AN ADVANCED SIGNAL PHASING SCHEME FOR DIVERGING DIAMOND INTERCHANGES

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**ABSTRACT**

A diverging diamond interchange (DDI), also called a double crossover diamond interchange (DCD), is a relatively new interchange type in the U.S. Due to its many advantages over the conventional diamond interchanges such as higher capacity and lower delay, there has been a growing interest in its analysis methodologies and operational strategies. DDIs have shown particularly high efficiency under conditions with heavy ramp traffic to arterial roads. However, as the DDI concept is relatively new in the U.S., the research on DDIs is still considered to be in the preliminary stage. In particular, most studies found in the literature only provided basic phasing schemes, which were not optimized for efficiency. Furthermore, there is a lack of commonly acceptable methodologies for obtaining critical signal control parameters, such as the cycle length, and phasing splits. Therefore, it is an area topic in need of much research in order to develop signal phasing schemes for achieving the optimized DDI performance. This study proposed a new methodology for signal phasing scheme design at DDIs. With minor adjustment, it can be applied to a variety of DDIs with different traffic demands and geometric configurations. The proposed phasing design approach was specifically designed for DDIs, so it could be employed as a guide or a reference for signal timing design at DDIs. At the DDI of Moana Ln and U.S. 395 in Reno, Nevada, the proposed scheme was evaluated and compared with various signal phasing schemes by using the hardware-in-the-loop simulation technology. The simulation results indicated that the proposed approach significantly outperformed the one currently being used at this interchange.

**Keywords:** Diverging Diamond Interchange, Traffic Signal Control, Signal Phasing Scheme, Webster Method, VISSIM.

**INTRODUCTION**

The DDI concept was first introduced to the United States by Gilbert Chlewicki in 2003 (1). The first DDI was constructed in Springfield, Missouri in 2009 (2). Before that, the only known DDIs existed in the communities of Versailles, Le Perreux-sur-Marne, and Seclin in France (3). Field and simulation studies have demonstrated many advantages of DDIs over the conventional diamond interchanges. As a result, seven states (Missouri, Utah, Georgia, Kentucky, Maryland, New York, and Nevada) have constructed DDIs with demonstrated operational improvements by the end of year 2012. More than 17 states are currently constructing DDIs in the U.S. (3).

Studies have been conducted to evaluate the performance of DDIs (1, 4-6). Most of these studies compared the performance of DDIs to conventional diamond interchanges and single point urban interchanges. Researchers sought signal timing plans either manually or by using Synchro to optimize signal timing for each traffic scenario. Since Synchro does not provide the specific function of signal timing optimization at DDIs, some of the research interpreted the two crossover intersections of DDIs as two separate intersections operated by two controllers (2). The microscopic simulation software of VISSIM was then employed to evaluate the performance. The existing research results have indicated that DDI works better than the conventional interchange designs in terms of delay, number of stops, queue length, and capacity (2, 6).
FHWA’s research suggested that the DDI signal timing could be two-phase control, with each phase dedicated for the alternative opposing movements (7). Signals at a DDI can be fixed time and may be fully actuated to minimize delay. Detectors can be used on all approaches of both crossovers, and durations of signal phases may vary on a cycle-by-cycle basis. A DDI can be operated by one controller or two. FHWA also indicated a three-phase signal operation by adding a dummy phase as the spacing between the crossover junctions is very close. Until now, there is no commonly accepted signal timing methodology specifically for DDIs. Therefore, professionals rely on personal experience or conventional approaches for other types of interchanges.

This study proposed a new methodology for the signal phasing scheme design at DDIs. With minor adjustment, it can be applied to a variety of DDIs with different traffic demands and geometric configurations. The proposed phasing design approach was specifically designed for DDIs, so it could be employed as a guide or a reference for signal timing design at DDIs. This research paper consists of four major sections. Section 1 provides a comprehensive literature review pertinent to DDIs. Section 2 briefly introduces the current operation scheme and presents a mathematical model for traffic signal timing design of DDIs based on Webster’s method. Section 3 compares performance of the current traffic signal operation scheme and signal timing by the proposed methodology with simulation results at the DDI of Moana Ln and U.S. 395 interchange in Reno, NV. The results indicated that the performance of the proposed scheme is better than the current operation scheme at the studied interchange under the traffic demand conditions. The last section offers conclusions of this study.

