Control Delay at Signalized Diamond Interchanges Considering Internal Queue Spillback

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ABSTRACT

Control delay is the primary measure for determination of the level of service of signalized intersections. The existing analytical delay models usually work well for isolated intersections, but not as effective when applied to diamond interchanges. The limited storage space between the two closely spaced intersections of a diamond interchange may cause queue spillback from the internal link to the outside roads. This property would give rise to unrealistic delay calculation for diamond interchanges with high traffic volumes. This technical document describes the development of a new analytical delay model that takes into account the effects of internal queue spillback at diamond interchanges. Simulation studies are also conducted to compare the effectiveness of the proposed approach with existing methods. The study shows that for low overlap time conditions, the proposed model tends to agree with the Synchro and Vissim simulation and is better than Elefteriadou’s method, which tends to over predict delay. For high overlaps, the Elefteriadou’s method, Vissim simulation, and the proposed model tend to agree, while Synchro diverges significantly by over estimating delay. The study contributes to the literature and practice by providing an open-source analytical model that can either be used as a standalone delay calculation model or as a supplement to the existing methods.

Keywords: diamond interchange, control delay, internal queue, spillback, traffic signal capacity
1. INTRODUCTION AND LITERATURE REVIEW

Control delay is the primary measure for determination of the level of service of signalized intersections. Simulation methods (1) are advantageous in conducting “what if” studies and testing the scenarios and phenomenon that may not occur or hard to capture in the field. Nonetheless, simulation approaches are usually less effective in providing generalized results. For that reason, a combined analytical and simulation approach would be ideal for developing control delay models.

In the past several decades, mathematical models for calculating (signalized) intersection delay have been studied extensively by numerous researchers. Notable works include, but are not limited to Beckmann et al. (2) and Webster (3) who developed and tested through simulation their fundamental delay models, and van Zuylen and Viti (4) who provided comprehensive summaries of analytical delay models and improved some of them. Currently, the commonly used method is described in Chapter 16 of the Highway Capacity Manual, 2000 edition (5), which came from the model developed by Fambro and Roupail (6). In review of these models, one can find that most of them were developed on the basis of the assumption that the subject intersection should not be blocked by queues spilled over from the downstream intersection, which is not realistic in the real world. Therefore, those models work reasonably well for isolated intersections, but not as effective when applied to diamond interchanges with two closely spaced intersections. Several researchers attempted to solve the problem by taking into account signal coordination in their models, such as Fambro (7) and van Zuylen (4), however, the control delay of two major external approaches (i.e., the through movements of the arterial street and the left-turn movements of the frontage roads) has not been satisfactorily formulated to date to reflect the real situation.

Diamond interchanges, like the one shown in Fig.1, are arguably the most commonly used interchange patterns in North America providing connections between the major highways and arterial streets in urban and suburban areas (8). One of the ordinarily used diamond interchange patterns is the tight urban diamond interchange (TUDI), where two traffic signals are installed on the arterial street to control the interchanging traffic (9). Diamond interchanges are often characterized by limited spacing between the signals (10). The two traffic signals are usually spaced between 200 ft and 1000 ft apart and the TTI three-phase and four-phase operations are typically applied (11) (12), as shown in Fig.2 and Fig.3. Because of the limited interior spacing, when the internal queue is overflowed, spillback occurs and blocks the external movements, which is shown in Fig.4. For standard TTI four-phase operation, the overlap time, which is defined when phase 2 and phase 8 are both in green, or phase 6 and phase 4 are both in green in Fig.1, is normally designed to be less than the time required to travel the internal space of the diamond interchange. In such a case, no internal queues will be present. If the overlap time is adjusted for any reason, for instance, to reduce the queue on external roads, the vehicles will be forced to stop between the two intersections. This situation generates queue in the interior space and may cause queue spillback.

