A Multi-Agent Approach to Real-Time Traffic Signal Optimisation

SUPHASAWAS NIGARNAJAGOOOL[1], HUSSEIN DIA[2]
1 Intelligent Transport Systems Research Laboratory, University of Queensland, QLD, Brisbane, Australia
2 Intelligent Transport Systems Research Laboratory, University of Queensland, QLD, Brisbane, Australia

1 Introduction

The application of advanced technologies including computers, electronics and communications holds great promise in improving traffic conditions, enhancing environmental quality and increasing economic productivity. Collectively known as Intelligent Transport Systems, these technologies are rapidly being accepted by road and transport authorities around the world as a viable alternative to reliance on building more roads to reduce congestion. Advanced Traffic Management Systems (ATMS), in particular, have been shown to reduce travel times, improve travel time reliability and reduce congestions and environmental emissions. The benefits of these systems have also been found to be a function of the accuracy and robustness of the underlying computer algorithms and optimisation techniques which provide various levels of intelligence to traffic signal control and operations. For this reason, traffic signal control is now considered an essential element of Intelligent Transport Systems.

Many cities around the world still implement fixed-time control traffic signal systems. These systems operate on a number of fixed or predetermined plans which are put into operation at different times of the day e.g. one set of plans would operate during morning peak, another during off peak and a third set of plans during evening peak hours. The disadvantage of these systems is that they do respond to changes in traffic demands and assume that traffic conditions during each of these periods will not change. If an accident happens between any two set of intersections, the traffic signal does not have the capacity to detect these changes or respond to such events. Advances in computer technologies and communications systems have now allowed for the introduction of various levels of intelligence in these systems by enabling them to collect traffic data about flows, speeds and travel times to enable the system to respond to incidents, accidents, road works and other events that may reduce the capacity of the road system. Examples of such technologies include commercially available adaptive traffic management systems such as SCATS and SCOOT in addition to other technologies being developed in research institutions such as the Traffic Responsive Urban Control (TRUC) system being developed at the Dynamic Systems and Simulation Laboratory, Technical University of Crete in Greece and the Agent-based traffic management systems being developed in the ITS Research Laboratory at the University of Queensland. The objective of our study is to demonstrate the feasibility of applying agent algorithms to develop a decentralised adaptive traffic control system. The controllers will be equipped with an optimisation model that is generally used to design pre-timed traffic signal timings. Interestingly, the origins of this signal optimisation model are found in the work by Webster in the early 1960s (Webster, 1962). However, the model was initially developed for operation in under-saturated conditions. A number of researchers introduced several changes to the model over the years and a variant of the model (described below) is currently implemented in the well-know aaSIDRA software (aaTraffic, 2004). In this paper, we demonstrate the applicability of our model using a microscopic traffic simulator (AIMSUN NG). The simulation approach provides an environment where different scenarios can be introduced and evaluated in a controlled setting without disrupting traffic conditions in the real world.

2 Modelling Signal Control Using Agents Algorithms

A number of studies have demonstrated the potential of agent technologies in the development and implementation of decentralised control architectures (Erol, 1998). Agents have the ability to solve problems in real-time by executing an action, predicting consequences of the action and evaluating the alternatives. By modelling the signal controllers as agents, it is possible to tune the actions of individual controllers through the agent concept of collaboration (van Kitwijk, 2002). A schematic of a decentralised traffic control system is shown in Figures 1 and 2 below. Each agent (intersection) is assigned a set of individual preferences or settings (these are to some extent similar to 'personalities' in some existing traffic control systems). These preferences include objectives, a set of pre-determined plans and algorithms to generate plans. To achieve some objective, agents execute a set of control plans to optimise performance. If the pre-determined plans are no longer optimal for a particular situation, agents apply a signal optimisation model to create new plans. When intersections are located close to each other, they can be grouped as shown in Figure 2 and assigned a common cycle time. In this paper, we will address the development and evaluation of the traffic signal optimisation algorithm that will be implemented within the agent-based controllers. The development of the agent techniques is the subject of another paper and won't be reported here.