METHODOLOGY

Proposed Operation

The proposed phasing scheme is for one controller operation and can be used with all control types: pre-timed, semi-actuated, and fully actuated. It is a general phasing scheme design methodology for DDIs. The proposed phasing scheme is shown in Figure 1 and Figure 2. The proposed operation operates very efficiently by applying overlapping phases extensively.

Phasing Scheme

As shown in Figure 1, phases 1 and 5 have the same split that is approximately equal to the travel time ($T_{11,12,7}$ sec) between locations of “11” and “7” (or between “4” and “8”) as depicted in Figure 3. Phases 1 and 5 start and terminate at the same time, so they have the same “Max 1” or “phase splits” in the controller settings for the same maximum recall. The two phases allow the southbound (SB) off-ramp to be released earlier by $T_{11,12,7}$ sec. Due to the early release, there are no stops at the next signal, which means improved capacity. Phase 7 serves as the green extension for phase 2, which allows the vehicles passing through location “5” in phase 2 to get to location “8”. These vehicles can then be served with green signal when phase 3 comes on, which reduces the control delay of the northbound (NB) off-ramp traffic. In addition, phase 7 is set to “Max Recall” for safety and its duration must be less than the minimum green of phase 4, which is set to be “Min Recall” in the controller. Another reason for setting “Max Recall” on phase 7 is to avoid the queue between nodes “3” and “4” exceeding the segment.
between these two nodes. Although Phase 8 is not assigned to any of the movements, the phase with a “Min Recall” ensures phases 3 and 7 not to come on at the same time as they are actually conflicting phases. Without phase 8, phase 7 will extend to run simultaneously with phase 3. If phase 8 is set up with “Max Recall, the signal operation the DDI is not efficiently as phase 3 cannot gap out when its pertinent traffic is low. Figure 2 shows the locations of phases in the proposed DDI phasing scheme. The SB on-ramp right-turn traffic and the NB on-ramp right-turn traffic are controlled by “yield” type in this study. They can also be controlled by overlap phases such as phase 4 for SB on-ramp right-turn traffic and overlapping phases 1 and 3 for NB on-ramp right-turn traffic. Therefore, the proposed phasing scheme can be easily adjusted for controlling other DDIs with different configurations.

Figure 1 Phases, Rings, and Barrier for Proposed Operation

Figure 2 Phase Location Diagram for Proposed Operation
Assumptions and Basic Strategies

Similarly to Webster’s method, the proposed operation assumes that the v/c ratios for critical lane groups are equal, and that the green times allocated to critical lane groups are assumed to be proportional to their saturation flow rates. Unlike the traditional Webster’s method, the principle of deriving a real cycle length and the real splits (the cycle length and the splits shown in a controller) of a DDI by the proposed operation is adding a certain of additional times (coming from overlapping phases) to the real cycle length for calculating its effective splits in proportion, and then deducting the additional times from the effective cycle length and splits to obtain the real cycle length and the real splits, which will be input into a controller.

The proposed method introduces phases 2 and 6 into the signal operation, which lead to two different timing schemes: phase 6 is one of the critical phases or phase 2 is the one of the critical phases. The criterion for selecting the critical phases between phases 2 and 6 is determined by the traffic demands and configurations of a DDI. The performance of these two schemes is different as their operation efficiencies are different. The flow chart in Figure 4 indicates the basic steps for deciding which scheme to use.
Figure 4: Overview of Proposed Operation

2. Signal Timing Scheme 1 of Proposed Operation

Signal timing scheme 1 is for the situation that phases 6, 4, and 3 are critical phases. Phase 1 is predetermined by the travel time of $T_{11,12,7}$ sec.