The delay model in Chapter 16 of HCM 2000 is used to estimate control delays at diamond interchanges by many traffic analysis tools, such as Synchro 5 and PASSER III. The results, however, do not realistically reflect the actual situation when spillback of internal queue occurs. For a diamond interchange operated by the TTI four-phase operation with long overlap time, the internal overflowing queue blocks the upstream intersection and increases the control delay of external movements significantly, but the existing delay model in the HCM 2000 doesn’t reflect the change. To address the effects of internal queue spillback, several methods were developed and employed in traffic analysis tools. Elefteriadou et al. (13) introduced a method, dubbed as the Elefteriadou model in this paper, to address this issue. However, this study finds that it tends to overestimate delay at low overlaps. Synchro 7 also introduced a new series of traffic analysis tools (called Queue Interactions), which looks at how queues may reduce capacity through spillback, starvation, and storage blocking between lane groups. A new queue delay factor is introduced to measure the additional delay incurred by the capacity reduction due to queues on short links. The new models are used for delay calculation of diamond interchanges by Synchro 7, but the specifics of this model were not published. This study finds that Synchro significantly over estimates delay for high overlaps.
The motive and objective of this study is to provide an open-source and accurate analytical delay formulation model for better operation of diamond interchanges under oversaturated conditions. To this end, an analytical model was developed with focus on taking into account the effect of internal spillback on the external movements. Since the effect on the arterial through movements and the ramp left-turn movements are similar, the study was emphasized on the delay of the arterial through movements under the TTI four-phase operation. Based on our analysis, it is revealed that the impact of internal spillback on delay varies depending on several variables including the signal timing, the arrival traffic flow rate of the arterial, the saturation flow rate of the upstream intersection, the saturation flow rate of the downstream intersection, the length of the initial internal queue and the distance of the internal space.
In the following section, we will describe in detail the proposed delay formulation model and the modification of the HCM 2000’s model. The new model was developed based on the classified traffic movement modes and the calculation of the lost green time and effective green time. All three delays, namely, the uniform control delay, incremental delay, and the total control delay per vehicle are predicted.

2. THE NEW ANALYTICAL DELAY MODEL FOR EXTERNAL MOVEMENTS OF DIAMOND INTERCHANGES

2.1 Classification of Traffic Movement Modes and Calculation of the Lost Green Time

In order to calculate the lost green time, the external traffic movements are classified into two modes. Mode 1 represents the case in which the traffic volume moving through the upstream intersection during overlap interval is less than the capacity of the internal space, as shown in Fig.5; Mode 2 represents the case that the traffic moving through the upstream intersection during the overlap interval is more than that the internal space can contain, as shown in Fig. 6. Calculation of the lost green time of these two traffic movement modes is introduced in the following two subsections.
FIGURE 6 Spillback Occurs during Overlap Time.

2.1.1 Traffic Movement Mode 1:
The traffic volume (per lane) moving through the upstream intersection during the overlap time is calculated by the following formula:

\[ V_{o-e} = \min\left(\frac{(o + r) \cdot q}{3600 \cdot n}, \frac{o \cdot s}{3600 \cdot n}\right) \]  

\( V_{o-e} \) = lane based traffic volume moving through the upstream intersection during overlap time;  
\( o \) = overlap time of the external arterial movement;  
\( r \) = red time per cycle for the external arterial through movement;  
\( q \) = arrival flow rate of the external arterial through movement;  
\( n \) = number of lanes of the lane group for the external arterial through movement;  
\( s \) = the saturated flow rate of the external arterial through movement.

Thus, the traffic movement mode 1 occurs when the following inequality is met:

\[ V_{o-e} \cdot L \leq l' \]  

\( L \) = the average length of space occupied by one vehicle in the queue;  
\( l' \) = the distance between the upstream intersection and the end of the internal queue;  
\( l' = l - Q_{il} \) \( \)  
\( l \) = length of the internal space of the subject diamond intersection;  
\( Q_{il} \) = length of the queue left in the internal space at the end of the last green time.

The through movement of the external arterial street will not be blocked in its green phase until the internal space overflows. Therefore, for movement mode 1, the external green time before being blocked by the internal spillback is longer than the overlap time and equals to the time needed to fill the internal space, which is expressed by the following formula:

\[ g_1 = \max\left(\frac{l' \cdot 3600 \cdot n}{q} - r, \frac{l' \cdot 3600 \cdot n}{s}\right) \]  

\( g_1 \) = green time of the external arterial before being blocked by the internal queue.

Since \( g_1 \) could not be longer than the green time \( g \), the formula (4) is adjusted:

\[ g_1 = \min(g, \max\left(\frac{l' \cdot 3600 \cdot n}{q} - r, \frac{l' \cdot 3600 \cdot n}{s}\right)) \]  

\( g \) = green time of the external arterial movement.
During the time interval of \( g_1 - o \), the vehicles in the internal space are discharged through the downstream intersection while the external traffic entering the internal space from the upstream intersection. Therefore, the internal queue length at the end of \( g_1 \) is

\[
Q_L = \max(0, I - \frac{(g_1 - o) \cdot s_d}{3600 \cdot n} \cdot L)
\]

(6)

\( Q_L \) = the internal queue length at the end of \( g_1 \);  
\( s_d \) = the saturated discharging traffic flow rate of the downstream intersection.

