FINAL REPORT

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THE FLORIDA DEPARTMENT OF TRANSPORTATION
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on Project

“Development of Recommendations for Arterial Lane Closures to Optimize Traffic Operations”

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from

The University of Florida

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METRIC CONVERSION CHART

U.S. UNITS TO METRIC (SI) UNITS

LENGTH

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km

METRIC (SI) UNITS TO U.S. UNITS

LENGTH

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi

TECHNICAL REPORT DOCUMENTATION PAGE

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16. Abstract

The 2010 FDOT Design Standards provide information regarding traffic control through work zones at multilane arterials. However, there are currently no quantitative guidelines for optimizing signal control around work zones. The objective of this project was to formulate suitable recommendations for the development of signal control plans, including phasing, signal timings, and channelization. The research focused on multilane arterial streets. These signal control optimization guidelines were developed distinguishing between three different cases:

- *Case 1: Lane Closure before the Intersection*. In this case, the work zone area blocks one or more lanes upstream of the intersection, and there is some distance from the work zone to the stop bar.
- *Case 2: Lane Closure at the Stop Bar*. This case can be further divided into two subcases: lane closure at the stop bar that causes changes in the type of the remaining lanes and lane closure at the stop bar that reduces the number of lanes but does not change the remaining channelization.
- *Case 3: Lane Closure at Some Distance Downstream from the Subject Intersection*. In this case, the work zone area will block some lanes in the middle of an arterial link between two intersections. There are three key parameters in this case: demand from the upstream intersection ($D_{upstream}$), capacity of the lane closure area ($C_{closure}$), and capacity of the downstream intersection ($C_{downstream}$).

Detailed guidelines were developed to optimize signal control around each of the work zone cases described above. A combination of field data and simulation was used to evaluate these guidelines and document their effectiveness under different demand conditions. Generally, signal retiming around work zones is warranted only when the work zone is expected to significantly impact operations and increase delay. This occurs when demand is high, approaching or exceeding capacity. If that is not the case, the existing signalization plan should be retained. The report provides the basic steps of the guidelines developed for each of the three cases, and the equations for performing the respective calculations.

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EXECUTIVE SUMMARY

The 2010 FDOT Design Standards provide information regarding traffic control through work zones at multilane arterials. However, there are currently no quantitative guidelines for optimizing signal control around work zones. The objectives of this project were to study the relationship between signal control and work zone organization at arterial streets and formulate suitable recommendations for the development of signal control plans, including phasing, signal timings, and channelization. The research focused on multilane arterial streets.

These signal control optimization guidelines were developed distinguishing between three different work zone cases:

- *Case 1: Lane Closure Before the Intersection.* In this case, the work zone area blocks one or more lanes upstream of the intersection, and there is some distance from the work zone to the stop bar. In this case, it is important that the green given to the approach does not significantly exceed the demand that is able to pass through the work zone. Otherwise, portions of the green time given to that approach would be wasted. The key issue for signal timing under this case is to efficiently use the available storage area (i.e., the area between the downstream end of the work zone and the signalized approach stop bar) so that the capacity of the approach is optimally used.
- *Case 2: Lane Closure at the Stop Bar.* This case can be further divided into two subcases: lane closure at the stop bar that causes changes in the type of the remaining lanes and lane closure at the stop bar that reduces the number of lanes but does not change the remaining channelization. When developing a signal timing plan for this case, the subject approach should first be considered for rechannelization based on the respective demand of the different movements. The next step is to retime the traffic signal to optimize the intersection operations based on the new channelization and per lane demand.
- *Case 3: Lane Closure at Some Distance Downstream from the Subject Intersection.* In this case, the work zone area will block some lanes in the middle of an arterial link between two intersections. There are three key parameters in this case: demand from the upstream intersection ($D_{upstream}$), capacity of the lane closure area ($C_{closure}$), and capacity of the downstream intersection ($C_{downstream}$). The values of the three parameters determine

at which area congested conditions may occur and which signal timing would need to be modified.

Detailed guidelines were developed to optimize signal control around each of the work zone cases described above. A combination of field data and simulation was used to evaluate these guidelines and document their effectiveness under different demand conditions. In this study, signal retiming around work zones was warranted only when the work zone was expected to significantly impact operations and increase delay. This occurs when demand is high, approaching or exceeding capacity. If that is not the case, the existing signalization plan should be retained. The following paragraphs provide the basic steps of the guideliness developed for each of the three cases, while Chapter 3 of the report provides the equations for performing the respective calculations.

Signal control optimization for Case 1:

a) Calculate the maximum green interval, which should be set equal to the time required to clear the queue between the stop bar and the downstream of the work zone. This green time in a cycle may not be long enough to meet the high demand of the subject approach. In that case, the required total green time can be split into two short intervals to ensure that each phase is not longer than the suggested maximum green, and the phase of another movement in between allows the queue to build up before the repeated green is given.
b) Calculate the throughput of the approach before and after splitting the phase.
c) Compare the before and after throughput, and select the signal timing that results in the higher throughput.

Signal control optimization for Case 2:

This lane closure case can be further divided into two subcases: a) lane closure at the stop bar causes change in the channelization type of the remaining lanes; b) lane closure at the stop bar reduces the overall number of lanes but there are no changes in channelization.

For the situation in which the lane closure causes changes in channelization, the phasing pattern along with the timing should be reconsidered. The Highway Capacity Manual (HCM) 2000, among others, provides detailed instructions for developing a phase plan and estimating the basic signal timing parameters. For developing a phase plan, the general guideline is that a simple two-phase control should be used unless conditions dictate the need for additional phases. For the configurations that do not result in changes in channelization, only the green interval for each phase needs to be adjusted according to the corresponding new per lane volume.

<u>Signal control optimization for Case 3:</u>

a) Compare $D_{upstream,}$ $C_{closure}$ and $C_{downstream}$ on an hourly basis to ensure that all the vehicles from the upstream intersection can be served ($D^h_{upstream}$ is the hourly demand from the upstream intersection in veh/h, it consists of the demand of the three movements at the upstream intersection; $C^h_{closure}$ is the hourly capacity of the lane closure area in veh/h; $C^h_{downstream}$ is the demand from the upstream intersection in veh/h).

If $D^h_{upstream} > C^h_{closure}$, the capacity of the lane closure area cannot meet the demand, resulting in queues building up and spilling back into the upstream intersection. In this case, a smaller g/C ratio (effective green time over cycle time ratio) of the corresponding upstream phases should be implemented to reduce the number of vehicles that enter the work zone area. If the *g/C* ratio cannot be reduced to the value calculated from the equation above, the analyst should consult the third step of the method. Alternatively, the construction work should be rescheduled for another time period when $D^h_{uptream}$ is less than $C^h_{closure}$.

If $D^h_{upstream} > C^h_{downstream}$, then not all vehicles can be served at the downstream intersection. Queues will keep building and finally spill back into the upstream intersection. There are two ways to address this problem. The first one is to reduce the upstream demand by using a smaller *g/C* ratio of the corresponding upstream phases. The second way is to increase the downstream capacity by increasing the *g/C* ratio of the downstream intersection. If the *g/*C ratio cannot be adjusted to the value calculated from Equation 16 or Equation 18, the analyst should consult the third step of the method.

Alternatively, it is suggested that the construction work should be rescheduled for a less congested time period when $D^h_{upstream}$ is less than $C^h_{downstream}$.

b) Compare $D_{upstream}$, $C_{closure}$ and $C_{downstream}$ on a per phase basis to avoid the occurrence of the spillback queues on a per phase basis. The analysis procedure discussed below should be conducted for each of the three upstream phases (left-turning, through, and right-turning).

c) Conduct further analysis for cases when the conditions in the previous steps cannot be met. The steps discussed above involve basic analysis for a general case when a lane closure is installed along an arterial link between two signalized intersections. The proposed methodology can accommodate some of the spillback that may occur. In cases when the recommended modifications cannot adequately address the expected operational conditions, it is possible that a full signal optimization would be able to address those. A full signal optimization can adjust the offset between intersections so that the queues can be better managed. However, in cases of severe congestion, it may be that there are no signalization improvements that can alleviate congestion when the work zone is installed. In those cases, it may be preferable to schedule the work zone for another time period, if feasible.

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1 INTRODUCTION

1.1 Background

The 2010 FDOT Design Standards provide information regarding traffic control through work zones at multilane arterials. Information is provided for work within the intersection (Index 615), near the intersection in the median or outside lanes (Index 616), near the intersection in the middle lane (Index 617), and for two-lane closures (Index 618). This information focuses on the placement of signs and markings, and does not provide any guidance with respect to the traffic signal control.

Previous research (FDOT Contract BD545-61, Elefteriadou et al., 2008) used simulation to estimate the capacity of various arterial work zone configurations, and to develop models for estimating the capacity under various design and work zone scenarios. One of the research conclusions was that the distance of the work zone to the downstream intersection affects the capacity of the entire arterial work zone. One can maximize the throughput of the intersection approach by using specific combinations of the g/C (green to cycle length) and the distance from the stop bar to the work zone. Also, the capacity of the arterial work zone is reduced when one of the movements is blocked by the other. The probability of such blockage increases when the g/C ratios are not optimal or when the channelization at the intersection is not optimal for the respective demands.

Because of the significant need for work zone closures and the restrictions placed on work zone lane closures, there is a need for better analytical tools and guidelines so that construction engineers can optimize traffic operations during lane closures. This research uses the findings of the previous research project and extends them to develop guidelines for traffic control plans at multilane arterials with work zones. It is expected that the proposed research will help improve traffic operations through arterial work zones by using the pavement remaining during construction more efficiently.

1.2 Objectives

The objectives of this project were to study the relationship between signal control and work zone organization at arterial streets and develop suitable recommendations for the development of signal control plans, including phasing, signal timings, and channelization. The research focused on multilane arterial streets.

1.3 Research Approach and Report Organization

To accomplish the research objectives we first reviewed the literature to identify and categorize specific cases of lane closures, based on their expected impacts and interactions with the signal control and channelization. Based on this review we categorized work zones into three broad categories. For each of these categories we developed preliminary signalization guidelines to be implemented when a work zone is present. Next, we identified two work zone locations and collected field data, to be used in a series of simulation experiments to test the guidelines developed. Lastly, we finalized our recommended guidelines based on the field observations and the results of the analysis.

Chapter 2 reviews the literature with respect to work zone types along multilane arterials and the existing guidelines related to signal control at work zones. Based on the literature and the expected specific impacts of different cases, multilane arterial work zones are categorized into three broad cases based on the location of the work zone relative to the signalized intersection. Chapter 3 develops and discusses the proposed guidelines for signal control optimization of each work zone lane closure case while Chapter 4 summarizes the data collection. Chapter 5 discusses the simulation testing of the recommended guidelines using the field data. Finally, Chapter 6 summarizes the conclusions and recommendations of the study.

2 IDENTIFICATION AND CATEGORIZATION OF LANE CLOSURES

This chapter first provides an overview of the literature review on work zone types and related signalization guidelines, and then discusses the resulting categorization of work zone types along multilane arterials and the existing guidelines related to signal control at work zones.

2.1 Literature Review on Work Zone Types for Multilane Arterials

The researchers first reviewed various state and national documents in order to identify and categorize various work zone cases. Part 6 of the Manual of Uniform Traffic Control Devices (MUTCD, FHWA, 2009) provides the national standard for all traffic control devices used during construction, maintenance, utility activities and incident management. For application of work zone traffic control devices at multilane arterial intersections, the Manual indicates that the lane closure scenarios can be generally classified into three categories according to the location of the work space with respect to the intersection area:

- near side
- far side
- in-the-intersection

The MUTCD provides guidance regarding the traffic control devices when the lane closure is on the near side of an intersection (Figure 2.1), right- or left-lane closure on the far side of an intersection (Figure 2.2 and 2.3), half road closure on the far side of an intersection (Figure 2.4), multiple lane closures at an intersection (Figure 2.5), closure in the center of an intersection (Figure 2.6), and closure at the side of an intersection (Figure 2.7). Sometimes, work spaces extend into more than one portion of the intersection. For these configurations the MUTCD indicates that an appropriate traffic control plan should be obtained by combining features shown in two or more of the typical guidance plans.

