Canadian Journal of Civil Engineering
Revue canadienne de génie civil

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Journal:	<i>Canadian Journal of Civil Engineering</i>
Manuscript ID	cjce-2016-0331.R1
Manuscript Type:	Article
Date Submitted by the Author:	25-Nov-2016
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Keyword:	adjacent signalized intersections, left-turn lane space, signal coordination, integrative design, delay minimization

Integrative design of left-turn lane space and signal coordination for two adjacent intersections

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Abstract: For two adjacent signalized intersections with short left-turn lanes, the capacity and delay for each intersection depend on not only the left-turn lane space but also the signal coordination strategy. In this paper, two optimization models are formulated to integrate the design of the left-turn lane space and signal coordination. While Model II can simultaneously optimize the short-lane space, green splits and offset under the equal-cycle constraint, Model I could not simultaneously obtain them. The simulation-based algorithm is proposed to seek the best offset in the global range when the common cycle length, green splits and short-lane length are all ascertained together with signal phase plan. Numerical examples are given to demonstrate these two models. The findings reveal that Model II is more effective and easier to apply than Model I. Finally, the application of Model II in engineering practice is also given.

Keywords: adjacent signalized intersections; left-turn lane space; signal coordination; integrative design; delay minimization; optimization models

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1. Introduction

At signalized intersections, to accommodate heavy left-turn traffic is often challenging. To satisfy the left-turn traffic demand, exclusive left-turn phases are usually provided together with exclusive left-turn lanes. Due to the limitation of road space, such left-turn lanes often exist in the form of short lanes or turn bays, referred to as short left-turn lanes or left-turn bays. In recent years, there have been a large number of studies on the operations of signalized intersections (Jin et al. 2009; Wong and Heydecker 2011; Xuan et al. 2011). In the *Highway Capacity Manuals* (HCM), short left-turn lanes are regarded as full exclusive left-turn lanes so that the effect of such a treatment on the capacity of the lane group with a short left-turn lane is ignored (TRB 2001). As a result, the capacity of the lane group may be overestimated, and the average delay per vehicle in the lane group may be underestimated, which may have a large impact on the signal timing optimization and intersection operations.

In the *Traffic Signals: Capacity and Timing Analysis*, it was first denoted that the saturation flow rate on a multi-lane approach could be reduced due to short lane effects (Akçelik 1981). The short lane effects would occur when the length of a lane was limited for queuing. Afterwards, some scholars found that the capacity of an intersection might be overestimated or underestimated by the existing methods, and they utilized the probability method to formulate the capacity estimation models for signalized intersections with a short right-turn or left-turn lane (Tian and Wu 2006; Wu 2007). Zhang and Tong (2008) also applied the probability method and regarded the length of left-turn bay as an independent variable to formulate the protected left-turn capacity model, the permitted left-turn capacity model and the adjacent through capacity model. On the basis of on-site experiments in the city of Vilnius, Lithuania, Klibavicius and Paliulis (2012) developed a methodology to model the short traffic lane capacity. Yin et al. (2011) formulated the probabilistic delay models for the left-turn and through vehicles under protected left-turn operations and with short left-turn bays. Additionally, Gowri and Sivanandan (2008) examined the effects of left-turn channelization on vehicle waiting times and found that the channelization length for the left-turn traffic had marginal impacts on vehicle waiting times.

To avoid the overflow and blockage due to the presence of short lanes, Kikuchi et al. ascertained the suitable length of the double short left-turn lanes (DLTLs) and the suitable length of the turn lanes when a single lane was divided into left-turn, through and right-turn lanes at a signalized intersection (Kikuchi et al. 2004; Kikuchi et al. 2007). For signalized intersections, Qi et al. (2012) determined the left-turn lane queue storage length and deceleration length by evaluating the analytical-based traffic models and the simulation-based methods. Yang and Zhou (2011) noted that the design length of left-turn lanes was inapplicable to signalized intersections and presented a new methodology to coordinate the geometric design of left-turn lanes with signal timing.

Signal coordination is very important for traffic signal control on an arterial street. To guarantee the most suitable progression scheme for different traffic flow patterns, Gartner et al. (1991) developed a multi-band optimization approach for arterial progression. This approach could generate a variable bandwidth on each directional road section. To consider the optimal traffic signal setting for an urban arterial road, Li and Chen (2009) introduced the synchronization rate and non-synchronization degree to construct a mathematical model and developed a new iterative algorithm. Considering the stochastic nature of field traffic and utilizing the high-resolution traffic

signal data, Hu and Liu (2013) proposed a data-driven arterial offset optimization model to improve traditional offset optimization for coordinated traffic signals. Mu et al. (2013) established a timed Petri net model with a traffic signal display module and a signal phase transition module, and a traffic flow model based on continuous Petri net with variable speeds. Based on these models, the traffic signal coordinated optimization of an urban arterial road could be carried out. Chen and Chang (2014) proposed a macroscopic simulation concept to capture the complex interactions between different types of vehicles so that the optimized signal timing plan could be provided for an arterial under heavy mixed traffic flows.

