Characterizing Signalized Intersection Performance Using Maximum Vehicle Delay

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Characterizing Signalized Intersection Performance using Maximum Vehicle Delay

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November 15, 2014

TRB Paper 15-0385

Word Count: 4927 words + 10 x 250 words/Figure-Table = 4927 + 2500 = 7427 words
ABSTRACT
Average delay is perhaps the most commonly used measure for characterizing the performance of signalized intersections. Current methodologies for estimating the average delay rely on the use of models based on volumes and green times. In practice, it is challenging to develop such real-time measurements of delay, due to the difficulty of accurately measuring vehicle arrivals and departures. However, measuring wait time after the first vehicle arrival during the red interval can be an important performance measure for low and moderate volume conditions. The maximum wait time performance measure provides an upper bound, or maximum, on individual vehicle delay during a given cycle and facilitates comparison between different types of operation. This paper demonstrates the effectiveness of this “maximum vehicle delay” performance measure with four different case studies, including split adjustment, implementation of coordination at a non-coordinated intersection, varying cycle length, and use of phase reservice. The paper concludes that maximum vehicle delay can be used to characterize the impact of timing adjustments, as well as the implementation of more unique controller features, on individual movements at the intersection.

MOTIVATION
Traditional measures of intersection delay, including total approach and average vehicle delay, can provide valuable information to traffic engineers when determining whether an intersection or phase is oversaturated. For example, average delay is used by the Highway Capacity Manual to determine intersection level of service (1). Typically, the average delay is calculated from a model based on 15-minute volumes and expected green times. However, an intersection may have low average delay, but individual vehicles may still experience long wait times. As agency budget and labor decisions are increasingly driven by customer satisfaction, it makes sense to consider delay measures at the individual vehicle level, especially for low- and medium-volume intersections, where traditional delay measures may not indicate a problem. Such measures should also provide meaningful information to the traffic engineer in the case of oversaturation on certain phases at higher-volume intersections.

HISTORICAL & REAL-TIME DELAY MEASUREMENTS
Models for estimating intersection delay are mostly based on theory developed by Webster (1966). He described a methodology for computing average delay for fixed-time and vehicle-actuated traffic signals, while indicating that field-collected data showed a standard deviation in delay values as 75% of the mean delay (2). Similarly, Akcelik (1988) proposed a revised model for computing average vehicle delay for undersaturated and oversaturated intersection conditions (3). These works provided the foundational theory upon which delay calculations in the 2010 Highway Capacity Manual are based, but they do not always provide an accurate portrayal of delay variability at the individual vehicle level.

Real-time data collection was pioneered in advanced control systems by measuring occupancy and cyclical flow profiles; many of these systems use surrogate measures of delay due to past data collection limitations (4), (5). Recently, high-resolution controller data has emerged as a new data source for developing performance measures (6). This data has been used to directly measure arrival profiles and develop real-time delay estimates, using techniques such as the input-output approach by Sharma (2008) (7). These bulk-vehicle delay estimates require advance detection, which is typically only available on coordinated approaches (and sometimes not at all).

A paper by Sunkari et al. (2012) proposed a portable data collection system that could measure the time from vehicle arrival to the start of green. This measure, referred to as “time to
service”, was intended to measure the responsiveness of a controller to detector calls on different phases. The authors demonstrated the use of this performance measure in real-world settings for one particular location (8).

Recently, Smith (2014) proposed a similar concept of “maximum vehicle delay”, which was defined as the maximum waiting time by any vehicle during a single cycle. For a particular phase, this often corresponded to the waiting time faced by the first arriving vehicle. However, the definition of maximum vehicle delay was broadened to include instances of vehicles turning right on red, as well as cases where split failures occur for an oversaturated phase. The performance measure was tested along a single low-volume corridor (9).

Based on a review of the literature, traditional delay measures can help to diagnose oversaturated or severely problematic intersections; however, they fail to consider the delay variability which may adversely impact drivers at otherwise adequately performing intersections. Until recently, delay has largely been computed on a total or average basis, with vehicle-specific measurements only obtained through software modelling and signal controller algorithms. A few recent studies have recognized the need for real-time measurement of individual vehicle delay, but the concept remains to be evaluated in a variety of real-world intersection environments. This paper develops the concept of maximum vehicle delay further, and implements the performance measure in a variety of case studies that demonstrate its utility.