The time used to discharge the internal queue \( Q_L \) is

\[
T_i = \frac{Q_L}{s_d} \cdot 3600 \cdot n
\]

(7)

\( T_i \) = time needed for discharging the internal queue \( Q_L \);  
To realistically calculate delay, the effect of time needed for a vehicle to travel the internal space should be considered, so the lost green time is calculated by the following formula:

\[
b = \min(g - g_1, T_i - T_{ir})
\]

(8)

\( b \) = the lost green time because of internal spillback.  
\( T_{ir} \) = average time needed for a vehicle to travel the internal space.

The green time of the external arterial after being blocked is

\[
g_2 = g - g_1 - b
\]

(9)

\( g_2 \) = green time of the external arterial after being blocked.

If \( b = 0 \), then \( g_1 = g \) and \( g_2 = 0 \).

2.1.2 Traffic Movement Mode 2:
The traffic movement mode 2 occurs when the following inequality is met:

\[
V_{O-E} \cdot L > I'
\]

(10)

For traffic movement mode 2, \( g_1 \) and \( g_2 \) are calculated using formula (5) and (9), while the lost green time \( b \) is

\[
b = \min(g - g_1, o - g_1 + \frac{L}{s_d} \cdot 3600 \cdot n - T_{ir})
\]

(11)

2.2 Calculation of the Effective Green Time and Effective v/c Ratio
With the lost green time \( b \) calculated, the effective green time is obtained by the following formula:

\[
g' = g - b
\]

(12)

\( g' \) = effective green time of the arterial through movement.

Since the green time is decreased from \( g \) to \( g' \), the capacity and v/c ratio of the lane group are changed.

\[
c' = \frac{c' \cdot g'}{g}
\]

(13)

\( c' \) = effective capacity of the lane group for the through moment of the external arterial street;  
\( c = \frac{s \cdot n \cdot g}{C} \);
2.3 Calculation of the Uniform Delay $d_1$

This section describes the modification of the delay model in Chapter 16 of the HCM 2000 to reflect the effect of queue spillback at diamond interchanges. The modification is made based on the changed values of the effective green time and v/c ratio of the through movement on the external arterial street.

According to the analysis in Section 2.1, the total green time for the through movement of the external arterial street is divided into two parts with regard to the effective green time (the second effective green time may be 0) due to the internal spillback. Thus, the delay for uniform arrivals ($d_1$) is calculated using the following process:

If $g_2 = 0$, the green time $g$ and v/c ratio $X$ in the formula 16-11 (in Chapter 16 of HCM 2000) are replaced by the effective green time $g'$ and effective v/c ratio $X'$ to get the following formula:

$$d_1 = \frac{0.5C(1-\frac{g'}{C})^2}{1 - [\min(1, X') \frac{g'}{C}]}$$

Else,

The uniform arrival flow is expressed by the following formula:

$$q' = \min(q, c')$$

$$Q_i = q'r$$

$Q_i$ = the queue length of the through movement on the external arterial street at the beginning of green time.

If $(r + g_1) \cdot q' \geq g_1 \cdot S$, the external queue is not cleared at the end of $g_1$

$$Q_2 = q'(r + g_1) - S \cdot g_1$$

$Q_2$ = the queue length of the external arterial street at the end of $g_1$.

$$Q_3 = Q_2 + b \cdot q'$$

$Q_3$ = the queue length of the external arterial street at the beginning of $g_2$.

$$d_1 = (0.5 \cdot r \cdot Q_1 + 0.5 \cdot (Q_1 + Q_2) \cdot g_1 + 0.5 \cdot (Q_2 + Q_3) \cdot b$$

$$+ 0.5 \cdot \frac{(r + g_1 + b) \cdot q' - S \cdot g_1}{S - q'} \cdot S) / (C \cdot q')$$

Else, the external queue is cleared at the end of $g_1$

$$Q_2 = 0$$

$$Q_3 = Q_2 + b \cdot q'$$

$$d_1 = 0.5 \cdot \frac{S \cdot r^2 + S \cdot b}{C \cdot (S - q')}$$

If $Q_3 > g_2 \cdot (S - q')$, the external queue is not cleared at the end of $g_2$.

This is the situation shown in Fig.7, which can be considered as an equivalent situation depicted in Fig.8.

Exchange values of $b$ and $r$. 
Exchange values of $g_1$ and $g_2$;

d_1$ is calculated by formulas (17), (18) and (19).

