13th COTA International Conference of Transportation Professionals (CICTP 2013)

Development of a Dynamic Control Model for Oversaturated Arterial Corridor

Gang Liu, Pengfei Li, Tony Z. Qiu*, Xu Han

Department of Civil and Environmental Engineering, University of Alberta, 9105 116th St, Edmonton, T6G 2W2, Canada

Abstract

In oversaturated condition, traffic queues that are unavoidable persist from cycle to cycle and the queue management capability is a critical feature in signal control. Inappropriate signal control and queue management strategies may lead to spillover and starvation with serious disruptive consequences. Compared with minimizing delay and stops in uncongested condition, the control policy in oversaturated condition comes to maximize system productivity and fully utilize storage capacity. This paper describes one dynamic control model to adjust the green phase duration, offset and cycle length by considering the saturation level, the queue length management and propagation along the arterial corridor. A VISSIM simulation test of one arterial corridor in City of Edmonton is used to validate the efficiency of the proposed control model.

© 2013 The Authors. Published by Elsevier Ltd.
Selection and peer-review under responsibility of Chinese Overseas Transportation Association (COTA).

Keywords: Arterial corridor; Oversaturated Condition, Dynamic control

1. Introduction

High levels of traffic congestion during peak periods are regular in busy arterials of major metropolitan areas, because the traffic demand approaches or exceeds the capacity of the arterial network. The realm of congested condition has been divided into two major categories: saturated and oversaturated conditions (Lee, et al., 1975). The saturation condition exists when queue has formed and/or is growing, but delay effect is local and no other intersection performance is affected by this queue. Generally, the oversaturated condition exists when the demand exceeds the capacity. An oversaturated network exists when the system is overloaded with heavy demand which exceeds the total capacity of the network. Oversaturated condition can produce unstable queues that block intersections and reduce capacity when it is most needed, which result in suboptimal utilization of the available

* Corresponding author. Tel.: +1-780-492-1906; fax: +1-780-492-0249.
E-mail address: zhijunqiu@ualberta.ca
facilities and degraded operational of efficiency. For example, it is already know that spillovers can cause the creation of additional congestion, lead to gridlocks on networks with closed loops, and lead the entire system to restricted mobility (Pignataro et al., 1978).

The identification of oversaturation and understanding of the characteristics are the prerequisite to manage oversaturated traffic signal systems. However, using the precise definition based on demand/capacity ratio is not an easy task in the real world by using the current data collection system. Previous research had developed different measures to identify and quantify the oversaturation, such as the queue development (Gazis, 1964; Abu-Lebdeh & Benekohal, 2003) and green phase utilization (Luyanda et al., 2003). Recently, Wu, Liu, & Gettman (2010) proposed quantifiable measures of the oversaturation by using high-resolution traffic signal data. The temporal detrimental effect was characterized by a residual queue at the end of a cycle and the spatial detrimental effect was referred to a spillover from the downstream intersection to upstream. Geroliminis & Skabardonis (2011) proposed a methodology for identifying queue spillovers by using data from conventional surveillance systems. The key idea was that queues discharge rate was smaller than the saturation flow when spillovers from a downstream link blocked vehicle departures from the upstream signal line. The abovementioned research makes a great contribution to developing signal control strategies under oversaturated condition, as different oversaturation types and levels may require different mitigation strategies.

In oversaturated condition, typical traffic control strategies, such as SIGSET (Allsop, 1971) and MAXBAND (Littke, 1966), do not work efficiently, since the control objectives need to be decidedly different when mobility is restricted. For example, delay minimization strategy that provides user-optimal delay minimization in uncongested condition can in some cases work not in favor of the minimizing total delay when system becomes oversaturated (Longley, 1968). Early studies, such as Gazis & Potts (1963), Gazis (1964), Gordon (1969), Singh & Tamura (1977), Pignataro et al. (1978), ITE (1988) and Rathi (1988), suggested that the objective function for signal control strategies in oversaturated condition should no longer be to minimize delay or some closely related parameter. Instead, the signal plans should be timed such that every green second should be serving traffic at its maximum flow rate. As the primary congestion is unavoidable, the control strategy should be aimed at avoiding or postponing the onset of secondary congestion. Mitigation of oversaturated condition also involves trade-offs between the storage of queues from the oversaturated routes to other less saturated routes.