CONCEPTUAL OVERVIEW OF MAXIMUM VEHICLE DELAY

This paper introduces a performance measure for estimating the upper bound on delay for an individual vehicle. This measure, referred to as maximum vehicle delay (MVD), is based on the time elapsed between the first detection event for a lane group, and the subsequent start of green for that lane group. Because MVD is derived solely from high resolution detection and phase timing data logged by the signal controller, this performance measure can be utilized for any approach with any type of detection, although it is particularly suited for lane groups where stop bar detection is present.

Figure 1 illustrates the maximum vehicle delay concept for a through movement. Figure 1a shows a trace of the detector and phase events as they are logged in the signal controller during a single cycle, while Figure 1b through Figure 1g show the arrival of several vehicles on the approach. In this example, the first vehicle arrives at 9:04:25.1, where the stop bar detector in the lane turns on (callout i). Over the next 29 seconds, two additional vehicles arrive (Figure 1c). At 9:04:54.4, the phase is served (callout ii). This results in a MVD value of 29.3 seconds, found by subtracting the detector on time from the start of green time. The queue begins moving at 09:04:55.6 (Figure 1d), and the last vehicle in queue begins moving at 09:04:59.9 (Figure 1e). At 09:05:01.0, the last vehicle in queue clears the detection zone, and the stop bar detector channel changes from on to off (callout iii). At 09:05:07.2, the queue has completely cleared the approach (callout iv), and the MVD calculation is reset. Note that the queue clearance event is defined as the first detector off time after the start of green, regardless of the interval that elapses until the next detector on time; that is, there is no clearance or passage time built into the performance measure.

The MVD calculation involves two timing elements: (implicitly) the time between the start of red (end of yellow) and the first detection event, and (explicitly) the time between the first detection event and the end of red (start of green). Figure 2 compares these two measures. Figure 2a shows an empirical and theoretical cumulative distribution function (CDF) of the times between start of red on the phase and the first detection event for that phase approach for each cycle during the analysis period, while Figure 2b shows the empirical and theoretical CDFs of MVD on the
FIGURE 1  Relation between vehicle arrivals, phase/detection behavior, and MVD. Eastbound through at US231 and State St. shown. The signal head for the phase is shown in the inset.
FIGURE 2 Comparison of (a) time between start of red and first vehicle detection and (b) time between first vehicle detection and start of green (MVD). Phase 2 at US231 & State St. from 0600 – 0900 shown.
same phase and time period. The time between start of red on a phase and the first vehicle detection is especially useful for validating the presence of split failures (in the case of queue progression being terminated by a red indicator, this time value would likely be negative, since the first detection event occurs before the start of red).

The sample CDFs are shown for the State Street intersection (Case Study #2) before coordination; they are used to validate the assumption of random vehicle arrivals at the intersection, and to show that MVD behaves consistently for intersections with and without arrivals that are influenced by external factors (such as coordination). In each plot, an empirical CDF is first computed, and a theoretical CDF is fitted to the data by minimizing the residual sum of squares (RSS). In both cases, the theoretical CDF corresponds to that of a negative exponential distribution, whose general form is as follows:

\[ y = 1 - e^{-\frac{x}{\mu}} \]  

(1)

where \( y \) is the probability of the measured variable being at or below the threshold value \( x \), and \( \mu \) is equal to the average value of \( x \), as well as the inverse of a rate parameter \( \lambda \) (that is, \( \lambda = \mu^{-1} \)).

In Figure 2a, the negative exponential distribution fits the empirical data quite well, and the best fit model indicates an average time between start of red and the first detection event of 16.3 seconds (denoted by \( \mu_1 \)). This is consistent with other reports in the literature that vehicle arrivals follow a Poisson process, in accordance with general queuing theory (10), (11). Similarly, the distribution in Figure 2b indicates an average MVD of 19.1 seconds (denoted by \( \mu_2 \)). The close fit with the negative exponential distribution indicates that MVD can also be considered as a Poisson process, suggesting that it is memoryless (that is, it generally does not depend on previous cycles).

**Comparison to Phase Red Time**

Figure 3a shows a 24-hour plot of Cycle Length, Red Time and MVD for an undersaturated protected-permitted left-turn movement. The cycle length at this location is a nominal 90 seconds throughout the day, with variations from cycle to cycle caused by the use of early yield (9). The red time for the left turn phase is roughly 70 seconds during these cycles. During the late night and early morning periods, the cycle tends to dwell on the mainline green, leading to very long cycle lengths and red times. Throughout most of the day, there are no vehicles present for this movement, as shown by the relatively few number of cycles for which a MVD was calculated (despite the lack of demand, the movement continues to be served by a permitted green because of dual entry.) The red time and the cycle length are both much greater than the individual MVD values, which range from 0 to no more than 60 seconds. This is also shown by a CDF (Figure 3b), representing the distribution of MVD measured from 09:00–15:00.