1) Cycle Length

For the traffic signal operation shown in Figure 1, the DDI traffic signal timing parameters must satisfy the relationship in the following equation:

$$g_3 + g_4 + g_6 = C_0 + T_{11,12,7} + T_{11,12,7} - L = C_0 + 2 * T_{11,12,7} - L$$  \hspace{1cm} (1)

where

- $g_3$: effective green time of phase 3 (s);
- $g_4$: effective green time of phase 4 (s);
- $g_6$: effective green time of phase 6 (s);
- $C_0$: sum of splits of phases 3, 4, and 6 shown in Figure 1 (s);
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\[ T_{11,12,7} \]: split of phase 1, fixed and determined by the travel time between the two signals “11” and “7” through nodes “11,” “12,” and “7” shown in Figure 3; and

\[ L: \] total lost time per cycle (s);

For this kind of DDI, the effective cycle length (\(C_e\)) for the three critical lane groups (SBL, eastbound (EB), and westbound (WB)) satisfy the following relationship with \(C_0\):

\[ C_e = C_0 + 2 * T_{11,12,7} \]  

(2)

Replacing \(g_3, g_4, g_6\) and \(C_0\) in Equations (1) and (2) yields:

\[ \sum_i \frac{y_i}{s_i} * \frac{C_e}{x_i} = C_e - L \]  

(3)

Then, based on the assumption, \(X = x_i = x_j\) for \(i \neq j\), the effective cycle length is:

\[ C_e = \frac{L}{\sum_i \frac{y_i}{s_i} * \frac{C_e}{x_i}} = \frac{L}{1 - \frac{\sum_i y_i}{X}} = \frac{L}{1 - \frac{Y}{X}} \]  

(4)

where

\[ y_i: \] flow ratio for critical lane group \(y_i\); and

\[ Y: \] sum of all critical lane groups.

For this traffic signal timing plan, the total lost time is:

\[ L = l_3 + l_4 + l_6 \]  

(5)

\[ l_i = l_{s,i} + l_{c,i} = l_{s,i} + Y_i + AR_i - e_i \]  

(6)

where

\[ l_i: \] lost time of critical lane group \(i\) \((i = 4\ and\ 6)\) (s);

\[ l_{s,i}: \] start-up lost time of critical lane group \(i\) \((i = 4\ and\ 6)\) (s);

\[ l_{c,i}: \] clearance lost time of critical lane group \(i\) \((i = 4\ and\ 6)\) (s);

\[ Y_i: \] yellow interval of critical lane group \(i\) \((i = 4\ and\ 6)\) (s);

\[ AR_i: \] all-red interval of critical lane group \(i\) \((i = 4\ and\ 6)\) (s); and

\[ e_i: \] extension of effective green time of critical lane group \(i\) \((i = 4\ and\ 6)\) (s).

\[ l_3 = l_{s,3} = 2 \]  

(7)

The lost time of phase 3 is 2 sec, since it is overlapped by phase 1. Phase 1 is overlapped by phase 2. Therefore, the westbound through traffic has start-up lost time of 2 sec as shown in Equation (7).

The real cycle length is computed by:

\[ C = C_e - L - T_{11,12,7} = \frac{L}{1 - \frac{Y}{X}} - L - T_{11,12,7} \]  

(8)

2) **Effective Green Times and Phase Splits**
Effective green times can be calculated by:

\[
g'_i = \frac{y_i}{\sum y_i} * (C_e - L)
\]  

(9)

where

\( g'_i \): effective green time for phase \( i \) (\( i = 3, 4, 6 \)).

The actual green time shown in Figure 1 can be obtained by:

\[
g_1 = \Phi_1 - Y_1 - AR_1
\]  

(10)

\[
g_3 = g'_3 - \Phi_1
\]  

(11)

\[
g_4 = g'_4
\]  

(12)

\[
g_2 = g_6 = g'_6 - \Phi_1
\]  

(13)

where

\( \Phi_1 \): is equal to \( T_{11,12,7} \) (s).