The above-mentioned studies indicate that the short lanes should be well designed to improve the capacity and level of service for an intersection because they have an assignable influence upon traffic flow operations. Nevertheless, the existing signal coordination techniques did not take short left-turn lanes into account. The authors have ever formulated the optimization models to acquire the optimal combination of effective green time and short-lane length for an isolated intersection and for two adjacent intersections with uncoordinated signals (Yao 2013a; Yao and Zhang 2013; Yao 2016). If two adjacent intersections are closely spaced so that coordinated signals have to be considered, the integrative design of left-turn lane space and signal coordination should be focused on. Its necessity will be interpreted by the next example. Considering the universality, Figure 1 illustrates two adjacent four-leg signalized intersections which are denoted by A and B, respectively. In Figure 1(a), each approach has a short left-turn lane, an exclusive left-turn lane, a through lane and a shared through-right lane at each intersection. The short left-turn lane on the westbound approach at intersection A and that on the eastbound approach at intersection B are both installed on the common road section between the two intersections (their lengths are denoted by D_k^A and D_l^B , respectively). The sum of the lengths of these two short lanes could not exceed the length of the common road section (denoted by D_0). In fact, there can be more than one exclusive left-turn or through lane or no through lane on each approach, or there can be no short left-turn lane on one to three approaches at each intersection. In addition, there can be no short left-turn lane at either or both of the two intersections. In Figure 1(b) (taking the east-west direction as an example), the signal phase plan may be permitted left-turn phasing, left-through phasing, exclusive left-turn phasing (an exclusive left-turn phase may also be provided after the through/right-turn phase for the subject street), leading and lagging green phasing or exclusively left-turn plus leading green phasing in the east-west or south-north direction at each intersection (FHWA 2008; Roess et al. 2008). The specific signal phase plan should be determined by the traffic demands for each intersection. Actually, there can be no exclusive left-turn lane on each approach at each intersection. If exclusive left-turn phasing is provided, the probability of overflow or blockage of the short lane may increase because the short left-turn or adjacent through lane could not be fully utilized (Yao 2016). In this case, vehicles in the short left-turn and adjacent through lanes should proceed together to relieve the overflow and blockage, that is, left-through phasing should be provided. If a shared left-through lane is installed on an approach, permitted left-turn phasing should be adopted. If a protected left-turn phase is designed for an approach, one or more left-turn lanes must be provided and there is no shared left-through lane on this approach. The above case can be simplified to consider one-way streets or T-intersections, and in that case the signal phase plan can be simpler. Signal coordination is to allocate the intersection time resources, whereas the design of left-turn lane length is to allocate the intersection space resources. Therefore, the integrative design of left-turn lane space and signal coordination can optimally allocate the time-space resources for two adjacent intersections.

2. Basic Principles

The basic assumptions are as follows in this paper. First, pre-timed and coordinated signals are adopted for the two given intersections and short left-turn lanes need be provided for heavy left turns during morning and afternoon peak hours. Second, the saturation flow rates and the total lost time can be determined, and the peak 15-min flow rates and the hourly volumes can be collected for each intersection. Third, no specific signal can control any right-turn traffic. Finally, the distance between adjacent intersections is shorter than one mile for signal coordination because it is mentioned that common practice is to coordinate signals less than one mile apart on major streets and highways in the *Traffic Engineering* (Roess et al. 2008). Such a distance of one mile may be too far for intersection spacing in an urban area. However, this critical value may be decreased based on actual situations. Since it is only a parameter, it will not affect the usage of the formulated models.

In general, travelers or traffic users pursue minimizing their travel delay for an isolated intersection, as does it for two adjacent intersections.

2.1. Calculation of Capacity

When considering the presence of short left-turn lanes and the design of signal phase plan (Yao and Zhang 2013), the capacity of a given lane group at a specified intersection can be computed as

$$Q_j^\eta = \begin{cases} \left[S_{ij}^\eta \left(\sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \right) + \phi_j^\eta S_{sj}^\eta D_j^\eta t / h \right] / \left(\sum_{i=1}^{n^\eta} g_{pi}^\eta + L^\eta \right), & \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \geq D_j^\eta t / h \\ \left(S_{ij}^\eta + \phi_j^\eta S_{sj}^\eta \right) \left(\sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \right) / \left(\sum_{i=1}^{n^\eta} g_{pi}^\eta + L^\eta \right), & \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta < D_j^\eta t / h \end{cases} \quad (1)$$

where $\eta \in \{A, B\}$; $L^\eta = n_d^\eta \cdot \bar{t}$ = total lost time for intersection η (s); n_d^η = number of discrete phases for intersection η ; $\bar{t}$ = average lost time for each phase (s); Q_j^η = capacity of lane group j at intersection η (pcu/h); S_{ij}^η = full-lane saturation flow rate for lane group j at intersection η (pcu/h); S_{sj}^η = short-lane saturation flow rate for lane group j at intersection η (pcu/h); n^η = number of signal phases for intersection η ; ϕ_{ij}^η = 0-1 variable to identify whether the movements in lane group j can pass through intersection η in phase i , $\phi_{ij}^\eta = 1$ if yes, or $\phi_{ij}^\eta = 0$ if no; g_{pi}^η = effective green time for phase i at intersection η (s); ϕ_j^η = 0-1 variable to identify whether a short left-turn lane can be installed in lane group j at intersection η , $\phi_j^\eta = 1$ if yes, or $\phi_j^\eta = 0$ if no; D_j^η = short-lane length for lane group j at intersection η (m); t = average saturation headway between consecutive vehicles (s); h = average queue spacing between consecutive vehicles (m).

By aggregating the capacity of all the lane groups, the *intersection capacity* can be expressed as

$$\tilde{Q}^\eta = \sum_{j=1}^{m^\eta} Q_j^\eta \quad (2)$$

where $\tilde{Q}^\eta$ = capacity of intersection η (pcu/h); m^η = number of lane groups at intersection η .