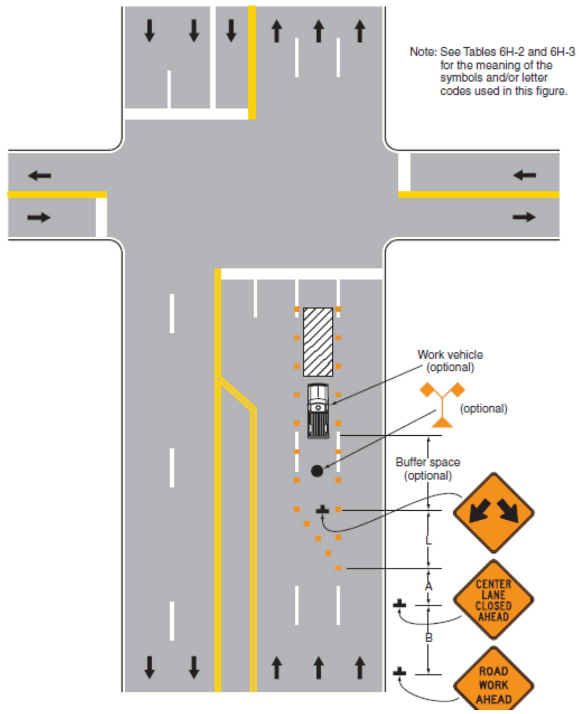

Figure 2.1 Lane Closure on the Near Side of an Intersection

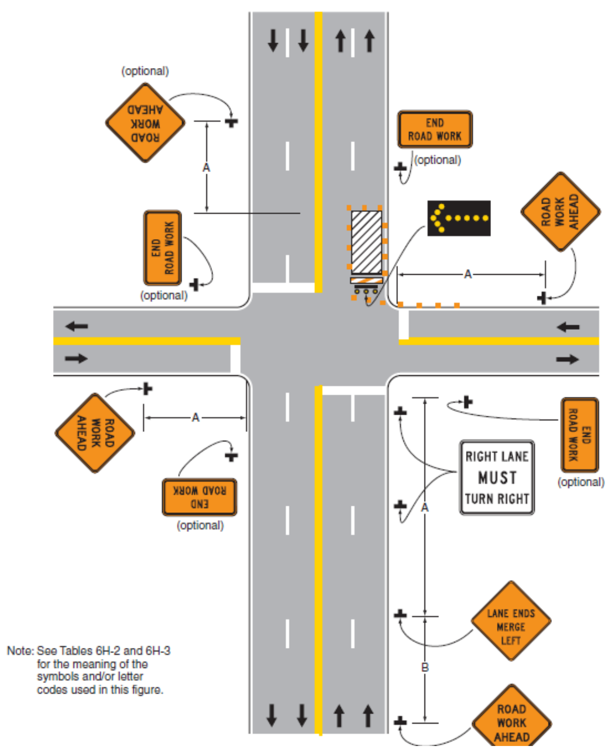

Figure 2.2 Right-Hand Lane Closure on the Far Side of an Intersection

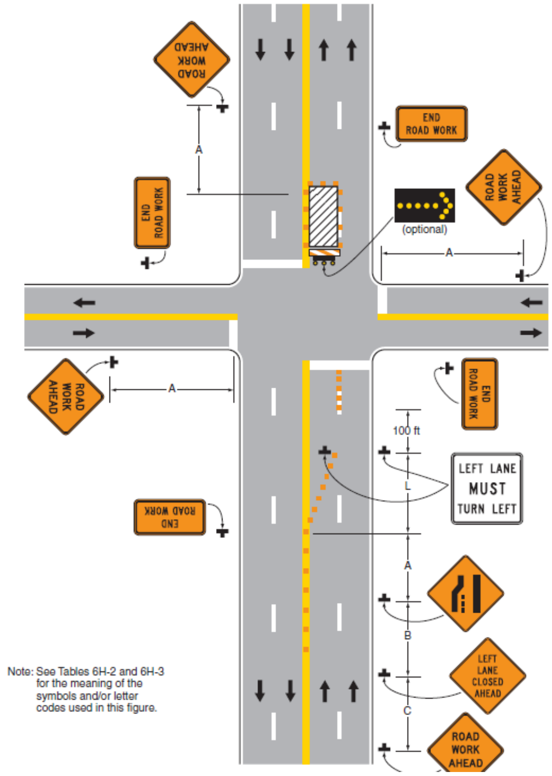

Figure 2.3 Left-Hand Lane Closure on the Far Side of an Intersection

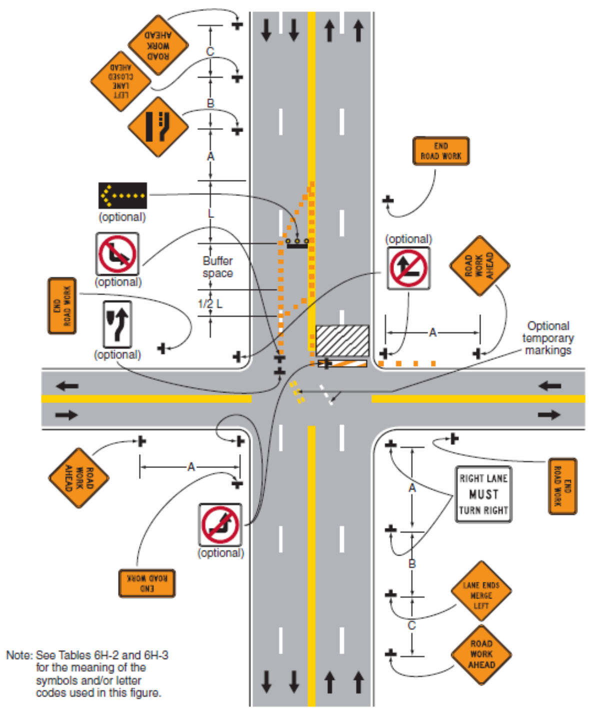

Figure 2.4 Half Road Closure on the Far Side of an Intersection

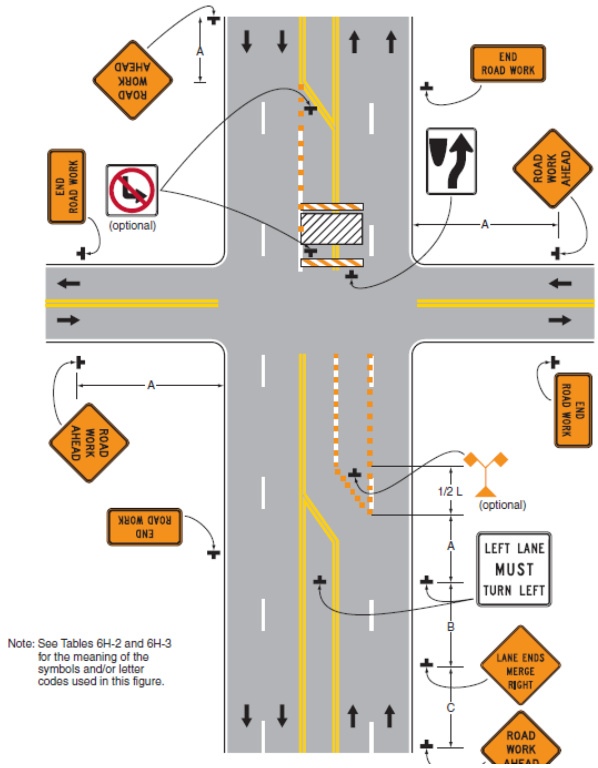

Figure 2.5 Multiple Lane Closures at an Intersection

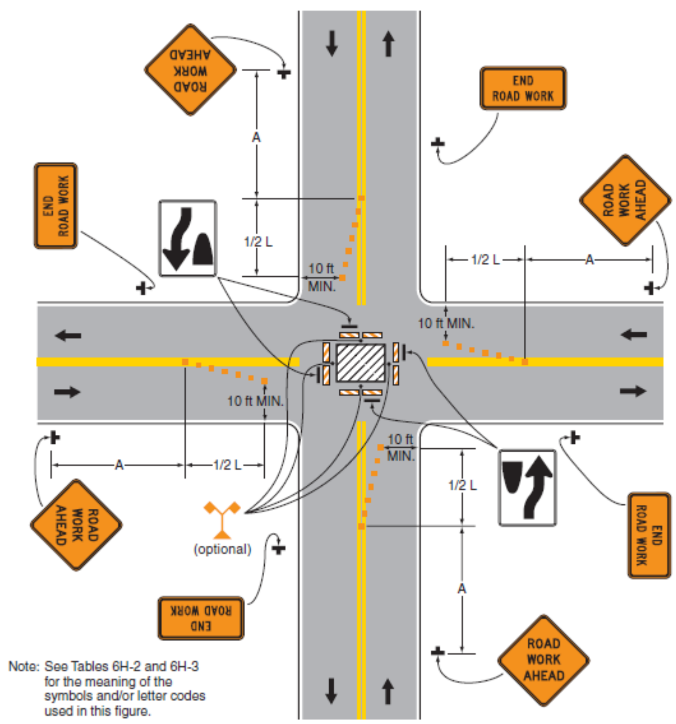

Figure 2.6 Closure in the Center of an Intersection

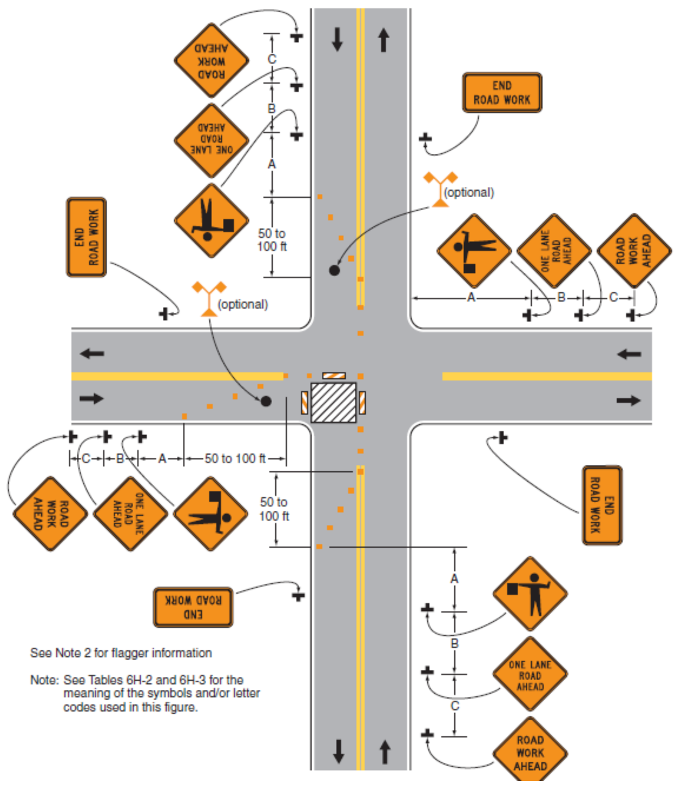

Figure 2.7 Closure at the Side of an Intersection

Based on the MUTCD, many states have developed their own standards and have categorized the lane closure scenarios in a similar way. The Florida Department of Transportation (2010 FDOT Design Standards) provides information regarding traffic control through work zones at multilane arterials for work zones within the multilane intersection (Index 615, conditions where vehicle, equipment, workers or their activities encroach on the pavement and require the closure of at least one median traffic lane), median or outside lane closed on the near side or far side of the intersection (Index 616), closure of the center lane near an intersection (Index 617), and for two-lane closures (Index 618, conditions where vehicle, equipment, workers or their activities encroach on the pavement requiring the closure of either the outside and center travel lanes or the median and center travel lanes). Compared to the MUTCD, the FDOT standards distinguish two different scenarios for the schematic shown in Figure 2.1: near side median or outside lane closure, and near side center lane closure. Then, the former is combined with far side left or right lane closure (Figure 2.2 and 2.3) into a new scenario (median or outside lane closed; Index 616). Closure of the center lane is treated as a separate scenario (Index 617). With respect to multiple lane closures (such as the one depicted in Figure 2.5) the FDOT standards consider only the two- lane closures on one side of the intersection (Index 618). The far side half road closure (Figure 2.4), the closure in the center of the intersection (Figure 2.6), and the closure at the side of an intersection (Figure 2.7) are not specifically considered in the FDOT standards. However, the FDOT standards consider another condition, namely work zones within the multilane intersection (Index 615).

The INDOT (Indiana Department of Transportation) Work Zone Traffic Control Handbook (2011) provides similar guidance on the placement of signs and markings for work zones at multilane arterial intersections. Lane closure scenarios that are considered in this handbook include through lane closure in advance of an intersection, lane closure on the far side of an intersection, turn lane closure at an intersection and closure in the center of intersection.

The intersection lane closures considered in the Temporary Traffic Control Zone Layouts Field Manual (2011) by the Minnesota Department of Transportation include lane closure on the far side of the intersection, lane closure on the near side of the intersection, left and right lane closures when work space is beyond the intersection, double lane closure at the intersection, and closure in the center of the intersection.

In addition to these handbooks and guidelines, Heaslip et al. (2011) studied how different factors affect the work zone capacity using simulation software. In their research, channelization at the intersection was considered as an important factor. The lane channelization scenarios that were considered in this study are illustrated in Figure 2.8. When estimating the work zone capacity under each channelization scenario, they only considered the number of lanes open and closed, but did not consider which specific lane(s) were closed.

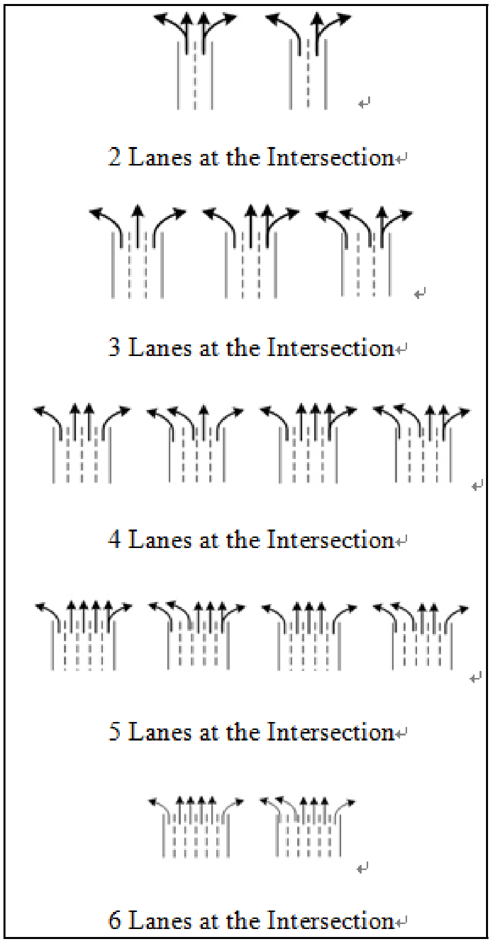

Figure 2.8 Lane Channelization Configurations (Heaslip et al., 2011)

2.2 Literature Review on Signal Control Guidelines at Multilane Arterial Work Zones

All of the current work zone traffic control manuals examined indicate that signal timing of the intersection within an impact area of the work zone should be revised to accommodate the changes in travel pattern, but none of them provide detailed guidelines to optimize traffic operations during the lane closure.

Hawkins et al. (1991) indicate that signal phasing and timing should be adjusted with each change in construction phasing, and that signal operation should be checked in the field after the adjustment. They also suggest that short cycle lengths may be useful in reducing queue backup into the intersection.

A guide for work zone analysis by Wisconsin Departmenr of Transportation (2009) indicates that during construction some of the lanes at the intersection will not be available for traffic to use and therefore intersection capacity will be significantly affected. In order to avoid queuing delays, they suggest making changes in the signal timing at the intersection. The guide also indicates that sometimes it is desirable to increase the cycle length to compensate for configurations when the left, through, and right turning vehicles are sharing a single lane after lane closure. Moreover, in order to maintain good traffic progression along the corridor, signal offsets should be adjusted to account for reduced travel speeds.

The Illinois Department of Transportation (IDOT) Bureau of Design and Environment (BDE) Manual (IDOT, 2011) and a Federal Highway Administration (FHWA) guide (2005) also suggest retiming the traffic signals within a work zone to optimize the intersection capacity. They suggest that adding or deleting signal phases may be required for changes in travel patterns. Also, they both point out that adding interconnection or improving coordination between traffic signals will move traffic through a work zone more efficiently.

A previous research project (FDOT BD545-61, Elefteriadou et al., 2008) used simulation to estimate the capacity of various arterial work zone configurations and to develop models for estimating intersection approach capacity under various design and work zone scenarios. The research found that the distance of the work zone to the downstream intersection affects the capacity of the entire arterial work zone. The throughput of the intersection approach can be maximized by using specific values of the *g/C* (green to cycle length) as a function of the distance from the stop bar to the work zone. The research also noted that the capacity of the arterial work zone is reduced when one of the movements is blocked by another. The probability of such blockage increases when the *g/C* ratio is not optimal or when the channelization at the intersection is not optimal for the respective demands.

Li and Elefteriadou (2012) proposed a method to maximize the throughput of a single-lane approach with a branched configuration (one-lane arterial with multiple lanes at the intersection approach). They found that for such an intersection approach, turn bays can be efficiently used only for a limited amount of time after the start of green, and setting the maximum green too high will result in loss of efficiency. The proposed method provided equations for estimating the maximum green that would maximize throughput and suggested repeating the phase twice in the cycle such that the multilane section can fill up each time before the green is given. The configuration studied in that research is similar to that of a work zone with a lane closure upstream of the intersection approach. Therefore, its findings are very useful and can be applied in this project.

2.3 Categorization of Specific Cases

Based on the literature and the potential signal control impact factors associated with the channelization, work zone lane closure cases are categorized into three cases. Each of these cases are described in this subsection.

2.3.1 Case 1: Lane Closure before the Intersection

In this case, the work zone area blocks one or more lanes upstream of the intersection, and there is some distance from the work zone to the stop bar. Thus, the work zone acts as a meter, and reduces the demand to the intersection. Thus, it is important that the green given to the approach does not significantly exceed the demand that is able to pass through the work zone. Otherwise, portions of the green time given to that approach would be wasted. The key issue for signal timing under this case is to efficiently use the available storage area (i.e., the area between the downstream end of the work zone and the signalized approach stop bar) so that the capacity of the approach is optimally used. Two types of configurations are distinguished:

a) The lane closure blocks the turning movement or the rightmost/leftmost lane, and may generate new or reconfigured turn bays (the left side of Figure 2.9 shows the “before” configuration while the right side shows the “after” configuration with the work zone in place).

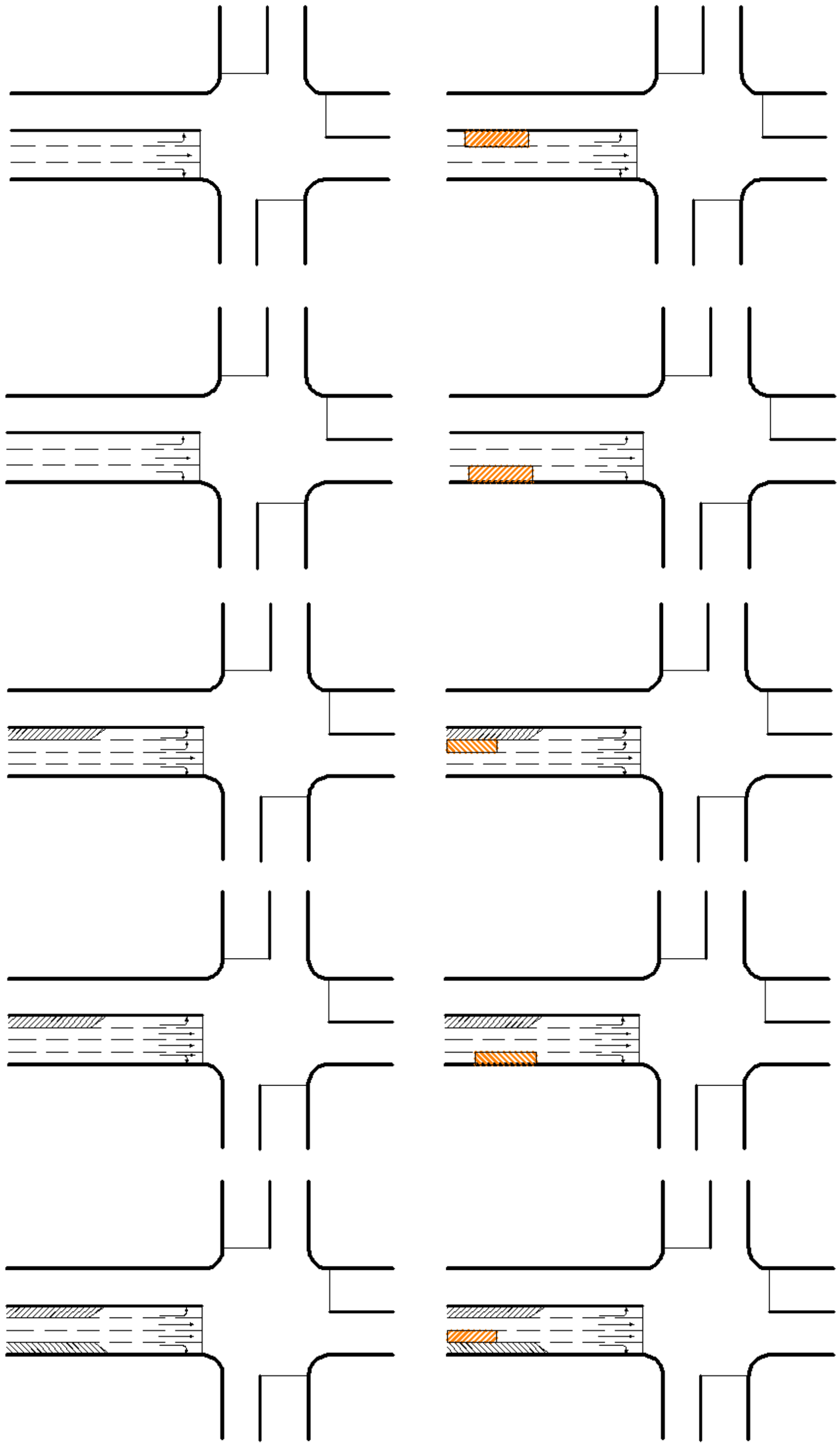

Figure 2.9 Lane Closure Affects a Turning Movement

b) The lane closure blocks the through movement. Figure 2.10 shows the respective "before" and "after" configurations.

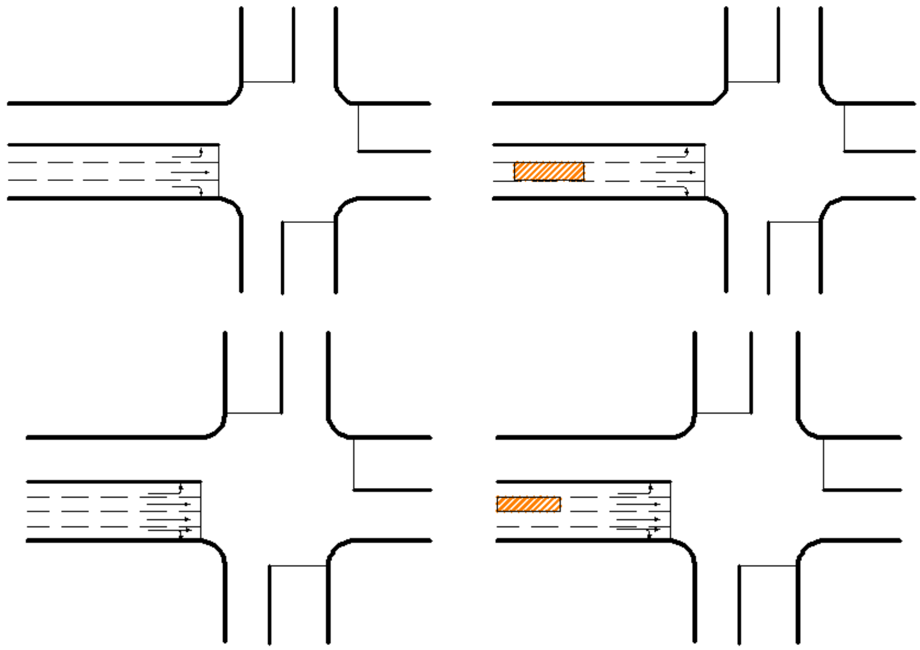

Figure 2.10 Lane Closure Affects the Through Movement

2.3.2 Case 2: Lane Closure at the Stop Bar

Based on the resulting channelization after the lane closure is in place, this case can be further divided into two subcases: lane closure at the stop bar that causes changes in the type of the remaining lanes and lane closure at the stop bar that reduces the number of lanes but does not change the remaining channelization.

Lane Closure at the Stop Bar Causes Change in the Type of the Remaining Lanes

In this subcase, the work zone area blocks one or more lanes at the intersection. This scenario is likely to result in changes in channelization at the intersection, and thus related changes in phasing and signal timings. The following three configurations are identified:

a) The lane closure results in the creation of a shared lane. This may result from closure of an exclusive right or left turn lane and the subsequent inclusion of the respective turning movements with a previously exclusive through lane. It may also result from closure of a through lane, and grouping of the through traffic with a previously exclusive left- or right-turning lane. Examples of such closures are shown in Figure 2.11.