The phase splits 3, 4, and 6 are:

\[
\Phi_i = g_i + Y_i + AR_i
\]  

(14)

The \( y_i \) in Equation (4) can be replaced by:

\[
y_3 = \max\left\{ \frac{v_{3,14,7}}{s_{3,14,7}}, \frac{v_{3,5,6}}{s_{3,5,6}} \right\}
\]  

(15)

\[
y_4 = \max\left\{ \frac{v_{4,1,8}}{s_{4,1,8}}, \frac{v_{4,10,9}}{s_{4,10,9}} \right\}
\]  

(16)

where

\( v_{3,14,7} \): traffic volumes of lane group between nodes “14” and “7” (veh/h);

\( s_{3,14,7} \): saturation flow rate of lane group between nodes “14” and “7” (veh/h);

\( v_{3,5,6} \): traffic volumes of lane group between nodes “5” and “6” (veh/h);

\( s_{3,5,6} \): saturation flow rate of lane group between nodes “5” and “6” (veh/h);

\( v_{4,1,8} \): traffic volumes of lane group between nodes “1” and “8” (veh/h);

\( s_{4,1,8} \): saturation flow rate of lane group between nodes “1” and “8” (veh/h);

\( v_{4,10,9} \): traffic volumes of lane group between nodes “10” and “9” (veh/h); and

\( s_{4,10,9} \): saturation flow rate of lane group between nodes “10” and “9” (veh/h).

3. **Signal Timing Scheme 2 of Proposed Operation**

Signal timing scheme 2 is for the situation that phases 2, 3, and 4 are critical phases. In this scheme, phase 1 is also predetermined by the travel time of \( T_{11,12,7} \) sec. Phase 1 serves SBL off-ramp traffic with phase 6. Phase 1 also serves the WBT through traffic. Phase 7 is neglected in the scheme 2 for studying the signal timing as phase 2 is the critical phase. It is not efficient to
Hu, Tian, Xu, and Andalibian calculate the cycle length and splits by adding phase 7 into the effective green time as phase 2 itself can serve NBL off-ramp traffic well, or in other words, phase 7 is not effective for the entire intersection although it can still add an additional green time to phase 2. All the symbols of the signal timing scheme 2 are same as the signal timing scheme 1.

1) Cycle Length

For the traffic signal operation shown in Figure 2, the DDI traffic signal timing parameters should satisfy the relationship in Equation 17:

\[ g_2 + g_3 + g_4 = C_0 + T_{11,12,7} - L = C_0 + T_{11,12,7} - L \]  \hspace{1cm} (17)

Under this condition, the effective cycle length for the three critical lane groups (NBL, EB, and WB) satisfy the following relationship with \( C_0 \):

\[ C_e = C_0 + T_{11,12,7} \]  \hspace{1cm} (18)

Replacing \( g_2, g_3, g_4 \) and \( C_0 \) in Equations (17) and (18) yields:

\[ \sum_{i}^{N} \frac{v_i}{s_i} \cdot \frac{C_e}{x_i} = C_e - L \]  \hspace{1cm} (19)

Then, based on the assumption, \( X = x_i = x_j \) for \( i \neq j \), the effective cycle length is:

\[ C_e = \frac{L}{1 - \frac{\sum_{i}^{N} v_i}{x}} = \frac{L}{1 - \frac{\sum_{i}^{N} v_i}{x}} = \frac{L}{1 - \frac{Y}{x}} \]  \hspace{1cm} (20)

The real cycle length is calculated as:

\[ C = C_e - L = \frac{L}{1 - \frac{Y}{x}} - L \]  \hspace{1cm} (21)

For this traffic signal timing plan, the total lost time is:

\[ L = l_2 + l_3 + l_4 \]  \hspace{1cm} (22)

\[ l_i = l_{s,i} + l_{c,i} = l_{s,1} + Y_i + AR_i - e_i \]  \hspace{1cm} (23)

\[ l_3 = l_{s,3} = 2 \]  \hspace{1cm} (24)

The lost time of phase 3 is only 2 sec, since it is overlapped by phase 1. Phase 1 is overlapped by phase 2. Therefore, the westbound through traffic has start-up lost time of 2 sec as shown in Equation (24).