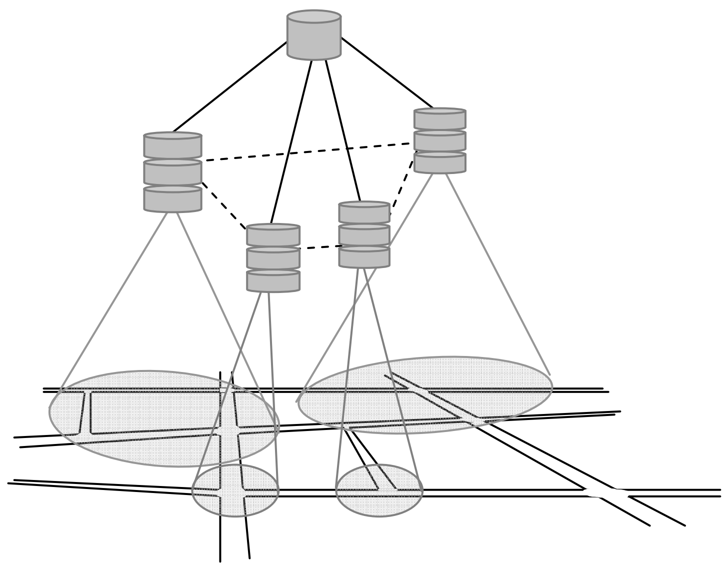


Figure 1: Agent-based Signal Controllers

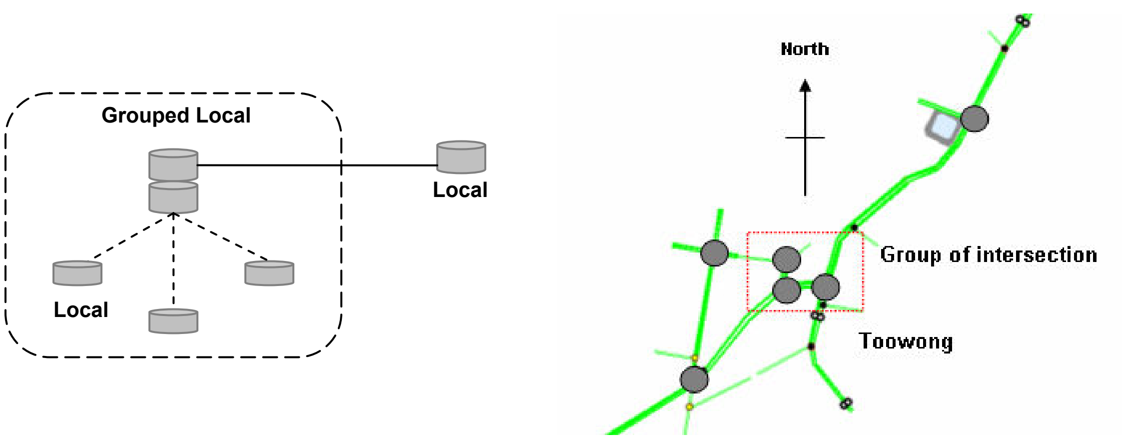


Figure 2: Group of Local Control Agents

3 Signal Timing Optimisation Model

Over the years, traffic engineers have used several methods for designing pre-timed isolated signals. For example, the Homburger and Kell's method utilised traffic volumes as the basis for allocating times to approaches with the constraint of keeping off-peak cycles as short as possible (e.g. 40 to 60 seconds) (Homburger, 1996). The highway capacity method, on the other hand, determines the traffic signal cycle length based on the capacity of lane groups.

The well known method which influenced the Australian and U.K. signal design practice is the Webster's method (Webster, 1962), which was introduced to obtain an optimum cycle time that produces the minimum delays to vehicles. The Webster method proposed the use of a "lost time" parameter (which represents the time lost before vehicles start to move) and the saturation flow (which is the maximum rate of discharge). The method has since become a well known technique to design signal timings for isolated intersections both in Australia and overseas (Kristy, 2003, Garber, 2001). Akcelik introduced several changes to the original formulation (Akcelik, 1984). The main modification included changing the core concept of 'phase-related' methods to 'movement-related' techniques. Consequently, an important aspect of this change was the use of 'movement lost time' instead of 'phase lost time' which led to a definition of the intersection's lost time as 'the sum of critical movement lost times' rather than 'the sum of phase lost times'. This new approach also facilitated a clearer understanding of the relationships between movement and signal phasing characteristics.

4 Movement-based Signal Timing Optimisation Model

This study applies the movement-related method proposed by Akcelik to determine dynamic traffic signal cycle times (e.g. every cycle) for use in the agent-based traffic control systems. A brief explanation of the approach, as implemented in this study, is presented next. Although these formulations are well established and reported in the literature, they are presented briefly in this paper for completeness. A full discussion of the technique is outside the scope of this paper but the reader is referred to (Akcelik, 1984, Nigarnjanagool, 2005) for a detailed explanation of the method.

4.1 Movement characteristics

The basic movement characteristics are illustrated in Figure 3. The main movement parameters are saturation flow, effective green time and lost time. The model assumes that when the signal changes to green, the flow across the stop line increases rapidly to saturation flow which remains constant until either the queue is exhausted or the green period ends. As indicated by the dotted line in Figure 3, the model replaces the actual departure flow curve by a rectangle of equal area whose height is saturation flow (s) and width is the effective green time (g). The start and end times of the effective green period for a movement are best defined with reference to phase change times. Start lag (a) is defined as the sum of the movement inter-green time (I) and start loss, and end lag (b) is defined simply as the end gain. The difference between the start and end lag time is defined as movement lost time (ℓ). It should be noted that the movement inter-green time is the inter-green time of the starting phase of the movement. Some of the equations relevant to describing movement-based signal optimisation are provided in Equations 1-10 below.

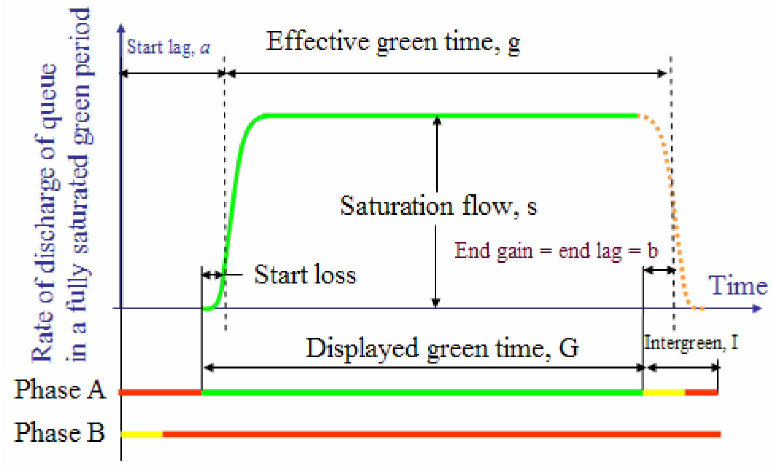


Figure 3: Basic movement characteristics

$$\ell = a - b \quad \text{....(1)}$$

From Figure 3, the relationship between the displayed green time (G) and the effective green time (g) is

$$g + \ell = G + I \quad \text{....(2)}$$

The sum of all phase inter-green and green times is the cycle time (c):

$$c = \sum (I + G) \quad \text{....(3)}$$

From Equation 2 and Equation 3, the similar relationship holds for movement parameters is

$$c = \sum (g + \ell) \quad \text{....(4)}$$

where the summation is for critical movements.

The movements which determine the capacity and timing requirements of the intersection are called critical movements. Sufficient time must be allocated to each critical movement to meet its capacity requirements in order to give all movements sufficient capacity. The technique to identify the critical movements is explained in Section 4.2.

