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Optimization of coordinated signal settings for hook-turn intersections

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SUMMARY

Hook turn (HT) is a unique traffic regulation rule for right-turning vehicles at intersections (in the system where driving is on the left), which was proposed in Melbourne to improve the safety level and operational efficiency of intersections. However, existing coordination plans for HT intersections are fixed and determined empirically, which restricts the further improvements of the efficiency. In this paper, mathematical models are developed for the calculation of the average vehicle delay, with consideration of the spillback phenomenon of HT vehicles in waiting areas. The platoon dispersion model is used to describe the traffic movements between coordinated intersections. With the aim of minimizing average delay of all vehicles, a mixed nonlinear integer model is developed for the optimal coordination plan, which is solved by a genetic algorithm due to the complexity of the model. Finally, a numerical example is built based on three HT intersections in downtown Melbourne, to verify the proposed methodology. Based on a comparison with the current signal plan, the optimal signal plan can significantly reduce the average delay as well as the number of spillbacks, in both the peak hour and off-peak hour cases. Copyright © 2015 John Wiley & Sons, Ltd.

KEY WORDS: hook turn; signal coordination; average vehicle delay; optimization model

1. INTRODUCTION

In a traffic system where driving is on the left side of the roads, the safety level and operational efficiency of intersections are highly affected by the right-turning vehicles. In the past several decades, many regulation schemes have been proposed and later widely implemented in the world; for instance, the U-turn scheme [1], dedicated lane/waiting area for right-turning vehicles in the intersection area [2], dedicated right-turning signal phase [3] and turning lane [4] and presignal [5]. This paper addresses a relatively innovative regulation scheme, termed as hook turn (HT).

Although HT is new to many other cities in the world, it has been successfully implemented in urban Melbourne for 60 years. Figure 1 indicates the geometry and phasing plan of two typical HT intersections in Melbourne city. We take the south leg of intersection A as an example to introduce the rule of HT scheme: the right-turning lane is relocated from the offside to the curbside. During the green time for this leg, the right-turning vehicles first enter the intersection and park at waiting area A1; then, after the signal of the side road turns to green, these vehicles will departure and leave the intersection. The vehicles waiting on the side lanes will follow these vehicles in the waiting area and cross the intersection. For the sake of presentation, these vehicles making the HTs are called as HT vehicles, and the approaching lane with HT vehicles is called as HT lane, for example, the left lane at the south leg of intersection A in Figure 1.

The HT scheme has been widely adopted and successfully implemented in downtown Melbourne, mainly because Melbourne has the world largest tram system. Most of the main roads in the city have

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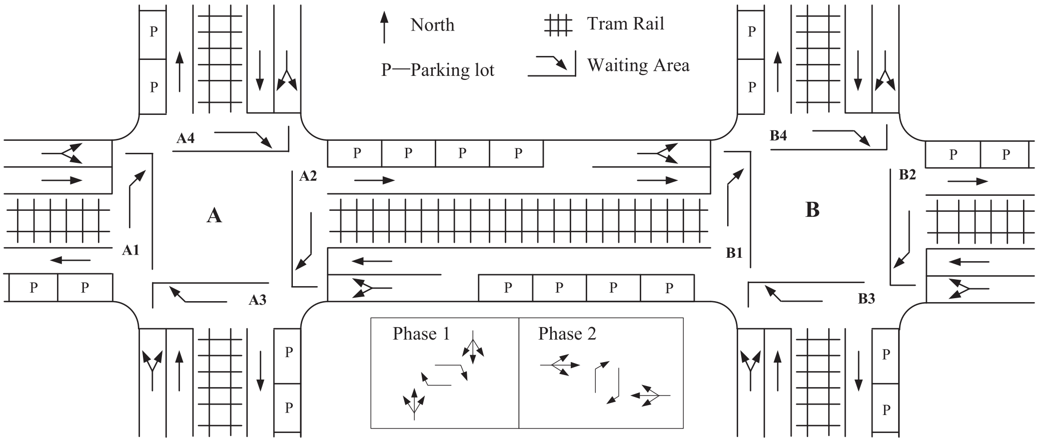

Figure 1. The typical geometry and phasing plan for hook-turn intersections in Melbourne.

bidirectional tram lines, which reduces the road space for mobile vehicles, such that dedicated lane and signal phase for right-turning vehicles are not suitable. This means that the right-turning vehicles have to share a signal phase with left-turning vehicles, through vehicles and trams, which leads to many conflict points and safety issues. Moreover, when the right-turning vehicles are dwelling and giving way to other vehicles, they block the approaching lane as well as through vehicles behind them, increasing the delay of through vehicles. Therefore, the HT scheme in Melbourne can (i) make better use of the intersection space and (ii) avoid the conflicts between right-turning vehicles and other traffic streams and thus reduces the delay of tram and through vehicles. Its successful implementations in urban Melbourne are solid evidence of its rationale and effectiveness.

The signal control mode inherently influences the operational efficiency of an intersection [6]. Although HT scheme has been implemented in Melbourne for tens of years, the studies on HT are still sparse. Most of the intersections in Melbourne still adopt fixed-time control mode (with no dynamic adjustments with regard to the traffic volumes). It is urgently needed to propose an algorithm for the optimal signal coordination plan of adjacent intersections with HT, which is an effective way to improve the travel efficiency of vehicles on arterial roads.

The remainder of this paper is organized as follows. Section 2 reviews the relevant literature on HT. Section 3 analyzes the special characteristics of traffic movements in HT intersections, in contrast with normal intersections. Section 4 establishes an algorithm for the optimal signal coordination plan, with the objective of minimizing average vehicle delay. Section 5 provides a real-world network example developed from three adjacent HT intersections in Melbourne. Based on real survey data, the numerical test in Section 5 is used to verify the proposed methodology. Section 6 finally concludes this study.

2. LITERATURE REVIEW

So far, the existing studies on HT mainly focus on the assessment and evaluation of HT schemes. For instance, in 2000, Andres O'Brien and Associates Pty. Ltd. [7] investigated the safety and operational effects of HTs, to determine whether the use of HTs should be continued, expanded, or reduced. It is concluded that HTs should be continued to be used in city area. They also used macroscopic modeling to examine the effects of removing HTs at seven junctions in the Melbourne central business district and replacing them with opposed right turns. By comparing the resultant degrees of saturation, it was found that four of these junctions would have lower capacity, whereas three had a slight improvement.