2.4 Calculation of the Control Delay

The green time $g$ and v/c ratio $X$ in the formula 16-10, 16-12, F16-1 and 16-9 in Chapter 16 of HCM 2000 are replaced by the effective green time $g'$ and effective v/c ratio $X'$ to develop the following formulas to estimate the control delay:

$$PF = \frac{(1 - P)f_{PA}}{1 - \left(\frac{g'}{C}\right)}$$ (21)

$PF$ = the uniform delay progression adjustment factor, which accounts for effects of signal progression.

$P$ = the proportion of vehicles arriving during green time.

$f_{PA}$ = supplemental adjustment factor for platoon arriving during green time.
\[d_2 = 900T[(X' - 1) + \sqrt{(X' - 1)^2 + \frac{8klX'}{c'T}}]\]  

\[d_2 = \text{incremental delay to account for the effect of random arrivals and oversaturation queues, adjusted for the duration of the analysis period and type of signal control; this delay component assumes that there is no initial queue for the lane group at the start of the analysis period (s/veh);}\]

\[T = \text{analysis duration;}\]

\[k = \text{incremental delay factor that is dependent on controller settings;}\]

\[l = \text{upstream filtering/metering adjustment factor.}\]

\[d_3 = \frac{1800Q_b(1 + u)t}{c'T}\]  

\[d_3 = \text{initial queue delay, which accounts for delay to all vehicles in the analysis period due to initial queue at the start of the analysis period (s/veh);}\]

\[Q_b = \text{initial queue at the start of period T (veh);}\]

\[u = \text{delay parameter;}\]

\[t = \text{duration of unmet demand in T(h);}\]

\[t = 0 \text{ if } Q_b = 0, \text{ else } t = \min\{T, \frac{Q_b}{c'[1 - \min(1, X')]}\}\]

\[u = 0 \text{ if } t < T, \text{ else } u = 1 - \frac{c'T}{Q_b[1 - \min(1, X')]\}}\]

The control delay per vehicle (s/veh) is

\[d = d_1(PF) + d_2 + d_3\]

The flow chart of the proposed control delay calculation model is demonstrated in Fig. 9.

3 EFFECTIVENESS OF THE NEW MODEL

This section describes the test of effectiveness of the new delay model. Owing to the difficulty to reproduce the studied cases in the field, we used Vissim simulation to develop the scenarios and compare the performance of the proposed model with the Elefteriadou’s model and Synchro 7. The simulation model used in the test was adapted from a previously developed simulation to evaluate the performance of diamond and X-pattern interchanges in Texas. Details about the development and calibration of the simulation model are beyond the scope of the study and can be found in (14). Different overlap time scenarios were developed, which, in concert with two hypothetical traffic demand profiles representing respectively moderate and congested traffic conditions, have produced various internal queue situations for the analysis.

The internal space of the subject diamond interchange is 280 ft. and the signal runs the TTI four-phase plan. The control delay of the through movement on the northbound arterial street (Phase 3 in Fig.11) was estimated by the new model, Synchro 7 and the Elefteriadou model. The results from these three methods were compared with the simulation results. Two traffic flow rates, 1577 veh/h and 2000 veh/h, were used in the experiment to examine the performance of the models under normal and high traffic volumes. For each traffic flow rate, 9 overlap time scenarios varying from 5s to 45s with a step increment of 5s were applied in the TTI four-phase signal operation. This was intended to produce enough internal queue cases for the test of model performance under various overlap intervals. Table 1 provides the data collected in the experimental study. The results are also shown in Fig.12 and Fig.13.

As shown in Figs. 12 and 13, the estimated control delay by the new model is very close to the simulated results as compared to the estimated values by Synchro 7 and Elefteriadou Model. At the flow rate of 1577 veh/h, the differences between the delay predicted by the three methods and the simulation are as follows. For the proposed model, the average and largest deviation between the calculated and the
simulated control delay are 21 s/vehicle and 46.5 s/vehicle, respectively; this is 116.1 s/vehicle and 397.7 s/vehicle for Synchro 7, and 84.6 s/vehicle and 109.7 s/vehicle for the Elefteriadou model. At the traffic flow rate of 2000 veh/h, the average and largest deviation between the new model and the simulation are 35.8 s/vehicle and 80.2 s/vehicle, respectively. The same deviations are 148.5 s/vehicle and 439.7 s/vehicle in Synchro 7 and 162.5 s/vehicle and 241.2 s/vehicle in the Elefteriadou model.