The time allocated to a movement is the sum of effective green time (g) and lost time (ℓ) and is given by:

$$t = g + \ell = I + G \quad \text{....(5)}$$

The required movement times can be calculated from

$$t = 100\, u + \ell \quad \text{....(6)}$$

100 is the first estimate of cycle time and u is the required green time ratio which is calculated to achieve maximum acceptable (practical) degree of saturation (x_p), and is given by:

$$u = y / x_p \quad \text{....(7)}$$

Movement flow ratio (y) is the ratio of arrival flow (q) to saturation flow (s) is given by:

$$y = q / s \quad \text{..} (8)$$

It must be noted that the movement time calculated from Equation 6 must satisfy the sum of fixed minimum effective green time and lost time as:

$$t \geq g_m + \ell \quad \text{..} (9)$$

The minimum displayed green time (G_m), therefore, can be calculated from the relationship

$$g_m + \ell = G_m + I \quad \text{..} (10)$$

4.2 Critical movement identification

The identification method is based on the comparison of the required movement time (t) values. If all movements were non-overlap movements, there would be one critical movement per phase. This would be the movement which requires the longest movement time in the phase. For the overlap movements, their movement time includes the green and inter-green times of all phases during which it has right of way. This method requires a phase-movement matrix as shown in Table 1, and a critical movement search diagram as illustrated in Figure 4. In the diagram, the nodes correspond to phase change events, and the links to movements.

Table 1: Phase-movement matrix

Movement	Starting Phase	Terminating Phase
1	A	C
2	A	B
3	B	C
4	C	A

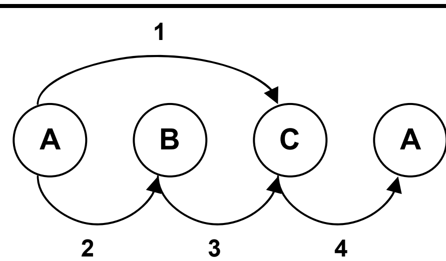


Figure 4: Critical movement search diagram

Then, intersection lost time (L), intersection green time ratio (U) and intersection flow rate (Y) which are the summation of respective critical movement parameters are calculated.

4.3 Cycle and phase timing determination

The optimum cycle time, which minimises a performance measure that is a function of total delays and number of stops for all critical movements at an isolated intersection, is calculated from the following formula:

$$c_o = (1.4+k)\, L + 6 / (1 - Y) \quad \text{..} (11)$$

where k = K/100 is the stop penalty parameter. This study selected a value of k which is commonly used to minimise cost (k = 0.2). The practical cycle time which ensures that the degrees of saturation of all movements are below specified maximum acceptable degree of saturation, $x < x_p$, is calculated from the following formula:

$$c_p = L / (1 - U) \qquad (12)$$

This paper selected a default value of x_p=0.9.

For a given cycle time (c) and total available green time ($c - L$), the available green time can be distributed to critical movements according to the formula:

$$g = u\,(c - L) / U \qquad (13)$$

For non-overlap movements, where the overlap movement is critical, the green time for non-overlap movements can be calculated by treating the critical movement time as a sub-cycle time, $c^* = g_c + \ell_c$, where g_c and ℓ_c are the critical movement green and lost times. Then available green time is ($c^* - L^*$), where L^* is the sum of non-overlap movement lost times. The green time distributed to non-overlap movement is as follows:

$$g = u\,(c^* - L^*) / U^* \qquad (14)$$

By modifying Equation 5, the displayed green time for a phase can be calculated from

$$G = (g + \ell) - I \qquad (15)$$

where ($g + \ell$) is the time allocated to a movement which receives right of way during the phase only, and I is the inter-green time of that phase.

Having introduced the reader to the theoretical background of the algorithm, the next section of this paper presents the application of this algorithm within microscopic traffic simulation to evaluate its performance under a variety of traffic conditions and scenarios.

5 Model Development

This section presents the model development, data requirements and traffic modelling tasks needed to set up the simulator and interface it to the signal optimisation model.

5.1 Model development and data requirements

This study used Brisbane's Western Corridor traffic simulation model as a test-bed for evaluating the performance of the optimisation model. A number of signalised intersections in the Toowong are were selected as shown in Figure 5.

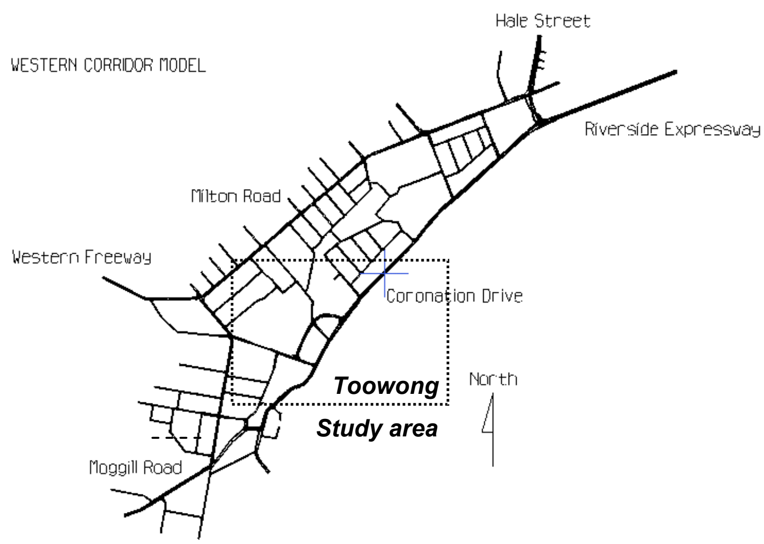


Figure 5: Schematic of the Brisbane Western Corridor model and selected study area

Microscopic simulation is characterised by a high level of modelling detail. The accuracy of the model will depend on the availability and quality of the input data. The following data was collected and used for model development in this study:

5.1.1 Network layout

Digitised maps (DXF format) and raster images showing locations of both signalised and un-signalised intersections, possible turning movements for each intersection, recommended turning speeds for each movement, visibility distances at junctions, presence of Stop or Yield signs and detector positioning were required. For each section, the road centrelines, number of lanes in each section of road, width of each lane, reserved lanes (where entry is allowed only for certain vehicle types), restrictions on lane changes (solid line markings), maximum speed for each section (or each lane if necessary), capacity (vehicles per hour for use in cost functions for the calculation of shortest paths), visibility distances, length of each section, slope and lane changing distances were also required.

