A Signal Coordination Algorithm for Adjacent Hook-turn Intersections

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Abstract

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 objective function. Finally, a numerical example is built based on three HT intersections in downtown Melbourne, to verify the proposed methodology. Based on a comparison of 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.

Keywords: 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 (1-2), dedicated lane/waiting-area for right-turning vehicles in the intersection area (3), dedicated right-turning signal phase (4). This paper addresses a relatively innovative regulation scheme, terms 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 off-side to the curbside. During the green time for this leg, the right-turning vehicles first enter the intersection and park at the 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 Hook turns are called as HT vehicles, and the approaching lane with HT vehicles is called as HT lane.

![Diagram of HT intersection in Melbourne](image)

FIGURE 1 The typical geometry and phasing plan for HT intersections in Melbourne

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 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 gives rise 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 (a) make better use of the intersection space, (b) 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 pays inherently influence on the operational efficiency of an intersection. 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 regards 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 Hook turn. Section 3 establishes an algorithm for the optimal signal coordination plan, with the objective of minimizing average vehicle delay. Section 4 provides a real-world network example developed from three adjacent HT intersections in Melbourne. Based on real survey data, the numerical test in Section 4 is used to verify the proposed methodology. Section 5 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. investigated the safety and operational effects of Hooke turns, to determine whether the use of Hook turns should be
continued, expanded or reduced. It is concluded that Hook turns should be continued to be used in
city area. They also used macroscopic modeling to examine the effects of removing hook turns at
seven junctions in the Melbourne central business district (CBD) and replacing them with opposed
right turns. By comparing the resultant degrees of saturation, it was found that four of these
junctions would have poorer capacity, whereas three had a slight improvement.

A seminal academic work on HT was provided by Currie and Reynolds (6). They presented a
review of the Melbourne hook turn and aimed to explore its impact on intersection operations.
Operational analysis of the traffic impacts of Hook turns 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 Hook
turns had better safety performance than conventional intersections. Hounsell and Yap (7)
compared the traffic performance of a hypothetical Hook-turn junction with an equivalent
conventional junction with opposed right turns using S-Paramics. It was found that the primary
advantage of Hook turns 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 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 (8-9). In urban Melbourne, some HT intersections also implements 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 (10) and Multi-band (11)) 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. DEVELOPMENT of SIGNAL COORDINATION ALGORITHM for HT 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, offsets of adjacent intersections.

The example shown in Figure 1 is adopted to explain the development of Signal Coordination Algorithm (SCA). Assume the common cycle length of intersections A and B is \( C_c \). At intersection A, the green time of phase 1 and 2 is \( g_{a1} \) and \( g_{a2} \) respectively, and the intergreen time of phase 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:

\[
C_c = \sum_{i=1}^{2} (g_{ai} + I_{ai}) = \sum_{i=1}^{2} (g_{bi} + I_{bi})
\]  

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 \( O_{ab} \). We know that \( 0 \leq O_{ab} < C_c \). If the green time of phase 2 at intersection A starts at \( T_{g2_a} \), then the green time of phase 2 at intersection B should start at \( T_{g2_b} + O_{ab} \). The offset of phase 2 in the direction of B to A is denoted by \( O_{ba} \), and we have:

\[
O_{ba} = C_c - O_{ab}
\]  

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 non-coordinated phase(s) are inherently different in terms of vehicle movements, thus we take them as two different cases in the discussions as follows.
3.1 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 the arrivals of vehicles at phase 1 follow Poisson distribution. The traffic streams controlled by phase 1 can be categorized into three types: (1) the traffic dwelling at the waiting area (A3, A4, B3 and B4 in Figure 1); (2) the traffic on the approaching HT lane; (3) the traffic on the dedicated through lane. The calculation of vehicle delay for each type of stream is further discussed in the following three sub-sections.