A seminal academic work on HT was provided by Currie and Reynolds [8]. They presented a review of the Melbourne HT and aimed to explore its impact on intersection operations. Operational analysis of the traffic impacts of HTs in Melbourne suggested that they acted to reduce congestion because turning traffic did not delay through vehicles. A series of safety analyses with crash data and conflict point analysis demonstrated that intersections with HTs had better safety performance than

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conventional intersections. Hounsell and Yap [9] compared the traffic performance of a hypothetical HT junction with an equivalent conventional junction with opposed right turns using S-paramics. It was found that the primary advantage of HTs was the removal of opposed right-turning vehicles that obstructed through traffic; overall network journey times were significantly lowered if the through movement was a dominant component of the junction flows.

Therefore, there are only few academic studies on HT, which mainly are focusing on the assessment of HT in contrast with other traffic regulation rules. To the best of our knowledge, there is no systematic study on HT regarding the optimization of signal timing plan or coordination between adjacent intersections.

Traffic signal coordination is an important traffic control mode, which can significantly reduce the delay of vehicles on the arterial roads by setting the common cycle length and signal offsets of adjacent intersections [10]. In urban Melbourne, some HT intersections also implement signal coordination, but the coordination plans are simply decided by the empirical experiences of the experts, which is fixed and suboptimal. It becomes an obstacle of further improving the efficiency of traffic movements. In addition, we can see from Figure 1 that the traffic movements at HT intersections are inherently different from those at a normal intersection. Thus, the existing signal coordination algorithms (e.g., MAXBAND method [11] and multiband [12]) for the normal intersections are not suitable for the coordination of HT intersections. Hence, this paper aims to propose an algorithm for the optimal signal coordination plan of adjacent intersections with HT. Minimizing the average delay of all the vehicles in the network is taken as the objective of such an algorithm.

3. TRAFFIC FLOW CHARACTERISTICS AT HOOK-TURN INTERSECTIONS

The traffic flow movements of the HT scheme are quite different with those of traditional scheme, mainly from the following two aspects:

(1) The HT vehicles can easily yield a spillback from the waiting area to the approaching lane.

We can see that the space of the waiting area is limited, which can only accommodate several vehicles. Hence, if the volume of HT vehicles is larger, the HT vehicles in the waiting area will spillback to the HT lane and block the movements of all the other vehicles on the HT lane, thus drastically increase the delay of these vehicles. In urban Melbourne, a waiting area of the HT vehicles usually can only accommodate three vehicles (standard sedans). Taking intersection A in Figure 1 as an example, the capacity of the waiting area in the intersection is endogenously determined by the number of lanes at the east/west leg.

On the other hand, the traditional/normal signal plan is not affected by such spillback. Figure 2 shows the queue discharge rates of the HT plan and normal signal plan. Figure 2(a) indicates the queue discharge rate of HT plan with spillback, while Figure 2(b) provides that of the normal plan. We can see that, at the HT intersection, when the spillback occurs, the discharge rate sharply drops to 0 pcu/h, implying that the remaining green time will be totally wasted. For the normal plan, the discharge rate is initially equal to the saturation flow rate S (pcu/h) and then reduces to the vehicle arrival rate q (pcu/h).

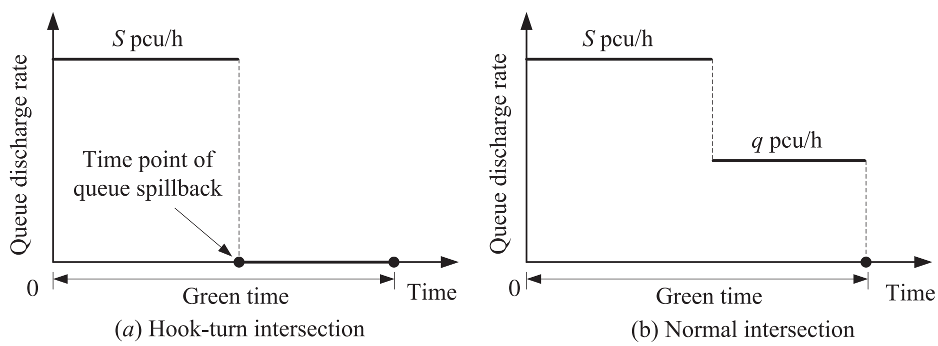

Figure 2. Comparison of the discharge rate during green phase of the approaching lane: (a) hook-turn intersection and (b) normal intersection.

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(2) There are two traffic streams at the coordinated phase of the HT intersection, with a gap in between.

In the coordinated control plan, the signal phase with the highest volume is usually selected as the coordinated phase, in order to enhance the efficiency of the vehicle movements in the network to a larger extend. As shown in Figure 1, if the ratio of through vehicles is high, then phase 2 can be set as the coordinated phase during the coordination control of intersections A and B. In this case, there are two traffic streams controlled by the coordinated phase: (i) the HT (right-turning) vehicles in waiting areas A1, A2, B1, and B2 and (ii) the through vehicles at the east/west leg. As per the rule of the HT maneuver, only after all the dwelling HT vehicles in the waiting areas are totally released, the through vehicles stopping on the side lane behind can start to move. Thus, the green time of phase 2 can be divided into two parts, parts I and II. There is a short time gap between them, which would affect the vehicle arrival pattern at the downstream intersection. This gap, hence, would affect the results of the optimal coordination plan (Figure 3).

Because of the unique characteristics of the HT plan, the traditional/existing coordination algorithms are not suitable for the coordination control of the HT plan, mainly because of the following:

(1) The maximum of the optimal cycle lengths should no longer be taken as the common cycle length for all the intersections.

In the traditional signal coordination algorithm, all the coordinated intersections take the common cycle length, in order to maintain a stable offset. The maximum of the individual cycle lengths is usually taken as the common cycle length. Here, the intersection with maximum cycle length is called as the critical intersection, and all the other intersections are termed as secondary intersections. The cycle lengths of the secondary intersections are prolonged, to increase their vehicle capacity as well as the width of green waves.

However, if the same approach is adopted for the HT intersections (increase the cycle length of the secondary intersections), the green times of all the phases at the secondary intersections will be correspondingly increased. Consequently, there will be more HT vehicles during the green time, inducing to spillback of the HT vehicles. Hence, the traditional coordination algorithm is not suitable for the HT intersections.

We use $C_{\max}$ to denote the maximal optimal cycle length and $C_{\min}$ the minimum. Let C_c denote the common cycle length. In the following three cases, we discuss about the influence of C_c on the signal coordination.

Firstly, $C_c > C_{\max}$. In this case, the cycle length of each coordinated intersection is increased. Longer green times of all the phases would induce to spillback of HT vehicles from each waiting area in the intersections.

Secondly, $C_{\min} \le C_c \le C_{\max}$. In this case, the cycle lengths of some intersections are increased, while the others are decreased. Increased cycle length would result in spillback. Reduced cycle length will reduce the vehicle capacity of the intersection and then induce to the oversaturation of the intersection.