5.1.2 Traffic demand data and signal optimisation model inputs

In AIMSUN, traffic demand data can be defined either by the combination of traffic flows at input sections and turning proportions at intersections or by Origin-Destination (O-D) matrices. For this study, route choice modelling was disabled to remove the influence of driver route choice and retain the same traffic conditions for all experiments. Therefore, traffic demand on the network was represented by traffic states (traffic volumes entering to sections and turning proportions at the intersections). Normal distributions were assumed.

As was discussed previously, basic movement characteristics including inter-green (I), minimum displayed green time (G_m), saturation flow (s) and practical degree of saturation (x_p) are all required as input to the model. Default values were used for these inputs (Akcelik, 1984). A value of 0.9 was selected as practical degree of saturation for all movements of all intersections. The traffic signal optimisation model also requires the input of traffic volumes for the network. These were obtained from simulated vehicle loop detectors located at the stop-line of every signalised intersection. The algorithm also implements a traffic volume prediction model to optimise signal timings for the next cycle. These were predicted using the average traffic volumes from three previous cycles. The traffic demand used in this study was for the morning peak starting from 6:45 A.M. to 8:45 A.M.

5.2 Simulation Parameters

The use of traffic simulation requires that several simulation parameters are specified. These include important parameters related to car following and lane changing algorithms and other global parameters that control the conduct of the experiments. Reaction time, simulation step per second and reaction time at stop were assigned value of 1, 1 and 1.35 seconds, respectively. The simulation was run for five replications to ensure statistical reliability. The simulation time was two hours with a 30- minute warm-up period to populate the network with vehicles and allow traffic conditions to stabilise before collection of statistics on performance.

6 Evaluation Using Microscopic Traffic Simulation

The signal optimisation algorithm (referred to in the remainder of this paper as dynamic signal optimisation model) was evaluated in two different scenarios which aimed to adjust the cycle time dynamically (every cycle) and compared the results with outputs from optimal fixed cycle times. The optimisation logic of the control strategy was based on the calculation of optimal signal timing and green time allocation for each movement at the end of every cycle by using predicted demands calculated based on the average volumes from previous cycles. First the algorithm was used to control isolated intersections assuming that the effects of upstream and downstream traffic from adjacent intersections are negligible. Second, the algorithm was applied to control seven signalised intersections in the study area using the signal optimisation model and traffic signal coordination.

6.1 Scenario 1: isolated junction without effects from adjacent intersections

The Toowong junction shown in Figure 6 was selected since it had a relatively large traffic volume and one of its approaches is a short link (approach number 1). During the morning peak, approaches 1 and 3 feed substantial traffic through the intersection towards the city.

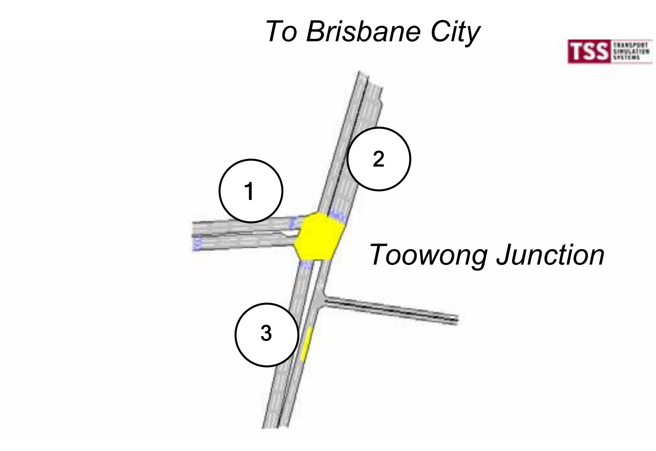


Figure 6: Toowong junction

The performance of the dynamic signal optimisation model was compared to an optimum fixed cycle time setting obtained by aaSIDRA. The comparative results are shown in Figure 7 where the dotted and solid lines represent dynamic and optimum fixed time control, respectively. The simulations were conducted for two hours with v/c = 1.0 and the majority of traffic was moving from approach 1 to city (left turning traffic from 1 to 2). The cycle time was updated every cycle based on traffic volumes. The plots shown in Figure 7 show how the dynamic cycle control algorithm increased the throughput across the intersection, especially during medium to heavy conditions, when compared to fixed time control. This demonstrates how simple improvements to the underlying core algorithms which control the operation of traffic signal systems can produce substantial benefits without the need to spend large amounts of funds to build or construct new roads to improve capacity.