3.1.1 Vehicle delay in the waiting area

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

$$Q_k = \min(Q_{\text{max}}, Q_j)$$

where $Q_j$ is the number of vehicles entering area $k$ from the adjacent HT lane $j$, which can be obtained by:

$$Q_j = \min\left[\frac{q_j C_j}{3600}, \frac{g_j S_j q_j}{(3600 q_j)}\right]$$

where $q_j$ is the total volume (number of arrival vehicles in one hour) of lane $j$, $q_{jr}$ is the volume of right-turning vehicles of lane $j$, $S_j$ denotes the saturation flow of lane $j$. They are all in the unit of pcu/h. The following two cases are discussed in terms of different $Q_{\text{max}}$ and $Q_j$ values.

Case 1: $Q_{\text{max}} \geq Q_j$

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

$$D'_{\text{lt}} = (0.5 g_2 + I_2) Q_k$$

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 \( D_{\text{st}}' \), can be calculated by:

\[
D_{\text{st}}' = 0.5 \left( \frac{3600 Q_k}{S_k} \right) Q_k
\]

\[
= 1800 \frac{Q_k^2}{S_k^2} \tag{6}
\]

where \( 3600 Q_k / S_k \) is the time required to release all the \( Q_k \) vehicles at waiting area \( k \). \( S_k \) is the saturation flow of waiting area \( k \). Therefore, the delay of right-turning vehicles at waiting area, denoted by \( D_{\text{st}} \), equals the sum of \( D_{\text{st}}' \) and \( D_{\text{st}}'' \).

**Case 2:** \( Q_{\text{largest}} < Q_j \)

In this case, during the green time of phase 2, the spillback will occur, and block approaching lane \( j \). In the case of spillback, the number of HT vehicles passing the stop line is \( Q_k + 1 \). According to the ratio of right-turning, through and left-turning vehicles, we can get the total number of vehicles passing the stop line, denoted by \( Q_j \):

\[
Q_j = (Q_k + 1) q_j / q_m \tag{7}
\]

When the spillback occurs, the elapsed green time of phase 2, denoted by \( g_j' \), is

\[
g_j' = \begin{cases} 
3600 Q_j / S_j & \text{if } Q_j < Q_m \\
q_j + 3600(Q_j - Q_m) / q_m & \text{if } Q_j \geq Q_m \tag{8}
\end{cases}
\]

where \( g_m \) is the time needed to release all the waiting vehicles at lane \( j \) with saturation flow rate, which can be given by:

\[
g_m = q_j (S_j - g_j) / (S_j - q_j) \tag{9}
\]

\( Q_m \) is the total number of vehicles released during time \( g_m \):

\[
Q_m = g_m S_j / 3600 \tag{10}
\]

Before phase 1 turns to green, the delay of the \( Q_k \) vehicles can be calculated by:

\[
D_{\text{st}}' = \left[ 0.5 g_j' + (g_j' - g_j'') + L_2 \right] Q_k \tag{11}
\]
After phase 1 turns to green, the delay of vehicles in waiting area $k$ can still be measured by Equation (6).

### 3.1.2 Delay of vehicles on the HT 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 the waiting area A1. This subsection only talks about the delay of the HT vehicles before passing the stop line, since the delay at the waiting area has been discussed in the subsection 3.1.1 above.

The spillback phenomenon often takes place in oversaturated intersections (12), which has inherent effect on the vehicle delay of the HT lane. Let $m$ denote the approaching HT lane under phase 1, and the volume of arrival vehicle is $q_m$ pcu/h. Let $Q_{p_{\text{max}}}$ 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: $Q_{p_{\text{max}}} \geq Q_{p_{\text{in}}}$

Here, $Q_{p_{\text{in}}}$ 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 (HCM) 2000 (13), the delay of vehicles waiting on lane $m$, denoted by $D_{m_{in}}$, can be obtained by:

$$D_{m_{in}} = q_mC_e/3600 \left[ \frac{0.5C_e(1-\lambda_m^2)}{1-[\min(1,x_{in})\lambda_m]} + 900T \left( x_{in} - 1 \right) + \sqrt{(x_{in} - 1)^2 + \frac{8Kx_{in}}{Cap_{m_{in}}T}} \right]$$

(12)

where $\lambda_m$ is the green split of phase 1, and $x_{in}$ is the degree of saturation of lane $m$ controlled by phase 1. $Cap_{m_{in}}$ 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 hour. $K$ is an adjustment parameter, usually taking 0.5.