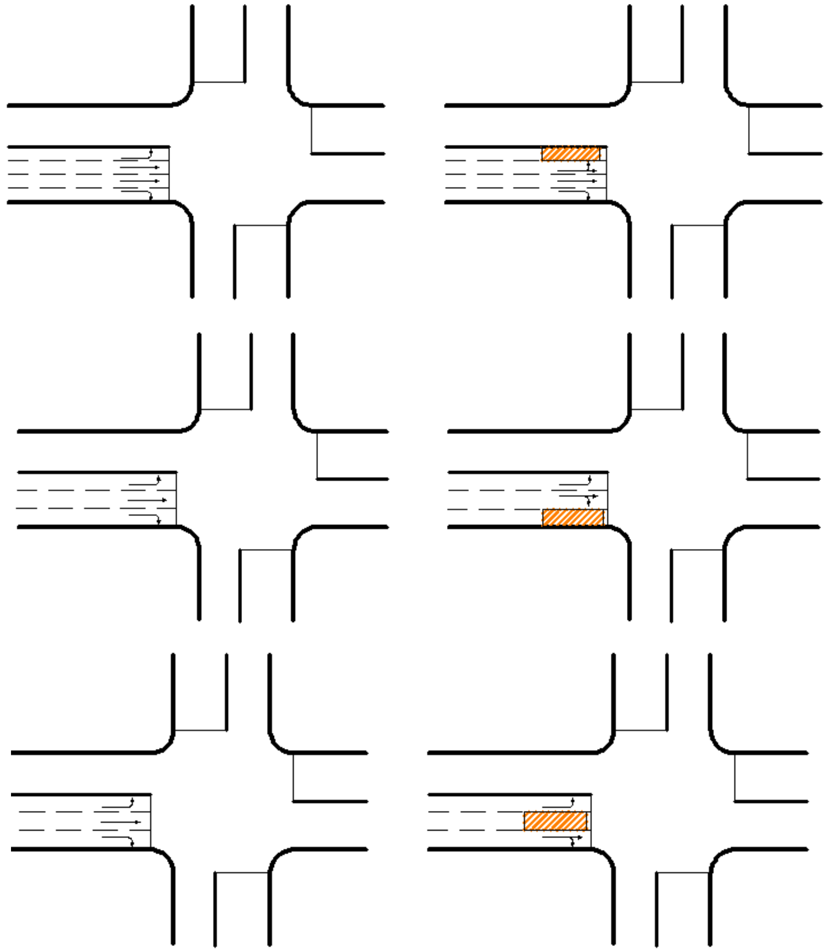

Figure 2.11 Lane Closure Results in the Creation of a Shared Lane

b) The lane closure results in the creation of an exclusive turn lane. This may occur when the work zone is located downstream of the intersection on the outside or median lane. Since the number of receiving lanes is reduced, the upstream through lane should also be rechannelized. In those configurations the operations of the turning movement affected would improve (since it won't have to share the lane with through traffic), but those of the through movement would deteriorate. One approach to mitigate this might be to reallocate some green time from the turning movement to the through movement. Examples of such closures are shown in Figure 2.12.

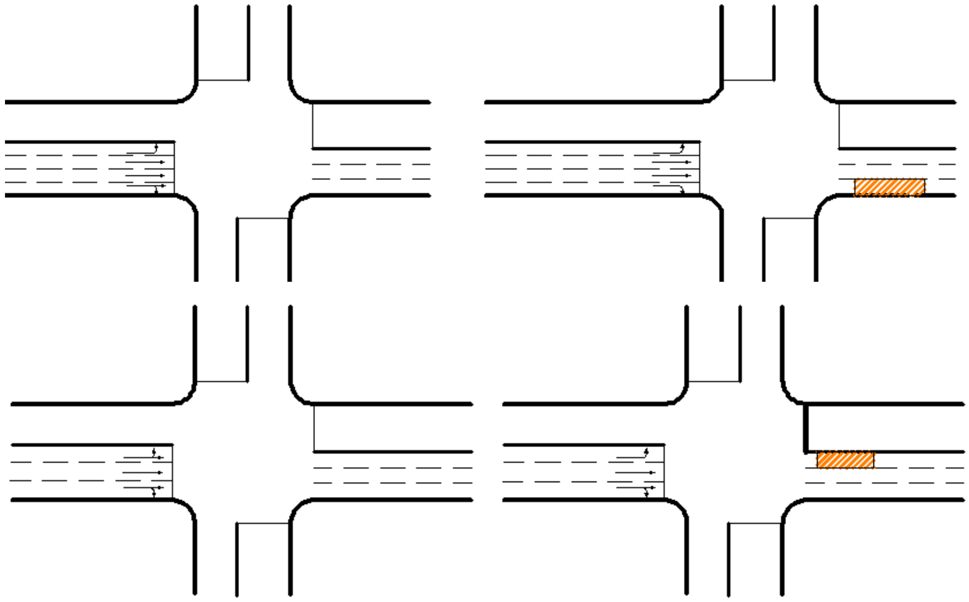

Figure 2.12 Lane Closure Results in the Creation of an Exclusive Turn Lane

c) In this subcase the work zone creates a one-lane approach, as shown in Figure 2.13. This is a special case of configuration (a) shown in Figure 2.11. The main difference is that there is only one phase serving the approach (and thus being affected by work zone operations) and there is no need to balance green times for different movements within the same approach.

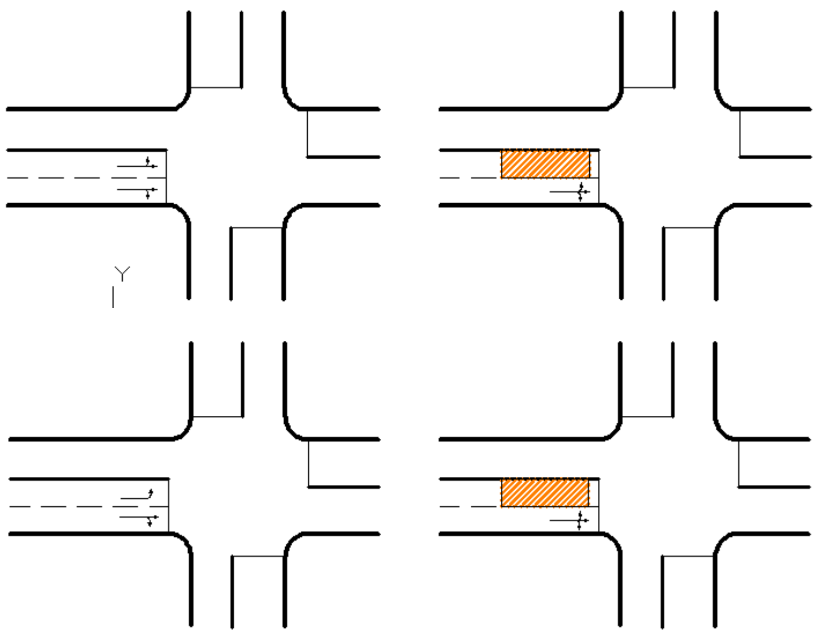

Figure 2.13 Lane Closure Results in the Creation of a One-lane Approach

Lane Closure at the Stop Bar Reduces the Number of Lanes But Does not Change the Remaining Channelization

In this subcase the lane closure reduces the number of lanes for a particular movement (left, through, or right) but does not result in changes in channelization for the remaining lanes. Two configurations are distinguished:

a) Lane closure on the near side of the intersection, as shown in Figure 2.14.

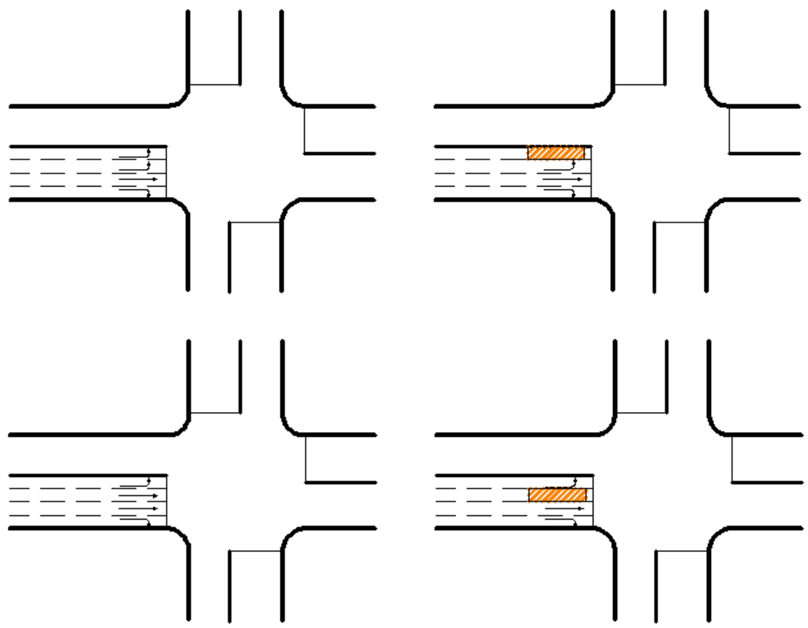

Figure 2.14 Lane Closure on the Near Side of the Intersection

b) The upstream lane is closed due to the work zone on the far side of the intersection, as shown in Figure 2.15

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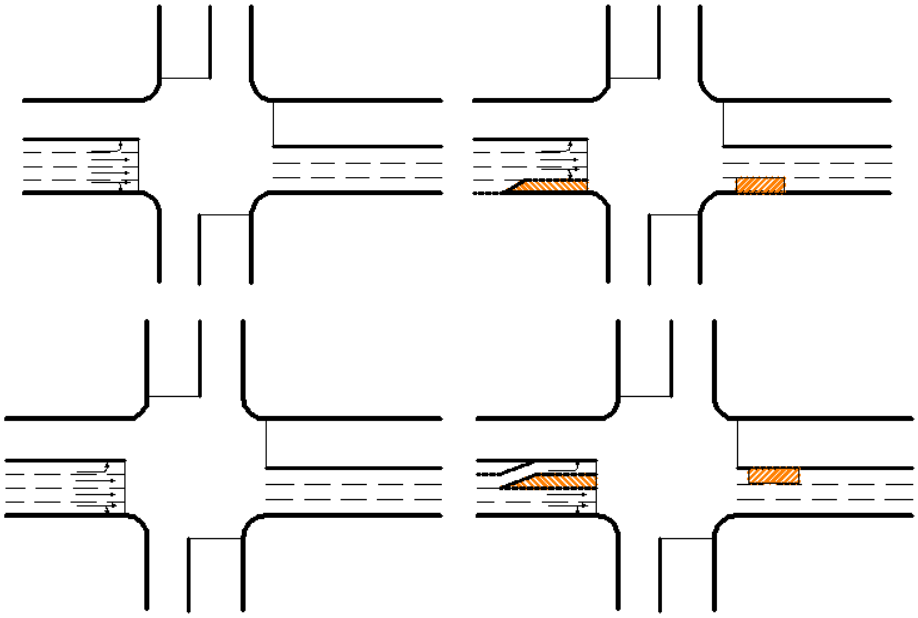


Figure 2.15 Lane Closure on the Far Side of the Intersection

2.3.3 Case 3: Lane Closure at Some Distance Downstream from the Subject Intersection

In this case, the work zone area will block some lanes in the middle of an arterial link between two intersections. Examples of this lane closure case are shown in Figure 2.16. The main difference between this case and the subcase depicted in Figure 2.12 and 2.15 (subcases of Case 2) is that in this case there is a relatively long distance between the upstream intersection and the work zone area, therefore, the lane closure will not have any direct impact on the upstream stop bar channelization. However, it is possible that queues may spill back into the intersection if the capacity of the work zone area is too low relative to the demand.

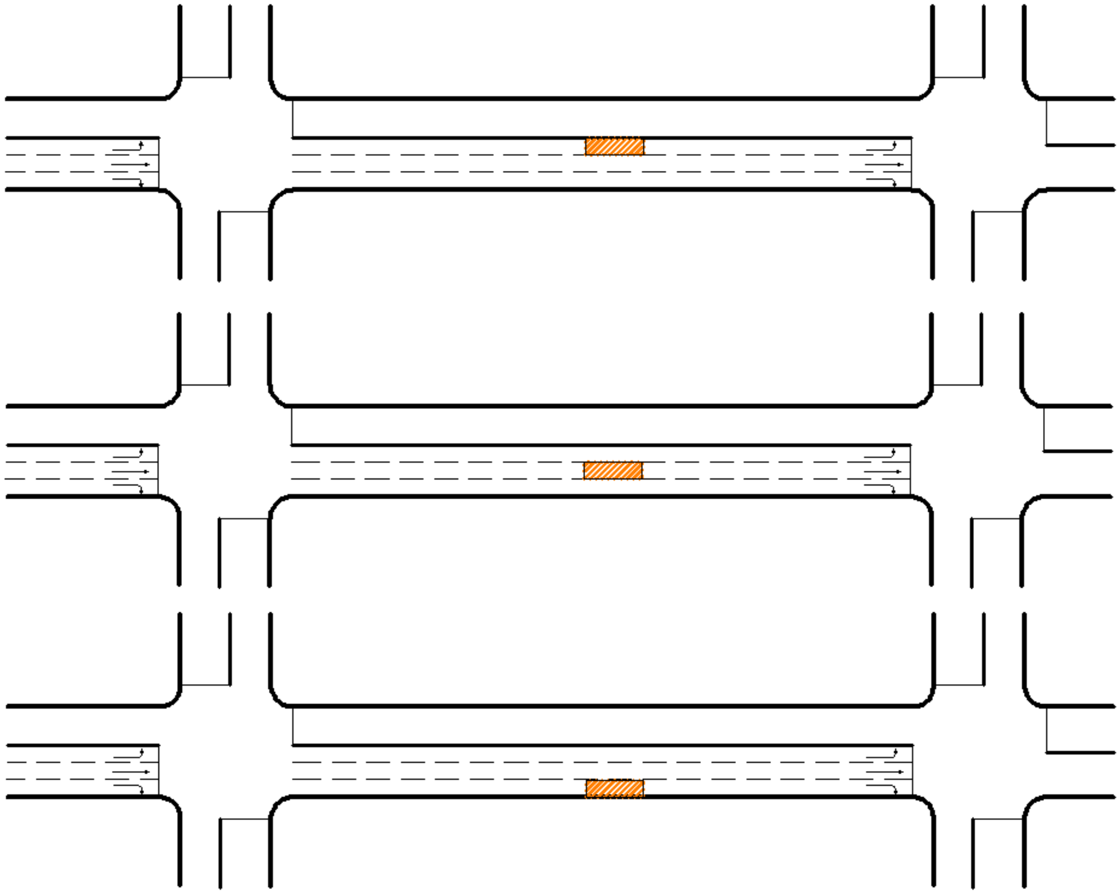


Figure 2.16 Lane Closure at Some Distance Downstream from the Subject Intersection

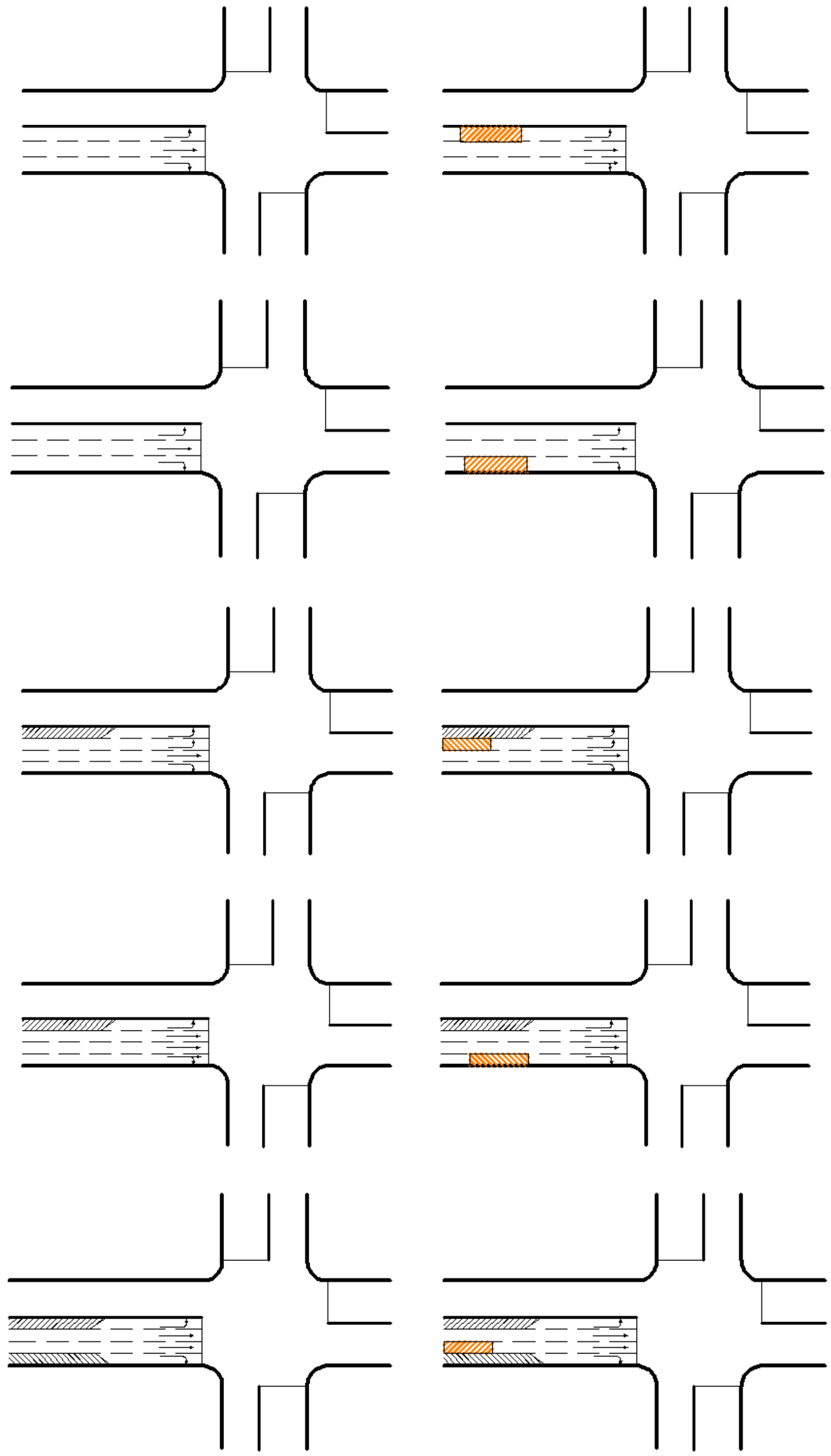
3 DEVELOPMENT OF GUIDELINES FOR OPTIMIZING SIGNAL CONTROL

As described in the previous chapter, three lane closure cases have been identified: lane closure before the intersection; lane closure at the stop bar; and lane closure at some distance downstream from the subject intersection. Based on the literature review discussed in the previous chapter and the specific channelization of each lane closure category, signal control guidelines for each of these cases are developed and described in the remainder of this chapter.