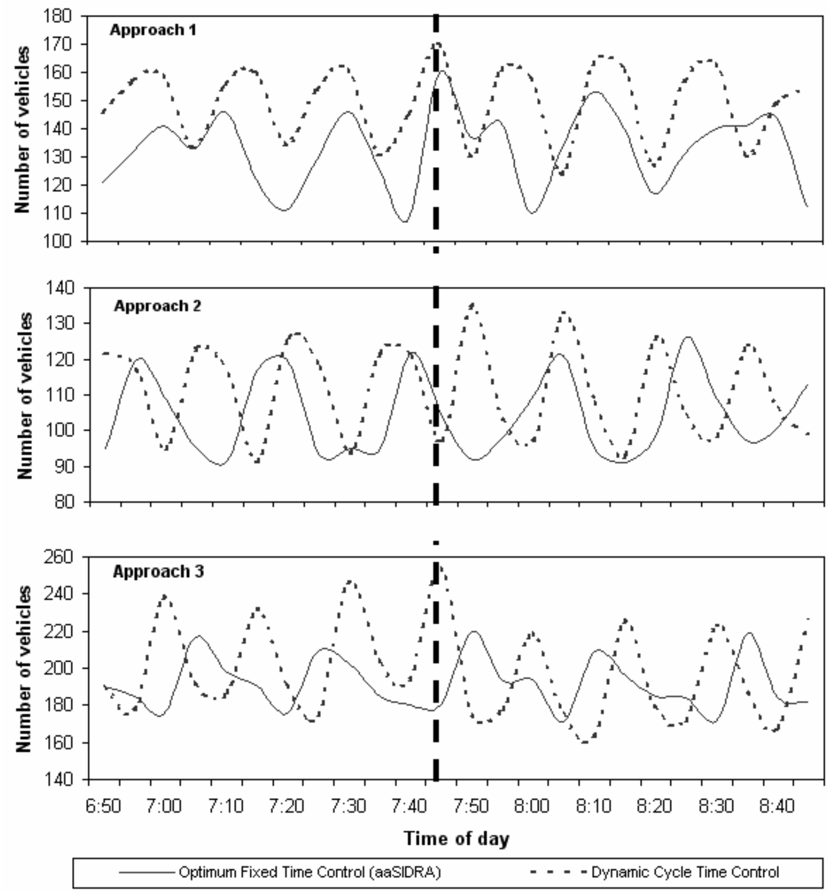
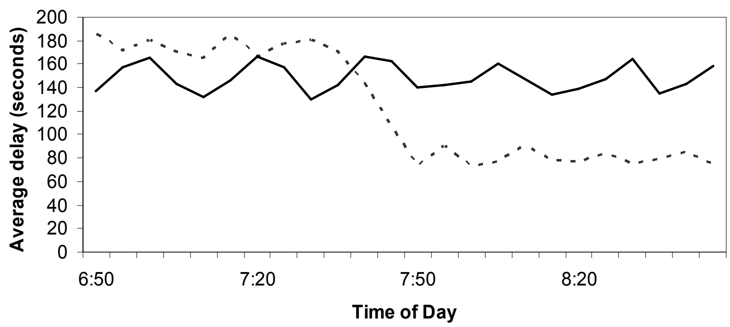


Figure 7: Comparative results of dynamic and optimum fixed cycle times by approach

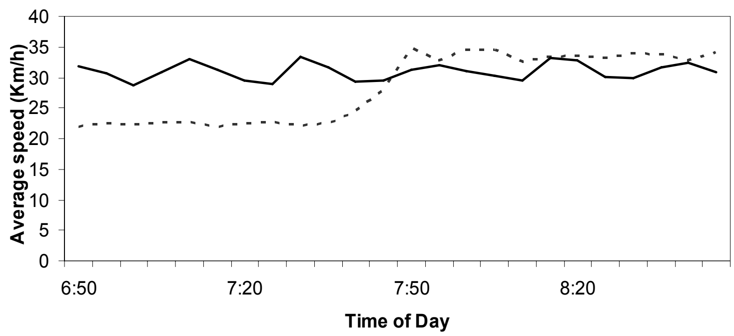
Figure 8 shows the average delay, average speed and average number of stops. The figure shows that dynamic cycle time control was superior to optimum fixed time control as traffic volumes started to increase (e.g. around 7:45 A.M.). In Figure 8(a), the dynamic cycle time control reduced delays by approximately 20 percent and produced higher average speeds as shown in Figure 8(b).

At the beginning of simulation, there were few vehicles entering the junction. Under fixed time control most of the vehicles were able to proceed through the junction without queuing. Since the dynamic cycle time control attempts to optimise cycle time based on predicted traffic volumes, the cycle times during the beginning periods resulted in a larger number of stops as shown in Figure 8(c). The dynamic cycle time control initially attempted to provide sufficient time for the major movements to pass through the junction. This clearly resulted in queues forming at the other approaches with less demand, where some of the vehicles on the minor approaches waited for more than a cycle to clear the intersection.

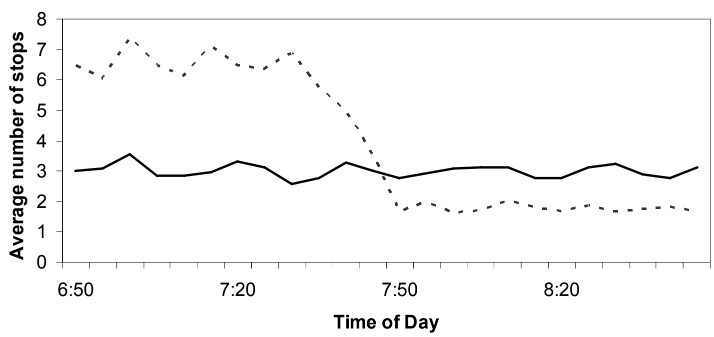
The number of stops under dynamic cycle time control was reduced to less than two. In fact, it was observed that the controller was reducing the queue length by a small amount every cycle by adjusting the cycle time and phase proportions to suit the new traffic demands for the next cycle. In other words, while fixed time control was operating the intersection using the same cycle and phase time proportions, the dynamic cycle time control was adjusting the time and proportions every cycle to suit the current demand for all movements as shown in Figures 9 and Figure 10.



a. Comparison of average delay time (seconds)



b. Comparison of average speed (Km/h)