#### Case 2: $Q_{p_{\text{max}}} < Q_{p_{\text{in}}}$
The spillback of HT vehicles will happen in this case. When there is a spillback, the 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 \( g_i^m \), can be calculated by Equation (8), while the red time can be regarded to be increased for \( g_i - g_i^m \) seconds. Herein, we still use Equation (12) to calculate the vehicle delay \( D_{m} \). However, the green time of phase 1 (denoted by \( g_i \)) should be replaced by \( g_i^m \), when used to calculate the green split, vehicle capacity of each phase, as well as the degree of saturation, etc.

3.1.3 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 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 due to the space limit.

3.2 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 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 3.1. 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 about the methodology for calculation of the vehicle delay, which is quite different from the methods in Section 3.1.

Without loss of generality, the west leg of intersection B is taken here to elaborate the calculation of vehicle delay. Let \( T_{agr2} \) 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 \( T_{agr2} \) equals \( T_{agr2} \) plus \( O_{aA} \). Hereafter, we use a standard time unit \( \Delta t \) to measure the cycle length and travel time, etc. The green time \( g_{a2} \) of phase 2 at intersection A is divided into \( U_{a2} \) intervals, i.e.,

\[
U_{a2} = g_{a2} / \Delta t
\]

(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 \( Q_r(i) \) can be obtained by:

\[
Q_r(i) = \begin{cases} 
S_r \Delta t / 3600 & \text{if } i \leq U'_{A2} \\
q_r \Delta t / 3600 & \text{if } i > U'_{A2} 
\end{cases}
\]  

(14)

Where \( S_r \) and \( q_r \) are the saturation flow rate and arrival flow rate of lane \( r \), \( U'_{A2} \) is the number of intervals of the green time that vehicles are released at saturation rate. Evidently, \( U'_{A2} \) is a portion of \( U_{A2} \), which is calculated by:

\[
U'_{A2} = \frac{q_r (C_r - q_{A2})}{(S_r - q_r) \Delta t}
\]  

(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, as well as 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:

\[
Q_{ab}(w) = \sum_{i=1}^{w} Q_r(i) F (1 - F)^{w-(i-1)}
\]  

(16)

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

\[
t = \frac{0.8 L_{ab} / V_{ab}}{\Delta t}
\]  

(17)

where \( L_{ab} \) is the length of the road segment between intersections A and B, \( V_{ab} \) is the average speed of the vehicles travelling between intersections A and B.

Figure 2 indicates the example of discharge pattern at intersection A and the arrival pattern at the intersection B, in the case of signal coordination. By means of Equations. (14) and (16), we can measure the 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.

![Diagram of vehicle discharge and arrival patterns at intersection A and B](image)

**FIGURE 2** Example of vehicle discharge and arrival patterns at coordinated phases

Assume that the green time of phase 2 at intersection B is divided to $U_{B_2}$ intervals by the standard time unit $\Delta t$. These $U_{B_2}$ intervals are numbered from $w'$, then the total number of arrival vehicles during the green time, denoted by $Q_{ab_2}$, equals:

$$Q_{ab_2} = \sum_{w=w'}^{w'+U_{B_2}-1} Q_{ab}(w)$$  \hspace{1cm} (18)

HCM2000 (10) has provided the average delay of vehicles on the road segment between two coordinated intersections:

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

where $\bar{d}_i$ 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:

$$PF = \frac{(1-P)f_{P\delta}}{1-\lambda_i}$$  \hspace{1cm} (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 $P$. 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 $P_{Bz}$, and $P_{Bz}$ equals:

$$P_{Bz} = \frac{Q_{Bz}}{(C_z q_z / 3600)} \quad (21)$$

Substituting the $PF$ in Equation (19) by $P_{Bz}$, we can easily obtain the average delay of vehicles on lane $z$, which are controlled by the phase 2 of intersection B.