3.1 Case 1: Lane Closure Upstream of the Intersection

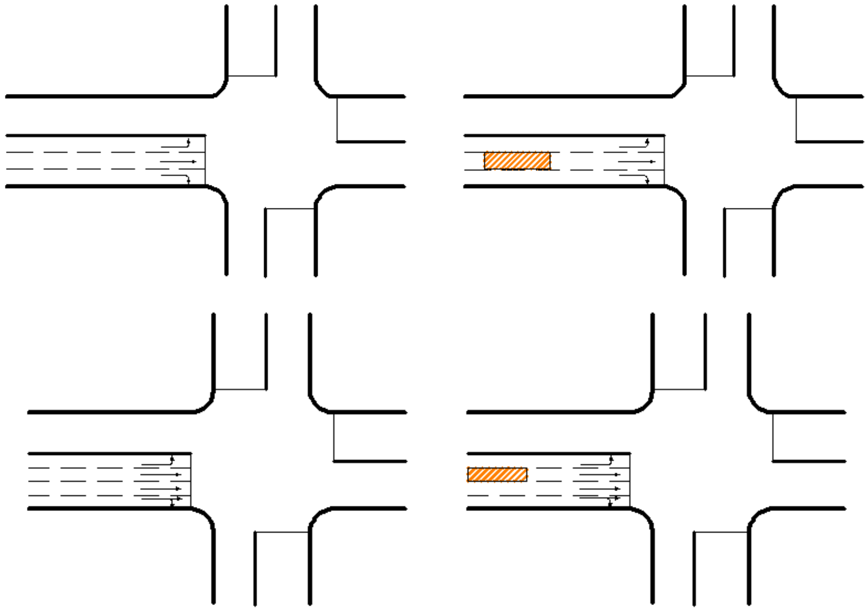
Based on which lane is closed, cases included in this category are further divided into lane closures that block the turning movement and lane closures that block the through movement. Figure 3.1 illustrates these two types of configurations.

Although those two lane closure subcases may affect different movements, the key issue for signal timing under both configurations is how to ensure the capacity of the approach at the stop bar is fully utilized. This problem is similar to the one studied by Li and Elefteriadou (2012). In that research, the authors developed a method for maximizing the throughput of a single-lane approach with a branched configuration (one-lane arterial with multiple turn lanes at the intersection approach). They found that for such configurations, the multilane section can be efficiently used only for a limited amount of time after the start of green, when all lanes discharge vehicles at their respective saturation flow rate. After the multilane portion has been cleared, the discharge rate of the entire approach is much lower than the theoretical capacity of the multilane portion. This problem is illustrated in Figure 3.2 and Figure 3.3. To increase the utilization of that approach, the authors recommended using a shorter green twice in the cycle, such that the multilane section can be filled up each time before the green is given.



a) Lane Closure that Blocks the Turning Movement and Generates New Turn Bays.

Figure 3.1 Configurations for the Case of Lane Closure before the Intersection



b) Lane Closure that Blocks the Through Movement

Figure 3.2 Configurations for the Case of Lane Closure before the Intersection (cont'd)

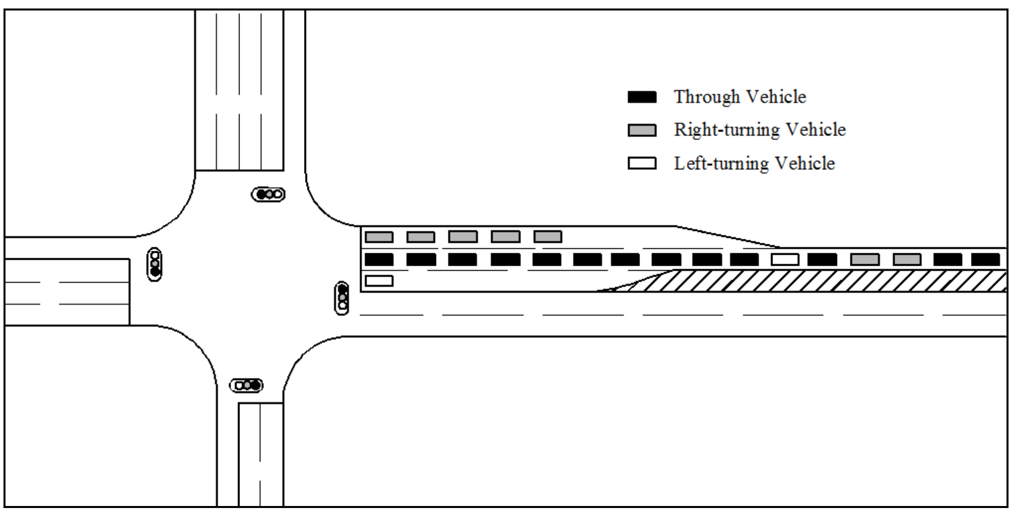


Figure 3.3 Queuing during the Red Interval (Li and Elefteriadou, 2012)

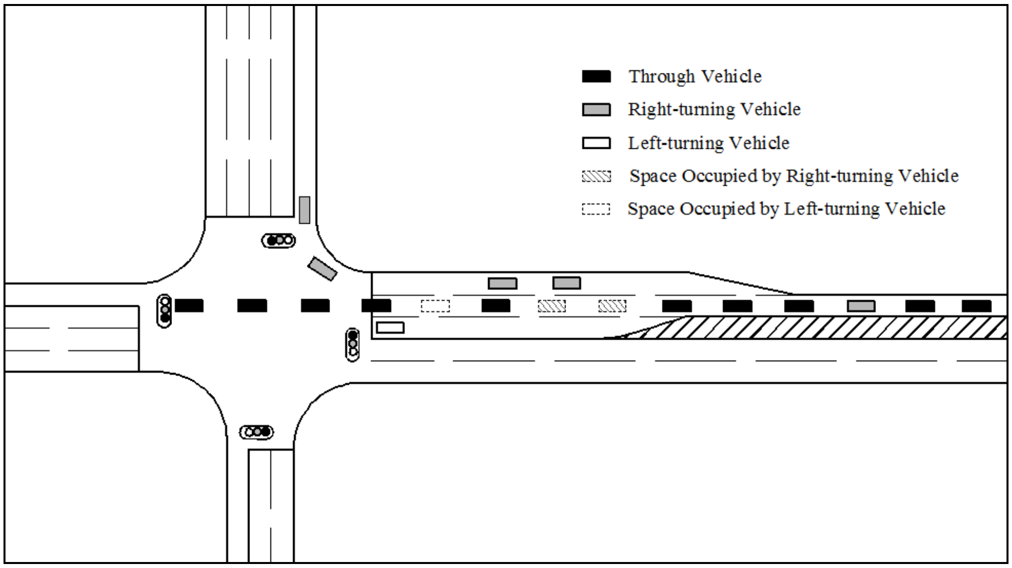


Figure 3.4 Lane Utilization during Queue Discharge (Li and Elefteriadou, 2012)

The configuration of this lane closure case, as well as the capacity utilization problem for the multilane approach at the stop bar is similar to the one studied in Li and Eleteriadou's research. Therefore, that method can also be applied in this research.

To maximize the discharge from the approach, the maximum green interval should be set equal to the time required to clear the queue between the stop bar and the downstream of the work zone:

$$G_{\max} = \frac{D}{V_L} \times S_h + L_T - Y - AR \tag{1}$$

where

G_{max}: Maximum green time for the approach with lane closure upstream of the intersection.

D: Length of the section between the stop bar and the downstream end of the work zone (ft)

V_L: Average space occupied by a queued vehicle (ft)

S_h: Saturation time headway (sec/veh)

L_T: Total lost time per phase (sec)

The lost time is:

$L_T = l_1 + l_2$ (2)

l_1: Start-up lost time per phase (sec).

l_2: Clearance lost time per phase (sec).

Y, AR: Length of the yellow and all red intervals, respectively (sec)

The maximum green interval calculated by this equation can guarantee the utilization of all the approach lanes. However, under congested conditions, this green time in a cycle may not be long enough to meet the high demand of the subject approach. In that case, the required total green time can be split into two short intervals to ensure that each phase is not longer than the suggested maximum green, and the phase of another movement in between allows the queue to build up before the repeated green is given.

Although the green time can be better used after being split into two short intervals, the additional phase will generate more lost time. In order to make sure the increased lost time will not offset the improvement in green time utilization, the throughput of the approach before and after splitting the phase should be compared. Equations 3 and 4 are developed for this comparison:

$$N_{before} = \frac{G_1 - l_1}{H_s} \times N + \frac{G_2 + Y + AR - l_2}{H_s} \times N_r \quad (3)$$

$$N_{after} = \frac{G_1 + G_2 + Y + AR - 2(l_1 + l_2)}{H_s} \times N = \frac{G_1 + G_2 + Y + AR - 2L_T}{H_s} \times N \quad (4)$$

where

N_{before}: Number of vehicles the subject approach can serve per cycle without splitting the green

N_{after}: Number of vehicles the subject approach can serve per cycle after splitting the green

H_s: saturation time headway (sec/veh)

N: Number of lanes of the subject approach at stop bar

N_r: Reduced number of lanes of the lane closure section

L_T: Total lost time per phase (sec)

l_1: Start-up lost time per phase (sec)

l_2: Clearance lost time per phase (sec)

G_1: Time period from the beginning of green to the time when vehicles stored in the branched section have cleared.

The relationship between G_1 and G_{max} is:

$$G_1 = G_{\max} + Y + AR - l_2 \tag{5}$$

G_2: The remaining green time for the subject phase.

G_2 is also:

$G_2 = G - G_1$ (6)

where

G: The actual green time before splitting the green.

There are two components in Equation 3. The first component is the throughput before vehicles stored in the multilane section have been cleared, and the second one is the throughput during the remaining effective green time which is actually only the number of vehicles discharged from the work zone. Equation 4 is used to compute the throughput after splitting the green phase. In that case, all the effective green time in the two phases is efficiently used by all the approach lanes, and therefore equation 4 is similar to the first component in Equation 3.

For the case of lane closures upstream of the intersection, the equations shown above can be used to estimate the maximum green and evaluate the feasibility of the phase splitting method. Further evaluation can be conducted using simulation software if necessary.

In summary, when developing signal timing plans for the case of a lane closure upstream of the intersection, the maximum green time of the subject approach should be evaluated using Equation 1. If the proposed maximum green time is greater than the value computed by Equation 1, splitting the phase into two short phases should be consider based on Equations 3 and 4.

Example

An example is presented below to illustrate the proposed method in more detail. The configuration of the studied approach (eastbound approach) before and after lane closure is demonstrated in Figure 3.4. The original signal timing for this intersection is shown in Table 3.1.

Figure 3.5 illustrates the corresponding phasing diagram of the intersection. The length of the section between the stop bar and the downstream end of the work zone is 450 ft. Start-up lost time and clearance lost time are both 2 seconds per phase. It is assumed that the average space occupied by a queued vehicle is 25 ft/veh and the saturation time headway is 2 sec/veh.

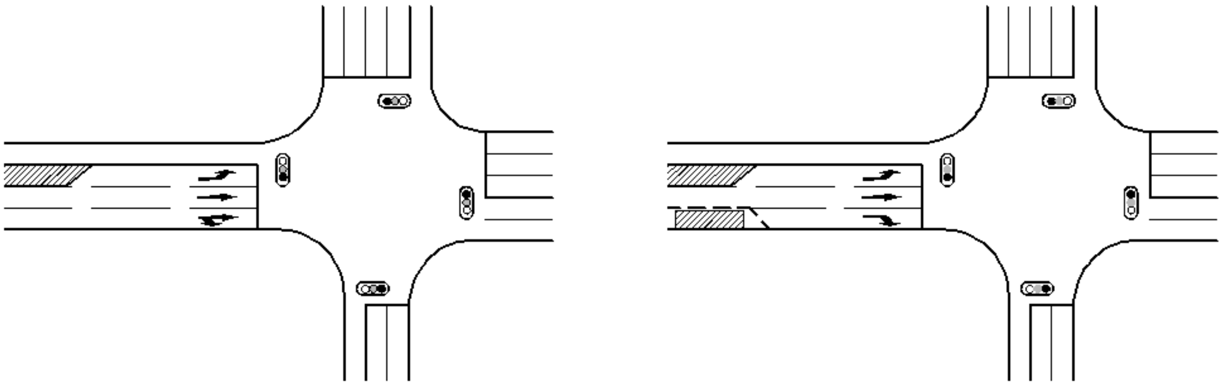


Figure 3.5 Approach Configuration before and after the Lane Closure

Table 3.1 Timing Plan Information for the Study Intersection

Phase	1	2	3	4	5	6	7	8
Min Green	4	15	7	6	4	15	4	4
Max Green	25	70	20	25	25	70	25	20
Yellow	3	3	3	3	3	3	3	3
Red	1	1	1	1	1	1	1	1

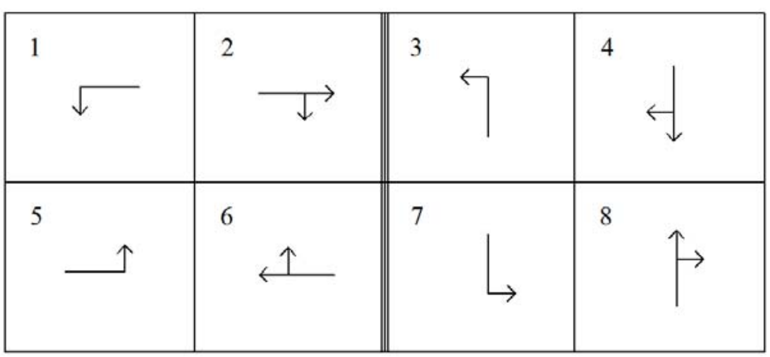


Figure 3.6 Phasing Diagram of the Study Intersection

Step 1. Calculate the theoretical maximum green time

The total lost time per phase is:

$$L_T = l_1 + l_2 = 2 + 2 = 4 \text{ sec} \tag{7}$$

Using equation 1, the maximum green time is:

$$G_{\max} = \frac{D}{V_L} \cdot S_h + L_T - Y - AR = \frac{450}{25} \times 2 + 4 - 3 - 1 = 36 \text{ sec} \tag{8}$$

Step 2. Split the original green and develop the new timing plan

In the original signal timing plan, the maximum green for the eastbound through is 70 sec, which is much longer than 36 sec. Therefore, split the original phase.

Splitting this phase will generate an additional 3-second yellow time and a 1-second red time. Subtracting these 4 seconds from the 70 seconds and dividing the remaining time into two phases results in 33 sec maximum green time for the two new phases, which is less than the theoretical maximum green (36 second) calculated by Equation 1. The cycle length for the intersection remains the same. The proposed timing plane is shown in Table 3.2 and Figure 3.6.

Table 3.2 Proposed Timing Plan Information for the Study Intersection

Phase	1	2	3	4	5	6	7	8
Min Green	4	15	7	6	4	15	4	4
Max Green	25	33	20	25	25	33	25	20
Yellow	3	3	3	3	3	3	3	3
Red	1	1	1	1	1	1	1	1

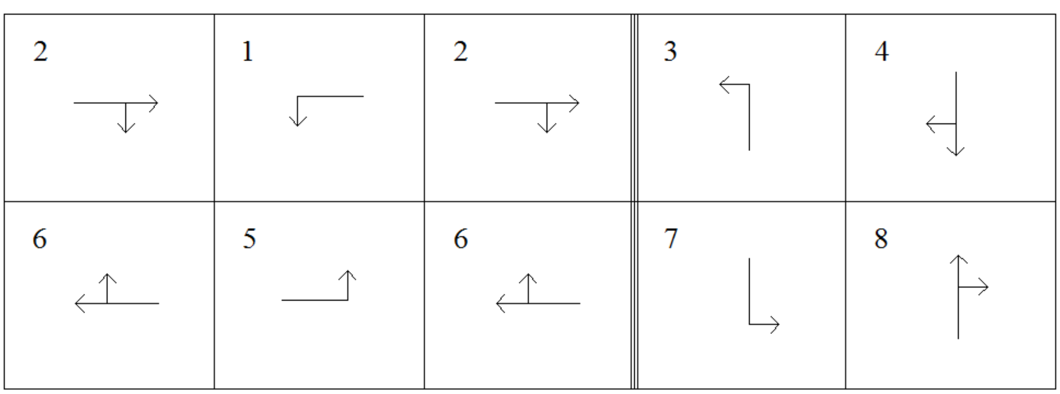


Figure 3.7 Proposed Phasing Diagram of the Study Intersection

Step 3. Calculate the throughput of the before and after scenario

To estimate the throughput in the "before" case, G_1 and G_2 are first calculated using Equation 5 and Equation 6:

$$G_1 = G_{\max} + Y + AR - l_2 = 33 + 3 + 1 - 2 = 35 \text{ sec} \tag{9}$$

$$G_2 = G - G_1 = 70 - 35 = 35 \text{ sec} \tag{10}$$

Based on the two values above, the throughput of the eastbound approach before implementing the proposed method can be determined using Equation 3:

$$\begin{aligned} N_{before} &= \frac{G_1 - l_1}{H_s} \times N + \frac{G_2 + Y + AR - l_2}{H_s} \times N_r \\ &= \frac{35 - 2}{2} \times 2 + \frac{35 + 3 + 1 - 2}{2} \times 1 = 51.5 \text{ vehicles} \end{aligned} \tag{11}$$

Note that, in the equation above, N is 2, and N_r is 1. This is because at the subject approach there are only through and right turning vehicles discharged during this green time.

Using Equation 4, the throughput in the "after" scenario is:

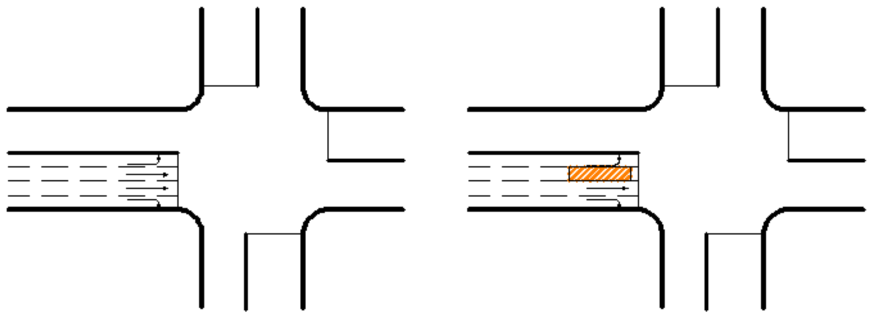
$$N_{after} = \frac{G_1 + G_2 + Y + AR - 2L_T}{H_s} \times N = \frac{70 + 3 + 1 - 2 \times 4}{2} \times 2 = 66 \text{ vehicles} \tag{12}$$

Step 4. Compare the before and after throughput, and determine the final timing plan

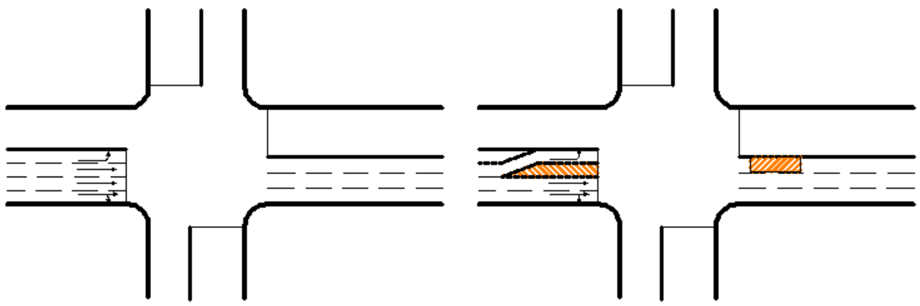
The calculation results above indicated an increased throughput for the subject approach after implementing the new timing scheme (66 vehicles vs. 51.5 vehicles). So, the proposed timing plan can be implemented in the field. To further investigate the potential impacts of the proposed approach on the performance of the approach, this method can also be analyzed and refined using simulation software if necessary.