c. Comparison of average number of stops

----- Optimal Fixed Time Control (aaSIDRA) - - - - Dynamic Cycle Time

Figure 8: Comparative results of dynamic and optimum fixed cycle time control

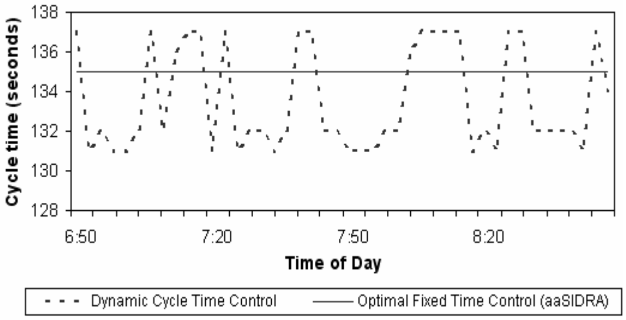


Figure 9: Cycle time variability

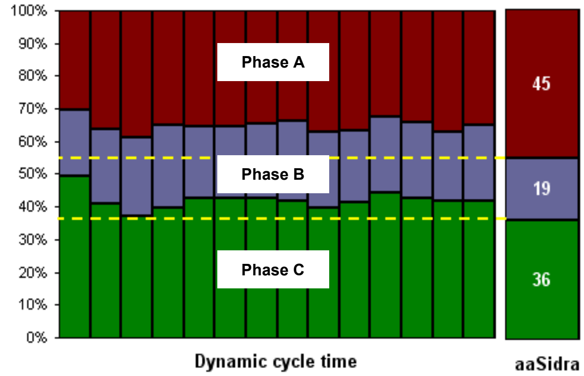
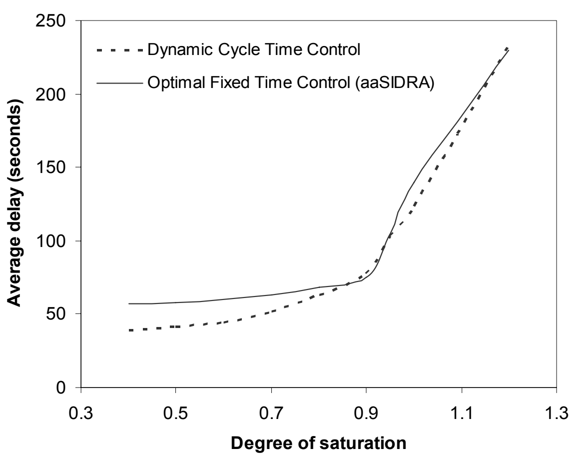
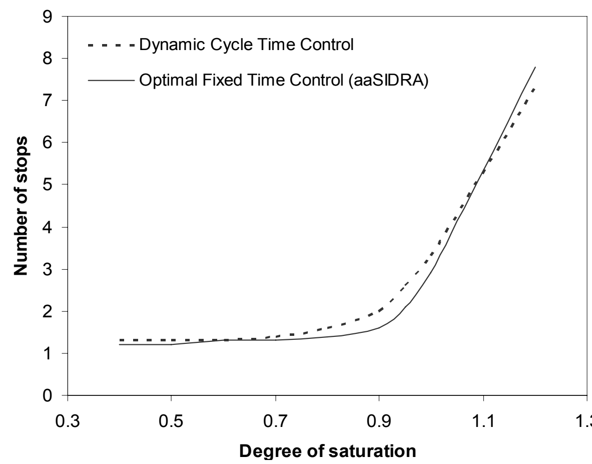


Figure 10: Phase time proportions for phases A, B and C

Figure 11 shows some sensitivity analysis results for the two control strategies when the degree of saturation was varied from light traffic (*v/c*= 0.4) to over saturated traffic conditions (*v/c* > 1.0). These figures show that dynamic cycle control which uses real-time traffic volumes to determine signal timings produced less delay in light traffic condition (*v/c* = 0.4 to 0.7). However, both control strategies performed in a similar manner when traffic volumes approached capacity.



a. Average delay for variable degrees of saturation



b. Average number of stops for variable degrees of saturation

Figure 11: Sensitivity analysis results

6.2 Scenario 2: commuting corridor with effects from adjacent intersections

In this scenario, the algorithm was used to operate seven coordinated signalised intersections in the Toowong traffic network shown in Figure 12. Among these intersections, three were classified as a group of intersections which meant that the common cycle time generated within the group will be assigned to all members of the group. The rest of the intersections were operated as a single controller since they are located far from each other. More than 5,000 vehicles per hour use this route during morning peak. The simulation was run for two hours with morning peak traffic state (section input and turning proportions at intersections).

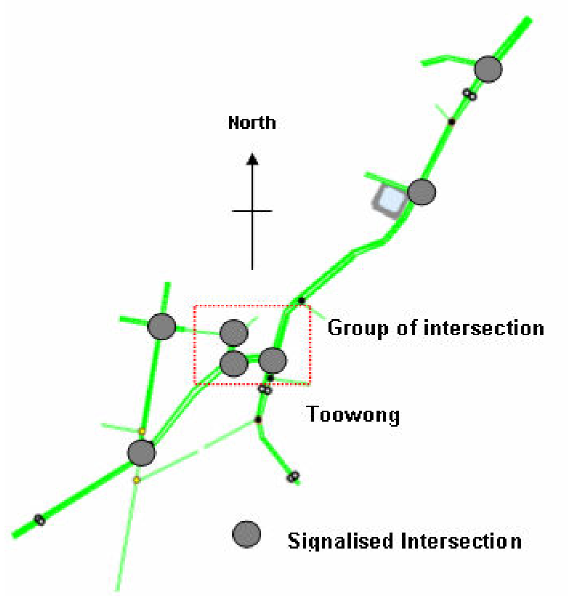


Figure 12: Commuting of Toowong

In this scenario, the algorithm was evaluated at intersections where there are effects from upstream and downstream intersections, which is more realistic and common than Scenario 1. This meant that the queue formations from downstream intersections could create spill-back or lane blocking conditions, which may result in unused green time and unnecessary delays. The dynamic cycle time control addressed this by considering measurements from loop detectors which helped detect whether traffic was passing through the intersection. The algorithm then assigned the minimum green time for that particular movement until conditions changed.

Figure 13 illustrates the coordination between two adjacent intersections. The arrows represent the direction of main flow inbound and outbound to the City through two major intersections (548 and 535). The coordination of this group of intersections is important to avoid queue spill-back and lane-blocking at intersections. The results of applying the optimisation model are shown in Figure 14. The coordinated dynamic control produced the lowest average delay per vehicle (seconds) at both intersections, whereas the fixed time control produced much larger values during the peak period (7:00 A.M. – 8:00 A.M.). The coordinated dynamic control approach provided the optimal cycle times and offsets between these two adjacent intersections in order to accommodated the predominant traffic volume. Although offsets were also used with the fixed time approach, the values were constant which meant that changes in traffic volumes cannot be accommodated. Cycle times in some periods might not be adequate for the actual traffic volume and provided insufficient green time which resulted in queue spill-back and lane-blocking at upstream intersections.