The method discussed above 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.

3.3 Optimization Model

Sections 3.1 and 3.2 above 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 travelling in the coordinated subarea, denoted by $\bar{D}$.

Evidently, $\bar{D}$ is a function of the common cycle length $C_c$ and green time $g_i$ of each phase $i$ at intersection $n$. From Equation (1), we can see that $C_c$ can be substituted by a function of $g_i$, which makes $\bar{D}$ only a function of $g_i$, denoted by function $(g_i)$. It should be pointed out that $\bar{D}$ is the average delay of all the vehicles in one hour rather than in one signal cycle.

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

$$\begin{align*}
\min \quad & \bar{D} = \text{function } (g_i) \\
\text{s.t.} \quad & g_{\min} - g_i \leq 0, \quad 1 \leq n \leq N, \quad 1 \leq i \leq 2 \\
& g_i - g_{\max} \leq 0, \quad 1 \leq n \leq N, \quad 1 \leq i \leq 2
\end{align*} \quad (22)$$

where $N$ is the total number of coordinated intersections in the study area.
3.4 Solution Algorithm

In the Equation (22), the objective function is complex. Hence, it is difficult to calculate its gradient and then use the gradient-based algorithms in the literature. For such sort of complex Engineering-based problem, Genetic Algorithm (GA) is a suitable solution algorithm, which does not require the calculation of gradient. The objective function values can be simply used for the solution searching. Therefore, GA is adopted in this paper to solve Equation (22). GA consists of three main processes, Selection, Crossover and Mutation. Due to the space limit, the details of GA are not further included here. Any interested readers can refer to references 16 and 17.

4. NUMERICAL EXPERIMENTS

To verify the proposed methodology, a case study is developed using VISSIM as numerical experiments. This case study is introduced in Section 4.1 as follows.

4.1. 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 3, these three intersections are adjacent, and the distance in-between is 235 and 228 meters. Each of them has two signal phases. The two intersections on Elizabeth street and William street both have four waiting areas, i.e., 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.

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 1. The green gap is 6s, including 3s yellow time and 3s all red time. The common cycle length is 90s. 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, 8s and 17s respectively.
**FIGURE 3** Layouts and phasing schemes of the three adjacent HT intersections in Melbourne

**TABLE 1** Current signal plans of the three adjacent HT intersections (unit: s)

<table>
<thead>
<tr>
<th>Intersection</th>
<th>$g_1$</th>
<th>$g_2$</th>
<th>$I_i$</th>
<th>$C_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Latrobe St. &amp; William St.</td>
<td>40</td>
<td>38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Latrobe St. &amp; Queue St.</td>
<td>37</td>
<td>41</td>
<td>6</td>
<td>90</td>
</tr>
<tr>
<td>Latrobe St. &amp; Elizabeth St.</td>
<td>38</td>
<td>40</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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 2. The waiting area for HT vehicles in the intersection can accommodate 3pcu. The saturation flow rate of waiting areas is 1200 pcu/h of right-turning vehicles, and 1520pcu/h of the other streams/lanes. The average travel speed of vehicles is 40 km/h.