It is, generally, known that queue dynamic information is vital in determining timing plans under oversaturated condition and queues should be taken into account either in the objective function or in the constraints. Over the past several decades, a large body of literature has been devoted on this vital issue. A number of fixed-time control strategies were established based on the anticipated oversaturated condition during a pre-set time. However, the anticipated traffic patterns, particularly in oversaturated condition, are seldom realized in the real-world exactly as they were planned. By the nature that fixed-time control strategies are based on historical data, they cannot adjust the signal timing dynamically with the queues and are not suitable to handle oversaturated condition (Papageorgiou et al., 2003). The need and trend are clearly toward real-time adaptive strategies, which are capable of optimizing signal control to satisfy the current traffic demand pattern. To take into account the complex interactions between traffic state evolution and key signal control parameters, a large number of researchers (Gartner et al., 1983; Henry et al., 1983; Boillot et al., 1992; Mirchandani & Head, 2001; Lo & Chow, 2004; Cai et al., 2009; He et al., 2012) have proposed the simulation-based dynamic approach. Recently, Li (2010) and Liu & Chang (2011) proposed the lane-based dynamic control model, which took into account complex flow interactions among different lane groups.

Parlis & Recker (2004) classified the simulation-based dynamic approach into three generalized categories: 1) store-and-forward models (SFM), 2) dispersion-and-store models (DSM), and 3) kinematic wave models (KWM). DSM has been empirically validated in several urban areas and is generally considered to be able to represent the realistic traffic flow in signalized networks, relatively. However, the accurate calibration of the dispersion model parameters is critical (Rakha & Farzaneh, 2006). KWM can describe the traffic flow dynamics more accurate; however, it seems it has a limited significance in interrupted traffic network flow, in contrast to the uninterrupted
freeway traffic flow (Parlis & Recker, 2004). The major disadvantage of KWM is that the creation of large dimensional state vectors results in high computational requirements. Hu et al. (2011) mentioned that it was hard for these approaches to be widely accepted and implemented by traffic engineers because of the complexity, although most of the methods were technically reasonable. Therefore, they proposed a simple forward-backward procedure to manage oversaturation.

SFM enables the mathematical description of the traffic flow process without use of discrete variables, so it allows for efficient optimization and control methods in real-time control for large-scale network. It has been used in various works notably for signal control (Diakaki et al., 2002, 2003; Aboudolas et al., 2009, 2010). SFM is unable to provide more accurate representation of traffic dynamics; however, oversaturation can be treated as an “emergency” and the best way to deal with this event is to adjust current signal timing to relieve traffic congestion as soon as possible (Hu et al., 2011). This paper proposes one SFM-based dynamic control model to minimize the risk of oversaturation and the spillover of link queues. The phase duration is decided at regular time intervals and meters traffic at intersections servicing oversaturated approaches to stabilize queue length. The coordination of signals along the arterial corridor is determined by considering the queue length ratio control and an ideal offset. One single cycle time is considered here to increase the corridor capacity as much as necessary to control the saturation level along the corridor. Finally, A VISSIM simulation test of one arterial corridor in City of Edmonton is used to validate the efficiency of the proposed control model.

2. Model Development

2.1. Phase Duration Control

The SFM allows only for split optimization, while cycle time and offsets must be delivered by other control algorithms. The signal phase duration meters traffic at oversaturated approaches to control and stabilize queue lengths over time. The following describes the traffic flows and queues on all links to obtain a measure of traffic conditions. Considering two intersections connected by one link (Figure 1), the traffic flow dynamic on the link is expressed as:

\[
Q_{(i-1,i)}(k+1) = Q_{(i-1,i)}(k) + T[I_{(i-1,i)}(k) - O_{(i-1,i)}(k) - S_{(i-1,i)}(k) + D_{(i-1,i)}(k)]
\]

Where, \(Q_{(i-1,i)}(k)\) is the number of vehicles within link \((i-1,i)\) at the time \(kT\); \(I_{(i-1,i)}(k)\) and \(O_{(i-1,i)}(k)\) are the inflow and outflow, respectively, in the sample period \([kT, (k+1)T]\); \(T\) is the discrete-time step and \(k = 0, 1, \ldots\) is the discrete-time index; \(S_{(i-1,i)}(k)\) and \(D_{(i-1,i)}(k)\) are the exit flow and the demand within the link, respectively. The SFM assumes that vehicles entering a link are either stored at the end of this link in case of red signal, or further forwarded to downstream links with spare storage capacity. Therefore, \(Q_{(i-1,i)}(k)\) also represents the number of vehicles in the queue within link \((i-1,i)\) at time \(kT\).