Thirdly, $C_c \le C_{\min}$. In such case, the common cycle length C_c is less than the optimal cycle length of each intersection. Hence, the degree of saturation will increase and reach an oversaturated state.

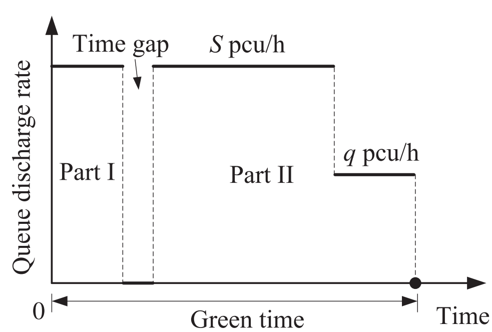

Figure 3. Vehicle discharging profile at the coordinated phase of hook-turn intersections.

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(2) Setting green split based on the principle of “equal degree of saturation” is not suitable for the HT plan.

For the signal control of normal intersections, the principle of equal degree of saturation is usually followed to set the green split at each intersection, in order to achieve a similar control status at each phase. However, if such a principle is still followed for the HT plan, then some phases will yield spillback, while other phases do not, inducing to a quite different control status among all the phases. This is because the spillback phenomenon is related to not only the green time length but also the ratio of right-turning vehicles at the HT lane. Yet, the principle of equal degree of saturation only concerns about the volume to saturation flow ratio of the HT lane. Because of the existence of waiting areas in the intersection, such a ratio can no longer reflect the nature of the traffic flows at the intersection.

Based on the aforementioned analysis, we can clearly see that the signal coordination of the HT case is quite different with that of the traditional case. The unique characteristics of HT scheme should be fully investigated, to develop a suitable coordination algorithm for the HT intersections.

4. DEVELOPMENT OF SIGNAL COORDINATION ALGORITHM FOR HOOK-TURN INTERSECTIONS

The objective of the signal coordination is to minimize the vehicle delay in the network. The decision variables in the optimization problem (also termed as coordination parameters) include the common cycle length, green time of each phase at each intersection, and offsets of adjacent intersections.

The example shown in Figure 1 is adopted to explain the development of signal coordination algorithm. We assume that the common cycle length of intersections A and B is C_c. At intersection A, the green time of phases 1 and 2 is g_{A1} and g_{A2}, respectively, and the intergreen time of phases 1 and 2 is I_{A1} and I_{A2}, respectively. At intersection B, the corresponding terms are denoted by g_{B1}, g_{B2}, I_{B1}, and I_{B2}, respectively. The common cycle length is defined as follows:

$$C_c = \sum_{i=1}^{2}(g_{Ai} + I_{Ai}) = \sum_{i=1}^{2}(g_{Bi} + I_{Bi}) \tag{1}$$

Without loss of generality, we set phase 2 as the coordinated phase. The offset of phase 2 in the direction of A to B is denoted by O_{BA}. We know that $0 \leq O_{BA} < C_c$. If the green time of phase 2 at intersection A starts at T_{Ags2}, then the green time of phase 2 at intersection B should start at T_{Ags2} plus O_{BA}. The offset of phase 2 in the direction of B to A is denoted by O_{AB}, and we have

$$O_{BA} = C_c - O_{AB} \tag{2}$$

In search for the optimal coordination plan, we should first establish the relationship between vehicle delay and the control parameters. Note that the coordinated phase and noncoordinated phase(s) are inherently different in terms of vehicle movements; thus, we take them as two different cases in the discussions as follows.

4.1. Vehicle delay of phase 1

The traffic streams controlled by phase 1 of intersections A and B are not affected by the offset. We assume that the arrivals of vehicles at phase 1 follow Poisson distribution. The traffic streams controlled by phase 1 can be categorized into three types: (i) the traffic dwelling at the waiting area (A3, A4, B3, and B4 in Figure 1); (ii) the traffic on the approaching HT lane; and (iii) the traffic on the dedicated through lane. The calculation of vehicle delay for each type of stream is further discussed in the following three subsections.

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4.1.1. Delay of vehicles in the waiting area

Let $Q_{Ak\max}$ denote the maximum number of vehicles that can be accommodated by waiting area k at intersection A. When phase 1 turns to green, the number of vehicles in the waiting area can be calculated by

$$Q_{Ak} = \min\left(Q_{Ak\,\max}, Q_{Akj}\right) \tag{3}$$

where Q_{Akj} is the number of vehicles entering area k from the adjacent HT lane j at intersection A, which can be obtained by

$$Q_{Akj} = \min\left[q_{Ajr}C_c/3600,\ g_{A2}S_{Aj}q_{Ajr}/\left(3600q_{Aj}\right)\right] \tag{4}$$

where q_{Aj} is the total volume (number of arrival vehicles in 1 hour) of lane j at intersection A, q_{Ajr} is the volume of right-turning vehicles of lane j at intersection A, and S_{Aj} denotes the saturation flow of lane j at intersection A. They are all in the unit of pcu/h. The following two cases are discussed in terms of different $Q_{k\,\max}$ and Q_{kj} values.

Case 1: $Q_{Ak\max} \geq Q_{Akj}$

In this case, it is not possible to have spillback from waiting area k. Thus, before phase 1 turns to green, the delay of all the Q_{Ak} vehicles can be calculated by Equation (5):

$$D'_{A1k} = (0.5g_{A2} + I_{A2})Q_{Ak} \tag{5}$$

where the average delay equals the half of the green time of phase 2 plus the intergreen time of phase 2. Then, after phase 1 turns to green, the vehicles in waiting area k will depart, and their delay, denoted by D''_{1k}, can be calculated by

$$\begin{aligned} D''_{A1k} &= 0.5(3600Q_{Ak}/S_{Ak})Q_{Ak} \\ &= 1800Q_{Ak}^2/S_{Ak} \end{aligned} \tag{6}$$

where $3600Q_{Ak}/S_{Ak}$ is the time required to release all the Q_{Ak} vehicles at waiting area k. S_{Ak} is the saturation flow of waiting area k at intersection A. Therefore, the delay of right-turning vehicles at waiting area, denoted by D_{A1k}, equals the sum of D'_{A1k} and D''_{A1k}.