3.2 Case 2: Lane Closure at the Stop Bar

This lane closure case includes two configurations. In the first configuration, the work zone is on the near side of the intersection (Figure 3.7a), while in the second one the work zone is on the far side right after the intersection (Figure 3.7b). For the latter, the work zone on the downstream side of the intersection reduces the number of receiving lanes; therefore, the corresponding upstream lane also has to be closed.



a) Work Zone on the Near Side of the Intersection



b) Work Zone on the Far Side Right after the Intersection

Figure 3.8 Case 2: Lane Closure at the Stop Bar

Since there is a lane closure at the stop bar, the capacity of the approach is reduced before the intersection. Therefore, the subject approach should first be considered to be rechannelized based on the respective demand of the approach movements. Otherwise, as Heaslip et al. (2011) indicated, the probability of movement blockage increases when the channelization at the intersection is not optimal for a work zone lane closure approach.

The FHWA Signalized Intersection Information Guide (Rodegerdts, et al., 2004) indicates that generally “channelization design should incorporate consideration of the design vehicle, roadway cross section, traffic volumes, vehicle speeds, type and location of traffic control, pedestrians, and bus stops. In addition to these design criteria, consideration should be given to the travel path; drivers should not have to sharply change direction in order to follow the channelization.”

Specifically for left-turning movements, the guide indicates that adopted guidelines and practices of local agencies should be reviewed when determining whether a left-turn lane is warranted. The key elements that should be considered include:

- Functional classification
- Prevailing approach speeds
- Capacity of an intersection
- Proportion of approach vehicles turning left
- Volumes of opposing through vehicles
- Design conditions
- Crash history associated with turning vehicles

This guide also indicates that as a rule of thumb, exclusive left-turn lanes are needed if the left-turn volume is greater than 100 vehicles in a peak hour, or if a left-turn volume is greater than 20 percent of the total volume of the approach.

Similar to left-turn lane warrants, the guide indicates the review of adopted guidelines and practices from local agencies when determining if a right-turn lane is warranted. Also, the following factors should be considered:

- Vehicle speeds
- Turning and through volumes
- Percentage of trucks
- Approach capacity
- Desire to provide right-turn-on-red operation
- Type of highway
- Arrangement/frequency of intersections
- Crash history involving right turns
- Pedestrian conflicts
- Available right-of-way.

Based on the percentage of right turning vehicles during the peak period, NCHRP 279 (Neuman, 1985) identifies warrants for right-turn lanes on four-lane, high-speed roadways in different states. Those warrants are shown in Table 3.3.

The Highway Capacity Manual (HCM) 2000 also indicates the probable need for an exclusive right turn lane if the right-turn volume exceeds 300 vehicles per hour and the adjacent mainline volume exceeds 300 vehicles per hour per lane. More detailed guidance for channelization of different movements is included in Chapter 12: Individual Movement Treatments, of the FHWA Signalized Intersection Information Guide (Rodegerdts, et al., 2004). Designers can follow the corresponding instructions.

Table 3.3 Right-turn Lane Volume Warrants (Neuman, 1985)

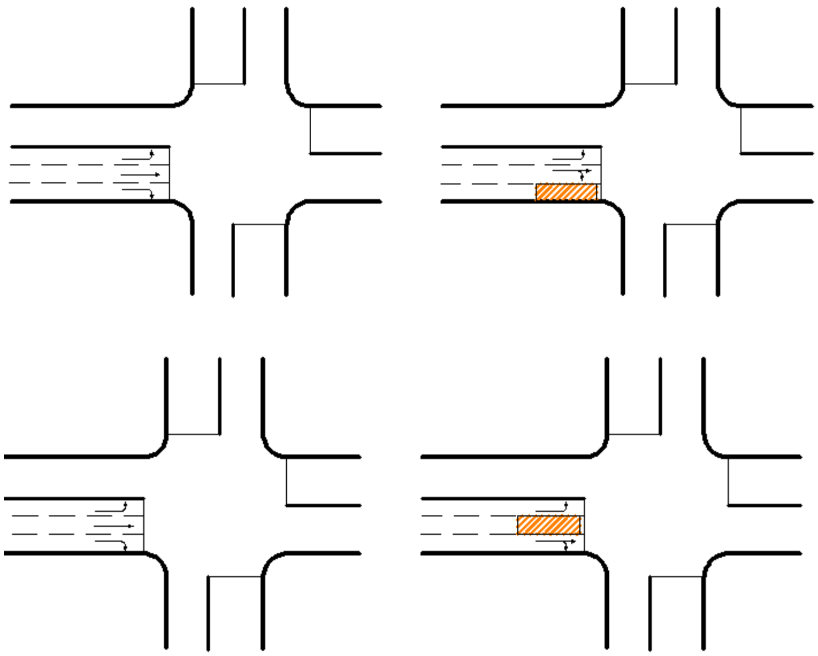
State	Conditions Warranting Right-Turn Lane off Major (Through Highway)		
	Through Volume	Right-Turn Volume	Highway Conditions
Alaska	N/A	DHV = 25 vph	
Idaho	DHV = 200 vph	DHV = 5 vph	2 lanes
Michigan	N/A	ADT = 600 vpd	2 lanes
Minnesota	ADT = 1,500 vpd	All	Design speed > 70 km/h (45 mph)
Utah	DHV = 300 vph	Crossroad ADT = 100 vpd	2 lanes
Virginia	DHV = 500	DHV = 40 vph	2 lanes
	All	DHV = 120 vph	Design speed > 70 km/h (45 mph)
	DHV = 1,200 vph	DHV = 40 vph	4 lanes
	All	DHV = 90 vph	
West Virginia	DHV = 500 vph	DHV = 250 vph	Divided highways
Wisconsin	ADT = 2,500 vpd	Crossroad ADT = 1,000 vpd	2 lanes

Notes: DHV = design hourly volume; ADT = average daily traffic; vph = vehicles per hour; vpd = vehicles per day

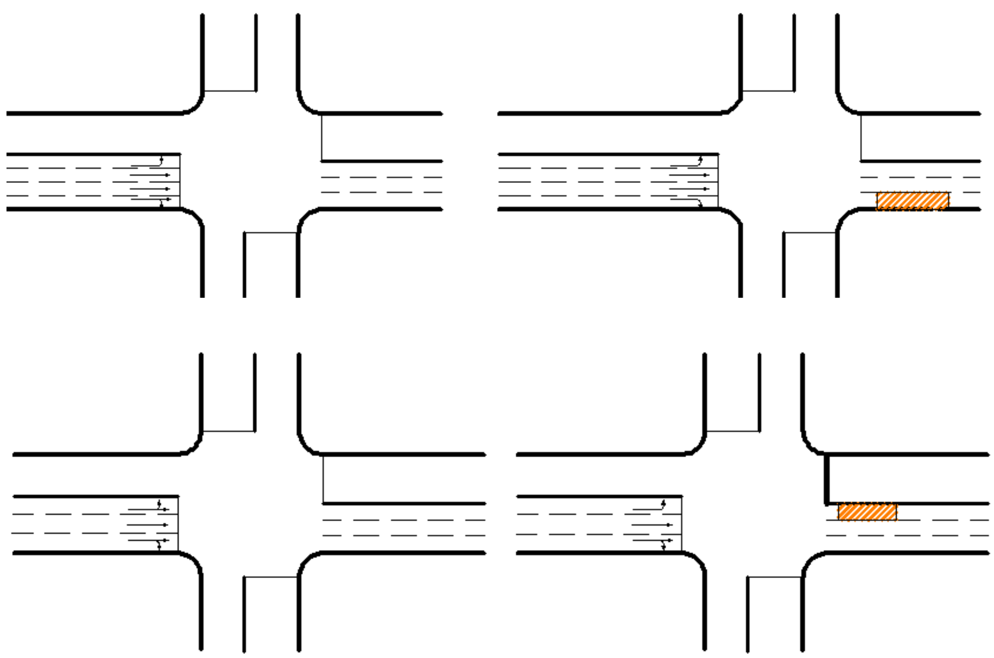
Based on the channelization results of the lane closure approach, this lane closure case can be further divided into two subcases: a) lane closure at the stop bar causes change in the channelization type of the remaining lanes; b) lane closure at the stop bar reduces the overall number of lanes but there are no changes in channelization. The former can be further divided into three configurations based on the treatment of the turning movements: turning vehicles have to share a lane with through vehicles; the original shared lane becomes an exclusive turn lane; the lane closure results in one lane at the approach. These subcases are illustrated in Figure 3.8 and Figure 3.9.

After the rechannelization, the travel pattern of the approach might change. Even if the channelization does not change, the per lane volume will be different after the lane closure. Therefore, the next step is to retime the traffic signals in order to optimize the intersection operation by reallocating green time.

The IDOT Bureau of Design and Environment Manual (2011) and the FHWA guide (2005) indicated that adding or deleting signal phases may be required for changes in travel patterns. A guide for work zone analysis by Wisconsin DOT (2009) indicated that sometimes it is desirable to increase the cycle length to compensate for the situation that the left, through, and right turning vehicles are sharing a single lane after lane closure.

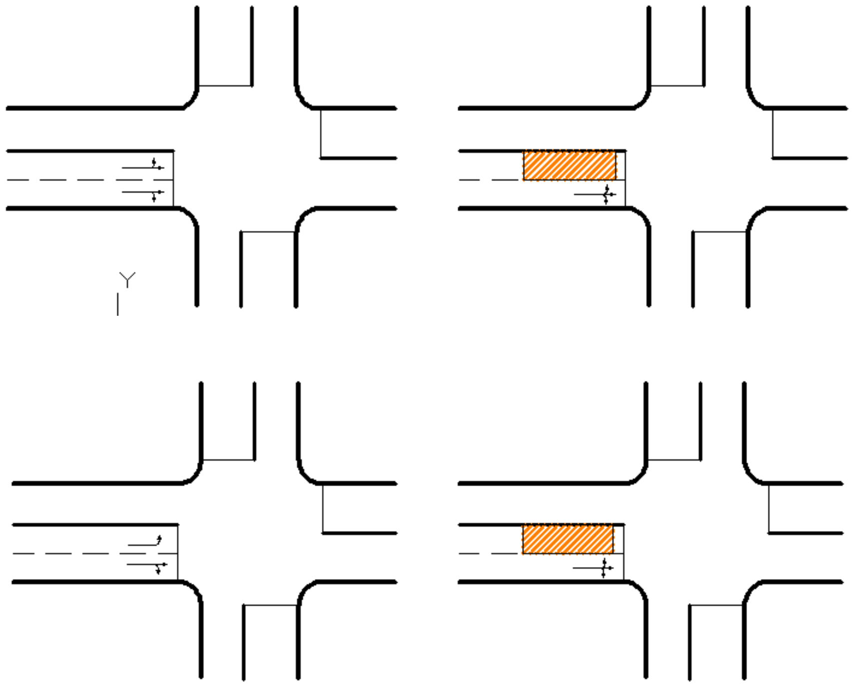


a) Turning Vehicles Have to Share A Lane with Through Vehicles



b) Original Shared lane Becomes An Exclusive Turn Lane

Figure 3.9 Lane Closures at the Stop Bar That Cause Channelization Changes in the Remaining Lanes



c) Lane Closure for a Two Lane Approach

Figure 3.10 Lane Closures at the Stop Bar That Cause Channelization Changes in the Remaining Lanes (cont'd)

For the configurations that do not result in changes in channelization, only the green interval for each phase needs to be adjusted according to the corresponding new per lane volume. However, for the situation in which the lane closure causes changes in channelization, the phasing pattern should also be reconsidered.

The HCM 2000 provides detailed instructions for developing a phase plan and estimating the basic signal timing parameters. For developing a phase plan, the general guideline is that a simple two-phase control should be used unless conditions dictate the need for additional phases. This is because as the number of phases increases, the percentage of lost time in a cycle also increases. HCM also indicates that local policy and practice are critical determinants in the development of the phasing scheme.

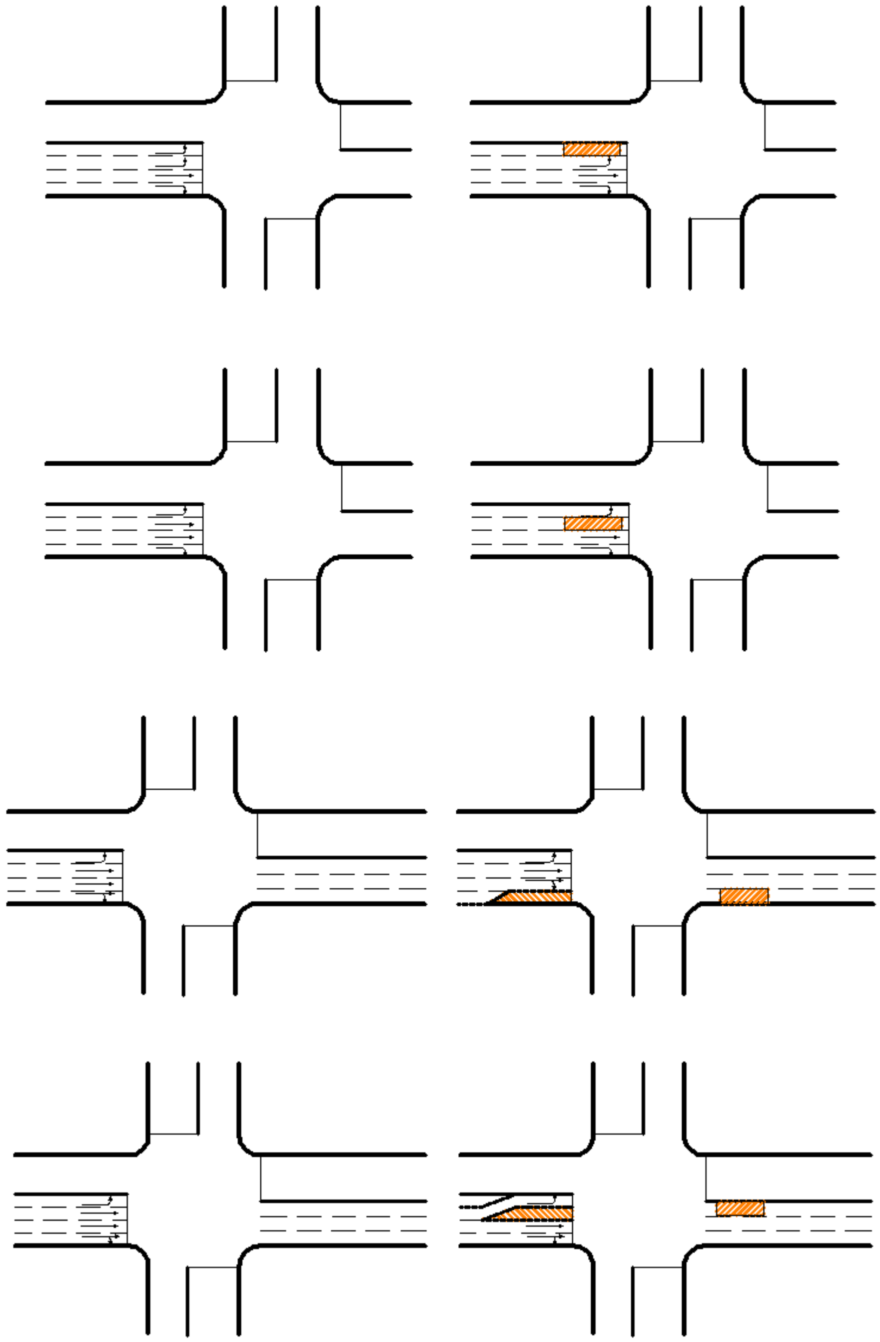


Figure 3.11 Lane Closures at the Stop Bar that Only Cause Reduction in the Number of Lanes

Once a phase plan has been established, signal timing can be estimated. The following two equations in HCM can be used for estimating cycle length and green times for pretimed signals.

$$C = \frac{LX_c}{\left[X_c - \sum_i \left(\frac{v}{s} \right)_{ci} \right]} \tag{13}$$

$$g_i = \frac{v_i C}{s_i X_i} = \left(\frac{v}{s} \right)_i \left(\frac{C}{X_i} \right) \tag{14}$$

where

C : cycle length (s);

L : lost time per cycle (s);

X_c : critical v/c ratio for the intersection;

X_i : v/c ratio for lane group i;

$(v/s)_i$: flow ratio for lane group i;

s_i : saturation flow rate for lane group i; and

g_i : effective green time for lane group i.

For actuated signal timing, the controller has several operating parameters that must be specified for each phase, such as maximum green, intergreen time, minimum phase time, etc. Local practice often plays an important part in determining those values.

More detailed guidelines for signal timing can be found in HCM 2000 (Chapter 16: Signalized Intersection) and in the Traffic Signal Timing Manual (FHWA, 2008). Designers can follow the corresponding procedures.

In summary, when developing a signal timing plan for the case of lane closures at the stop bar, the subject approach should first be considered for rechannelization based on the respective demand of the different movements. The next step is to retime the traffic signals to optimize the intersection operations based on the new channelization and per lane demand.

3.3 Case 3: Lane Closure at Some Distance Downstream from the Subject Intersection

In this case, the work zone area will block some lanes in the middle of an arterial link between two intersections. The difference between this case and the subcase depicted in Figure 3.8-b (subcase of Case 2) is that in this case, there's a relatively long distance between the upstream intersection and the work zone area, therefore, the lane closure will not have any impact on the upstream stop bar channelization. However, queues may spill back into the intersection if the capacity of the work zone area is too low for the demand. The difference between this case and the case depicted in Figure 3.1 (Case 1) is that in this case, the distance between the work zone area and the downstream intersection is relatively long, and the vehicles have already been distributed to their respective lanes before they reach the stop bar, therefore, all the lanes can be efficiently used.