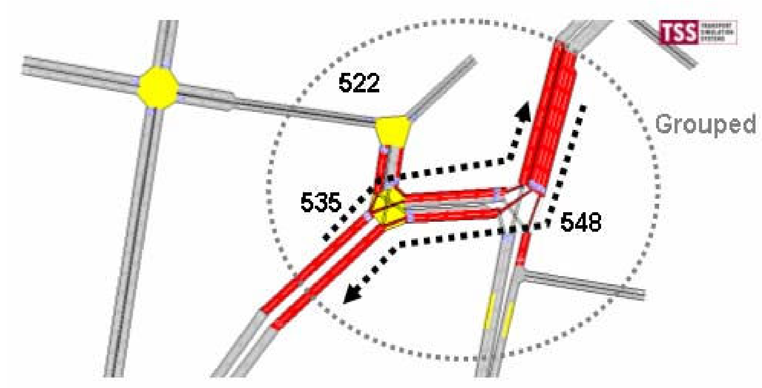
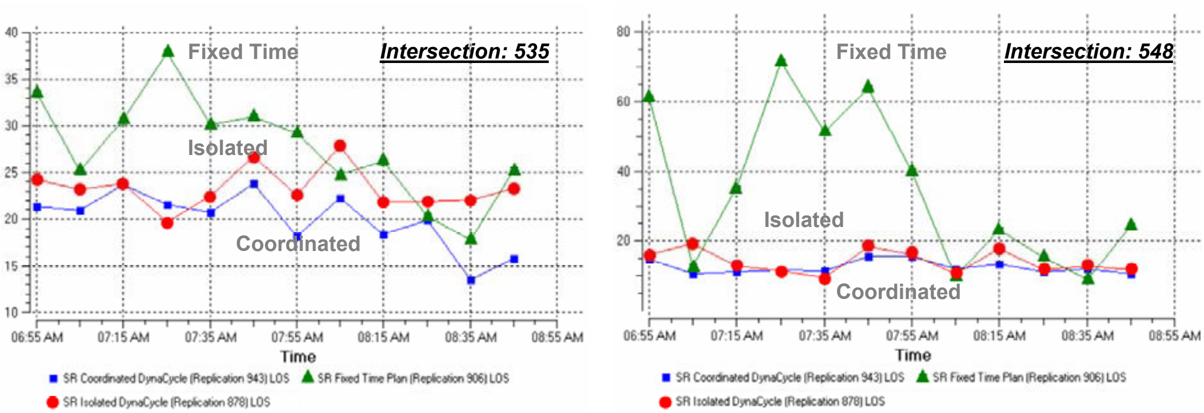


Figure 13: Group of three coordinated intersections (522, 535 and 548)



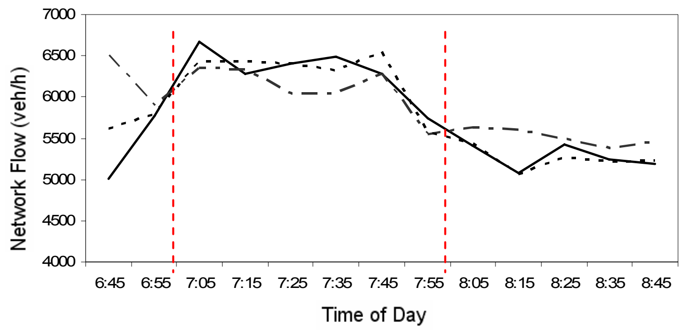
a. Average delay per vehicle (seconds) at intersection 535

b. Average delay per vehicle (seconds) at intersection 548

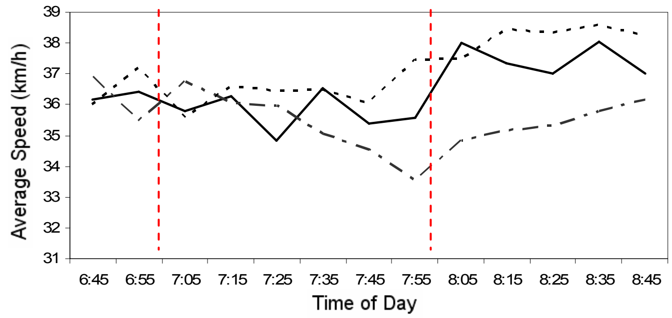
Figure 14: Time series of the intersections level of service

The comparative results of three control strategies including optimal fixed time plan, dynamic cycle time control and coordinated control are shown in Figure 15. Five replications were run for statistical reliability and the periodic statistical results were recorded every 10 minutes. From figure 15 (a), the peak hour can be considered as the period of 7:00 A.M. to 8:00 A.M. During these heavy traffic conditions (after 7:00 AM) the network was filled by a large number of vehicle and the traffic volume approached capacity. The coordinated dynamic control produced better results in terms of delays as shown in Figure 15 (c). The isolated dynamic control approach performed worst under heavy traffic conditions probably because it did not take into account influences from other intersections in the network.

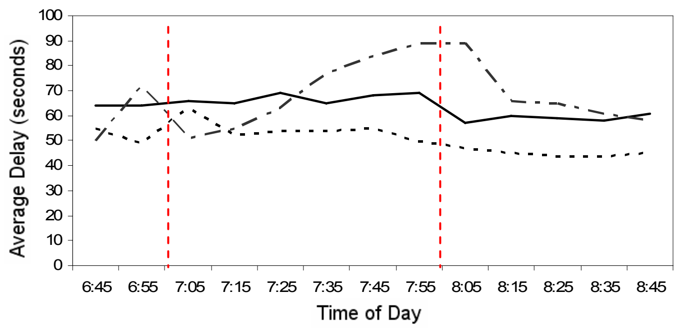
During medium and light traffic conditions (after 8.00 A.M.), the coordinated dynamic control was aiming to provide sufficient time for all movements which resulted in delays on the approaches with less demand. During medium traffic conditions the dynamic cycle time control performed better than fixed time plans. The results presented in Figure 15 also show that average speeds, delays and number of stops for the dynamic control scenario were superior to fixed time control (e.g. delays were reduced by more than 10 percent). It should be noted here that the improvements were not better than the first scenario due to the effects from upstream and downstream intersections.