2.2. Calculation of Delay

Based on the *Highway Capacity Manual 2000* (HCM 2000) (TRB 2001), the average delay for a

given lane group at a specified intersection can be expressed as

$$\begin{aligned} d_j^\eta &= d_{1j}^\eta (PF_j^\eta) + d_{2j}^\eta + d_{3j}^\eta \\ d_{1j}^\eta &= \left[0.5 \left(\sum_{i=1}^{n^\eta} g_{pi}^\eta + L^\eta \right) (1 - u_j^\eta)^2 \right] / \left[1 - \min(1, x_j^\eta) u_j^\eta \right] \\ d_{2j}^\eta &= 900T \left[(x_j^\eta - 1) + \sqrt{(x_j^\eta - 1)^2 + (8kI_j^\eta x_j^\eta) / (Q_j^\eta T)} \right] \\ d_{3j}^\eta &= 1800(1 + \lambda_j^\eta) t_{wj}^\eta Q_{bj}^\eta / (Q_j^\eta T) \end{aligned} \quad (3)$$

$$\text{where } \lambda_j^\eta = \begin{cases} 0, & t_{wj}^\eta < T \\ 1 - \frac{Q_j^\eta T}{Q_{bj}^\eta [1 - \min(1, x_j^\eta)]}, & t_{wj}^\eta \geq T \end{cases}; \quad t_{wj}^\eta = \begin{cases} 0, & Q_{bj}^\eta = 0 \\ \min \left\{ T, \frac{Q_{bj}^\eta}{Q_j^\eta [1 - \min(1, x_j^\eta)]} \right\}, & Q_{bj}^\eta \neq 0 \end{cases};$$

$$u_j^\eta = \left(\sum_{i=1}^n \phi_{ij}^\eta g_{pi}^\eta \right) / \left(\sum_{i=1}^{n^\eta} g_{pi}^\eta + L^\eta \right); \quad d_j^\eta = \text{average delay for lane group } j \text{ at intersection } \eta \text{ (s/pcu);}$$

d_{1j}^η = uniform delay component for lane group j at intersection η (s/pcu); PF_j^η = progression adjustment factor for lane group j at intersection η ; d_{2j}^η = overflow delay component for lane group j at intersection η (s/pcu); d_{3j}^η = delay due to pre-existing queue for lane group j at intersection η (s/pcu); u_j^η = green ratio for lane group j at intersection η ; $x_j^\eta = q_j^\eta / Q_j^\eta$ = v/c ratio for lane group j at intersection η ; q_j^η = demand flow rate for lane group j at intersection η (pcu/h); T = analysis period (h); k = incremental delay factor for actuated controller settings, 0.50 for all pre-timed controllers; I_j^η = upstream filtering/metering adjustment factor for lane group j at intersection η ; λ_j^η = delay parameter for lane group j at intersection η ; t_{wj}^η = duration of unmet demand for lane group j at intersection η (h); Q_{bj}^η = initial queue at the start of analysis period for lane group j at intersection η (pcu).

At any specified intersection, the upstream filtering/metering adjustment factor should be set to 1.0 for a given lane group without upstream signals. For a given lane group with upstream signals, this factor can be calculated as

$$I_j^\eta = \begin{cases} 1.0 - 0.91(x_{wj}^\eta)^{2.68}, & x_{wj}^\eta \leq 1.0 \\ 0.09, & x_{wj}^\eta > 1.0 \end{cases} \quad (4)$$

where x_{wj}^η = weighted v/c ratio of all upstream movements contributing to the volume for lane group j at intersection η , and computed as a weighted average with the v/c ratio of each contributing upstream movement weighted by its volume (TRB 2001).

By aggregating the average delays for all the lane groups, the *intersection delay* can be given by

$$\bar{d}^\eta = \sum_{j=1}^{m^\eta} q_j^\eta d_j^\eta / \sum_{j=1}^{m^\eta} q_j^\eta \quad (5)$$

where $\bar{d}^\eta$ = average delay for intersection η (s/pcu).

Then, the *total delay* can be calculated as

$$TD = \bar{d}^A \cdot \sum_{j=1}^{m^A} q_j^A + \bar{d}^B \cdot \sum_{j=1}^{m^B} q_j^B \quad (6)$$

where TD = total delay (s); $\bar{d}^A$ = average delay per vehicle for intersection A (s/pcu); $\bar{d}^B$ = average delay per vehicle for intersection B (s/pcu); q_j^A = demand flow rate for lane group j at intersection A (pcu/h); q_j^B = demand flow rate for lane group j at intersection B (pcu/h); m^A = number of lane groups for intersection A; m^B = number of lane groups for intersection B.

2.3. Constraints

At a specified intersection, the effective green time for the lane group with a short left-turn lane should not be less than the queue full discharge time for the short lane, that is

$$\sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \geq D_j^\eta t / h, \quad \phi_j^\eta = 1 \quad (7)$$

For any intersection, the effective green time for each lane group should not be less than a minimum, namely

$$\sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \geq g_{\min} \quad (8)$$

where $g_{\min}$ = minimum effective green time (s).

For each intersection, the summation of the total effective green time and the total lost time is the cycle length which should not be less than a minimum and greater than a maximum, i.e.

$$C_{\min} \leq \sum_{i=1}^{n^\eta} g_{pi}^\eta + L^\eta \leq C_{\max} \quad (9)$$

where $C_{\min}$ = minimum cycle length (s); $C_{\max}$ = maximum cycle length (s).