In contrast, Figure 3c shows a 24-hour plot for a protected-permitted left turn that is often oversaturated throughout the day. The cycle length at this location is approximately 120 seconds. Here, many of the MVD values fall above the red time and even the cycle length, denoted by callout \( i \). These represent situations where the detector is occupied for a longer time than either the red time or the cycle length. The CDF (Figure 3d) reveals that about 10% of the cycles during the 09:00–15:00 time period exhibit these characteristic oversaturation MVD values, which are considerably higher than the undersaturated values. In the oversaturated regime, the values of MVD are less likely to correspond to true individual vehicle delays, but the high amount of
separation from the undersaturated values makes the resulting CDF useful for identifying the
frequency of split failures.

As Figure 3 shows, at low- and medium-volume intersections, MVD allows the engineer
to identify cases of extreme delay for individual drivers. At high-volume intersections, it provides
a measure of oversaturated conditions, which facilitates the identification of recurrent split failures
in order to reduce overall driver delay.
STUDY METHODOLOGY

To demonstrate how MVD responds to varying traffic conditions and timing scenarios, a series of case studies are examined here to illustrate the impacts of changes to signal timing. Table 1

FIGURE 3  Comparison of (a) MVD over a 24-hour period, showing undersaturated conditions, (b) the corresponding CDF of MVD from 0900-1500, (c) a 24-hour plot of MVD showing oversaturated conditions, and (d) its corresponding CDF from 0900-1500.
provides a comprehensive summary and schedule of the activities undertaken. For each case study, MVD was calculated from high-resolution signal controller data at the intersection (12). The start of red events were recorded for each phase, and the vehicle detection events were recorded for each lane group (13).

**TABLE 1 Schedule of Field Data Collection for MVD for Various Intersection and Treatment Combinations**

<table>
<thead>
<tr>
<th>Start</th>
<th>End</th>
<th>TOD Plan</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>July 15, 2013</td>
<td>July 19, 2013</td>
<td>0900 to 1500</td>
<td>No adjustments</td>
</tr>
<tr>
<td>July 29, 2013</td>
<td>August 2, 2013</td>
<td>0900 to 1500</td>
<td>Split adjustments on phase 3/8</td>
</tr>
<tr>
<td>December 2, 2013</td>
<td>December 6, 2013</td>
<td>0600 to 0900; 1500 to 1900</td>
<td>No coordination (free mode)</td>
</tr>
<tr>
<td>April 21, 2014</td>
<td>April 25, 2014</td>
<td>0600 to 0900; 1500 to 1900</td>
<td>Coordination on phase 2/6</td>
</tr>
<tr>
<td>May 9th, 2013</td>
<td>May 9th, 2013</td>
<td>1900 to 2200</td>
<td>104s Cycle Length</td>
</tr>
<tr>
<td>May 22nd, 2013</td>
<td>May 22nd, 2013</td>
<td>1900 to 2200</td>
<td>108s Cycle Length</td>
</tr>
<tr>
<td>July 2nd, 2013</td>
<td>July 2nd, 2013</td>
<td>1900 to 2200</td>
<td>112s Cycle Length</td>
</tr>
<tr>
<td>June 19th, 2013</td>
<td>June 19th, 2013</td>
<td>1900 to 2200</td>
<td>116s Cycle Length</td>
</tr>
<tr>
<td>July 24th, 2013</td>
<td>July 24th, 2013</td>
<td>1900 to 2200</td>
<td>120s Cycle Length</td>
</tr>
<tr>
<td>May 13th, 2013</td>
<td>May 13th, 2013</td>
<td>1900 to 2200</td>
<td>124s Cycle Length</td>
</tr>
<tr>
<td>US231 &amp; Martin Jischke Dr.</td>
<td>February 3, 2014</td>
<td>February 3, 2014</td>
<td>0900 to 1500</td>
</tr>
<tr>
<td>US231 &amp; Martin Jischke Dr.</td>
<td>February 4, 2014</td>
<td>February 4, 2014</td>
<td>0900 to 1500</td>
</tr>
</tbody>
</table>

**CASE STUDY #1: SIDE STREET SPLIT ADJUSTMENT**

US 31 and 126th St. in Carmel, Indiana was selected as a study location for the impact of side street split adjustment on MVD. US 31 is a major signalized corridor that runs north-south through the Indianapolis Metropolitan Area, while 126th St. provides arterial movements for traffic from major retail development to the east and a mix of retail and residential development to the west.