![Fig. 1. A link Connecting Two Intersections \(i-1\) and \(i\)](image)
The outflow \( O_{(i\rightarrow j)}(k) \) equals to the saturation flow \( S_{(i\rightarrow j)} \) if the link has the right of way, and equals to zero otherwise. The major characteristic of SFM is a critical simplification that the discrete-time step \( T \) is equal to \( C \), which is the cycle length (Aboudolas et al., 2009). It results in a continuous average outflow, although the model is not aware of short-term queue oscillations due to the green-red switching within a cycle. The average outflow for each period is given by

\[
O_{(i\rightarrow j)}(k) = (G_{(i\rightarrow j)}(k) - l_{(i\rightarrow j)}) \cdot S_{(i\rightarrow j)} / C
\]  

(2)

where \( G_{(i\rightarrow j)}(k) \) is the green phase duration and \( l_{(i\rightarrow j)} \) is the lost time.

The inflow \( I_{(i\rightarrow j)}(k) \) is given by

\[
I_{(i\rightarrow j)}(k) = \frac{(G_{(i\rightarrow j)}(k) - l_{(i\rightarrow j)}) \cdot S_{(i\rightarrow j)}}{C} + \sum t_{w} \cdot \frac{C - G_{(i\rightarrow j)}(k) - l_{(i\rightarrow j)}}{C} \cdot S_{(i\rightarrow j)}
\]  

(3)

where \( t_{w} \) is the turning rates from the links that enter the intersection \( i-1 \).

Replacing Equations (3) and (2) to (1), we can get

\[
Q_{(i\rightarrow j)}(k + 1) = Q_{(i\rightarrow j)}(k) + T\left[\frac{(G_{(i\rightarrow j)}(k) - l_{(i\rightarrow j)}) \cdot S_{(i\rightarrow j)}}{C} + \sum t_{w} \cdot \frac{C - G_{(i\rightarrow j)}(k) - l_{(i\rightarrow j)}}{C} \cdot S_{(i\rightarrow j)}
\]

\[
-(G_{(i\rightarrow j)}(k) - l_{(i\rightarrow j)}) \cdot S_{(i\rightarrow j)} / C - S_{(i\rightarrow j)}(k) + D_{(i\rightarrow j)}(k)
\]

(4)

From Equation (4) it is obvious that only green phase duration is the control variable in SFM. Offset ad cycle time have no impact within the SFM and must be either fixed or updated in real-time independently. In this paper, the control model aims to maintain a stable queue, which implies that \( Q_{(i\rightarrow j)}(k + 1) = Q_{(i\rightarrow j)}(k) \). Then it reveals that the determination of the green phase duration at an intersection depends on the duration of the green phase at the upstream and downstream intersections. Thus, the computation of phase durations in the control model cannot be viewed as intersection-specific parameters.

There are other three constraints for green phase duration control. One constraint is to avoid secondary queue, as shown in Equation (5). Another constraint is that the maximum queue length certainly must not exceed the available block length, as shown in Equation (6). Equation (7) introduces the constraint of minimum green time.

\[
\frac{G_{(i\rightarrow j)}(k) - l_{(i\rightarrow j)}}{C} \cdot S_{(i\rightarrow j)} \geq Q_{(i\rightarrow j)}(k)
\]  

(5)

\[
0 \leq Q_{(i\rightarrow j)}(k) \leq Q_{(i\rightarrow j)}^{\text{max}}
\]

(6)

\[
G_{(i\rightarrow j)}(k) \geq g_{(i\rightarrow j)}^{\text{min}}
\]

(7)

where \( Q_{(i\rightarrow j)}^{\text{max}} \) is the maximum admissible queue length and \( g_{(i\rightarrow j)}^{\text{min}} \) is the minimum permissible green time.