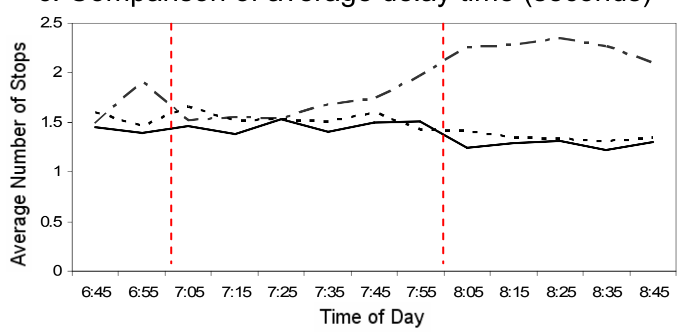
a. Comparison of network flow (veh/h)



b. Comparison of average speed (km/h)



c. Comparison of average delay time (seconds)



d. Comparison of number of stop

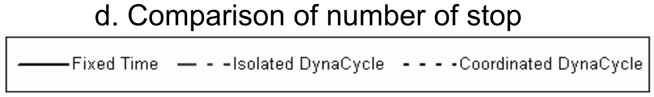


Figure 15: Comparative results of three control strategies (fixed time plan, isolated dynamic cycle time and coordinated dynamic cycle time)

One of the unique features of AIMSUN NG is that it provides a decision table for comparison purposes based on weighted mean calculations (Table 2). Each performance measure is assigned a weight (positive where higher values of the measure are better e.g. speed and negative where lower values of the measure are better e.g. travel time). All measures were given a default weight of 0.167 in this study. AIMSUN NG then provides an aggregate weighted index where higher values represent better performance. From the simulator results, the coordinated dynamic control received the highest aggregated weighted index (0.68 compared to 0.43 for the fixed time control) which represents an improvement of 58 percent over fixed time control.

Table 2: AIMSUN NG decision table

Decision Table

	Weight	Coordinated DynaCycle (966)	Fixed Time (961)	Isolated DynaCycle (960)
Speed	0.167	37.1777	36.428	35.4788
Flow	0.167	5796	5769	5889
Density	0.167	12.7961	13.7001	15.6566
Travel Time	-0.167	111	126	134
Delay Time	-0.167	49	64	72
# Stops	-0.167	1.46454	1.3923	1.91229
Aggregated weighted index		0.682374	0.429256	0.334

7 Conclusions and Future Research Directions

The aim of this work was to demonstrate the benefits of agent algorithms in optimising traffic signal control systems using traffic simulation. The dynamic cycle time control introduced in this paper is based on existing and well established fixed cycle time optimisation techniques. The study evaluated the performance of the algorithm and its potential for optimising network-wide traffic conditions based on dynamic demand data provided by loop detectors. The dynamic control strategy was tested and evaluated using the AIMSUN NG traffic simulator. The optimisation logic of the control strategy was based on the calculation of optimal signal timing and green time allocation for each movement at the end of every cycle by using predicted demands calculated based on the average volumes from previous cycles. The application was evaluated in two different scenarios. In the first scenario, an isolated intersection which operated without influences from downstream intersections was investigated. In the second scenario, the algorithm was tested on seven signalised intersections. Results for both scenarios showed that the performance of the coordinated dynamic cycle control was superior to fixed time control and provided better throughput across the intersections. The performance of the dynamic control strategy started to suffer under heavy traffic conditions but remained superior to fixed time control. In addition, by applying the linear optimisation model for coordination of intersections, the average delay per vehicle at the intersection was reduced. Results obtained from the simulator showed that the dynamic signal optimisation approach has the potential to produce an improvement of 58 percent over the fixed cycle time approach based on an aggregated weighted index which considers speed, flow, density, travel time, delay and number of stops. Current research efforts are focused on extending the evaluation framework by testing larger and different network layouts.

8 References

AATRAFFIC (2004) Introducing aaSIDRA 2.1 (http://www.aatraffic.com/downloads/Introducing_aaSIDRA.pdf).

AKCELIK, R. (1984) Traffic Signal: Capacity and Timing Analysis. *Australian Road Research Report, Australia,* N0. 123.

EROL, K., LEVY, R. AND WENTWORTH, J. (1998) Application of Agent Technology to Traffic Simulation. *Complex Systems, Intelligent Systems and Interfaces, Nimes, France.*

GARBER, N. J., AND HOEL, L. A. (2001) *Traffic and Highway Engineering,* USA, Books/Cole Thomson Learing.

HOMBURGER, W. S. (1996) *Fundamentals of Traffic Engineering,* Berkeley, USA, University of California at Berkeley.

KRISTY, C. J. A. L., B. L (2003) *Transportation Engineering: An Introduction,* New Jersey, USA, Prentice Hall.

NIGARNJANAGOOL, S. A. D., H (2005) Evaluation of A Dynamic Signal Optimisation Control Using Traffic Simulation. *Journal of International Association of Traffic and Safety Sciences,* Vol 29, No 1**,** 22-30.

PILLAI, R. S., RATHI, A. K. AND COHEN, S. L. (1998) A restricted branch-and-bound approach for generating maximum bandwidth signal timing plans for traffic network. *Transportation Research Part B,* Vol 32, No 8**,** 517-529.

TSS (2006) AIMSUN 5.0 Microsimulator User Manual. Transport Simulation Systems.

VAN KITWIJK, R. V. A. V. K., P. (2002) Coordination of Traffic Management Instruments Using Agent Technology. *Transportation Research Part C,* Vol 10**,** 455-471.

WALLACE, C. E., COURAGE, K. G., REAVES, D. P., SHOENE, G. W., EULER AND WILBUR, A. (1998) *TRANSYT-7F User' Manual Release 6 Technical Report, Prepared for FHWA by the Transportation Research Center*, University of Florida, Gainesville.

WEBSTER, F. V. A. C., B. M. (1962) *Traffic Signal, Road research technical paper 56,* London, UK, Her Majesty's Stationary Office.