The different configurations included in this case are illustrated in Figure 3.10. There are three key parameters in this case: demand from the upstream intersection ($D_{upstream}$), capacity of the lane closure area ($C_{closure}$) and capacity of the downstream intersection ($C_{downstream}$). The values of the three parameters determine at which area (one of the three areas shown in Figure 3.11) congested conditions may occur and which signal timing would need to be modified. There are three steps for addressing this case. In the first step, the hourly values of these three parameters are compared to ensure that all vehicles can be served by the facility. If either of the two capacity values are less than the hourly demand, queues will occur and will eventually spill back into the upstream intersection. The guidelines developed provide methods to either retime the upstream intersection to re-package the demand or retime the downstream intersection to increase the capacity of that movement. In the second step, demand and capacity are compared for each phase (phases associated with the three highlighted upstream movements as shown in Figure 3.11). Due to the fluctuating vehicle arrivals caused by the upstream signal, spillback queues are still possible to appear during some phases, even if the hourly demand is less than capacity. The analysis in the second step helps prevent this from happening. The third step is optional and it addresses scenarios where the first two steps cannot result in undersaturated conditions. The three steps of the proposed analysis method for this case are described in more detail in the following paragraphs.

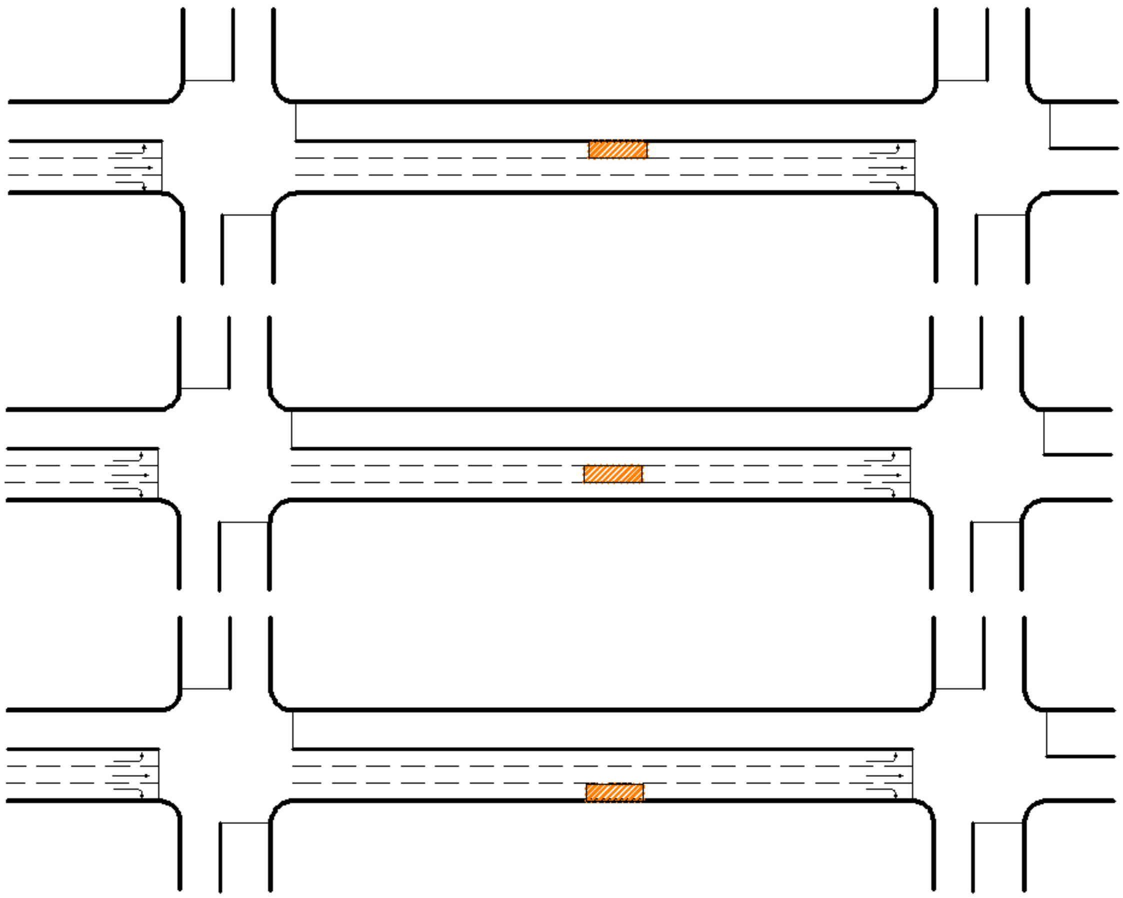


Figure 3.12 Configurations for the Case of Lane Closure between Intersections

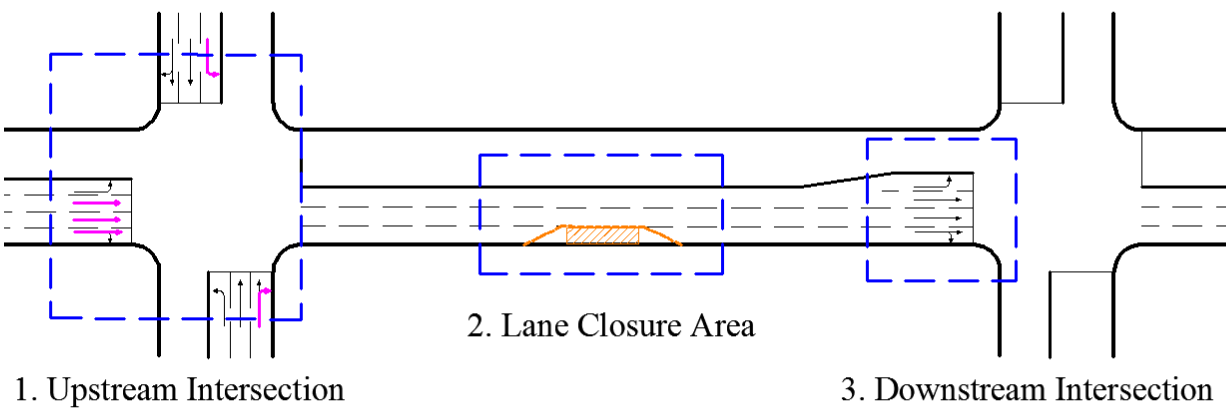


Figure 3.13 Three Important Analysis Areas for Case 3

Step 1. Compare $D_{upstream}$, $C_{closure}$ and $C_{downstream}$ on hourly basis.

In this step, the demand and capacity should be compared on an hourly basis (the units of the parameters are veh/h) to ensure that all the vehicles from the upstream intersection can be served. The values of the three parameters have to satisfy the following constraints:

1) $D^h_{upstream} \le C^h_{closure}$;

2) $D^h_{upstream} \le C^h_{downstream}$.

where

$D^h_{upstream}$: hourly demand from the upstream intersection (veh/h), it consists of the demand of the three movements at the upstream intersection as shown in Figure 3.11;

$C^h_{closure}$: hourly capacity of the lane closure area (veh/h);

$C^h_{downstream}$: demand from the upstream intersection (veh/h).

If $D^h_{upstream} > C^h_{closure}$, the capacity of the lane closure area cannot meet the demand, resulting in queues building up and spilling back into the upstream intersection. In this case, a smaller g/C ratio of the corresponding upstream phases should be implemented to reduce the number of vehicles that enter the work zone area. The maximum g/C ratio can be calculated using the following equation.

$$(g/C)_{\max,upstream} = (g/C)_{existing,upstream} \times \frac{C^h_{closure}}{D^h_{upstream}} \tag{15}$$

$(g/C)_{max,\ upstream}$: maximum g/C ratio of the upstream intersection;

$(g/C)_{existing,\ upstream}$: existing g/C ratio of the upstream intersection;

Other variables are as previously defined.

If the g/C ratio cannot be reduced to the value calculated from the equation above, the analyst should consult Step 3 of the method. Alternatively, the construction work should be rescheduled for another time period when $D^h_{uptream}$, is less than $C^h_{closure}$.

If $D^h_{upstream} > C^h_{downstream}$, then not all vehicles can be served at the downstream intersection. Queues will keep building and finally spill back into the upstream intersection. There are two ways to solve this problem. The first one is to reduce the upstream demand by using a smaller g/C ratio of the corresponding upstream phases. The second way is to increase the downstream capacity by increasing the g/c ratio of the downstream intersection.

If the first method is used, similar to Equation 15, the maximum g/C ratio of the upstream intersection can be calculated using the following equation.

$$(g/C)_{\max,upstream} = (g/C)_{existing,upstream} \times \frac{C^h_{downstream}}{D^h_{upstream}} \tag{16}$$

All variables are as previously defined.

If the upstream demand cannot be reduced, increasing the downstream capacity is another way to satisfy the constraints. Capacity of the downstream intersection can be calculated by the following equation.

$$C^h_{downstream} = s_{downstream} \times g/C \tag{17}$$

where

$S_{downstream}$: saturation flow rate of the studied approach at downstream intersection (veh/h);

g : length of the green interval (sec);

C: cycle length of the downstream intersection (sec).

Therefore, the minimum g/C ratio of the downstream intersection should be:

$$(g/C)_{\min,downstream} = D^h_{upstream} / s_{downstream} \tag{18}$$

$(g/C)_{min,\ downstream}$: minimum g/C ratio of the downstream intersection;

All other variables as previously defined.

If the *g/C* ratio cannot be adjusted to the value calculated from Equation 16 or Equation 18, the analyst should consult Step 3. Alternatively, it is suggested that the construction work should be rescheduled for a less congested time period when $D^h_{upstream}$ is less than $C^h_{downstream}$.

Step 2. Compare $D_{upstream}$, $C_{closure}$ and $C_{downstream}$ on a per phase basis.

The objective of step 1 is to ensure the upstream hourly demand will not exceed the capacity of the downstream sections. However, satisfying the constraints in step 1 only suggests that the vehicles can all be served on an hourly basis. Due to the vehicle arrivals fluctuation, queues may still appear in each phase, and it is possible to spill back into the upstream intersection. In order to avoid the occurrence of the spillback queues, the per phase demand and capacity should be compared in this step. The analysis procedure discussed below should be conducted for each of the three upstream phases (left-turning, through, and right-turning).

The three parameters analyzed in this step are the per phase demand from the upstream intersection ($D^p_{upstream}$, veh/phase), per phase capacity of the lane closure area ($C^p_{closure}$, veh/phase) and the number of upstream vehicles that can be absorbed by the downstream intersection per phase ($N^p_{downstream}$, veh/phase). The per phase capacity of the lane closure area, which represents the number of vehicles that can be discharged from the lane closure area during each phase, can be calculated using the following equation:

$$C^p_{closure} = DR_{sat} \times g_{upstream} \tag{19}$$

where

DR_{sat}: saturation discharge rate of the lane closure area (veh/sec);
$g_{upstream}$: green time of the studied phase at the upstream intersection(sec).

The parameter $N^p_{downstream}$ represents the number of upstream vehicles that can be discharged by the downstream intersection without a stop at the downstream stop bar:

$$N^p_{downstream} = P_{discharge} \times D^p_{upstream} \tag{20}$$

where

$P_{discharge}$: percentage of upstream vehicles that can be discharged by the downstream intersection without a stop at the downstream stop bar;

$D^{p}_{upstream}$: per phase demand from the upstream intersection (veh/phase).

The calculation for the parameter $P_{discharge}$ depends on the coordination of the two intersections. If the two intersections are coordinated, $P_{discharge}$ can be calculated using Equation 21.

$$P_{discharge} = \frac{\left(\min(g_{upstream}, g_{downstream}) - \max(offset - \frac{L}{v_{avg}}, 0)\right) \times s_{downstream}}{g_{upstream} \times s_{upstream}} \tag{21}$$

where

$g_{upstream}$: green time of the subject phase at the upstream intersection (sec);

$g_{downstream}$: green time of the corresponding downstream phase (sec);

offset: offset of the downstream intersection relative to the upstream intersection (sec);

L : distance from the upstream stop bar to the end of downstream queue (ft);

v_{avg} : vehicles' average speed (ft/sec);

$S_{downstream}$: saturation flow rate of the subject approach at downstream intersection (veh/sec);

$S_{upstream}$: saturation flow rate of the subject approach at upstream intersection (veh/sec).

If the two intersections are not coordinated, the worst case should be assumed. Therefore, we assume no upstream vehicle can be discharged by the downstream intersection without a stop and $P_{discharge}$ equals to 0 in this case.

In order to avoid the occurrence of spillback queues, the following constraints should be satisfied for the parameters calculated using the equations above.

1) If $D^{p}_{upstream} \leq C^{p}_{closure}$, $D^{p}_{upstream} \leq N^{p}_{downstream} + C_{storage,downstream}$;

2) If $D^p_{upstream} > C^p_{closure}$, $D^p_{upstream} \le C^p_{closure} + C_{storage,closure}$.

where

$C_{storage,downstream}$:storage capacity of the link between the two intersections (veh);
$C_{storage,closure}$: storage capacity of the section upstream of the lane closure area (veh);
Other variables as previously defined.

If $D^p_{upstream} \le C^p_{closure}$, all the vehicles from the upstream intersection can be absorbed by the lane blockage section. The lane closure does not have much impact on traffic. $D^p_{upstream}$ should be compared with the summation of $N^p_{downstream}$ and the link storage capacity ($C_{storage,downstream}$). If $D^p_{upstream} > N^p_{downstream} + C_{storage,downstream}$, queues at the downstream stop bar will spill back into the upstream intersection. In this case, the offset needs to be adjusted to increase $N^p_{downstream}$. Alternatively, $D^p_{upstream}$ should be reduced by reducing the cycle length. However, since $D^p_{upstream} \le C^p_{closure}$, the demand is relatively low in this case. Also, the arterial links are usually long enough to store the vehicles, therefore, this situation is not very common.

If $D^p_{upstream} > C^p_{closure}$, there are more vehicles than that can be discharged from the work zone area in one phase. In this situation, queues will form at the lane closure area. In order to avoid spillback, $D^p_{upstream}$ should not exceed the summation of $C^p_{closure}$ and $C_{storage,closure}$ (storage capacity of the section upstream of the lane closure area). Using the calculation for $D^p_{upstream}$ and $C^p_{closure}$, the constraint can be written as follows:

$$\frac{D^h_{upstream}}{3600 / C_{upstream}} \le DR_{sat} \times g_{upstream} + C_{storage,closure} \tag{22}$$

where

$D^h_{upstream}$: hourly upstream demand for the subject phase (veh/h);

$C_{upstream}$: cycle length of the upstream intersection (sec).

Other variables as previously defined.

The cycle length can be written as:

$$C = \frac{g}{g/C} \tag{23}$$

Using the above equation of cycle length, Equation 22 can be written as:

$$g_{upstream} \leq \frac{C_{storage,closure}}{\left(\dfrac{D^{h}_{upstream}}{3600 \times (g/C)_{upstream}} - DR_{sat} \right)} \tag{24}$$

All variables as previously defined.

In equation 24, if we assume that the g/C ratio does not change, only the value of $g_{upstream}$ is unknown. Therefore, if $D^{p}_{upstream} > C^{p}_{closure} + C_{storage,closure}$, the phase length of the upstream intersection should be reduced based on Equation 24.

<u>Step 3 (optional). Further analysis for severely congested condition.</u>

The steps discussed above involve basic analysis for a general case when a lane closure is installed along an arterial link between two signalized intersections. The proposed methodology can accommodate some of the spillback that may occur. In cases when the recommended modifications cannot adequately address the expected operational conditions, it is possible that a full signal optimization would be able to address those. A full signal optimization can adjust the offset between intersection so that the queues can be better managed. However, in cases of severe congestion, it may be that there are no signalization improvements that can alleviate congestion when the work zone is installed. In those cases, it may be preferable to schedule the work zone for another time period, if feasible.

4 DATA COLLECTION

This chapter describes the data collection that was undertaken for this project. The field data were collected so that we could accurately replicate operations at both the no-work-zone scenarios as well as the work-zone configurations in simulation. Once these configurations were modeled in simulation, we then tested the effectiveness of the guidelines developed and described in Chapter 3.

Traffic data were collected at two intersections with and without work zones, along Archer Road, in Gainesville, Florida. Sketches of these two intersections are provided in Figures 4.1 and 4.2. The intersection at Archer Rd. and SW 75th St. is to the west of the intersection at Archer Rd. and SW 63rd Blvd.