Case 2: $Q_{Ak\max} < Q_{Akj}$

In this case, during the green time of phase 2 at intersection A, the spillback will occur and block approaching lane j. In the case of spillback, the number of HT vehicles passing the stop line is $Q_{Ak}+1$. According to the ratio of right-turning, through, and left-turning vehicles, we can obtain the total number of vehicles passing the stop line, denoted by Q_{Aj}:

$$Q_{Aj} = (Q_{Ak} + 1)q_{Aj}/q_{Ajr} \tag{7}$$

When the spillback occurs, the elapsed green time of phase 2, denoted by g_{A2}^{so}, is

$$g_{A2}^{so} = \begin{cases} 3600Q_{Aj}/S_{Aj} & \text{if } Q_{Aj} < Q_{Ajs} \\ g_{Ajs} + 3600\left(Q_{Aj} - Q_{Ajs}\right)/q_j & \text{if } Q_{Aj} \geq Q_{Ajs} \end{cases} \tag{8}$$

where g_{Ajs} is the time needed to release all the waiting vehicles at lane j with saturation flow rate, which can be given by

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$$g_{Ajs} = q_{Aj}(C_c - g_{A2})/\left(S_{Aj} - q_{Aj}\right) \tag{9}$$

Q_{Ajs} is the total number of vehicles released during time g_{Ajs}:

$$Q_{Ajs} = g_{Ajs}S_{Aj}/3600 \tag{10}$$

Before phase 1 turns to green, the delay of the Q_{Ak} vehicles can be calculated by

$$D'_{A1k} = \left[0.5g^{so}_{A2} + \left(g_{A2} - g^{so}_{A2}\right) + I_{A2}\right]Q_{Ak} \tag{11}$$

After phase 1 turns to green, the delay of vehicles in waiting area k $\left(D''_{A1k}\right)$ can still be measured by Equation (6). The total delay of vehicles in waiting area k at intersection A (D_{A1k}) is the sum of D'_{A1k} and D''_{A1k}.

4.1.2. Delay of vehicles on the hook-turn lane

The HT lane is usually a shared lane of left-turning and right-turning vehicles. After its signal turns to green, the HT vehicles then enter the waiting area; for instance, the HT vehicles at the south leg in Figure 1 should enter waiting area A1. This subsection only talks about the delay of the HT vehicles before passing the stop line, because the delay at the waiting area has been discussed previously in Subsection 4.1.1.

The spillback phenomenon has inherent effect on the vehicle delay of the HT lane. Let m denote the approaching HT lane under phase 1 at intersection A, and the volume of arrival vehicle is q_{A1m} pcu/h. Let $Q_{Ap\max}$ denote the maximum number of vehicles that can be accommodated by waiting area p in the intersection. Calculation of the vehicle delay is further classified into the following two cases:

Case 1: $Q_{Ap\max} \geq Q_{Apm}$

Here, Q_{pm} is the number of vehicles entering waiting area p during the green time of phase 1, which can be obtained by Equation (4). In this case, it is not possible to have a spillback. In accordance with the Highway Capacity Manual 2010 [13], the delay of vehicles waiting on lane m, denoted by D_{A1m}, can be obtained by

$$D_{A1m} = q_{A1m}C_c/3600\left\{\frac{0.5C_c(1-\lambda_{A1})^2}{1-[\min(1,x_{A1m})\lambda_{A1}]} + 900T\left[(x_{A1m}-1) + \sqrt{(x_{A1m}-1)^2 + \frac{8Kx_{A1m}}{Cap_{A1m}T}}\right]\right\} \tag{12}$$

where λ_{A1} is the green split of phase 1 at intersection A and x_{A1m} is the degree of saturation of lane m controlled by phase 1. Cap_{A1m} is the vehicle capacity of lane m, which equals the product of saturation flow rate and the green split of this lane. T is the length of the total analysis period, a default value of which is 0.25 h. K is an adjustment parameter, usually taking 0.5.

Case 2: $Q_{Ap\max} < Q_{Apm}$

The spillback of HT vehicles will happen in this case. When there is a spillback, lane m will be blocked, and the remaining green time of this phase will be wasted. The effective green time of lane m denoted by g^{so}_{A1} can be calculated by Equation (8), while the red time can be regarded to be increased for $g_{A1} - g^{so}_{A1}$ seconds. Herein, we still use Equation (12) to calculate the vehicle delay D_{A1m}. However, the green time of phase 1 (denoted by g_{A1}) should be replaced by g^{so}_{A1}, when used to calculate the green split, vehicle capacity of each phase, as well as the degree of saturation.

4.1.3. Delay of vehicles on the dedicated through lane

The dedicated through lane is not affected by the HT scheme; thus, the vehicle movement on this lane is the same as that at a normal intersection. Equation (12) can be adopted to calculate the vehicle delay in this case, and the details are not further covered here because of the space limit. The total delay of vehicles arrive at through lane under phase 1 is denoted as D_{A1t}.

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4.2. Vehicle delay of phase 2

For the example indicated in Figure 1, phase 2 is the coordinated phase. Note that not all the vehicles controlled by phase 2 are affected by the offset of the signal coordination. For example, the traffic streams on the west leg of intersection A and east leg of intersection B are out of the coordinated subarea; thus, their delay can be calculated using the methodology discussed in Section 4.1. However, the traffic arrival patterns on the east leg of intersection A and west leg of intersection B are highly affected by the offset. This section thus introduces the methodology for calculation of the vehicle delay, which is quite different from the methods in Section 4.1.

Without loss of generality, the west leg of intersection B is taken here to elaborate the calculation of vehicle delay. Let T_{Ags2} denote the start time of the green signal of phase 2 at intersection A, and then the start time of phase 2 at intersection B T_{Bgs2} equals T_{Ags2} plus O_{BA}. Hereafter, we use a standard time unit Δt to measure the cycle length and travel time. The green time g_{A2} of phase 2 at intersection A is divided into U_{A2} intervals, that is,

$$U_{A2} = g_{A2}/\Delta t \tag{13}$$

Let r denote the through lane at west leg of intersection A. Then, at the ith interval at the green time of phase 2, the number of vehicles released from lane r, denoted by $Q^s_{Ar}(i)$, can be obtained by

$$Q^s_{Ar}(i) = \begin{cases} S_{Ar}\Delta t/3600 & if \;\; i \leq U^s_{A2} \\ q_{Ar}\Delta t/3600 & if \;\; i > U^s_{A2} \end{cases} \tag{14}$$

where S_{Ar} and q_{Ar} are the saturation flow rate and arrival flow rate of lane r and U^s_{A2} is the number of intervals of the green time that vehicles are released at saturation rate. Evidently, U^s_{A2} is a portion of U_{A2}, which is calculated by

$$U^s_{A2} = \frac{q_{Ar}(C_c - g_{A2})}{(S_{Ar} - q_{Ar})\Delta t} \tag{15}$$

The vehicles released at the U_{A2} intervals from intersection A will disperse, because of the different travel speeds. The degree of dispersion is decided by the distance between intersections A and B and the average speed of the vehicles. This paper uses the Robertson's model [14, 15] to define the dispersion of the traffic stream, which is

$$Q_{dB}(w) = \sum_{i=1}^{w-t} Q^s_{Ar}(i)F(1-F)^{w-t-i} \tag{16}$$

where $F = \frac{1}{1+0.35t}$ is the dispersion parameter of the traffic flow. $Q_{dB}(w)$ is the number of arrival vehicles at the downstream intersection in interval w. t is the 80th percentile of the travel time between two intersections, which is also measured by the number of intervals in terms of standard time unit Δt, namely

$$t = (0.8L_{AB}/V_{AB})/\Delta t \tag{17}$$

where L_{AB} is the length of the road segment between intersections A and B and V_{AB} is the average speed of the vehicles traveling between intersections A and B.