The lengths of the short left-turn lanes on the common road section should satisfy the following expression, so we have

$$\phi_k^A D_k^A + \phi_l^B D_l^B \leq D_0, \quad \delta_k^A = 1 \& \delta_l^B = 1 \quad (10)$$

where ϕ_k^A = 0-1 variable to identify whether a short left-turn lane can be installed in lane group k at intersection A, $\phi_k^A = 1$ if yes, or $\phi_k^A = 0$ if no; ϕ_l^B = 0-1 variable to identify whether a short left-turn lane can be installed in lane group l at intersection B, $\phi_l^B = 1$ if yes, or $\phi_l^B = 0$ if no; D_k^A = short-lane length for lane group k at intersection A (m); D_l^B = short-lane length for lane group l at intersection B (m); D_0 = length of the common road section (m); δ_k^A = 0-1 variable to identify whether lane group k at intersection A is located on the common road section, $\delta_k^A = 1$ if yes, or $\delta_k^A = 0$ if no; δ_l^B = 0-1 variable to identify whether lane group l at intersection B is located on the common road section, $\delta_l^B = 1$ if yes, or $\delta_l^B = 0$ if no. When $\phi_k^A = 0$ and $\phi_l^B = 0$, there is actually no such a constraint.

Considering signal coordination and equal cycle length for two adjacent intersections, it yields

$$\sum_{i=1}^{n^A} g_{pi}^A + L^A = \sum_{i=1}^{n^B} g_{pi}^B + L^B \quad (11)$$

where g_{pi}^A = effective green time for phase i at intersection A (s); g_{pi}^B = effective green time for phase i at intersection B (s); n^A = number of phases for intersection A; n^B = number of phases for intersection B; L^A = total lost time per signal cycle for intersection A (s); L^B = total lost time per signal cycle for intersection B (s).

For any intersection, the effective green time for each phase and the length of each short left-turn lane should all be greater than or equal to zero, i.e.

$$g_{pi}^{\eta} \geq 0 \quad (12)$$

$$D_j^{\eta} \geq 0, \phi_j^{\eta} = 1 \quad (13)$$

3. Optimization Model I

3.1. Progression Adjustment Factor

For a given lane group at a specified intersection, the progression adjustment factor can be computed as (TRB 2001)

$$PF_j^{\eta} = \min \left(1, \frac{(1 - r_{pj}^{\eta}) f_{pj}^{\eta}}{1 - u_j^{\eta}} \right) \quad (14)$$

where r_{pj}^{η} = platoon ratio for lane group j at intersection η ; f_{pj}^{η} = supplemental adjustment factor for platoon arrival during the green phase for lane group j at intersection η .

When delay is estimated for future coordination, favorable progression should be assumed as a base condition for coordinated lane groups (except left turns), and random arrivals should be assumed for all uncoordinated lane groups and movements released from exclusive left-turn lanes in exclusive phases. Thus, r_{pj}^{η} and f_{pj}^{η} can be set to 1.333 and 1.15, respectively for favorable progression, and 1.000 and 1.00, respectively for random arrivals (TRB 2001).

When two adjacent intersections are considered for coordinated signals, the offset will have an effect on the progression adjustment factor for the coordinated movement at the downstream intersection. However, the offset could not be directly expressed by Eq. (14). According to the conventional signal timing methods (Akçelik 1981), the cycle length and green ratios for each intersection are firstly computed by utilizing the mathematical formulas for isolated intersections, and then the common cycle length is determined for all the intersections and the green ratios are modified using the common cycle length. Next, the offset between successive signals is calculated for optimum progression. Similarly, the following two steps are proposed in this paper. The first step is to determine the optimal values of the effective green time per phase and the short left-turn lane length of each lane group for two adjacent intersections. The second step is to determine the optimal offset between these two intersections.

3.2. Determination of Effective Green Time and Short-Lane Length

If not considering coordinated signals and minimizing the total delay for two adjacent intersections, the optimization problem is to minimize Eq. (6) under Eqs. (7)–(13), so we have

$$\begin{aligned} &\text{minimize} \quad TD = f(g_{pi}^A, D_j^A, g_{pi}^B, D_j^B) \\ &\text{subject to} \quad \text{Eqs. (7)–(13)} \end{aligned} \quad (15)$$

where $f(g_{pi}^A, D_j^A, g_{pi}^B, D_j^B)$ = function of variables g_{pi}^A , D_j^A , g_{pi}^B and D_j^B ; D_j^A = short-lane length for lane group j at intersection A; D_j^B = short-lane length for lane group j at intersection B.

In Eq. (15), the decision variables are the effective green time per phase and the length of each

short left-turn lane, the objective is to minimize the total delay for all vehicles and nonlinear, and the constraints are linear inequality and equality. Thus, Eq. (15) is a nonlinear single-objective optimization problem with linear inequality and equality constraints. In this paper, the *fmincon* function provided in the *MATLAB* software is used to solve Eq. (15).

3.3. Calculation of Ideal Offsets

Before proceeding to present the algorithm, the following definition is given. The *offset* refers to the difference between the green initiation time for the first signal phase at the upstream intersection and that at the downstream intersection. Here, the offset is expressed as a positive number between zero and the common cycle length.

To conveniently interpret the procedure, we denote the platoon from intersections A to B by PT1 and that from intersections B to A by PT2, respectively, as shown in Figure 2(a). In this paper, it is assumed that the platoons are moving as they go through the upstream intersections (Roess et al. 2008). Figures 2(b) and 2(c) indicate the determination of the feasible range of an offset using the time-space diagram. Here MP1 refers to a movement in the first phase, and MT1 and MT2 refer to the movements relating to PT1 and PT2, respectively. As shown in Figure 2(b), the ideal offset required by PT1 can be expressed as

$$O_{BA} = \text{mod}\left(\frac{s_{AB}}{v_{AB}}, C_c\right) + OP_A - OP_B \quad (16)$$

where O_{BA} = ideal offset required by PT1 (s); s_{AB} = distance between the stop lines at intersections A and B along the direction of PT1 (m); v_{AB} = average running speed of a platoon from intersections A to B (m/s); C_c = common cycle length (s); mod = modulus operator which means the remainder after division; OP_A = difference between the green initiation time for movement MT1 and that for movement MP1 at intersection A (s); OP_B = difference between the green initiation time for movement MT1 and that for movement MP1 at intersection B (s).