FIGURE 9 Flow Chart of the New Control Delay Calculation Model.
FIGURE 10 Traffic Volumes of the Researched Diamond Interchange.

FIGURE 11 TTI Four-phase Signal Timing of the Researched Diamond Interchange.

TABLE 1 Delay Comparison at Different Overlap Times

<table>
<thead>
<tr>
<th>Arrival Flow Rate = 1577veh/h</th>
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</thead>
<tbody>
<tr>
<td>Overlap time(s)</td>
<td>5</td>
<td>10</td>
<td>15</td>
<td>20</td>
<td>25</td>
<td>30</td>
<td>35</td>
<td>40</td>
</tr>
<tr>
<td>Delay by New Model(s/veh)</td>
<td>39.0</td>
<td>39.1</td>
<td>39.8</td>
<td>41.3</td>
<td>49.2</td>
<td>105.7</td>
<td>151.7</td>
<td>151.7</td>
</tr>
<tr>
<td>Delay by Synchro 7(s/veh)</td>
<td>41.4</td>
<td>41.7</td>
<td>43.7</td>
<td>61</td>
<td>109.5</td>
<td>176.8</td>
<td>264.3</td>
<td>383.8</td>
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<tr>
<td>Delay by Elefteriadou Model(s/veh)</td>
<td>135.6</td>
<td>138.8</td>
<td>142.1</td>
<td>145.5</td>
<td>148.9</td>
<td>152.4</td>
<td>155.9</td>
<td>159.4</td>
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<tr>
<td>Simulation delay (s/veh)</td>
<td>35.1</td>
<td>35.2</td>
<td>36.4</td>
<td>36.1</td>
<td>39.2</td>
<td>76.3</td>
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<td>108.1</td>
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</table>

<table>
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<tr>
<th>Arrival Flow Rate = 2000 veh/h</th>
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<td>25</td>
<td>30</td>
<td>35</td>
<td>40</td>
</tr>
<tr>
<td>Delay by New Model(s/veh)</td>
<td>41.8</td>
<td>43.8</td>
<td>48.9</td>
<td>81.6</td>
<td>148.8</td>
<td>241.8</td>
<td>295.6</td>
<td>295.6</td>
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<tr>
<td>Delay by Synchro 7(s/veh)</td>
<td>56.8</td>
<td>71.9</td>
<td>111.4</td>
<td>161.2</td>
<td>222.9</td>
<td>299.7</td>
<td>398.2</td>
<td>529.1</td>
</tr>
<tr>
<td>Delay by Elefteriadou Model(s/veh)</td>
<td>275.3</td>
<td>279.5</td>
<td>283.8</td>
<td>288.1</td>
<td>292.5</td>
<td>296.9</td>
<td>301.4</td>
<td>305.9</td>
</tr>
<tr>
<td>Simulation delay (s/veh)</td>
<td>38.4</td>
<td>38.3</td>
<td>44.1</td>
<td>81</td>
<td>129.1</td>
<td>190.5</td>
<td>217.3</td>
<td>215.4</td>
</tr>
</tbody>
</table>

It can also be found from the figures that, with the overlap time increased from 5 seconds to 45 seconds, the trend of the estimated control delay by the new model is consistent with the trend of simulated results, while the estimation by Synchro 7 and the Elefteriadou model varies depending on low and high overlap times. It appears that for low overlaps, Elefteriadou’s method tends to over predict delay, while the new model, the Synchro and Vissim simulation tend to agree. For high overlaps, however, the new model, Elefteriadou’s method and Vissim simulation tend to agree, while Synchro diverges significantly by over estimating delay.
FIGURE 12 Delay Calculated by Different Models With an Arrival Traffic Flow Rate of 1577 veh/h.

FIGURE 13 Delay Calculated by Different Models with an Arrival Traffic Flow Rate of 2000 veh/h.

5 CONCLUSIONS
An analytical model for calculating control delay of diamond interchanges with consideration of the effect of internal queue spillback was developed. The performance of the new model was examined in traffic simulation and compared with the existing methods. The study shows that for low overlaps, the proposed model tends to agree with the Synchro and Vissim simulation and is better than Elefteriadou’s method, which tends to over predict delay. For high overlaps, the Elefteriadou’s method, Vissim simulation, and the proposed model tend to agree, while Synchro diverges significantly by over estimating delay. The proposed methodological approach, along with the detailed derivation of the formulas and the carefully designed calculation flow chart, would be helpful for researchers and practitioners to further study and effectively operate diamond interchanges.

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