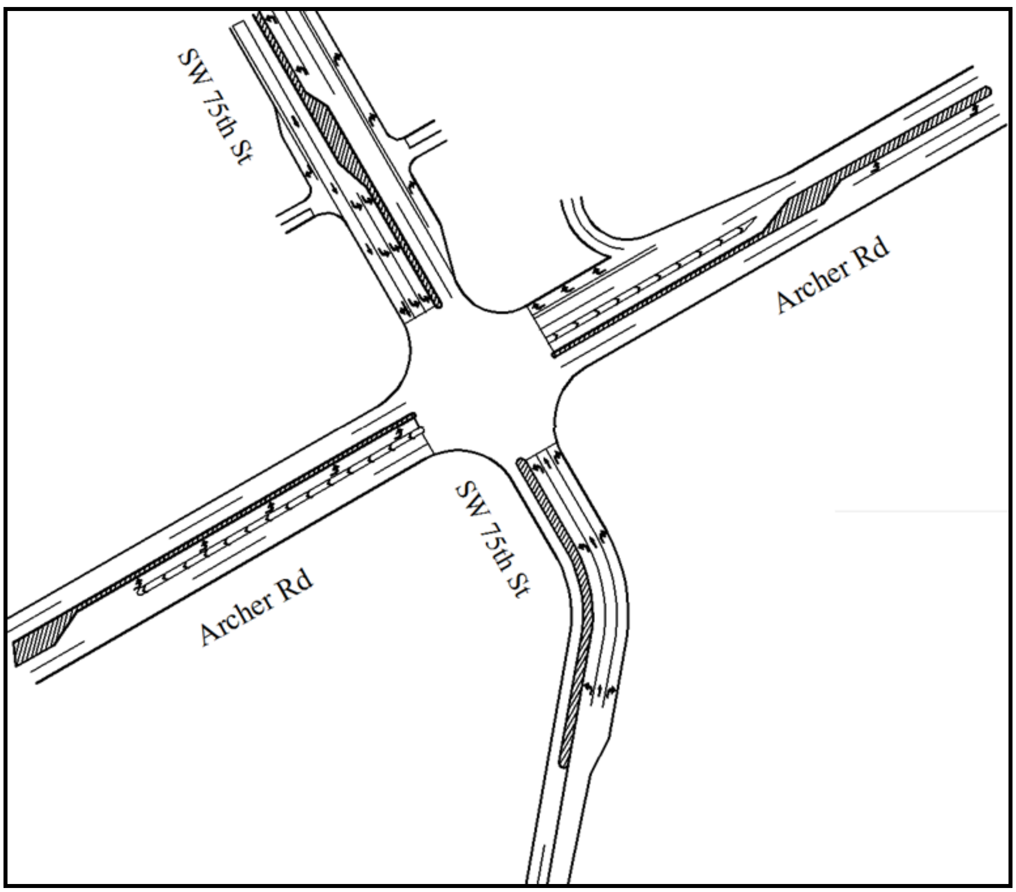


Figure 4.1 Sketch of the Intersection at Archer Rd and SW 75th St

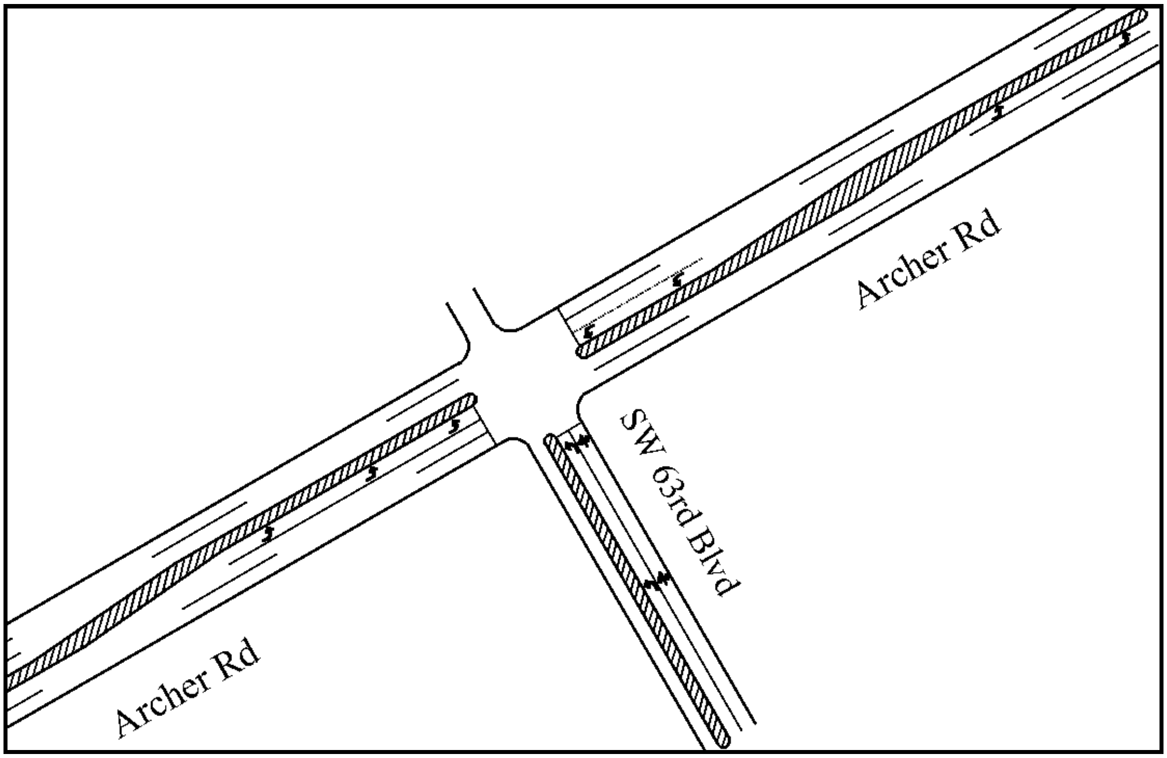


Figure 4.2 Sketch of the Intersection at Archer Rd and SW 63rd Blvd

Existing signal timing schemes for those two intersections were obtained from the Traffic Operations Division of the City of Gainesville, Florida. Both intersections are fully actuated and do not coordinate with each other. All volume and queuing data were collected at 15-minute intervals.

4.1 Data Collection for the Intersection of Archer Rd. at SW 75th St.

The first intersection studied is Archer Rd. at SW 75th St in Gainesville, Florida. The subject approaches are the eastbound (a two-lane approach with a left-turn bay at the stop bar) and westbound (a two-lane approach with a left-turn bay and a right-turn bay at the stop bar) approaches of Archer Rd. The following data were collected at this intersection:

1. Queuing data (for eastbound and westbound approach) and volume data (for all approaches) before the work zone was installed. These data were collected through a camera installed at the intersection by the City of Gainesville. The data were collected on typical weekdays (with clear weather, no incidents) from 8:00 pm to 10:00 pm in May and June, 2012.

2. Queuing and volume data at the westbound approach while a lane closure was present. Figure 4.3 provides a sketch of the lane closure configuration. This configuration corresponds to Case 2 (Lane Closure at the Stop Bar) presented in Chapter 3 of this report. Queuing data for this case were collected using a camera installed by the research team at the upstream end of the lane closure. Volume data (turning movements) were also collected by a camera installed at the intersection by the research team , as the City of Gainesville equipment was disconnected during construction. The data collection was conducted on April 26th, 2012, from 8:30 pm to 10:00 pm.

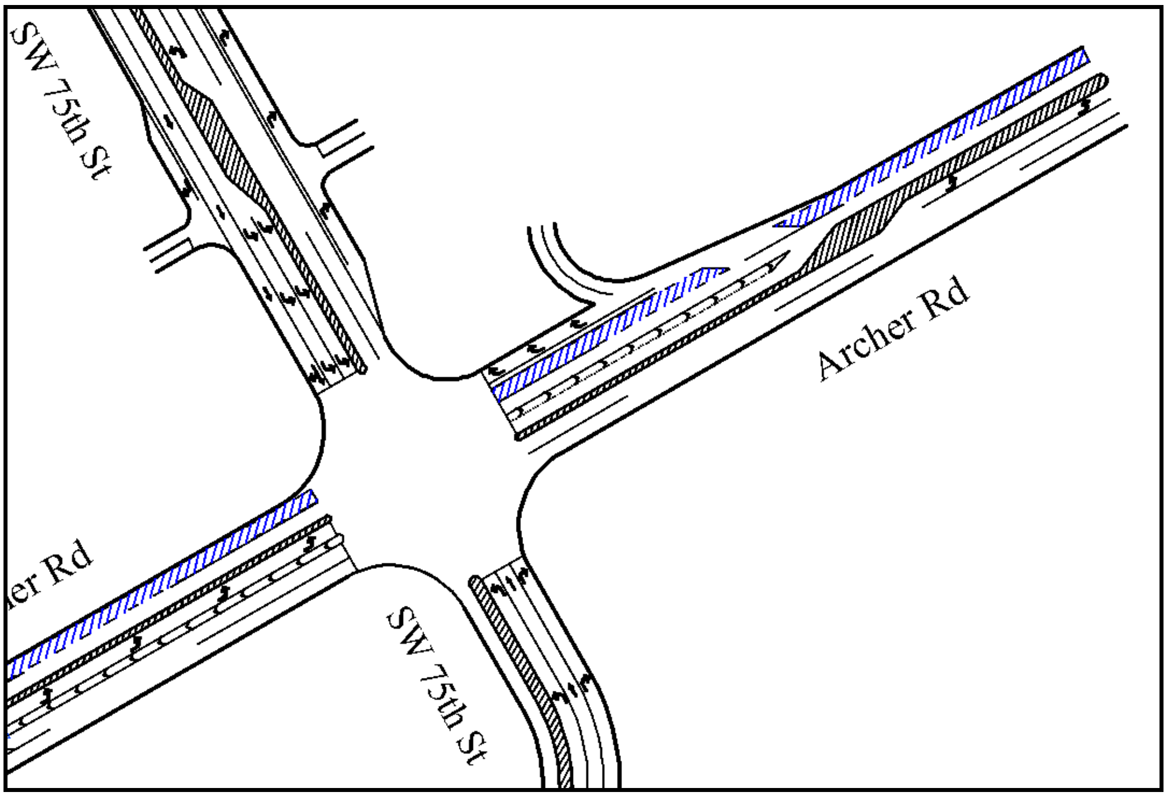


Figure 4.3 Sketch of the Lane Closure in Westbound Archer Rd at the Intersection of Archer Rd and SW 75th St

3. Queuing data (for the eastbound approach) and volume data (for all approaches) of the intersection during a lane closure along its eastbound approach. Figure 4.4 shows a sketch of this lane closure configuration. This is also a Case 2 lane closure type. Similar to the case shown in Figure 4.3, queuing data for this case were collected using a camera at the upstream end of the lane closure, and volume data were collected by a camera at the intersection. Data were collected from 8:30 pm to 10:00 pm on March 28th, 2012.

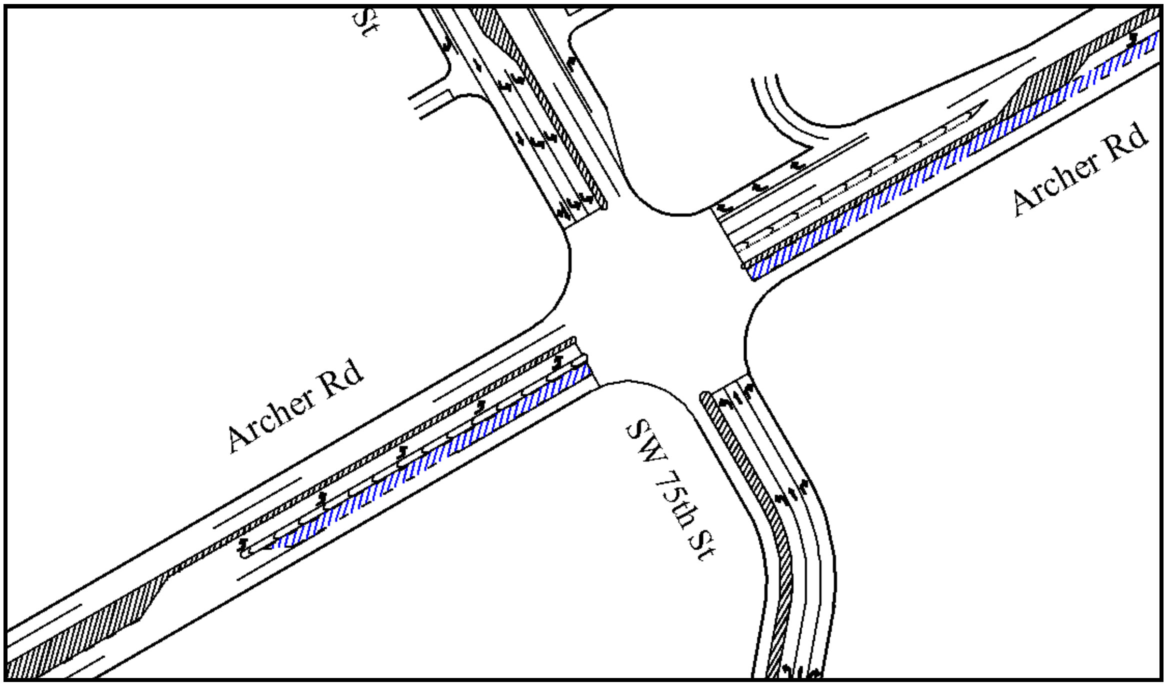


Figure 4.4 Sketch of the Lane Closure in Eastbound Archer Rd at the Intersection of Archer Rd and SW 75th St

4.2 Data Collection for the Intersection of Archer Rd. at SW 63rd St.

The second intersection is Archer Rd. at SW 63rd St. in Gainesville, Florida. The subject approach is the westbound one along Archer Rd. as shown in Figure 4.2 (a two-lane approach with a left-turn bay at the stop bar). The intersection has three major approaches while the southbound approach is a driveway. The volume originating from this driveway is very low and the phase serving this approach was usually skipped during the data collection. Therefore, this intersection can be treated as a T-intersection for the majority of the analysis period. The following data were collected at this intersection:

1. Queuing data (for the westbound approach) and volume data (for all approaches) before the work zone was installed. Queuing data were collected through a camera at the upstream end of the westbound queue. Volume data were collected using a camera at the intersection. The data were collected on June 20th, 2012, a typical weekday with clear weather and no incidents, from 8:00 pm to 10:00 pm.

2. Queuing and volume data of the westbound approach during a lane closure of the

shoulder lane. Figure 4.5 shows a sketch of this lane closure configuration. This is also a Case 2 lane closure. Queuing data were collected using a camera at the upstream end of the lane closure, and volume data were collected by a camera at the intersection. The data collection was conducted on April 26th, 2012 from 8:30 pm to 10:00 pm.

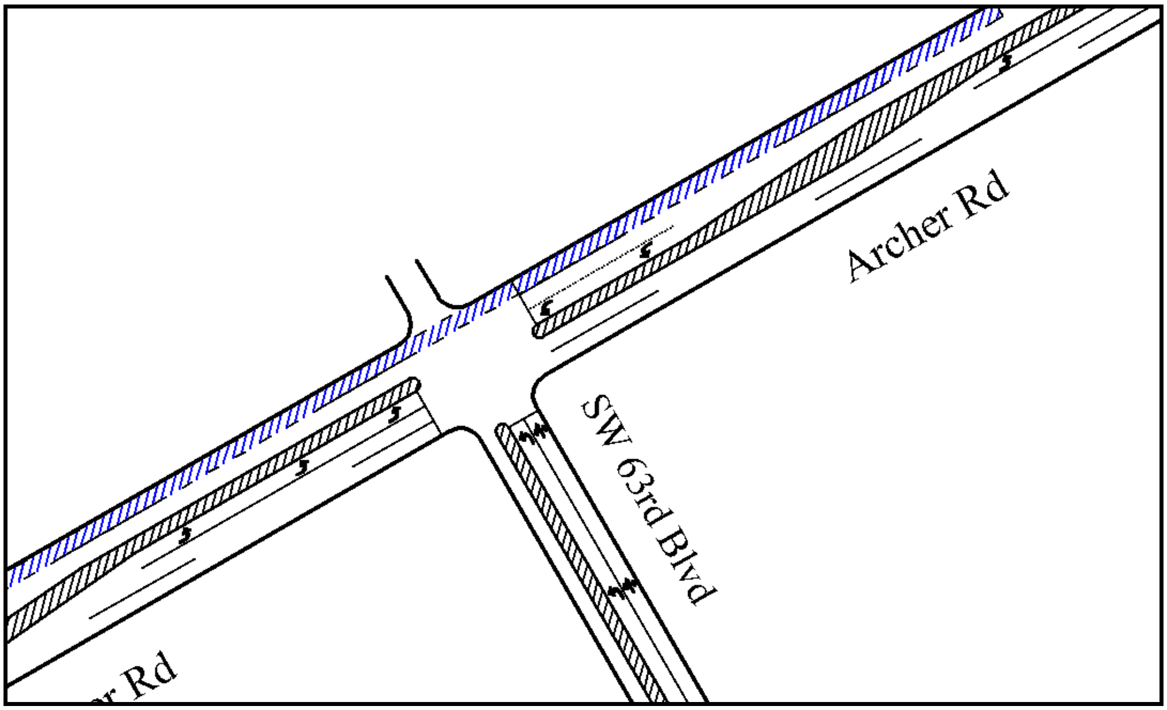


Figure 4.5 Sketch of the Lane Closure in Westbound Archer Rd at the Intersection of Archer Rd and SW 63rd Blvd

5 SIMULATION ANALYSIS OF THE PROPOSED GUIDELINES

This chapter presents the testing of the proposed guidelines using the data collected at the two intersections described in Chapter 4. The CORSIM^{TM} microsimulator was used to test the proposed guidelines. The intersections were first simulated and calibrated in CORSIM to ensure the simulator can adequately replicate the existing field conditions both with and without work zones. Next, several types of work zones were simulated at the two sites and the proposed signalization schemes under the guidelines of Chapter 3 were implemented to evaluate their effectiveness. The first subsection summarizes the calibration effort, while the second one describes the implementation and testing of the guidelines for each of the three work zone cases.

5.1 Model Calibration

The two sites were replicated in CORSIM, with and without the work zones installed. To perform the calibration, observed queuing data from each site were compared to the simulation results for every 15-minute-period. All the simulated values shown in the following sections are averages of 10 runs.

5.1.1 Calibration of the Two Sites without Work Zones

Table 5.1 and Table 5.2 show the calibration results of this scenario for the westbound and eastbound approaches of the first intersection, while Table 5.3 shows the calibration results for the westbound approach of the second intersection. The results are reported for these approaches because these are the ones where operations with work zones were also observed. As shown in these tables, for both sites, the simulated queuing values are similar to the field values.

Table 5.1 Calibration for the Westbound Approach at Archer Rd and SW 75th St. (Without Lane Closure)

Queue Length	Field Data (TH)			Simulation Results		Field Data (LT)			Simulation Results	
	Max Vehicles in Queue	Min Vehicles in Queue	Average of the Max Queue	Max Vehicles in Queue	Average of the Max Queue	Max Vehicles in Queue	Min Vehicles in Queue	Average of the Max Queue	Max Vehicles in Queue	Average of the Max Queue
20:00–20:15	6	3	6.10	7	4.07	2	0	1.20	3	1.77
20:15–20:30	6	1	3.67	6	3.79	4	0	1.86	4	2.45
20:30–20:45	8	1	6.10	7	4.08	3	0	1.63	4	2.69
20:45–21:00	6	0	5.10	7	3.98	2	0	1.38	4	2.25
21:00–21:15	5	1	3.67	6	3.79	3	0	1.80	3	1.87
21:15–21:30	4	1	3.67	5	3.29	3	0	1.38	3	1.79
21:30–21:45	3	0	1.75	5	2.95	2	0	1.14	2	1.62
21:45–22:00	2	0	1.92	5	2.88	1	0	1.00	2	1.53

Notes: TH=through movement; LT= left turning movement.

Table 5.2 Calibration for the Eastbound Approach of Archer Rd and SW 75th St. (Without Lane Closure)

Queue Length	Field Data (TH)			Simulation Results		Field Data (LT)			Simulation Results	
	Max Vehicles in Queue	Min Vehicles in Queue	Average of the Max Queue	Max Vehicles in Queue	Average of the Max Queue	Max Vehicles in Queue	Min Vehicles in Queue	Average of the Max Queue	Max Vehicles in Queue	Average of the Max Queue
20:00–20:15	5	1	2.90	4	2.09	2	1	1.38	4	1.67
20:15–20:30	4	1	2.75	4	2.20	3	1	1.50	5	1.89
20:30–20:45	6	1	3.25	4	2.14	2	1	1.42	5	2.05
20:45–21:00	5	1	2.83	5	2.11	2	1	1.38	3	1.42
21:00–21:15	6	1	3.00	4	2.10	2	1	1.43	2	1.4
21:15–21:30	4	1	2.17	4	2.10	2	1	1.20	3	1.21
21:30–21:45	4	1	2.43	2	2.00	1	1	1.00	4	1.28
21:45–22:00	5	1	1.83	2	2.00	1	1	1.00	3	1.25

Table 5.3 Calibration for the Westbound Approach at Archer Rd and SW 63rd Blvd. (Without Lane Closure)

Queue Length	Field Data (WB, TH)			Simulation Results	
	Maximum Vehicles in Queue	Minimum Vehicles in Queue	Average of the Max Queue	Maximum Vehicles in Queue	Average of the Max Queue
20:00–20:15	16	1	6.00	12	4.63
20:15–20:30	13	1	4.83	12	3.65
20:30–20:45	14	1	4.67	9	3.73
20:45–21:00	9	1	4.00	11	3.51
21:00–21:15	10	1	4.11	10	3.55
21:15–21:30	7	1	3.30	10	3.06
21:30–21:45	6	1	2.80	13	2.87
21:45–22:00	8	1	2.90	9	3.1

Notes: WB=westbound approach; LT= left turning movement.