US 31 has consistently high traffic volumes throughout the day, with an AADT of 36,000 (14). A previous study noted a large number of split failures on the 126th Street approaches, including the westbound through and left-turn movements (15). In response, the split of phase 3 was increased from 16% to 20%, while the split of phase 8 was increased from 20% to 24% of the cycle length. The time was taken from phases 2 and 6, which each saw a 4% decrease. The splits of phases 4 and 7 were unchanged. The MVD was calculated for each phase based on a week of data collected before and after the split adjustments. Figure 4 provides a summary of the effects of these split adjustments on MVD, expressed through a series of empirical CDFs by phase. Also shown in this figure are the split adjustments made to each phase.

The greatest reductions in MVD occurred on phases 3 and 8, for which the split percentages were increased. The median delay values decreased by 5 and 9 seconds, respectively. The variance in the delay time distributions did not substantially change, as shown by the same general shapes.
of the distributions. On phase 3, the number of split failures was substantially reduced; the CDFs show that before the split adjustments, approximately 20% of the cycles were oversaturated on this phase. This reduced to less than 5% of cycles after the split adjustments, as evidenced by the leftward shift in the CDF.

A point of clarification on the use of MVD for detecting split failures is that it will only work on phases where stop bar detection is present. The first vehicle arrival time can still be determined on phases with advance detection only, but unless excessive queuing is present on the phase, it is unlikely that the advance detector will remain continuously occupied throughout the red phase. The calculated MVD will this almost always be less than the phase red time, even when several vehicles remain at the stop bar after the end of green.

A slight reduction in delay can also be seen on phase 7, the eastbound left turn. Although the split of phase 7 did not change, the reduction in green time on phase 6 led it to be called earlier in the cycle. This could potentially explain the decrease in MVD, since the earlier start of phase 7 meant that vehicles coming from intersections further to the west would be more likely to see a green upon initial arrival. Finally, the MVD increased very slightly on the mainline protected left turn phases, although the overall distribution functions are largely identical.

**FIGURE 4** Case study #1, split adjustments: graphs of empirical cumulative distribution of MVD during 0900-1500 at US31 & 126th St.

**CASE STUDY #2: IMPLEMENTING COORDINATED PROGRESSION**

The US 231 and State Street intersection, west of the Purdue University campus in West Lafayette, was used to assess MVD in comparing coordinated versus non-coordinated operation. The US 231 corridor opened to traffic in September 2013, and was operated in fully-actuated mode until January 2014. At this time, coordination was implemented along the north-south mainline.
movements (phases 2 and 6). Two weeklong data collection periods were used to compare intersection performance before and after implementing coordination.

Figure 5a shows the changes in MVD before and after coordination for the AM peak (0600–0900), while Figure 5b shows the changes for the PM peak (1500–1900). Overall, coordination increased MVD for all phases. This is not surprising, because the coordinated cycle length of 90 seconds was considerably longer than the effective cycle lengths during fully-actuated operations, which was typically around 50 seconds. The corresponding increases in red times result in the increases in MVD. This was most pronounced on the side street phases where the peak direction through movements (phase 4 for AM peak, phase 8 for PM peak) saw delay increases of 30s and 27s, respectively. Minor increases in delay were also reported on the mainline phases, ranging from 2s to 9s per cycle. Alternatively, the agency could consider running this intersection at a half cycle length of 45s, relative to the other intersections in the corridor, to try and reduce delay on the side street phases.

Interestingly, the coordinated phases do not exhibit particularly different behavior from the other phases, despite receiving a heavy share of the green time as a result of coordinated signal timing. Although there is a potential for many of the arrivals on those phases to belong to coordinated platoons, the first vehicle is likely to be a random arrival from an upstream side-street entry. For those vehicles, the wait time is still longer during coordination than during fully-actuated operations.