2.2. Offset Control

According to the LWR theory (Lighthill & Whitham, 1955), shock waves are generated by the traffic signal, which cause congested conditions to develop near the stop line during the red interval and capacity conditions to occur in the period during which the queue discharges at the saturation flow rate. When the queue has dissipated, the rest of the platoon that arrives during the green time crosses the intersection stop line with no interference from the traffic signal. Figure 2 is a space-time diagram displays queue propagation under oversaturated condition. At the beginning of effective green \( T_{g}(m) \), the front of this residual queue begins to discharge at the saturation flow rate and a discharge shockwave \( \alpha \) moves propagating upstream from the stop line of intersection \( i \). At time \( T_{g}(m) \), the platoon from intersection \( i-1 \) entering the approach \( (i-1,i) \) encounters a residual queue \( Q_{(i\rightarrow j)}(k) \) at intersection \( i \). A backward-moving shock wave \( \beta \) is created by the stoppage caused by the residual queue. Therefore, subsequent entering vehicles encounter stoppage. Whether these two shock waves \( \alpha \) and
\( \omega_i \) could intersect with each other depends on the the discontinuity between the saturated discharging traffic flow at intersection \( i \) and the traffic arrival from intersection \( i-1 \). A new residual queue \( Q_{(i-1,i)}(k+1) \) is formed sometime after the start of the red light of the next cycle when the queuing shock wave \( \omega_i \) meets the traffic arrival. The shock waves and queue dynamic described above will repeat from cycle to cycle.

\[
\begin{align*}
Q_{(i-1,i)}(k) & \quad \omega_i \\
Q_{(i,i)}(k+1) & \quad \omega_f
\end{align*}
\]

**Fig. 2. Shock wave Profile and Queue Dynamic under Oversaturated Condition**

Intuitively, the offset need to be designed to control the interaction between incoming platoons and residual queues in order to use the available storage capacity and maximize throughput. It depends on the input-output flow balance and queue length control on each oversaturated approach at every cycle. As shown in Figure 2, stoppage should be avoided. In addition, queue spillover blocks intersections and starvation has the tardy arrival of traffic at the stop line, which both waste green time and should be avoided (as shown in Figure 3). One objective of the dynamic control model is to avoid the abovementioned phenomena and achieve continuous saturated flow rate and maximization of throughput. The dynamic model can control the input-out balance and queue length value at the optimal value.

**Fig. 3. Spillover and Starvation under Oversaturated Condition**
From Figure 2 we can see that if the discharged platoon from intersection \( i - 1 \) joins the tail of the downstream residual queue at the time when the tail has reached its free flow speed, there will be no stoppage and starvation (Girianna and Benekohal, 2003). This ideal signal offset allows the leading vehicle in the incoming platoon to just avoid encountering with the residual queue, yet allows it to reach the stop line one headway after the last vehicle in the residual queue discharges, thereby avoiding stoppage and starvation.

This offset can be calculated as the following equation.

\[
\phi_{\text{ideal}}(k) = \frac{L_{i-1}(i,i)}{v_{i-1}(i,i)} - \left[ \frac{v_{i-1}(i,i) + \omega_i}{\omega_i} \right] Q_{i-1}(i,i)(k)
\]

(8)

where \( L_{i-1}(i,i) \) is the link length; \( v_{i-1}(i,i) \) is the mean speed of the leading vehicle of the incoming platoon; \( r_{i-1}(i,i)(k) \) is the residual queue length ratio.

The value of \( \phi_{i-1}(i,i)(k) \) can be either negative or positive. The negative value means the green start time at intersection \( i - 1 \) is leading the green start time at intersection \( i \) and corresponds to a smaller queue length, while the positive value means the green start time at intersection \( i - 1 \) is lagging the green start time at intersection \( i \) and corresponds to a heavier queue length. We define the direction from intersection \( i - 1 \) to intersection \( i \) as the primary direction. As we can see from Equation (8) the calculated offset is related to the residual queue length ratio, it is possible to control the residual queue lengths ratio on all links at an optimized value in order to satisfy the objective of maximizing productivity. With the abovementioned analysis, the relationship of the two offsets depends on the magnitude of queue length along the primary directions. The problem comes to how to balance the queue ratio control for the two directions. These issues will be discussed later.