Similarly, as shown in Figure 2(c), the ideal offset required by PT2 can be expressed as

$$O'_{BA} = C_c - \text{mod}\left(\frac{s_{BA}}{v_{BA}}, C_c\right) + ON_A - ON_B \quad (17)$$

where O'_{BA} = ideal offset required by PT2 (s); s_{BA} = distance between the stop lines at intersections B and A along the direction of PT2 (m); v_{BA} = average running speed of a platoon from intersections B to A (m/s); ON_A = difference between the green initiation time for movement MT2 and that for movement MP1 at intersection A (s); ON_B = difference between the green initiation time for movement MT2 and that for movement MP1 at intersection B (s).

3.4. Simulation-Based Algorithm

Comparing Eqs. (16) with (17), the ideal offset required by PT1 is usually different from that required by PT2. Therefore, it is necessary to seek the optimal offset which is the best for both intersections. As shown in Figure 3, a simulation-based algorithm is given to seek the optimal combination of short-lane space, green splits, common cycle length and offset for two adjacent intersections during a specified period. A global optimal offset may be found when the difference between the ideal minimum and maximum offsets is less than a threshold (e.g. 15 s). Otherwise, a

local optimal offset may be found.

4. Optimization Model II

4.1. Modification of Progression Adjustment Factor

As stated earlier, for all uncoordinated lane groups and movements released from exclusive left-turn lanes in exclusive phases, the progression adjustment factor should be set to 1.0. However, for coordinated lane groups, this factor can also be expressed as (TRB 2001)

$$PF_j^\eta = \min \left(1, \frac{(1-P_j^\eta)f_{pj}^\eta}{1-u_j^\eta} \right) \quad (18)$$

where P_j^η = proportion of vehicles arriving on green for lane group j at intersection η .

Also, the supplemental adjustment factor for platoon arrival during the green phase can be calibrated by

$$f_{pj}^\eta = \begin{cases} 1, & r_{pj}^\eta \leq 0.5 \\ 0.93, & 0.5 < r_{pj}^\eta \leq 0.85 \\ 1, & 0.85 < r_{pj}^\eta \leq 1.15 \\ 1.15, & 1.15 < r_{pj}^\eta \leq 1.5 \\ 1, & 1.5 < r_{pj}^\eta \leq 2 \\ 1, & r_{pj}^\eta > 2 \end{cases} \quad (19)$$

where $r_{pj}^\eta = P_j^\eta / u_j^\eta$.

The ideal offset between coordinated signals can be given by (Roess et al. 2008)

$$O = \begin{cases} \text{mod}(s_{BA} / v_{BA}, C_c), & \text{mod}(s_{BA} / v_{BA}, C_c) \geq o \text{ \& } \eta = A \\ \text{mod}(s_{BA} / v_{BA}, C_c) + C_c, & \text{mod}(s_{BA} / v_{BA}, C_c) < o \text{ \& } \eta = A \\ \text{mod}(s_{AB} / v_{AB}, C_c), & \text{mod}(s_{AB} / v_{AB}, C_c) \geq o \text{ \& } \eta = B \\ \text{mod}(s_{AB} / v_{AB}, C_c) + C_c, & \text{mod}(s_{AB} / v_{AB}, C_c) < o \text{ \& } \eta = B \end{cases} \quad (20)$$

where O = ideal offset between intersection η and the upstream intersection (s).

The actual offset between coordinated signals can be expressed as

$$o = \begin{cases} C_c - o_{BA} + ON_A - ON_B, & \eta = A \\ o_{BA} - OP_A + OP_B, & \eta = B \end{cases} \quad (21)$$

where o = actual offset between intersection η and the upstream intersection (s); o_{BA} = offset between intersections B and A, namely, the difference between the green initiation time of the first phase at intersection B and that at intersection A (s).

Then, the proportion of vehicles arriving on green can be calculated as (Lu and Xu 2008)

$$P_j^\eta = \begin{cases} 1 - \frac{q_j^\eta t_{ij}^\eta}{3600 N_{pj}^\eta}, & N_{pj}^\eta \leq \frac{q_j^\eta \left(t_{ij}^\eta + \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \right)}{3600} \& o + \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \leq O < o + C_c \\ \frac{q_j^\eta \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta}{3600 N_{pj}^\eta}, & N_{pj}^\eta > \frac{q_j^\eta \left(t_{ij}^\eta + \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \right)}{3600} \& o + \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \leq O < o + C_c \\ 1 - \frac{q_j^\eta t_{ij}^\eta}{3600 N_{pj}^\eta}, & t_{ij}^\eta < C_c - \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta + \frac{q_j^\eta \left(C_c - \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \right)}{S_j^\eta - q_j^\eta} \& o \leq O < o + \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \\ 1 - \frac{q_j^\eta \left(C_c - \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \right)}{3600 N_{pj}^\eta}, & t_{ij}^\eta \geq C_c - \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta + \frac{q_j^\eta \left(C_c - \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \right)}{S_j^\eta - q_j^\eta} \& o \leq O < o + \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \end{cases} \quad (22)$$

where $S_j^\eta = \begin{cases} \left[\frac{S_{ij}^\eta \left(\sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \right) + \phi_j^\eta S_{sj}^\eta D_j^\eta t / h}{\left(\sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \right)} \right], & \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta \geq D_j^\eta t / h \\ S_{ij}^\eta + \phi_j^\eta S_{sj}^\eta, & \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta < D_j^\eta t / h \end{cases};$