2) Effective Green Times and Phase Splits

Effective green times can be calculated by:

\[ g'_i = \frac{v_i}{\sum_{i}^{N} v_i} \cdot (C_e - L) \]  \hspace{1cm} (25)

The actual green time illustrated in Figure 1 can be obtained by:

\[ g_1 = \Phi_1 - Y_1 - AR_1 \]  \hspace{1cm} (26)
CASE STUDY

The proposed schemes have proved to work well with one controller operation at DDIs (8). In this study, the interchange performance with the proposed scheme was compared with the performance of the existing traffic operation.

Site Description

The interchange of Moana Lane and U.S. 395 is a diverging diamond interchange in Reno, NV, as shown in Figure 5. The interchange has high right-turn traffic demands on the off-ramps. No-turn-on-red is used on the southbound off-ramp due to the dual-lane geometry. Other peak hour volumes of the interchange are shown in Figure 5.

Existing Traffic Operation and Performance

The current actuated-coordinated signal timing plans running at the studied interchange were developed by the City of Reno in 2012, with consideration of the heavy right-turn movements on the off-ramps and southbound no-turn-on-red. The current signal phasing scheme has been evaluated by Hu et al. in September, 2012 (8). The existing timing plan is displayed in Figure 6 and Figure 7.
Phases 4 and 1 are not conflicting phases. There is no accurate way to determine the phase 1’s split. Therefore, there is no accurate quantitative method to derive the cycle length for the current method as well. The cycle lengths of the actuated-coordinated signal timing plans at the DDI are provided as the existing actuated-coordinated plans for the adjacent signals: 110-sec cycle for AM and 130-sec cycle for PM. In this study, the phase 1 split is assumed to be 10% of the cycle length. Phases 2, 3, and 4 share the remaining time in proportion to their critical saturation flow ratios. Table 1 shows each phase split in year 2015 AM and PM peak hours.

Table 2 provides the yellow and all-red intervals of the two actuated-coordinated plans for current operation used in VISSIM simulation models.
Table 1 Cycle Lengths and Phase Splits of Current and Proposed Operations

<table>
<thead>
<tr>
<th></th>
<th>Current Operation</th>
<th>Proposed Operation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2015 AM</td>
<td>2015 PM</td>
</tr>
<tr>
<td></td>
<td>2015 AM</td>
<td>2015 PM</td>
</tr>
<tr>
<td>Cycle (sec)</td>
<td>110</td>
<td>130</td>
</tr>
<tr>
<td></td>
<td>110</td>
<td>130</td>
</tr>
<tr>
<td>Phase 1 (sec)</td>
<td>11</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Phase 2 (sec)</td>
<td>23</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>29</td>
</tr>
<tr>
<td>Phase 3 (sec)</td>
<td>23</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>17</td>
<td>32</td>
</tr>
<tr>
<td>Phase 4 (sec)</td>
<td>53</td>
<td>59</td>
</tr>
<tr>
<td></td>
<td>53</td>
<td>59</td>
</tr>
<tr>
<td>Phase 5 (sec)</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Phase 6 (sec)</td>
<td>23</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>29</td>
</tr>
<tr>
<td>Phase 7 (sec)</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>Phase 8 (sec)</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td></td>
<td>58</td>
<td>79</td>
</tr>
</tbody>
</table>

Table 2 Clearance Intervals Used in VISSIM Simulation Models of Current Operation

<table>
<thead>
<tr>
<th>Operation</th>
<th>Phase</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Current Operation</td>
<td>Critical Movement</td>
<td>10-&gt;9</td>
<td>3-&gt;8</td>
<td>3-&gt;8</td>
<td>12-&gt;7</td>
<td>None</td>
<td>12-&gt;7</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>Yellow (sec)</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Red (sec)</td>
<td>1.5</td>
<td>3.5</td>
<td>3.5</td>
<td>2.5</td>
<td>2.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Proposed Operation</td>
<td>Critical Movement</td>
<td>3-&gt;8</td>
<td>3-&gt;8</td>
<td>None</td>
<td>12-&gt;7</td>
<td>None</td>
<td>10-&gt;11</td>
<td>5-&gt;4</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td>Yellow (sec)</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Red (sec)</td>
<td>3.5</td>
<td>3.5</td>
<td>2.5</td>
<td>2.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>0</td>
</tr>
</tbody>
</table>