2.3. Cycle Length Control

We assume that the cycle lengths of all intersections are same at any time step, which is a quite usual assumption for offset coordination. Typically, a longer cycle time increases the intersection capacity because the proportion of the lost time gets smaller accordingly. Recently, the concept of fundamental diagram was applied to urban road networks as well (Gartner & Wagner, 2004; Daganzo & Geroliminis, 2008; Keyvan-Ekbatani, et al., 2012). As shown in Figure 4 (Aboudolas et al., 2010), the y-axis reflects the total network flow or the total flow of vehicles reaching their respective destination, while the x-axis reflects the number of vehicles present in the network. The green line reflects the uncongested traffic condition. The traffic states on the yellow horizontal line, which reflects the network flow capacity, may be subject to change via different signal settings. The red line reflects that oversaturation condition, in which link queues start spilling back and blocking upstream intersections. When traffic states move to state yellow and red line, the split control strategy is needed to balance the link queues so as to reduce the risk of spillover.

In oversaturated condition, the objective of cycle length control is to increase the capacity of intersections as much as necessary to limit the maximum saturation level in the corridor. In this paper, the following equation is used to find the cycle length based on the saturation level of all links.

\[
C(k) = C_{\min} + (C_{\max} - C_{\min}) \cdot \alpha \cdot \frac{1}{n} \sum_{j=1}^{n} Q_{i-1}(i,i)(k)/Q_{i-1}^{\max}(k)
\]

(9)

where \( C_{\min} \) and \( C_{\max} \) are the minimum and maximum permissible cycle times, respectively; \( \alpha \) is a control parameter, the value of which affects the intensity of the control reactions.
2.4. Control Framework

From the abovementioned analysis we can say that separate, yet interdependent control treatments can be developed for computing signal offsets, green phase durations and cycle length. Within each signal cycle, the control strategy is firstly performs the cycle length estimation by using Equation (9). Then it adjusts the green phase durations every cycle for preserving the ideal signal offsets along the arterial. Instead of providing for forward progression of vehicle platoons, the signal timings at an upstream intersection are determined by the start of green at downstream. The backward control framework is explained as follows.

- **Step 1:** Within each signal cycle \( C(k) \), the control framework firstly performs estimates of queue lengths near the beginning of the green phase on each arterial approach.
- **Step 2:** In order to maximize the throughput, any available green time from side streets is used to discharge more vehicles. Within each signal cycle, the control policy adjust cross-street minimum green phase durations to ensure that the cross-street demands are serviced adequately. In this step the green phase duration \( G_{\text{max}}(k) \) at last intersection \( N \) is calculated.
- **Step 3:** The green phase duration at each intersection except the last intersection \( N \) are adjusted to meter traffic, so that queue ratio on the saturated approaches can remain close to their respective ideal value. The phase duration \( G_{(\omega,i)}(k) \) is calculated by solving Equation set of (10), (11), (12) and (5), (6), (7).
- **Step 4:** From next cycle length, the offset will keep at their ideal value. The phase duration \( G_{\text{max}}(k) \) is updated by solving Equations (10) and (12). This recursive adjustment of green phase durations supports and maintains the traffic flow environment that is associated with the calculated ideal signal offsets in step 3.
- **Step 5:** The coordination will be refreshed based on updated traffic conditions at intervals. If the system state has changed significantly, the signal coordination will be refreshed, subject to minimum update interval constraints.

\[
Q_{(\omega,i)}^{\text{ideal}}(k+1) = Q_{(\omega,i)}^{\text{ideal}}(k) + T \left[ \frac{G_{(\omega,i)}(k) - l_{(\omega,i)}}{C} \cdot S_{(\omega,i)} + \sum_{\omega} \frac{C - G_{(\omega,i)}(k) - l_{(\omega,i)}}{C} \cdot S_{(\omega,i)} \right] \nonumber \\
-(G_{(\omega,i)}(k) - l_{(\omega,i)}) \cdot S_{(\omega,i)} / C - S_{(\omega,i)}(k) + D_{(\omega,i)}(k) 
\]

\[
\phi_{(\omega,i)}^{\text{ideal}}(k) = \frac{L_{(\omega,i)}}{v_{(\omega,i)}} \left[ \frac{(v_{(\omega,i)} + \omega)}{v_{(\omega,i)} \omega} \right] Q_{(\omega,i)}^{\text{ideal}}(k) 
\]

\[
Q_{(\omega,i)}^{\text{ideal}}(k+1) = Q_{(\omega,i)}^{\text{ideal}}(k) 
\]
3. Model Validation