Figure 4 indicates the example of discharge pattern at intersection A and the arrival pattern at intersection B, in the case of signal coordination. By means of Equations (14) and (16), we can measure the

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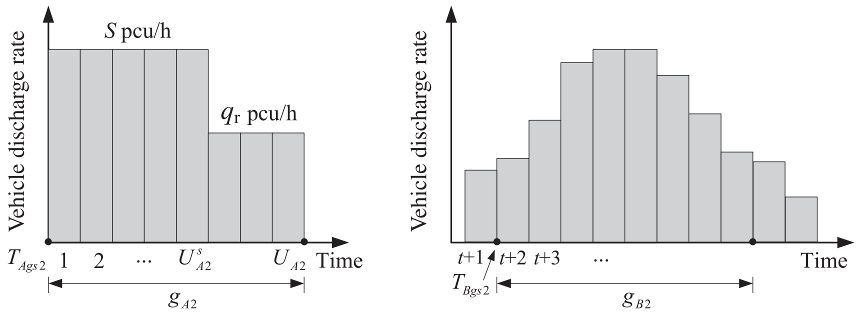

(*a*) Discharge pattern at intersection A (*b*) Arrival pattern at intersection B

Figure 4. Example of vehicle discharge and arrival patterns at coordinated phases: (a) discharge pattern at intersection A and (b) arrival pattern at intersection B.

number of arrival vehicles to intersection B at each time interval. Then, in light of the start time and end time of the green time of phase 2 at intersection B, we can calculate the total number of arrival vehicles during this green time.

Assume that the green time of phase 2 at intersection B is divided into U_{B2} intervals by the standard time unit Δt. These U_{B2} intervals are numbered from w'; then the total number of arrival vehicles during the green time, denoted by Q_{dB2}, equals

$$Q_{dB2} = \sum_{w=w'}^{w'+U_{B2}-1} Q_{dB}(w) \tag{18}$$

Highway Capacity Manual 2010 [13] has provided the average delay of vehicles on the road segment between two coordinated intersections:

$$\overline{d}_i = PF \cdot \frac{0.5C(1-\lambda_i)^2}{1-[\min(1,x_i)\lambda_i]} + 900T\left[(x_i-1)+\sqrt{(x_i-1)^2+\frac{8Kx_i}{Cap_iT}}\right] \tag{19}$$

where $\overline{d}_i$ is the average delay of the vehicles controlled by phase i. PF is the uniform delay progression adjustment factor, which accounts for effects of signal coordination (signal progression).

Good signal coordination plan will result in a high proportion of vehicles arriving on the green phase at the downstream intersection. Progression primarily affects uniform delay; thus, the adjustment is only made on the first term in Equation (19). The value of PF can be determined by

$$PF = \frac{(1-P)f_{PA}}{1-\lambda_i} \tag{20}$$

where P is the ratio of vehicles arriving on green. f_{PA} is supplemental adjustment factor for platoon arriving during green, which can be set as 1.0. Hence, PF is mainly determined by the value of P. Based on Equation (18), the value of P can be easily obtained.

Let z denote the approaching lane at west leg of phase 2 at intersection B. The ratio between the number of arrival vehicles during the green time of phase 2 and the total number of arrival vehicles in one cycle is denoted by P_{B2z}, and P_{B2z} equals

$$P_{B2z} = Q_{dB2}/\left(C_c q_z/3600\right) \tag{21}$$

Substituting PF in Equation (19) by P_{B2z}, we can easily obtain the average delay of vehicles on lane z, which are controlled by phase 2 of intersection B. The average vehicle delay on lane z is denoted by D_{B2z}. The total vehicle delay on lane z equals $\overline{d}_{B2z}C_c q_{Bz}/3600$.

The method discussed previously provides the calculation of the vehicle delay controlled by the coordinated phase at intersection B. Note that the vehicle delay at each leg of intersection A can be calculated similarly, which is not further repeated here.

4.3. *Optimization model*

The foregoing Sections 4.1 and 4.2 took an in-depth analysis of the characteristics of HT scheme as well as the calculation of vehicle delay in each particular case. Based on this analysis, we can obtain the average delay of all the vehicles traveling in the coordinated subarea, denoted by $\overline{D}$. Evidently, $\overline{D}$ is a function of the common cycle length C_c and green time g_{ni} of each phase i at intersection n. From Equation (1), we can see that C_c can be substituted by a function of g_{ni}, which makes $\overline{D}$ only a function of g_{ni}, denoted by $\Phi\ (g_{ni})$. It should be pointed out that $\overline{D}$ is the average delay of all the vehicles in 1 hour rather than in one signal cycle.

Taking minimizing the average vehicle delay $\overline{D}$ as the objective, we can build the following optimization model:

$$\begin{aligned} &\min \quad \overline{D} = \Phi\ (g_{ni}) \\ &s.t. \\ &\quad g_{ni\min} - g_{ni} \leq 0,\ 1 \leq n \leq N, 1 \leq i \leq 2 \\ &\quad g_{ni} - g_{ni\max} \leq 0,\ 1 \leq n \leq N, 1 \leq i \leq 2 \end{aligned} \tag{22}$$

where N is the total number of coordinated intersections in the study area.

We proceed to discuss about the specific expression of the objective function $\Phi\ (g_{ni})$. The average vehicle delay $\overline{D}$ of intersections A and B equals

$$\overline{D} = \sum_{n=A}^{B} \left(\sum_{\substack{i=1\\k=1}}^{\substack{k=K_{ni}\\i=2}} D_{nik} + \sum_{\substack{i=1\\m=1}}^{\substack{m=M_{ni}\\i=2}} D_{nim} + \sum_{\substack{i=1\\t=1}}^{\substack{m=M_{ni,t\neq z}\\i=2}} D_{nit} + \sum_{z=1}^{Z_n} D_{n2z} \right) \Big/ \sum_{n=A}^{B} Q_n \tag{23}$$

where D_{nik} is the total delay of vehicles in waiting area k under phase i at intersection n. D_{nim} is the total delay of vehicles in HT lane m under phase i at intersection n. D_{nit} is the total delay of vehicles in the dedicated through lane t, which is not affected by the offset. D_{n2z} is the total delay of vehicles in the dedicated through lane z, which is affected by the offset. Q_n is the total number of vehicles arriving at intersection n in one cycle, and it equals the sum of the products of the arrival volume of each lane and the cycle length. K_{ni} is the number of waiting areas under phase i at intersection n. M_{ni} is the number of HT lanes under phase i at intersection n. T_{ni} is the number of dedicated through lanes that is not affected by the offset. Z_n is the number of dedicated through lanes that is affected by the offset at intersection n.