Proposed Operation

Based on the traffic volumes provided by NDOT, the proposed signal operation is from the schemes presented in this study shown in Figures 1, 2, and 4. Its cycle lengths and phase splits are summarized in Table 1. In fact, the cycle lengths based on the proposed operations are close to the current operation’s cycle lengths in both AM and PM peak periods. In order to eliminate the cycle length’s effects on the current and proposed operations, the proposed cycle lengths are assumed as the current method’s cycle lengths both in AM and PM peak periods. These splits were acquired by the methodologies for proposed operations introduced in this study. The study uses the clearance intervals in VISSIM simulation models as shown in Table 2.

Table 2 These intervals are defined based on proposed operations and the specific geometry conditions at the Moana DDI. The proposed signal operation is an actuated-coordinated control plan, which has been successfully tested in a Naztec 2070 traffic controller (10).
Simulation Results

Each VISSIM model ran five simulations with random seeds. The warm-up time in the simulation models was 300 sec, followed by 3600 sec of run time with data compiled and collected for each traffic movement at the Moana DDI. The average delays of five runs of the current and proposed operations as well as the compared results of their average delays are presented in Table 3. The average maximum queues of the five runs of the current and proposed operations as well as their average maximum queues are provided in Table 4. The percent values in Table 3 are obtained by taking the difference of the average delays from the proposed operation and the current operation and then dividing by the average delay found in the current operation. For example, the value “11%” shown in the table is obtained by \((17.9-16.1)/16.1*100\%\). The value “16.1” comes from the current operation average delay of traffic movement “10->9” during the AM peak hour. Similarly, the number “17.9” is the proposed operations average delay of the same traffic movement in the period. Based on simulation results, proposed operations brought about less average delay for most movements but greater average delay of movement “10->9” compared to the operation developed by the City of Reno for 2015 AM and PM peak hour periods. The reason for less delay of movement “10->9” by the current operation is that it adds phase 1 to this traffic movement compared to the proposed operation. The average delay for all vehicles of the proposed operation dropped by 17% and 28% in AM and PM peak hours, respectively, compared to the current operation.

Similar to the data in Table 3, Table 4 summarizes the results of average maximum queues. The average maximum queue of movement “10->11” under the proposed operation decreased by 15% and 44% in the AM and PM peak hours, respectively, compared to the current operation. The average maximum queue of movement “5->4” in the proposed operations reduced 21% in both the AM and PM peak hour over the current operation, specifically from 222.1 to 174.8 feet in the AM peak and from 308.9 to 245.5 feet in the PM peak. Of the other traffic movements including “1->8,” “1->2,” “12->7,” and “14->7,” the proposed operation performed better than the current operation during the peak periods. However, proposed operations increased the maximum queue by 0% and 24% of movement “10->9,” and increased 11% and 14% of the maximum queue of movement “3->8” in the AM and PM peak hour periods to 69.0 and 84.3 feet, respectively. The same reason for greater average delay of traffic movement “10->9” brought about the larger average maximum queue of this movement. The reason for longer maximum queues of movement “3->8” experienced with the proposed operation is that this operation allows the release of northbound left (NBL) traffic earlier, which stops in front of node “8.” The maximum queues of this movement were less than 84.3 feet. This maximum queue is acceptable for reducing the delay in front of node “4.” Of the other movements, both operations performed well with no noticeable differences for their low traffic volume, even though they seemed much different.
Table 3 Average Delays from VISSIM Simulation Models (sec/veh)