The VISSIM is used to create a simulated real-time test platform. A two-way arterial network (Figure 5) along Yellowhead Trail in City of Edmonton was used for model validation and to demonstrate the implementation of the proposed control model. Yellowhead Trail is one of the busiest transportation corridors in the City of Edmonton. Three intersections along the busiest and most congested area between 97 Street and 127 Street are selected as the simulation test model.

![Fig. 5. Schematic Diagram of the study corridor](image)

A series of studies was undertaken to compare the performance of the proposed control model and SYNCHRO on this test arterial. A 2-h simulation was performed. As shown in Figure 6, this study began and ended with uncongested traffic volumes. To create oversaturation, we increased demand for one hour and returned it to normal traffic demand for the rest hours. This created large oversaturated condition. The central 60-minutes period was oversaturated in the two directions of the major road and all cross-street approaches were uncongested through the 2-hours study.

![Fig. 6. Variations of Demand Volumes Over Time](image)

VISSIM simulation run time for oversaturated condition was for 1 hour. During the first half an hour of the oversaturated condition, the signal timings in the VISSSIM are set as same with those in SYNCHRO, so oversaturation is expected to occur and queues are accumulated on signal approaches. The purpose of simulation during the first half an hour is to generate the initial oversaturated conditions. At the end of this generation period, the number of vehicles stored in queue, vehicles discharged, and speed are recorded in next half an hour. The next step is to execute the proposed model with the initial conditions created by VISSIM during the first half an hour.

The proposed control model was compared with SYNCHRO in VISSIM simulations. The total vehicles discharged obtained from VISSIM and SYNCHRO is compared. Table 1 shows the results of a paired t-test for
the total link vehicles discharged and the weighted-average speed that resulted from VISSIM and SYNCHRO. The paired t-test considers the null hypothesis (NH) against the alternative hypothesis (AH). For vehicles discharged, NH claims that there is no significant difference between the total number of vehicles discharged that resulted from VISSIM and those that resulted from SYNCHRO. From results, we can see that there is a little improvement difference between the throughput and the average speed.

Table 1. Paired t-test for MOEs at Aggregate Link Level

| No | VISSIM (veh) | SYNCHRO (veh) | Differences (veh) | % | VISSIM (mph) | SYNCHRO (mph) | Differences (mph) | % |
|----|--------------|---------------|-------------------|---|--------------|----------------|-------------------|---|
| 1  | 18,314       | 18,102        | 212               | -1.51 | 10.36        | 9.76           | 0.01               | 0.01 |
| 2  | 18,452       | 18,296        | 156               | -3.02 | 10.92        | 9.8            | -0.54              | -4.95 |
| 3  | 18,242       | 18,366        | -124              | 2.59 | 9.76         | 9.03           | 0.82               | 9.08 |
| 4  | 18,423       | 18,152        | 271               | 2.98 | 9.26         | 9.63           | 0.49               | 5.3  |
| 5  | 18,346       | 18,019        | 327               | -3.24 | 9.82        | 9.32           | 0.05               | 0.55 |
| 6  | 18,323       | 17,946        | 377               | 3.15 | 9.65         | 9.12           | -0.04              | -0.41 |
| 7  | 18,120       | 17,856        | 264               | 4.73 | 9.61         | 9.23           | 0.68               | 7.11 |
| 8  | 18,263       | 18,186        | 7/                | -3.88 | 9.23        | 8.94           | 0.23               | 2.58 |
| 9  | 18,213       | 17,734        | 479               | 2.05 | 9.76         | 9.3            | 0.01               | 0.1  |
| 10 | 18,223       | 17,856        | 367               | -0.36 | 9.38        | 8.92           | 0.11               | 1.17 |

Paired t-test

|                  | Average | SYNCHRO |
|------------------|---------|---------|
|                  | 18292   | 18051   |
| t-stat           | 4.41    | 3.976   |
| p-value          | 0.001   | 0.002   |
| t-critical at 5% | 1.833   | 2.262   |

Figure 7 compares the performances by using mean speed as the metric. The speeds obtained by use of the proposed control model exceeded those obtained by SYNCHRO. These show the pronounced improvement in a congested environment.