Based on the discussions in Sections 4.1 and 4.2, we know that the delay of each vehicle stream in Equation (23) equals

$$D_{nik} = \begin{cases} \left(0.5g_{n(i+1)} + I_{n(i+1)}\right)Q_{nk} + 1800Q_{nk}^2/S_{nk} & \text{if } Q_{nk\max} \geq Q_{nkj} \\ \left[0.5g_{n(i+1)}^{so} + \left(g_{n(i+1)} - g_{n(i+1)}^{so}\right) + I_{A(i+1)}\right]Q_{nk} + 1800Q_{nk}^2/S_{nk} & \text{if } \ Q_{nk\max} < Q_{nkj} \end{cases} \tag{24}$$

$$D_{nim} = q_{nim}C_c/3600\left\{\frac{0.5C_c(1-\lambda_{ni})^2}{1-[\min(1,x_{nim})\lambda_{ni}]} + 900T\left[(x_{nim}-1) + \sqrt{(x_{nim}-1)^2 + \frac{8Kx_{nim}}{Cap_{nim}T}}\right]\right\} \tag{25}$$

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where $\lambda_{ni} = \begin{cases} g_{ni}/C_c & \text{if } Q_{np\max} \geq Q_{npj} \\ g_{ni}^{so}/C_c & \text{if } Q_{np\max} < Q_{npj} \end{cases}$.

$$D_{nit} = q_{nit}C_c/3600\left\{\frac{0.5C_c(1-\lambda_{ni})^2}{1-[\min(1,x_{nit})\lambda_{ni}]} + 900T\left[(x_{nit}-1)+\sqrt{(x_{nit}-1)^2+\frac{8Kx_{nit}}{Cap_{nit}T}}\right]\right\} \tag{26}$$

$$D_{n2z} = \left\{PF\cdot\frac{0.5C(1-\lambda_{n2})^2}{1-[\min(1,x_{n2})\lambda_{n2}]} + 900T\left[(x_{n2}-1)+\sqrt{(x_{n2}-1)^2+\frac{8Kx_{n2}}{Cap_{n2}T}}\right]\right\}C_c q_{n2z}/3600 \tag{27}$$

where PF is the function of offsets. x_{ni} is the saturation degree of phase i at intersection n. Assuming that lane m is the critical lane of phase i, then x_{ni} equals

$$x_{ni} = x_{nim} = S_{nm}g_{ni}/(C_c q_{nm}) \tag{28}$$

where S_{nm} is the saturation flow rate of lane m and q_{nm} is the arrival traffic volume of lane m.

As shown in Equation (1), C_c is the sum of green times and intergreen times of all phases. Thus, from Equation (28), we can see that there is a nonlinear mapping between x_{ni} and g_{ni}. With the increase of g_{ni}, x_{ni} decreases quickly. After g_{ni} reaches a certain value, the decrease of x_{ni} becomes slower.

4.4. Solution algorithm

Equations (24–28) define the objective function $\Phi\ (g_{ni})$ of model (22). We can see that this objective function is highly nonlinear, nonconvex, and also a stepwise function. The green time g_{ni} takes integer values; thus, model (22) is a mixed nonlinear integer program. Hence, it is a nondeterministic polynomial-time-hard problem, which is difficult and inefficient to be solved by any gradient-based algorithms in the literature.

For such sort of complex engineering-based problem, some existing heuristics are more suitable to be taken as the solution method, which does not require the calculation of gradient. In this section, we adopt the genetic algorithm (GA). GA consists of three main processes: selection, crossover, and mutation. For the addressed problem, each chromosome defines a signal green time plan g_{ni}, which can be conveniently used to calculate the value of objective function $\Phi\ (g_{ni})$ and the selection process. The initial generation of chromosomes as well as crossover and mutation is all conducted with the aids of pseudorandom numbers. Because of the space limit, the details of GA are not further included here. Any interested readers can refer to References [16–21].

5. NUMERICAL EXPERIMENTS

To verify the proposed methodology, a case study is developed using VISSIM (Planung Transport Verkehr AG, Karlsruhe, Germany) as numerical experiments. This case study is introduced in Section 5.1 as follows.

5.1. Design of the experiments

The study area includes three intersections in downtown Melbourne, which are the junctions of Latrobe Street with William Street, Queue Street, and Elizabeth Street, respectively. As shown in Figure 5, these three intersections are adjacent, and the distance in-between is 235 and 228 m. Each of them has two signal phases. The two intersections on Elizabeth Street and William Street both have four waiting areas, that is, all the four legs have HT maneuvers. The intersection on Queue Street has HT maneuvers for the east and west legs (controlled by phase 2), while the south and north legs have dedicated lane for the right-turning vehicles. This example is then adopted to test the proposed coordination control algorithm.

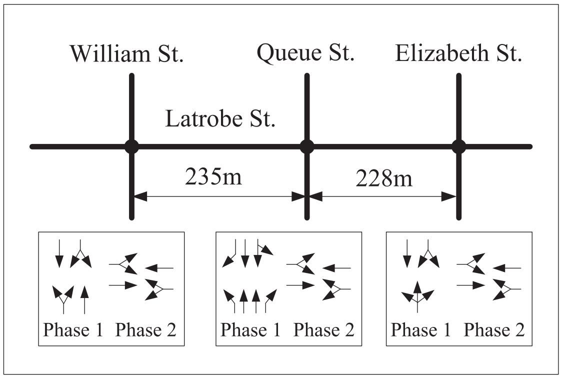

Figure 5. Layouts and phasing schemes of the three adjacent hook-turn intersections in Melbourne.

In practice, the three intersections currently all adopt fixed traffic signal plan, taking phase 2 as the coordinated phase, and the signal plan is indicated in Table I. The green gap is 6 s, including 3 s yellow time and 3 s all red time. The common cycle length is 90 s. As to the offsets, we take the east/west bound as an example; the green light of phase 2 at the intersection on William Street, Queue Street, and Elizabeth Street starts at time 0, 8, and 17 s, respectively.

In the morning peak (8:15–8:45) and off-peak hour (9:30–10:00), the traffic volume data were collected by a field survey, which are provided in Table II. The waiting areas for HT vehicles can accommodate 3 pcu. Saturation flow rates of waiting area, HT lane, and through lane are 1200, 1340, and 1520 pcu/h, respectively. The average travel speed is 40 km/h.