The purpose of coordination is to improve traffic flows along a corridor by aligning vehicle arrivals with the green period of the coordinated phases. Figure 6 shows the empirical CDFs of these travel times during the AM and PM peak periods in both directions, as measured using Bluetooth re-identification (16). It can be clearly seen that corridor travel times are reduced after the implementation of coordination. Most evaluations of coordinated signal operations focus primarily on corridor travel times and arrivals on green. Thus, as illustrated here, MVD does provide a necessary perspective for balancing those aspects of performance with potential increases in delay for the non-coordinated phases, which are often overlooked by the field engineer.
FIGURE 5 Case study #2, before and after coordination: graphs of empirical cumulative distribution of MVD during (a) 0600-0900 and (b) 1500-1900 at US231 and State St.
FIGURE 6 Case study #2, before and after coordination: graphs of empirical cumulative distribution of corridor travel time during (a) 0600-0900, northbound, (b) 0600-0900, southbound, (c) 1500-1900, northbound, and (d) 1500-1900, southbound on US 231.

CASE STUDY #3: INCREASING CYCLE LENGTH WITHIN A COORDINATED SYSTEM

The third case study considers the impacts of varying intersection cycle lengths, and illustrates how MVD can identify anomalous controller behavior. The study intersection was SR37 and 126th Street, located north of I-69 on the northeast side of Indianapolis. This intersection has multiple medium- and high-volume movements due to its proximity to I-69 and substantial residential and
commercial development. AADT on the mainline is approximately 35,000, while the side street corridor sees daily traffic volumes approaching 6,000.

Figure 7 provides an overview of the MVD calculations for varying cycle length. As the cycle length increases, the MVD also increases. This aligns with expectations, since a longer cycle length corresponds to longer red times occurring on those phases. The increase is more pronounced for some phases than others. Phase 1, for example, sees the median MVD increase from 50s to 78s (callout ii), while the increase for phase 3 is only from 50s to 58s (callout iii). Despite an expected increase in MVD with longer cycle lengths, there is no corresponding decrease in the percentage of split-failed cycles, as initially expected (since longer cycle lengths serve more green time to individual phases). While the effect is subtle in this analysis (the percentage of MVD values exceeding the cycle length was low to begin with), it provides some evidence that higher cycle lengths may not necessarily be an effective solution for oversaturated intersections.

The MVD performance measure was also used to identify anomalous controller behavior. In Figure 7, the cumulative distributions of MVD are consistently different for the 120s cycle length. At approximately the 80th percentile mark, the distribution makes a sharp turn to the right (callout i), indicating that the phases experience very long delays during approximately 20% of the cycles over the 1900-2200 time period. This is in great contrast with the other cycle lengths, which do not exhibit this behavior. As mentioned earlier, the upper end of the distribution is an indicator of split failure, and warrants a deeper look into the operation.

Figure 8 provides a more detailed look at this phenomenon. Figure 8a shows the empirical CDF of the measured cycle lengths, as determined from the actual high resolution signal controller data. These CDFs show that while the cycle lengths are centered on their nominally programmed values, there are instances where they run for a shorter or longer period, due to events such as timing plan transitions or ped calls pushing the intersection out of coordination. The beginning and ending of a cycle is considered to occur whenever the signal transitions from the side-street block of phases (3,4,7,8) to the mainline block of phases (1,2,5,6).

As expected, the vertical portion of the distribution corresponds to the programmed cycle length, as illustrated by callout ii. The tails of the distribution correspond to instances where the effective cycle length was longer or shorter than the programmed value. This is a result of the use of actuated coordination (17), (18), which enables the coordinated phase to gap out under certain conditions. The ability for the phase to gap out produces variation in the actual cycle length, while still maintaining a fixed portion of the cycle length during which the coordinated phase green is scheduled. Some of the variation might also be attributable to controller transition at the beginning of the timing plan.

The cumulative distribution for the 120s cycle length shows that approximately 20% of the cycles were running at or greater than 200s, as shown by the significant horizontal portion of the CDF, (callout i). This is in contrast with the distribution of measured cycle lengths for the other days, where the vertical section of the distribution corresponds to the nominally programmed cycle length for that day (callout ii). Figure 8b and Figure 8c show plots of cycle length over time for operations at 104s and 120s, respectively. The long cycle lengths occurring during the day of 120s operation were caused by a coordination configuration problem that caused the controller to repeatedly (and unexpectedly) fall into transition. The impact can clearly be seen in the high proportion of cycles having long MVD.
FIGURE 7 Case study #3, 104s – 124s cycle length sweep: graphs of empirical cumulative distribution of MVD during 1900 – 2200 at SR37 & 126th St.
FIGURE 8  Case study #3, graphs of (a) empirical cumulative distribution functions of cycle lengths during 1900 – 2200 at SR37 & 126th St. Also shown are the distribution of cycle lengths over the timing plan for the (b) 104s cycle length and (c) 120s cycle length.