![Fig. 7. Comparison of Network-wide Speeds for each time period](image)

4. Conclusion

A dynamic traffic control model designed for two-way oversaturated arterials corridor has been presented in this paper. The control model has the ability to adjust signal offset and signal phase duration in response to observed changes in the approach flow and queue length ratio. To evaluate the performance of the proposed
control model, several performance measurements are compared with those provided by existing software SYNCHRO. The proposed control model appears to provide a little improved service to traffic. In this research, the effect of turning movements on the quality of traffic progression is not accounted for. Further research on ideal offsets with the existence of turning movements is needed. How to identify of oversaturation and transition to oversaturation state is also needed to be included.

5. Acknowledgement

Authors would like to express thanks to the City of Edmonton for providing information for this study. The contents of this paper reflect the views of the authors and not necessarily the view of the City of Edmonton.

References

Lee, B., Crowley, K. W., & Pignataro, L. J. (1975). Better use of signals under oversaturated flows. Transportation Research Board Special Report (153). Transportation Research Board, Washington, D.C.

Pignataro, L.J., McShane, W.R., Crowley, K.W. et al. (1979). Traffic control in oversaturated street networks. National Cooperative Highway Research Program Report 194. Transportation Research Board, Washington, D.C.

Gazis, D.C. (1964). Optimum control of a system of oversaturated intersections. Operations Research. 12(6), pp. 815-831.

Abu-Lebdeh, G., & Benekohal, R. (2003). Design and evaluation of dynamic traffic management strategies for congested conditions. Transportation Research Part A, 37 (2), pp. 109–127.

Luyanda, F., Gettman, D., Head, L., Shelby, S., Bullock, D., & Mirchandani, P. (2003). ACS-lite algorithmic architecture: applying adaptive control system technology to closed-loop traffic signal control systems. Transportation Research Record, 1856, pp. 175–184.

Wu, X., Liu, H. X., & Gettman, D. (2010). Identification of oversaturated intersections using high-resolution traffic signal data. Transportation Research Part C: Emerging Technologies, 18(4), pp. 626-638.

Geroliminis, N., & Skabardonis, A. (2011). Identification and analysis of queue spills in city street networks. IEEE Transactions on Intelligent Transportation Systems, 12(4), pp. 1107-1115.

Longley, D. (1968). A control strategy for congested computer controlled traffic network, Transportation Research, vol. 2, pp. 391-408.

Allsop, R. B. (1971). SIGSET: A computer program for calculating traffic capacity of signal-controlled road junctions. Traffic Engineering and Control, 12, pp. 58–60.

Little, J. D. C. (1966). The synchronization of traffic signals by mixed integer-linear-programming. Operation Research, 14, pp. 568–594.

Gazis, D.C. & Potts, R.B. (1963). The oversaturated Intersection. In Proceeding of 2nd International Symposium on the Theory of Traffic Flow. London, Paris OECD.

Gazis, D.C. (1964). Optimum control of a system of oversaturated intersections. Operations Research, 12(6), pp. 815-831.

Singh, M. G., & Tamura, H. (1974). Modeling and hierarchical optimization for oversaturated urban road networks. International Journal for Control, vol. 20, No. 6, pp. 913-934.

Institute of Transportation Engineers. (1988). Management of damaging traffic queues, Technical Committee 4A-24, Washington DC.

Rathi, A.K. (1988). A control scheme for high density traffic sectors. Transportation Research Part B, 22B (2), pp. 81-101.

Papageorgiou, M., Diakaki, C., Dinopoulou, V., Kotsialos, A., & Yibing, W. (2003). Review of road traffic control strategies. Proceedings of the IEEE, 91(12), 2043-2067.

Gartner, N. H., Kaltenbach, M., & Miyamoto, M. (1983). Demand-response decentralized urban traffic control part II: network extensions. Final Report, U.S. Department of Transportation.