5.2. *Optimal signal coordination plan*

The settings of GA are as follows. Population size is 50. Reproduction operator is binary tournament selection. Crossover operator is uniform crossover, and the probability is 0.8. Mutation operator is creep mutation operator, with a probability of 0.05. The maximal number of generation is 100. The program is coded in Matlab (MathWorks Inc., Natick, MA, USA) to solve the optimal timing plan.

Table I. Current signal plans of the three adjacent hook-turn intersections (unit: s).

Intersection	g_1	g_2	I_i	C_c
Latrobe St. and William St.	40	38	6	90
Latrobe St. and Queue St.	37	41		
Latrobe St. and Elizabeth St.	38	40		

Table II. The traffic volumes obtained from field surveys (unit: pcu/h).

Intersection		Peak hour			Off-peak hour		
		Left	Through	Right	Left	Through	Right
Latrobe St. and William St.	South leg	152	482	182	116	236	138
	North leg	214	420	160	162	344	124
	East leg	306	562	150	254	458	98
	West leg	210	432	112	174	352	84
Latrobe St. and Queue St.	South leg	146	620	118	116	528	90
	North leg	194	742	176	124	654	136
	East leg	258	484	134	222	392	86
	West leg	234	444	130	188	388	72
Latrobe St. and Elizabeth St.	South leg	126	274	96	112	228	68
	North leg	208	416	138	168	340	114
	East leg	246	442	120	204	368	76
	West leg	212	436	112	176	356	64

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The minimum green time is set as 15 s, and the maximum green time is 60 s. There are three intersections, and each one has two phases; thus, totally, we have six variables for the optimization. The results are tabulated in Table III.

We can see that in the morning peak, the common cycle length is 88 s, which is close to 90 s in real world. The offset between the three intersections in Figure 3 (from left to right) is 83 and 7 s, respectively. In the off-peak hour, the common cycle length is 73 s, which is significantly less than 90 s in reality, and the offset values are 69 and 3 s, respectively.

5.3. *Results evaluation*

In order to fully contrast the current signal plan and the optimal signal plan in Table III, we proceed to establish two projects in VISSIM and use the simulation results to comprehensively evaluate their performances. The project with current/practical signal plan is termed as scheme I, and the one with optimal signal plan is scheme II. The network shown in Figure 3 is adopted to build the two projects, with adjusted saturation flow rate, surveyed traffic volume as well as the traffic regulation plan.

The following four indexes are adopted to quantify the performance of each scheme:

- E1: average delay of through vehicles on the east/west leg.
- E2: average delay of HT vehicles.
- E3: average number of spillbacks that occurred in one waiting area per hour.
- E4: average delay of all the vehicles traveling in the study area.

The modeling procedure of an HT intersection in VISSIM is significantly different from that of a normal intersection. This is mainly because the unique phasing scheme resulted from HT. Let us take intersection A in Figure 1 as an example to illustrate the modeling procedure. In practice, vehicles in waiting areas A1, A2, and west and east approaching lanes are controlled by phase 1. There are two phases at intersection A. After phase 1 turns to green, vehicles in waiting areas A1 and A2 first leave the intersection. Then vehicles on west and east approaching lanes depart and leave the intersection. However, in VISSIM environment, if we set the same phasing scheme for an HT intersection, vehicles on east and west approaching lanes would enter the intersection when signal light of phase 1 turns to green, even before the vehicles in waiting areas have left the intersection. Thus, in VISSIM environment, we set four phases for intersection A. Vehicles in waiting areas A1 and A2 are controlled by phase 1. Vehicles on west and east approaching lanes are controlled by phase 2. In south and north directions, there are also two signal phases. The intergreen time of phases 1 and 3 is all set as 0 s. Besides, a loop detector is placed at the stop line of each waiting area. When no vehicle is detected by the detector, the current green light would be terminated. The difference between simulation and practice is minimized by this approach.

In the VISSIM simulations, the random seed would affect the probabilistic distribution of the arrival vehicles. Thus, we take five independent runs with different random seeds, to remedy the effects of random seed on the results. The values of random seeds are set as 20, 30, 40, 50, and 60 in each run. The average results of these five runs are taken for the evaluation. Each run is operated for 4500 s, and the data between 300 and 3900 s are collected for the analysis.

There are 10 waiting areas in the example, as shown in Figure 3. To detect the spillback phenomenon, a detector is set between the waiting area and corresponding approaching HT lane. So, if a vehicle dwells at the detector area for sufficiently long (larger than 10 s), it is recognized that there is a spillback of HT vehicles.

Table III. The optimal signal coordination control plan (unit: s).

	Peak hour				Off-peak			
Intersection	g_1	g_2	C_c	Offset	g_1	g_2	C_c	Offset
Latrobe St. and William St.	36	40	88	—	30	31	73	—
Latrobe St. and Queue St.	32	44		83	27	34		69
Latrobe St. and Elizabeth St.	35	41		7	29	32		3

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Table IV provides the evaluation and comparison results, where “Improvement” means how much the optimal plan has reduced the index values compared with the practical plan. Taking index E4 as an example, for the morning peak hour, the value of scheme I is 44.6 s, while the value of scheme II is 39.3 s that is 10.8% lower than scheme I. For the off-peak hour, the E4 index value of optimal plan is 16.7% lower than that of the practical plan, showing a more significant improvement. Note that the other three indexes also show the same trend (the improvement in off-peak hour is more significant than in the peak hour).

Performance in terms of each index is further analyzed as follows:

(1) E1: average delay of through vehicles on the east/west leg

The average delay of scheme II in terms of E1 is obviously less than that of scheme I, and there are two reasons: Firstly, the optimization model accounts for the dispersion of the traffic platoon; thus, the resultant offset can better suit the arrival pattern of the vehicles. Secondly, the model considers about the influence of HT vehicles in the waiting area (e.g., areas A1 and A2 in Figure 1) on the through vehicles at east/west leg. We clearly indicate that during the green time of the coordinated phase, the released traffic has two streams with a temporal gap in-between. However, these two aspects are not considered in designing the current signal pattern at the intersections in Melbourne.

The outperformance of scheme II is more evident in the off-peak hour, compared with the peak hour case. This is because the signal plan being operated in downtown Melbourne is a fixed scheme, which is not dynamically changing subject to the traffic volumes. Such a fixed plan functions well during the peak hour, yet for the off-peak hour, the common cycle length is clearly too long inducing to a higher vehicle delay.

(2) E2: average delay of HT vehicles

The value of index E2 varies following the same trend as E1 yet with mild amplitude. The change is mild because the HT vehicles need to stop twice at the intersection (before the stop line and then in the waiting area); thus, the coordination plan could not significantly reduce their delay. In addition, the capacity of each waiting area is quite limited, leaving little space for the coordination plan to perform. This feature is more evident during the peak hours. Thus, at the morning peak hour, the outperformance of scheme I is much mild.