N_{pj}^η = number of vehicles in a platoon for lane group j at intersection η (pcu); $t_{ij}^\eta = C_c - O + o$ = difference between the arrival time of the first vehicle and the end of red for lane group j at intersection η (s); $t_{ij}^\eta = 3600 \frac{N_{pj}^\eta}{q_j^\eta} + O - o - \sum_{i=1}^{n^\eta} \phi_{ij}^\eta g_{pi}^\eta$ = difference between the arrival time of the last vehicle and the start of red for lane group j at intersection η (s); S_j^η = equivalent saturation flow rate for lane group j at intersection η (pcu/h).

4.2. Optimization of Effective Green Time, Short-Lane Length and Offset

Since the progression adjustment factor for each lane group is a function of the offset between two adjacent intersections, the average delay per vehicle and the total delay for all vehicles are all the functions of the offset between these two intersections. Thus, the following constraint must be added:

$$0 \leq o_{BA} < \sum_{i=1}^{n^\eta} g_{pi}^\eta + L^\eta \quad (23)$$

When considering coordinated signals and minimizing the total delay for all vehicles, the optimization problem becomes to minimize Eq. (6) under Eqs. (7)–(13) and (23), so we have

$$\begin{aligned} & \text{minimize} \quad TD = f(g_{pi}^A, D_j^A, g_{pi}^B, D_j^B, o_{BA}) \\ & \text{subject to} \quad \text{Eqs. (7)–(13), (23)} \end{aligned} \quad (24)$$

where $f(g_{pi}^A, D_j^A, g_{pi}^B, D_j^B, o_{BA})$ = function of variables g_{pi}^A , D_j^A , g_{pi}^B , D_j^B and o_{BA} .

In Eq. (24), the decision variables are the effective green time per phase, the length of each short left-turn lane and the offset, the objective is also to minimize the total delay for all vehicles and

nonlinear, and the constraints are also linear inequality and equality. Thus, Eq. (24) is also a nonlinear single-objective optimization problem with linear inequality and equality constraints. In this paper, it is also solved by the *fmincon* function in the *MATLAB* software.

5. Numerical Examples

5.1. Traffic Data

It is assumed that the study intersections are controlled by pre-timed signals, thus the incremental delay factor is 0.50 (TRB 2001). The analysis period is considered to be 1 h. Considering no pre-existing queue, so the initial queue at the start of analysis period is 0 for each lane group. Based on the literature, the average lost time is 3.5 s for each phase, and the average saturation headway and queue spacing between successive vehicles are 2 s and 6 m, respectively (Yao 2013b); the minimum effective green time is 10 s (RTRA 2003); the minimum and maximum cycle lengths are 60 and 150 s, respectively (TRB 2001); the amber time is 3 s for each phase and the all-red time is 2 s for each phase transition (FHWA 2008; Roess et al. 2008).

The following example is to demonstrate the proposed models. Focusing on the short left-turn lanes on the common road section between adjacent intersections, Figure 4(a) illustrates the intersection layout. Table 1 lists the peak 15-min flow rates and hourly volumes for these two intersections. Considering the distribution of traffic streams at each intersection, the phase diagrams are designed, as illustrated in Figure 4(b). At intersection A, the exclusively left-turn plus leading green phasing is used in the east-west direction and the exclusive left-turn phasing is used in the south-north direction. At intersection B, the leading and lagging green phasing is used in the east-west direction and the exclusive left-turn phasing is used in the south-north direction. M1~M8 represent different traffic movements. In view of the *Traffic Engineering* (Roess et al. 2008), the number of discrete phases is 4 for each intersection. In addition, the saturation flow rates for an exclusive left-turn lane, a through lane and a shared through-right lane are set to 1810, 1850 and 1810 pcu/h, respectively (Akçelik 1981).

The microscopic traffic simulation software *VISSIM* is utilized to simulate intersection operations. To evaluate traffic flow operations in the peak hour, the hourly volume for each lane group is used as the flow input in *VISSIM*. In order to fit the saturation flow rates, the additive parts of safety distance are set to 2.45, 2.40 and 2.45 for exclusive left-turn, through and shared through-right lanes, and the multiplicative parts of safety distance are set to 3.45, 3.40 and 3.45 for exclusive left-turn, through and shared through-right lanes, respectively. Traffic flow is composed of 100% passenger cars, and the desired speed is 50 km/h. The mode of multi-run is used and the number of runs is set to 10. The random seeds adopt 8, 18, 28, 38, 48, 58, 68, 78, 88 and 98. To eliminate the impact of system initialization on the simulation results, the simulation length is set to 4200 s and the period of data collection is from 600 to 4200 s.

5.2. Analysis of Optimization Results

Applying the developed optimization models, the effective green time for each lane group, the length of each short left-turn lane, and the common cycle length can be obtained. Considering the

fluctuation of traffic flow, the peak 15-min flow rate is regarded as the demand flow rate when solving the optimization models. However, the hourly volume is regarded as the demand flow rate to calculate the intersection performance indices. Table 2 lists the optimization results of Models I and II. It reveals that: (1) the difference between the common cycle lengths obtained by utilizing Models I and II is 2 s; (2) the capacity, total delay, average delay and degree of saturation for each intersection obtained by Models I and II are very close; (3) the optimal offset obtained from Model I ranges from 52 to 121 s, and the optimal offset obtained from Model II is 84 s.