<table>
<thead>
<tr>
<th>Peak Hours</th>
<th>AM</th>
<th>PM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Movement</td>
<td>Current</td>
<td>Proposed</td>
</tr>
<tr>
<td>10-&gt;9</td>
<td>16.1</td>
<td>17.9</td>
</tr>
<tr>
<td>10-&gt;11</td>
<td>45.8</td>
<td>33.5</td>
</tr>
<tr>
<td>1-&gt;8</td>
<td>21.7</td>
<td>16.3</td>
</tr>
<tr>
<td>1-&gt;2</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>3-&gt;8</td>
<td>7.7</td>
<td>8.7</td>
</tr>
<tr>
<td>3-&gt;2</td>
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<td>0.3</td>
</tr>
<tr>
<td>14-&gt;13</td>
<td>0.6</td>
<td>0.5</td>
</tr>
<tr>
<td>14-&gt;7</td>
<td>51.7</td>
<td>47.6</td>
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<tr>
<td>5-&gt;6</td>
<td>0.8</td>
<td>0.7</td>
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<tr>
<td>5-&gt;4</td>
<td>48.7</td>
<td>32.5</td>
</tr>
<tr>
<td>12-&gt;13</td>
<td>1.4</td>
<td>1.3</td>
</tr>
<tr>
<td>12-&gt;7</td>
<td>5.9</td>
<td>1.5</td>
</tr>
<tr>
<td>All</td>
<td>18.3</td>
<td>15.1</td>
</tr>
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</table>

Table 4 Average of the Maximum Queue from Simulation Models (ft)

<table>
<thead>
<tr>
<th>Peak Hours</th>
<th>AM</th>
<th>PM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Movement</td>
<td>Current</td>
<td>Proposed</td>
</tr>
<tr>
<td>10-&gt;9</td>
<td>288.2</td>
<td>287.2</td>
</tr>
<tr>
<td>10-&gt;11</td>
<td>206.4</td>
<td>175.3</td>
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<tr>
<td>1-&gt;8</td>
<td>250.7</td>
<td>229.9</td>
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<td>1-&gt;2</td>
<td>41.7</td>
<td>30.8</td>
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<td>62</td>
<td>69</td>
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<tr>
<td>3-&gt;2</td>
<td>13.5</td>
<td>37.7</td>
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<td>0</td>
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<tr>
<td>14-&gt;7</td>
<td>274.8</td>
<td>279.7</td>
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<tr>
<td>5-&gt;4</td>
<td>222.1</td>
<td>174.8</td>
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<td>7.8</td>
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<tr>
<td>12-&gt;7</td>
<td>101.8</td>
<td>95.6</td>
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</table>

TRB 2014 Annual Meeting  Paper revised from original submittal.
CONCLUSION

Since there is no widely accepted signal timing methodology for DDIs, in the literature, traffic engineers and researchers designed DDI signal timing plans based on their personal experience and approaches for conventional interchanges, which have been proven to be inefficient. Some of the signal phasing schemes in existing research did not consider the capability of signal controllers, so they are not readily implementable in the field.

Because DDIs are a relatively new interchange design, there is a limited amount of related literature. Most DDI related reports do not include detailed descriptions of feasible signal phasing schemes. Where signal phasing was actually discussed, most reports did not follow the NEMA convention, thus the phasing schemes were not readily implementable in the field. This paper presented an innovative phasing scheme and a methodology for determining cycle length, splits, and phase intervals for DDIs with consideration of the capacity of signal controllers. The proposed phasing scheme employed the conception of overlap phases, which was inspired by the TTI-4 signal strategy for conventional diamond interchanges. The phasing scheme and the timing methodology had been fully tested by Hu et al. (8) for its validity under pre-time, fully actuated, and actuated-coordinated control modes. The proposed phasing scheme and timing methodology can also be easily modified to meet the specific DDI requirements under a variety of geometric characteristics and traffic demand conditions. A case study was conducted using the first DDI site in Nevada. The simulation results by VISSIM showed that the proposed operation outperformed what was initially implemented by reducing vehicle delays.

Further research identified through this study includes the following. First, the relationship between cycle length and interchange level delay needs to be further addressed. Second, the relationship between traffic demands, phasing scheme, and signal timing parameters, needs additional investigation. Finally, the effect of signal spacing on DDI’s performance needs to also be thoroughly researched.
REFERENCES