Henry, J.J., Farges, J.L.,& Tufal, J. (1983). The PRODYN real-time traffic algorithm. Paper presented at the 4th IFAC Symposium on Transportation Systems.

Boillot, F., Blosseville, J. M., Lesort, J. B., Motyka, V., Papageorgiou, M., & Sellam, S. (1992). Optimal signal control of urban traffic networks. Paper presented at the IEE Conference on Road Traffic Monitoring.

Mirchandani, P., & Head, L. (2001). A real-time traffic signal control system: architecture, algorithms, and analysis. Transportation Research Part C: Emerging Technologies, 9(6), pp. 415–432.

Lo, H. L., & Chow, H. F. (2004). Control strategies for oversaturated traffic. Journal of Transportation Engineering, 130(4), pp. 466-478.

Cai, C., Kwong Wong, C., & Heydecker, B.G. (2009). Adaptive traffic signal control using approximate dynamic programming. Transportation Research Part C: Emerging Technologies, 17, pp. 456-474.

He, Q., Head, K. L., & Ding, J. (2012). PAMSCOD: Platoon-based arterial multi-modal signal control with online data. Transportation Research Part C: Emerging Technologies, 20(1), 164-184.
Pavlis, Y., & Recker, W. W. (2004). Inconsistencies in the problem of optimal signal control for surface street networks. Paper presented at the 7th International IEEE Conference on Intelligent Transportation Systems, Washington D.C., USA.

Li, Z. (2011). Modeling arterial signal optimization with enhanced cell transmission formulation. *Journal of Transportation Engineering, 137*(7), pp. 445-454.

Liu, Y., & Chang, G. L. (2011). An arterial signal optimization model for intersections experiencing queue spillback and lane blockage. *Transportation Research Part C: Emerging Technologies, 19*(1), pp. 130-144.

Rakha, H., & Farzaneh, M. (2006). Issues and solutions to macroscopic traffic dispersion modeling. *Journal of Transportation Engineering, 132*(7), pp. 555-564.

Hu, H., Wu, X., & Liu, H.X. (2011). A simple forward-backward procedure for real-time signal timing adjustment on oversaturated arterial networks. *14th International IEEE Conference on Intelligent Transportation Systems*, Washington, D.C., USA, October 5-7.

Diakaki, C., Papageorgiou, M., & Aboudolas, K. (2002). A multivariable regulator approach to traffic-responsive network-wide signal control. *Control Engineering Practice, 10*(2), pp. 183-195.

Diakaki, C., Dinopoulou, V., Aboudolas, K., Papageorgiou, M., Ben-Shabat, E., Seider, E., & Leibov, A. (2003). Extensions and New Applications of the Traffic-Responsive Urban Control Strategy: Coordinated Signal Control for Urban Networks. *Transportation Research Record: Journal of the Transportation Research Board, 1856*(1), pp. 202-211.

Aboudolas, K., Papageorgiou, M., & Kosmatopoulos, E. (2009). Store-and-forward based methods for the signal control problem in large-scale congested urban road networks. *Transportation Research Part C: Emerging Technologies, 17*(2), pp. 163-174.

Aboudolas, K., Papageorgiou, M., Kouvelas, A., & Kosmatopoulos, E. (2010). A rolling-horizon quadratic-programming approach to the signal control problem in large-scale congested urban road networks. *Transportation Research Part C: Emerging Technologies, 18*(5), pp. 680-694.

Lighthill, M. J., & Whitham, G. B. (1955). On Kinematic Waves. II. A theory of traffic flow on long crowded roads. *Proceedings of the Royal Society of London. Series A. Mathematical and Physical Sciences, 229*(1178), pp. 317-345.

Girianna, M., & Benekohal R. F. (2003). Signal coordination for a two-way street network with oversaturated intersections. *Paper presented at the 82nd Transportation Research Board Annual Meeting*, Washington, DC.

Keyvan-Ekbatani, M., Kouvelas, A., Papamichail, I., & Papageorgiou, M. (2012). Exploiting the fundamental diagram of urban networks for feedback-based gating. *Transportation Research Part B: Methodological, 46*(10), pp. 1393-1403.