5.3. Analysis of Simulation Results

According to the optimization results, the actual green time for each lane group and the design length of each short left-turn lane are listed in Table 3 for Models I and II. The *VISSIM* software is used to construct two simulation models. As indicated in Figure 2(a), PT1 and PT2 refer to the eastbound and westbound through movements, respectively. In both simulation models, the distances between the upstream and downstream stop lines are 335 and 342 m for the eastbound and westbound through movements, respectively. The desired speed of vehicles is set to 50 km/h, which is assumed to be the average running speed of a platoon on the arterial street.

For Model I in which the common cycle length is 136 s, the simulation-based algorithm has to be used to obtain the optimal offset. Because the difference between the ideal minimum and maximum offsets is greater than 15 s, in view of Figure 3, the step increment is set to 10 s, and the modified minimum and maximum offsets are 60 and 120 s, respectively, i.e., the offset ranges from 60 to 120 s with a 10-s increment during the simulation. To analyze the impacts of different offsets on traffic flow operations, four performance indices (i.e. vehicle throughput, average delay, average number of stops and average queue length) are selected.

Figure 5 shows the comparison of the four performance indices under the different offsets when using Model I. Each data point in this figure is the average value from the multi-run simulation. It can be seen that (1) the average delay is more sensitive to the offset than the other three performance indices; (2) the optimal offset for intersection A may be different from that for intersection B; and (3) the optimal offset which generates the minimum average delay or queue length is 80 s for each intersection. Therefore, the obtained suboptimal offset is 80 s for both intersections.

To further seek the optimal offset in the global range, the tolerance of the offset adopts 5 s on the basis of Figure 3. Then, the simulation model is repeatedly run when the common cycle length is 136 s and the offset ranges from 75 to 85 s with a 1-s increment. Figure 6(a) shows the relationship curves between the average delay and offset when using Model I. Each illustrated data point is the average value from the multi-run simulation. Based on the average delay for both intersections, the optimal offset is 78 s for Model I.

Using Model II, the optimal offset which is 84 s can be directly obtained. To testify whether such an offset of 84 s is indeed suitable, based on Figure 3, the tolerance of the offset adopts 5 s. Then, the simulation model is repeatedly run when the common cycle length is 138 s and the offset ranges from 79 to 89 s with a 1-s increment. Figure 6(b) shows the relationship curves between the average delay and offset when testifying Model II. Each illustrated data point is also the average value from the multi-run simulation. Based on the simulation results, the offset of 82 s can minimize the average delay for both intersections. The average delay for both intersections resulted from the

offset of 84 s is 40.15 s; and the average delay for both intersections resulted from the offset of 82 s is 39.97 s. The former is only slightly greater than the latter and such a difference is 0.18 s. Therefore, it can be proven that the optimal offset obtained from Model II is suitable for traffic flow operations. That is to say, Model II is effective and reliable to simultaneously optimize the green splits, offset and short-lane space for two adjacent intersections. Thus, Model II can be used to directly provide the optimum signal timing plan in practice.

Comparing Figures 6(a) with 6(b), the best signal timing plan obtained by using Model I is slightly better than that obtained by testifying Model II, which denotes these two optimization models can all optimally allocate the time-space resources for two adjacent intersections. However, the workload of Model I is far more than that of Model II because Model I is more laborious than Model II, which denotes Model II is more effective and easier than Model I for use.

By integrating Table 2 with Figure 6, Table 4 lists the signal timing parameters and the statistical indices of average delay from the simulation models. In this table, the maximum and minimum are the maximal and minimal average delays for one intersection or for both intersections when the offset varies within the specified range, respectively; and the extreme difference is the maximum minus the minimum. It can be seen that (1) the extreme differences of the average delay are all less than 1 s for each intersection and for both intersections; and (2) the parameters and statistical indices from Model I are all close to those from Model II.

6. Discussions and Suggestions

As discussed earlier, Optimization Model I need to use two steps to obtain the optimal combination of short-lane space, common cycle length, green splits and offset by utilizing the simulation-based algorithm, while Optimization Model II can use one model to directly obtain the optimal combination of these parameters. Furthermore, the optimal offset obtained from Model II is very close to the best offset found by utilizing the simulation-based algorithm. According to the analysis, the workload of Model II is far less than that of Model I. Therefore, it is recommended that Model II should be used for practical applications. It is worth mentioning that whereas this paper focuses on the case in which there is at least one short left-turn lane at two adjacent intersections, the formulated models are also applicable to the case in which there is no short left-turn lane at two adjacent intersections.

The above-mentioned findings are acquired for the analysis period of 1 h. Generally speaking, traffic flow always fluctuates and different signal phase schemes are usually designed for different analysis periods. However, short-lane space of an intersection may be unchanged for several months or years. Therefore, Figure 7 demonstrates the procedure to apply the recommended model. In Part A of the first stage, the optimal combination of short-lane length, green splits, common cycle length and offset can be obtained for each analysis period. In Part B, the optimal offset can be adjusted by applying the simulation-based algorithm. If the workload need to be reduced or the simulation tool is not available in some actual situations, Part B can be skipped or neglected. In the second stage, the required length of each short left-turn lane can be determined for the whole study period. Finally, the signal phase plan and timing parameters can be determined for each analysis period.

Furthermore, the developed models can be applied in the following conditions: (1) the distance between adjacent intersections does not exceed one mile; (2) pre-timed and coordinated signals

need be adopted through the entire day or in the morning and afternoon peak periods.

