and Traffic Engineering Handbook, 1985, Institute of Traffic Engineers Prentice Hall, Englewood Cliffs, N.J.

Turchetta, 1999, US Department of Transportation, Federal Highway Administration. http://www.fhwa.dot.gov/environment/global.htm (Accessed on October 14, 2007)

Turner, D., 1999, ''A call to arms: are we ready for tomorrow's challenge when we are still struggling with today?'', Institute of Transportation Engineers, ITE Journal, January, pp. 26–31.

Van Dam, D., Van Dobben, H. F., Ter Braak, C. F. J., and De Wit, T., 1986, "Air pollution as a possible cause for the decline of some phanerogamic species in The Netherlands", Plant Ecology, Vol. 65, No. 1, pp. 47-52

Wilhelm, M. H., 2004, "Motor Vehicle-Related Air Pollution and Adverse Birth Outcomes in Los Angeles County, California, 1994-2000", Dissertation, pp. 10-23.

Wong, S. C., 1997, "Group-Based optimization of signal timing using parallel computing", Transportation Research, Vol. 5, No. 2, pp. 123-139.

Wong, C. K., and Wong, S. C., 2001, "Lane-based optimization of signal timings for isolated junctions", Transportation Research, Part B, No. 37, 2003, pp. 63-48.

Wong, C.K., Wong, S.C., and Tong, C.O., 2001, "Lane-based optimization method for minimizing delay of isolated signal controlled junctions", 7th International Conference on Applications of Advanced Technology in Transportation, 5–7 August 2002, Cambridge, Massachusetts, USA.

Wong, C. K., and Wong, S. C., 2005, "The advantages of the multi-period lane-based optimization method for traffic signal design", Proceedings of the Eastern Asia Society for Transportation Studies, Vol. 5, pp. 358-1366.

Wong, S. C., and Yong, C., 1999, "An iterative group-based signal optimization scheme for traffic equilibrium networks", Journal of Advanced Transportation, Vol. 33, No. 2, pp. 201-217.

Wong, S. C., Yang, C., and H. K. Lo, 1999, "A path-based traffic assignment algorithm based on the TRANSYT traffic model", Transportation Research, Part B: Methodological, Vol. 35, No. 2, February 2001, pp. 163-181.

Yan, H., and Lam, W. H., 1996, "Optimal road tolls under conditions of queuing and congestion", Transportation Research Part A: Policy and Practice, Vol. 30, No. 5, pp. 319-332.

NOMENCLATURE

BMINLP	Binary-Mix-Integer-Non-Linear Programs
CO	Carbon Monoxide
CO2	Carbon Dioxide
DTA	Dynamic Traffic Assignment
DWTD	Distance-Weighted Traffic Density
EPA	Environmental Protection Agency
GVW	Gross Vehicle Weight
HC	Hydro Carbon
HDDV	Heavy-duty diesel vehicles, 8501+ lb GVW
HDGV	Heavy-duty gasoline-fueled vehicles, 8501+ lb
ITE	Institute of Traffic Engineers
ITS	Intelligent Transportation Systems
IVHS	Intelligent Vehicle-Highway System
LDDT	Light-duty diesel trucks, up to 8500 lb GVW
LDDV	Light-duty diesel vehicles, up to 6000 lb GVW
LDGT1	Light-duty gasoline-fueled trucks, up to 8500 lb GVW
LDGV	Light-duty gasoline-fueled vehicles, up to 6000 lb GVW
LWB	Low Weight Birth
MC	Motorcycles
MMTCE	Million Metric Tons of Carbon Equivalent
MPEC	Mathematical Program with Equilibrium Constraints
NOx	Nitrogen Oxides
NSDP	Network Signal Design Problem
PGM	Projected Gradient Method
SCATS	Sydney Coordinated Adaptive Traffic System
SCOOT	Split Cycle Offset Optimization Technique
TCU	Through car Equivalents
VMT	Vehicle Miles Traveled
VOC	Volatile Organic Compounds

$$C_w = 5 + 1.5L / \left(1 - \sum Y_{ci}\right)$$

C_w	Optimal cycle length [s]
$C_{\min}$	Minimum cycle time [s]
$C_{\max}$	Maximum cycle time [s]
L	Total lost time [s]
Y_{ci}	Intersection critical flow ratio

$$C = \left[(N \cdot t_1)\right] / 1 - V_c / \left[PHF \cdot v/c \cdot (3600/h)\right]$$

C	Cycle time [s]
h	Saturation headway [s]
N	Number of signal phases
PHF	Peak-hour factor
S	3600/h [veh/h]
t_1	Lost time per phase [s]
v/c	Required volume-to-capacity ratio
V_c	Sum of critical lane volumes [veh/h]

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