CASE STUDY #4: COORDINATED PHASE SPLIT EXTENSION & SIDE STREET PHASE RE-SERVICE

In addition to evaluating traditional timing plan changes, MVD can also be used to assess the impacts of advanced or proprietary controller features. One such feature is phase reservice, variations of which are available in some controllers. This feature enables the controller to leave the coordinated phase to serve non-coordinated phases, when there is enough time to carry out that
maneuver before the scheduled beginning of coordinated green. This can potentially serve non-coordinated phases multiple times in the same cycle (19).

The case study intersection used here was US 231 and Martin Jischke Drive (approximately 1 mile south of the US 231 and State St. intersection referenced in the coordinated operation case study). This is a 3-leg intersection, with the mainline coordinated movements controlled by phases 2 and 6, while phase 8 controls the side street movement. Phase 1 corresponds to the southbound protected left movement. Because of sight distance restrictions in the southbound direction, left turns are only allowed during the protected phase.

The impacts of phase reservice on MVD can be seen in Figure 9. It did not appear to reduce the delay time on phase 8. However, the MVD on phase 1 decreased substantially, with median delay times reduced by approximately 12s. This is somewhat in contrast to the expectation, which was that the controller would leave the mainline phases (2 and 6) to serve the side street phase (8), which comes next in the local sequence. In effect, the phase reservice feature circumvents the sequence rules and can select any phase for service. The delay time reductions on the protected mainline left turn phase (1), and lack of reduction on phase 8, show that the controller tended to favor phase 1 over phase 8. Bench testing showed that when the feature was used, if calls on phase 1 and 8 were placed simultaneously, there were certain times in the cycle when phase 1 would be served before phase 8. In contrast, when the feature was disabled, phase 8 would always receive priority. Most likely, additional timing constraints such as phase backup prevention rules would force the controller to serve phase 8 first. Nevertheless, this case study demonstrates the need for performance measures to verify the impacts of various control features.

**FIGURE 9** Case study #4, with and without phase reservice: graphs of empirical cumulative distribution functions of MVD during 0900 – 1500 at US231 and Martin Jischke Dr.
CONCLUSION

A new performance measure, maximum vehicle delay, was proposed for evaluating intersection behavior in four different case studies:

1. The MVD performance measure can be used to accurately assess minor intersection changes, such as side-street split adjustments (Figure 4). It can also be used to identify split failures on individual phases, and quantify the reductions in both individual and total driver delay.

2. MVD clearly illustrated the impacts of implementing coordination for non-coordinated phases. In the case study, MVD increased very slightly on the mainline through and left turn phases, while the side street phases saw substantial delay increases (Figure 5). Ultimately, this information enables a more complete analysis of intersection operations to be balanced against an evaluation of progression (Figure 6), enabling trade-offs between coordinated and non-coordinated phases to be characterized.

3. MVD was useful for identifying controller issues in a coordinated corridor. It was observed that increasing the intersection cycle length consistently resulted in increased MVD for the mainline protected left and side street phases (Figure 7); however, the coordinator did not perform as programmed, warranting further investigation into the cause (Figure 8). In this case study, MVD was also used to show that longer cycle lengths do not necessarily correspond to fewer split failures.

4. Finally, MVD was used to demonstrate the impact of a less commonly implemented controller feature, phase reservice. In contrast with a priori expectations, phase reservice did not reduce delay on the side street by a substantial amount (Figure 9). However, MVD was reduced on the mainline protected left turn phase, a result of the logic for this feature inherent to the controller.

Future research should focus on determining how MVD can be used together with other performance measures to provide a comprehensive evaluation of single intersection behavior. This might include the development of an intersection-level MVD value, computed as some sort of a weighted average of the individual phase MVDs. Furthermore, this study omitted phases with advance detection only from the MVD calculations; there is the possibility to explore whether another measure, such as volume to capacity (v/c) ratios, could serve as a suitable proxy measure of MVD in cases where stop-bar detection is absent. More generally, other performance measures such as occupancy ratio or total waiting vehicles on the side streets, could also be compared. Finally, it should be determined if the stochastic nature of this performance measure can be effectively parameterized for various scenarios, and whether this can be useful for computing more accurate total delay measures for the intersection.

ACKNOWLEDGEMENT

This work was supported by the Indiana Department of Transportation. The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein, and do not necessarily reflect the official views or policies of the sponsoring organizations. These contents do not constitute a standard, specification, or regulation.

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