7. Conclusions

For a signalized intersection with short left-turn lanes, the capacity and level of service may heavily lie on the length of each short left-turn lane and the effective green time per phase. For two adjacent intersections with short left-turn lanes, the capacity and level of service may depend on not only these two factors but also the offset. To seek the optimal combination of green splits, offset and short-lane space, two single-objective optimization models are developed. Optimization Model I can first obtain the optimal combination of short-lane space, green splits and common cycle length, and then acquire the optimal offset via the simulation-based algorithm. On the other hand, Optimization Model II can directly obtain the optimal combination of short-lane space, green splits, common cycle length and offset. Numerical examples are provided to demonstrate the model application. By analyzing the optimization and simulation results, Model II is more effective and easier to apply than Model I. Thus, Model II is recommended for use. Also, the flowchart is presented to utilize the recommended model in practice.

The contributions of this paper are as follows: (1) the optimization models are constructed to optimally allocate the time-space resources for two adjacent intersections with short left-turn lanes and coordinated signals, and these models are also suitable for the case with no short left-turn lane; (2) the optimization models are compared by numerical examples and the procedure of applying the recommended model is illustrated.

In the future, we will extend the recommended optimization model in this paper to consider an arterial intersection group which consists of more than two intersections.

Acknowledgements

This work was supported by the National Natural Science Foundation of China under Grant Nos. 51578111 and 51308092. The authors thank the anonymous reviewers for the valuable comments.

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Table 1. Peak 15-min flow rates and hourly volumes for the intersections

Parameters		Intersection A			Intersection B		
		Left-turn	Through	Right-turn	Left-turn	Through	Right-turn
Peak 15-min flow rate (pcu/h)	Eastbound	440	780	110	500	620	180
	Southbound	320	800	130	330	710	100
	Westbound	560	600	140	470	840	90
	Northbound	340	670	170	350	640	100
Hourly volume (pcu/h)	Eastbound	400	700	100	450	550	150
	Southbound	300	730	120	280	630	90
	Westbound	500	540	120	420	750	80
	Northbound	300	600	150	320	580	80

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Table 2. Optimization results using Models I and II

Parameters / Indices	Model I		Model II	
	Intersection A	Intersection B	Intersection A	Intersection B
Common cycle length (s)	136		138	
Capacity (pcu/h)	5414.39	5372.58	5422.88	5381.14
Total delay ($\times 10^4$ s)	29.11	27.47	29.29	27.69
Average delay (s/pcu)	63.83	62.72	64.22	63.22
Degree of saturation	0.84	0.82	0.84	0.82
Offset (s)	[52, 121]		84	

Note: The degree of saturation is a weighted average with the volume to capacity ratio for each lane group weighted by its volume. Assume that the offset is a positive integer, and the listed data are obtained from the calculated values by omitting decimal fractions smaller than 0.5 and counting all others, including 0.5, as 1.

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Table 3. Signal timing parameters and short-lane length for the simulation models

Model	Intersection	Parameters	Lane group							
			M1	M2	M3	M4	M5	M6	M7	M8
I	A	Actual green time (s)	31	27	24	34	22	36	24	34
		Short-lane length (m)	—	—	—	—	72	—	—	—
	B	Actual green time (s)	22	40	25	29	33	29	25	29
		Short-lane length (m)	72	—	—	—	—	—	—	—
II	A	Actual green time (s)	32	26	25	35	22	36	25	35
		Short-lane length (m)	—	—	—	—	72	—	—	—
	B	Actual green time (s)	22	41	25	30	34	29	25	30
		Short-lane length (m)	72	—	—	—	—	—	—	—

Note: Actual green time means the duration of the green indication for a lane group, and is equal to the effective green time plus the start-up lost time and then minus the amber time for the lane group. Assume that the actual green time is a positive integer here; the calculated values are converted into the design values by omitting decimal fractions smaller than 0.5 and counting all others, including 0.5, as 1. The design length of a short left-turn lane is obtained from the calculated value by using the formula $D'_j = \text{ceil}(D_j/h) \times h$. Here, D'_j and D_j are the design and calculated values of the short-lane length, respectively; “ceil” is a mathematical operator, and $\text{ceil}(x)$ rounds x to the nearest integer towards infinity.

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Table 4. Statistical characteristics of average delay from the simulation models

Model	Common cycle length (s)	Range of offset (s)	Statistical indices	Average delay (s/pcu)		
				Intersection A	Intersection B	All
I	136	[75, 85]	Maximum	40.25	39.32	39.67
			Minimum	39.62	38.84	39.33
			Extreme difference	0.63	0.48	0.34
II	138	[79, 89]	Maximum	41.22	39.73	40.44
			Minimum	40.44	39.44	39.97
			Extreme difference	0.78	0.29	0.47

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Figure 1. Intersection layout and phase diagrams

- (a) Intersection layout
- (b) Phase diagrams in the east-west direction at each intersection

Figure 2. Determining the range of ideal offset via time-space diagram

- (a) The two platoons controlled by coordinated signals
- (b) An ideal offset required by PT1
- (c) An ideal offset required by PT2

Figure 3. Flowchart for applying Model I**Figure 4. A study case**

- (a) Intersection layout
- (b) Phase diagrams

Figure 5. Comparison of four performance indices under different offsets for Model I

- (a) Vehicle throughput
- (b) Average delay
- (c) Average number of stops
- (d) Average queue length

Figure 6. Average delay versus offset

- (a) Model I
- (b) Model II

Figure 7. Flowchart of using the recommended model

(a)

(b)

(a)

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5
Intersection A					
Intersection B					

(b)

(a)

(b)

(c)

(a)

(b)