(3) E3: average number of spillbacks that occurred in one waiting area per hour

In the morning peak, the number of spillback is 6.8 under scheme II, which is 18.1% lower than 8.3 of scheme I. During the off-peak hour, scheme II is 22.2% lower than scheme I. Note that the volume of right-turning vehicles is high during the morning peak, inducing to more spillbacks, whereas there are less right-turning vehicles during the off-peak hours, giving rise to lower possibility of spillback. Yet, the spillback still occurs during off-peak hour, because a better coordination plan needs to increase the cycle length as well as the green time of coordinated phase, and longer green time eventually induces to more spillback. Hence, to deal with this trade-off between the spillback and coordination plan, the proposed methodology aims to find an optimal solution.

(4) E4: average delay of all the vehicles traveling in the study area

Compared with scheme I, scheme II can improve the E4 index by 10.8% and 16.7% under the morning peak and off-peak hours, respectively. The value of E4 index is inherently related to E1 and E3.

Table IV. Comparison of the two schemes in terms of four indexes.

	Morning peak hour				Off-peak hour			
Scheme	E1 (s)	E2 (s)	E3	E4 (s)	E1 (s)	E2 (s)	E3	E4 (s)
Scheme I	34.8	52.2	8.3	44.6	25.3	39.8	3.6	34.7
Scheme II	29.9	46.8	6.8	39.8	19.6	34.4	2.8	28.9
Improvement	14.1%	10.3%	18.1%	10.8%	22.5%	13.6%	22.2%	16.7%

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We can see that, for E1 and E3, the outperformance of scheme II is more evident in off-peak, compared with the peak hour case. Therefore, the index E4 follows the same changing trend with E1 and E3.

6. CONCLUSIONS

Hook turn is a unique traffic regulation scheme to control the movements of right-turning vehicles at the intersection. This paper developed a signal coordination algorithm for adjacent intersections with HT schemes. A real-case example and field survey were finally adopted to test the proposed methodology. The results indicated that, compared with the current signal plan, the optimal signal plan can significantly improve the operation of the intersection in terms of all the four different indexes, in both peak hour and off-peak hour cases.

It should be pointed out that although the HT scheme can avoid the conflict between right-turning vehicles and other traffic streams (especially trams) and improve the safety level, it has very limited capacity for HT vehicles in the waiting area. Hence, if the volume of right-turning vehicles is high, the spillback will occur and then drastically increase the delay. Therefore, HT is only suitable for those intersections with lower volume of right-turning vehicles. In urban Melbourne, because of the characteristics of the Origin-Destination (OD) trips and network structure, the volume of right-turning vehicles is not high. These features give rise to successful implementations of HT scheme in Melbourne for more than 50 years. Yet, for other cities, an in-depth analysis is necessary on the traffic flows and networks, before fully introducing the HT schemes.

In this paper, simulations were employed to test the effectiveness of the established algorithm. Although simulation projects were carefully calibrated, there is inevitably small difference between simulation and practice. Besides, the delay models developed in this paper are not validated using field data. Future research will be focused on validating the delay models and implementing the algorithm to real HT intersections.

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7. LIST OF SYMBOLS AND ABBREVIATIONS

7.1. Symbols

q	vehicle arrival rate, pcu/h
S	Saturation flow rate, pcu/h
$C_{\max}$	maximal optimal cycle length, s
$C_{\min}$	minimal optimal cycle length, s
C_{c}	common optimal cycle length, s
g_{Ai}	green time of phase i at intersection A, s
g_{Bi}	green time of phase i at intersection B, s
I_{Ai}	intergreen time of phase i at intersection A, s
I_{Bi}	intergreen time of phase i at intersection B, s
O_{BA}	offset between intersections A and B in the direction of A to B, s
O_{AB}	offset between intersections A and B in the direction of B to A , s
T_{Agsi}	the start time of green light of phase i at intersection A, s
$Q_{Ak\max}$	maximum number of vehicles that can be accommodated by waiting area k at intersection A, pcu
Q_{Akj}	number of vehicles entering area k from the adjacent HT lane j at intersection A, pcu
q_{Aj}	total volume (number of arrival vehicles in one hour) of lane j at intersection A, pcu/h
q_{Ajr}	volume of right-turning vehicles of lane j at intersection A, pcu/h
S_{Aj}	saturation flow of lane j at intersection A, pcu/h

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D'_{A1k}	delay of all the Q_{Ak} vehicles before phase 1 turns to green, s
D''_{1k}	delay of vehicles in waiting area k after phase 1 turns to green, s
D_{A1k}	delay of right-turning vehicles at waiting area k, s
g^{so}_{A2}	elapsed green time of phase 2 when the spillback occurs, s
g_{Ajs}	the time needed to release all the waiting vehicles at lane j with saturation flow rate, s
Q_{Ajs}	total number of vehicles released during time g_{Ajs}, pcu
λ_{Ai}	green split of phase i at intersection A
Cap_{A1m}	vehicle capacity of lane m under the control of phase 1 at intersection A, pcu
T	length of the total analysis period, s
K	adjustment parameter
U_{A2}	A phase 2 green time of intersection is divided into several periods
Δt	standard time unit, s
$Q^s_{Ar}(i)$	number of vehicles released from lane r at the *ith* interval at the green time of phase 2 at intersection A, pcu
U^s_{A2}	number of intervals of the green time that vehicles are released at saturation rate at intersection A
$Q_{dB}(w)$	number of arrival vehicles at the intersection B in interval w, pcu
t	80 percentile of the travel time between two intersections
L_{AB}	length of the road segment between intersections A and B, m
V_{AB}	average speed of the vehicles travelling between intersections A and B, m/s
$\overline{d}_i$	average delay of the vehicles controlled by phase i, s
PF	uniform delay progression adjustment factor
P	ratio of vehicles arriving on green to total arrival vehicles in one cycle
f_{PA}	supplemental adjustment factor for platoon arriving during green
P_{B2z}	the ratio between the number of arrival vehicles during the green time of phase 2 and the total number of arrival vehicles in one cycle
$\overline{D}$	average delay of all the vehicles travelling in the coordinated subarea, s
Φ	average delay function
$g_{ni\,\min}$	minimal green time of phase i at intersection n, s
$g_{ni\,\max}$	maximal green time of phase i at intersection n, s
N	total number of coordinated intersections in the coordinated area
x_{ni}	saturation degree of phase i at intersection n

7.2. Abbreviations

HT	Hook turn
OD	Origin-Destination
CBD	Central business district
pcu	Per car unit
SCA	Signal coordination algorithm
HCM	Highway Capacity Manual
GA	Genetic algorithm

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