**TABLE 2** The traffic volumes obtained from field surveys (unit: pcu/h)

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Peak hour</th>
<th>Off-peak hour</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Left</td>
<td>Through</td>
</tr>
<tr>
<td>Latrobe St. &amp; William St.</td>
<td>South leg 152</td>
<td>482</td>
</tr>
<tr>
<td></td>
<td>North leg 214</td>
<td>420</td>
</tr>
<tr>
<td></td>
<td>East leg 306</td>
<td>562</td>
</tr>
<tr>
<td></td>
<td>West leg 210</td>
<td>432</td>
</tr>
<tr>
<td>Latrobe St. &amp; Queue St.</td>
<td>South leg 146</td>
<td>620</td>
</tr>
<tr>
<td></td>
<td>North leg 194</td>
<td>742</td>
</tr>
<tr>
<td></td>
<td>East leg 258</td>
<td>484</td>
</tr>
<tr>
<td></td>
<td>West leg 234</td>
<td>444</td>
</tr>
<tr>
<td>Latrobe St. &amp; Elizabeth St.</td>
<td>South leg 126</td>
<td>274</td>
</tr>
<tr>
<td></td>
<td>North leg 208</td>
<td>416</td>
</tr>
<tr>
<td></td>
<td>East leg 246</td>
<td>442</td>
</tr>
<tr>
<td></td>
<td>West leg 212</td>
<td>436</td>
</tr>
</tbody>
</table>
4.2. 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 to solve the optimal timing plan.

The minimum green time is set as 15s, and the maximum green time is 60s. There are three intersections and each one has two phases, thus totally we have 6 variables for the optimization. The results are tabulated in Table 3. We can see that in the morning peak, the common cycle length is 88s, which is close to the 90s in real world. The offset between the three intersections in Figure 3 (from left to right) is 83s and 7s, respectively. In the off-peak hour, the common cycle length is 73s, which is significantly less than the 90s in reality, and the offset values are 69s and 3s respectively.

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Peak hour</th>
<th>Off-peak</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(g_1)</td>
<td>(g_2)</td>
</tr>
<tr>
<td>Latrobe St. &amp; William St.</td>
<td>36</td>
<td>40</td>
</tr>
<tr>
<td>Latrobe St. &amp; Queue St.</td>
<td>32</td>
<td>44</td>
</tr>
<tr>
<td>Latrobe St. &amp; Elizabeth St.</td>
<td>35</td>
<td>41</td>
</tr>
</tbody>
</table>

4.3. Results Evaluation

In order to fully contrast the current signal plan and the optimal signal plan in Table 3, 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 travelling in the study area.

In the VISSIM simulations, the random seed would affect the probabilistic distribution of the arrival vehicles. Thus, we take 5 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 4500s, and the data between 300s and 3900s 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 10s), it is recognized that there is a spillback of HT vehicles.

Table 4 provides the evaluation and comparison results, where the “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 44.6s, while the value of Scheme II is 39.3s 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).

<table>
<thead>
<tr>
<th>Scheme</th>
<th>Morning Peak Hour</th>
<th>Off-peak Hour</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E1 (s)</td>
<td>E2 (s)</td>
</tr>
<tr>
<td>Scheme I</td>
<td>34.8</td>
<td>52.2</td>
</tr>
<tr>
<td>Scheme II</td>
<td>29.9</td>
<td>46.8</td>
</tr>
<tr>
<td>Improvement</td>
<td>14.1%</td>
<td>10.3%</td>
</tr>
</tbody>
</table>

We 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 have two streams with a temporal gap in-between. However, these two aspects seem are not considered in designing the current signal pattern at the intersections in Melbourne.

The outperformance of the 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 the 8.3 of Scheme I. During the off-peak hour, the 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 tradeoff between the spillback and coordination plan, the proposed methodology aims to find an optimal solution.

(4) E4: Average delay of all the vehicles travelling in the study area.

Compared with the Scheme I, the Scheme II can improve the E4 index for 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. 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.

5. CONCLUSIONS

Hook turn (HT) 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 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, due to the characteristics of the OD trips and network structure, the volume of right-turning vehicles is not high. These feature result in a successfully 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 the next step, two issues about HT scheme should be further studied: (i) Traffic crash risk evaluation at HT intersection (18-25); (ii) Development of adaptive traffic signal control algorithm for HT intersection based on loop detector or GPS data (26-27).

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