5.1.2 Calibration of the Two Sites with the Work Zones

Three different work zone scenarios at the two intersections were simulated: lane closure in the westbound approach of Archer Rd and SW 75th St, lane closure in the westbound approach of Archer Rd and SW 63rd Blvd, and lane closure in the eastbound approach of Archer Rd. and SW 75th St.

For the first two lane closure scenarios, the calibration results are shown in Table 5.4 and Table 5.5. Based on these two tables, the simulated queuing values are similar to the field values for the intersection at SW 75th St. The results for the third work zone are shown in Table 5.6. For this scenario the simulated queue lengths are much less than the field data. This is caused by the manually controlled extended red time, which allows the paving crew to set up the equipment in the field, at the beginning of the construction. Therefore, for this intersection further tests will be conducted based on the calibrated no-work-zone simulation model.

Table 5.4 Calibration for the Westbound Approach of Archer Rd and SW 75th St. (Lane Closure in the Westbound Approach)

	Field Data			Simulation Results	
	Maximum Vehicles in Queue	Minimum Vehicles in Queue	Average of the Max Queue	Maximum Vehicles in Queue	Average of the Max Queue
8:30–8:45	16	3	9.75	17	9.03
8:45–9:00	17	1	6.00	13	7.37
9:00–9:15	17	1	7.73	17	8.91
9:15–9:30	17	0	5.46	14	8.4
9:30–9:45	13	1	4.17	13	7.36
9:45–10:00	10	0	2.13	10	5.17

Table 5.5 Calibration for the Eastbound Approach of Archer Rd and SW 75th St. (Lane Closure in the Eastbound Approach)

Queue Length	Field Data			Simulation Results	
	Maximum Vehicles in Queue	Minimum Vehicles in Queue	Average of the Max Queue	Maximum Vehicles in Queue	Average of the Max Queue
8:30–8:45	4	1	2.25	4	1.73
8:45–9:00	5	1	2.50	4	1.43
9:00–9:15	5	1	1.73	3	1.35
9:15–9:30	4	1	1.18	3	1.21
9:30–9:45	5	1	2.00	3	1.4
9:45–10:00	3	1	1.33	2	1.1

Table 5.6 Calibration for the Westbound Approach of Archer Rd and SW 63rd St. (Lane Closure in the Westbound Approach)

Queue Length	Field Data			Simulation Results	
	Maximum Vehicles in Queue	Minimum Vehicles in Queue	Average of the Max Queue	Maximum Vehicles in Queue	Average of the Max Queue
8:30–8:45	34	22	29	12	9.5
8:45–9:00	39	20	28.25	12	9.3
9:00–9:15	40	17	30.25	18	11.2
9:15–9:30	36	10	17.83	11	7.4
9:30–9:45	16	1	8.43	11	7.3
9:45–10:00	14	1	7.43	7	5.6

5.2 Tests of the Proposed Guidelines

This section presents the tests of the proposed guidelines based on the calibrated simulation model. Due to the stochastic nature of CORSIM™, a relatively large number of runs are required in order to estimate the performance measures with reasonable accuracy. Therefore, performance measures discussed below are all obtained from 10 simulation runs.

5.2.1 Case 1: Lane Closure before the Intersection

As discussed in Chapter 3, the key issue for this case is to maximize the throughput at the stop bar. Therefore, to develop the signal timing plan for the intersections of this case, the maximum green time of the subject approach should be compared with the value calculated by Equation 1. If the proposed maximum green time is greater than the value computed by Equation 1, splitting the phase into two short phases should be considered based on Equation 3 and Equation 4. Work-zone scenarios at two different intersections are tested for this case. The first one is the lane closure in the westbound approach of Archer Rd and SW 75th St, and the second is the lane closure in the westbound approach of Archer Rd and SW 63rd Blvd.

Lane Closure in the Westbound before the Intersection of Archer Rd and SW 75th St

A sketch of the configuration of this scenario is provided in Figure 5.1. As shown in the figure, there are originally two through lanes with a left turn pocket and a right turn pocket in this approach. The lane closure is implemented before the intersection in one through lane, resulting in a configuration of a one through lane approach with a four-lane section at the stop bar. The data collection site did not have such a lane closure configuration during the construction period. Therefore, the calibrated no-work-zone simulation model was used and the lane closure was implemented in the simulation. Volume data for the most congested 15 minutes (8:45pm-9:00pm) were used as the demand for this test.

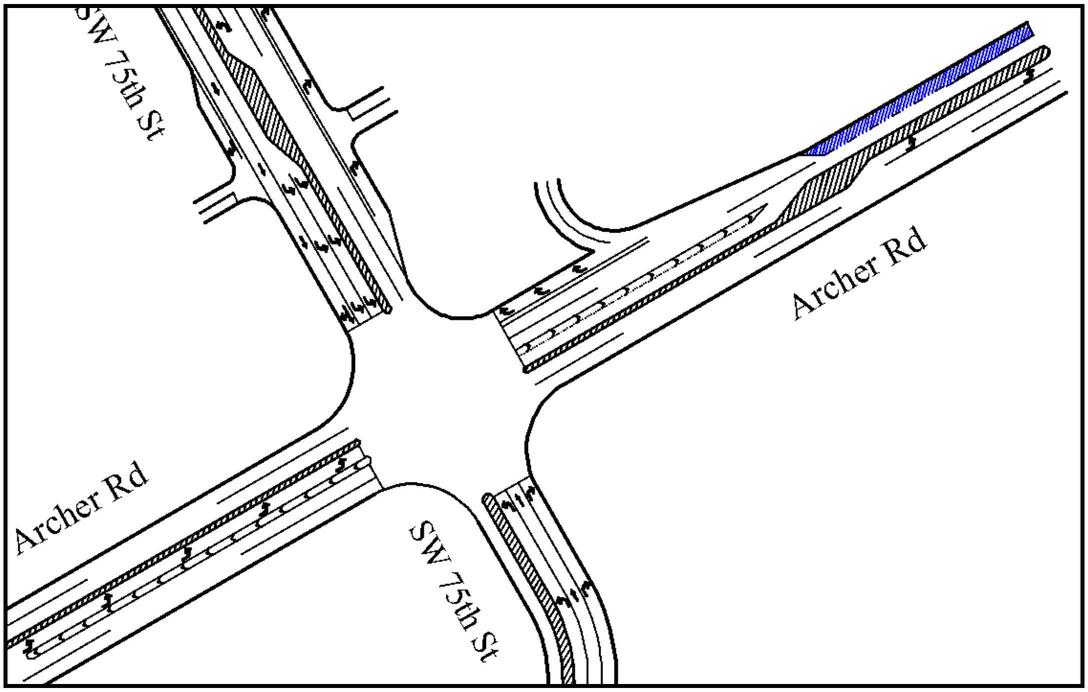


Figure 5.1 Sketch of the Lane Closure in Westbound Archer Rd before the Intersection of Archer Rd and SW 75th St.

Performance measures were first compared between the no-work-zone scenario and the work-zone scenario. The existing actuated signal timing was implemented in both scenarios. Table 5.7 provides the comparison results. From this table it can be inferred that there are very small changes in the control delay and the throughput after implementing the work zone in the simulation model. The weighted average control delay for the entire intersection has only increased by 0.63 seconds. This is because the data were collected under uncongested conditions, with very low demands. Therefore, even if one of the two lanes were closed, the remaining capacity is still able to accommodate the demand and the existing signal timing works well. Therefore, in this low demand situation, there is no need to change the existing signal timing plan.

Table 5.7 Comparison between the Before and After Lane Closure Scenarios Implementing Existing Actuated Signal Timing

a) Performance Measures before Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	30.93	6.68	4.06	25.65	6.38	2.64	17.38	32.34	4.24	23.24	20.55	5.61	10.63
Throughput	9.00	43.90	1.30	19.70	116.00	51.00	1.20	4.10	5.70	45.00	5.20	15.00	

b) Performance Measures after Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	27.12	6.39	2.68	27.35	7.04	2.71	17.86	31.18	4.75	23.83	19.28	7.97	11.26
Throughput	9.80	43.20	1.00	20.50	113.10	51.40	1.20	4.70	5.30	46.80	4.80	13.60	

Notes: EB=eastbound approach; WB=westbound approach; NB=northbound approach; SB=southbound approach;LT= left turning movement; TH=through movement; RT=right turning movement.

In order to test the proposed method in more congested conditions, the volumes for the four approaches were increased manually in the simulation model. In order to simulate heavy demand conditions, 2.8 times the original demand was used as input in the simulation. The resulting performance measures for the before and after lane closure scenario under this demand condition are shown in Table 5.8. The control delay per vehicle for the westbound through movement increased significantly (18.63 sec vs. 29.01 sec) after implementing the lane closure, and the weighted average delay for all approaches of the intersection increased by 5.14 sec (21.0%). In this case, the existing signal timing cannot handle the high demand in the westbound approach, and the proposed method should be implemented to make better use of the green time and reduce congestion.

The proposed method suggests splitting the existing westbound through phase into two short phases. However, although CORSIM™ can simulate actuated signal control, it cannot handle more than 8 phases using dual-ring phasing. The proposed signal timing scheme, which has more than 8 phases after the phase-splitting, cannot be implemented into the simulation model directly. Therefore, an equivalent pretimed signal timing scheme was developed to match the existing actuated one, and the proposed method was then implemented using pretimed control.

Based on Equation 1, the maximum green time for the westbound through approach should not be greater than 30 seconds. However, in the equivalent pretimed signal timing plan, the green time is 50 seconds, which is greater than the theoretical value. Therefore, splitting and repeating the original westbound through phase should be considered.

Table 5.9 shows the performance measures before and after splitting the westbound through phase in the equivalent pretimed simulation model. The control delay per vehicle decreases by 4 seconds (17.1%) and 21.5 seconds (18.3%) for the westbound through and left movement, respectively, after implementing the proposed method. The weighted average delay for the entire intersection decreases by 3.89 seconds. This improvement is consistent with the results calculated using Equation 3 and 4 (55 veh/cycle before implementing the proposed method vs. 67.5 veh/cycle after implementing the proposed method).

Table 5.8 Comparison between the Before and After Lane Closure Scenarios using the Increased Volume Data

a) Performance Measures before Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	50.93	13.55	15.37	97.91	18.63	5.35	48.10	41.67	5.99	36.85	28.53	18.25	24.52
Throughput	28.3	123.5	2.9	53.2	319.6	144.8	3.4	12.0	16.9	130.6	12.9	38.5	

b) Performance Measures after Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	53.73	13.79	9.53	114.98	29.01	5.17	49.77	42.72	5.92	37.12	28.59	18.65	29.66
Throughput	29.9	122.7	2.8	53.9	312.1	141.4	3.5	12.1	16.8	127.9	13.1	37.6	

Table 5.9 Comparison between Before and After Implementing the Proposed Method

a) Performance Measures before Implementing the Proposed Method (Using the Equivalent Pretimed Signal Timing)

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	50.08	14.64	10.34	117.53	29.24	4.44	38.15	44.11	5.26	37.73	35.54	22.06	29.66
Throughput	28.3	122.5	3.0	50.8	310.4	144.4	4.1	11.2	17.1	131.3	13.8	38.7	

b) Performance Measures after Implementing the Proposed Method (Based on the Equivalent Pretimed Signal Timing)

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	43.61	9.08	11.00	96.07	24.25	4.46	37.98	44.45	5.44	37.32	35.67	19.97	25.77
Throughput	29.1	124.0	3.3	55.6	314.5	147.8	4.2	10.8	17.3	130.0	13.0	37.4	

Lane Closure at the Westbound Approach before the Intersection of Archer Rd and SW 75th St

A sketch of the configuration of this scenario is provided in Figure 5.2. The lane closure is implemented at the westbound approach, which has the highest demand. The configuration of this approach is very similar to the one discussed in the previous section. The only difference is that, for this intersection very few vehicles turn right from the westbound approach, and therefore, there is not an exclusive right turn lane at the stop bar. As shown in Figure 5.2, there is only one through lane at the lane closure with a three-lane section at the stop bar. Since the field site did not have such a lane closure configuration during the construction period, the simulation network for the after lane closure scenario was developed based on the calibrated no-work-zone model. Volume data for the most congested 15 minutes (8:00pm-8:15pm) was used as the demand for this case.

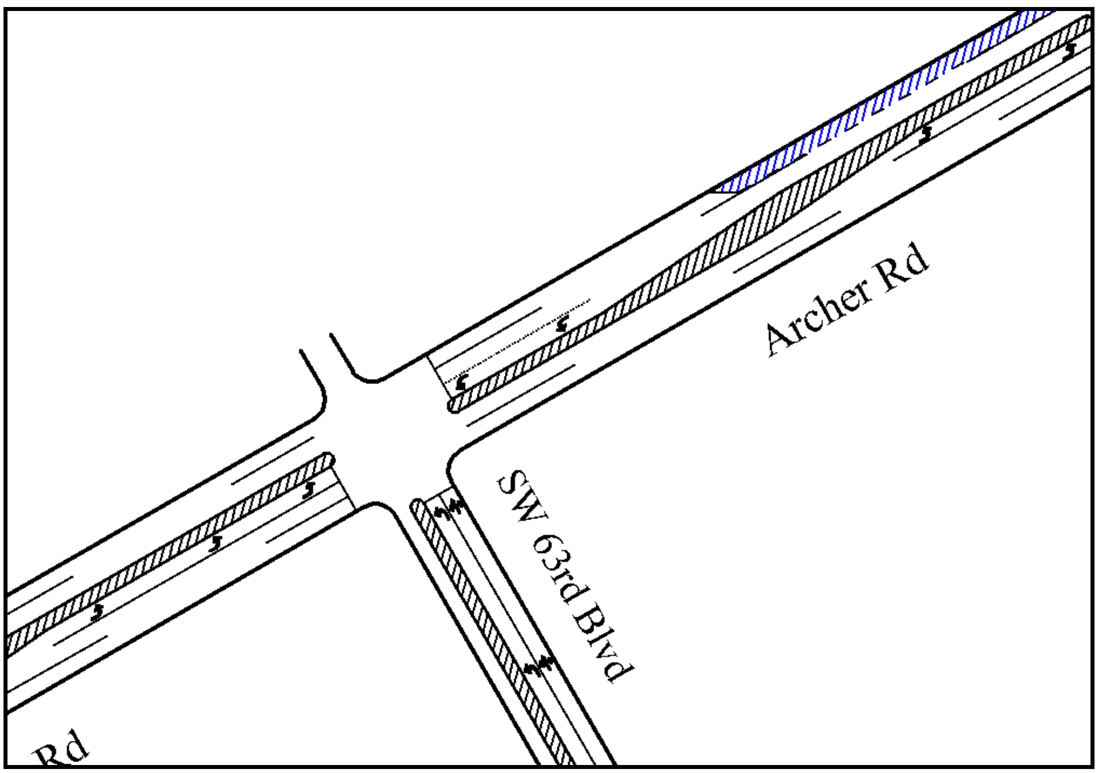


Figure 5.2 Sketch of the Lane Closure in Westbound Archer Rd before the Intersection of Archer Rd and SW 63rd Blvd

The results of the comparison of the performance measures between the no-work-zone scenario and the work-zone scenario are shown in Table 5.10.

Table 5.10 Comparison between the Before and After Lane Closure Scenarios Implementing Existing Actuated Signal Timing

a) Performance Measures before Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	25.59	5.52	6.23	23.16	6.00	-	29.01	-	5.43	27.01	-	-	8.31
Throughput	1.4	91.1	5.0	3.5	170.2	-	27.1	-	5.7	2.0	-	-	

b) Performance Measures after Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	24.03	5.59	6.53	22.43	7.05	-	28.41	-	4.92	31.30	-	-	9.02
Throughput	1.6	89.9	5.7	4.2	169.1	-	28.3	-	5.0	2.0	-	-	

This intersection has three major approaches while the southbound approach is a driveway. Therefore, the volume originating from or approaching this driveway is very low, and the demand was zero for the four movements (westbound right turn, northbound through and southbound through and right turn) during the study time period. For the remaining movements, there are very small changes in the control delay and the throughput after implementing the work zone in the simulation model. The average control delay for the entire intersection only increases by 0.71 second (from 8.31 seconds to 9.02 seconds). Similar to the scenario discussed in the previous section, this small change in performance measures is due to the low westbound demand, which is far less than the capacity of the approach. Therefore, there is no need to change the signal timing under existing demand conditions.

In order to test the proposed method in more congested conditions, the volumes of the four approaches were manually increased in the simulation model. For this intersection, the original demand was increased by a factor of 3.2 in the simulation model to replicate high demands. The resulting performance measures for the before and after lane closure scenario under this demand condition are shown in Table 5.11. The per vehicle control delay for the westbound through movement increases from 15.83 seconds to 26.09 seconds after implementing the lane closure. The weighted average delay for all approaches of the intersection is also significantly increased (from 18.21 seconds to 23.57 seconds). The proposed method was then implemented in the work zone model to alleviate congestion.

Since the proposed timing plan has more than 8 phases which CORSIM™ cannot model, the proposed signal timing scheme cannot be implemented into the simulation model directly. Therefore, to be able to directly compare the before and after scenarios on the same basis, an equivalent pretimed signal timing scheme was developed to match the existing actuated one, and the proposed method was then implemented using pretimed control.

Based on Equation 1, the maximum green time for the westbound through approach should not exceed 24 seconds. However, in the equivalent pretimed signal timing plan the green time for that phase is 50 seconds. Therefore, it is recommended to split the green into two phases. Based on Equations 3 and 4, the calculation results showed that after splitting the phase, the number of vehicles that are discharged from the westbound approach at each cycle increase from 37 veh/cycle to 45 veh/cycle. The cycle length is the same as the existing timing plan. The proposed method was implemented in the simulation model. Table 5.12 shows the performance measures before and after splitting the westbound through phase in the equivalent pretimed simulation model.

Based on the results shown in Table 5.12, the control delay per vehicle is decreased by 4.12 seconds (14.9%) for the westbound through movement, and the weighted average delay for the entire intersection is decreased by 3.47 seconds (15.0%) after implementing the proposed method. This improved result is consistent with the calculations based on Equations 3 and 4.

Table 5.11 Comparison between the Before and After Lane Closure Scenarios using the Increased Volume Data

a) Performance Measures before Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	44.75	11.68	11.69	46.62	15.83	-	46.83	-	9.60	28.80	-	-	18.21
Throughput	6.2	288.6	16.8	18.1	528.6	-	91.8	-	16.9	8.7	-	-	

b) Performance Measures after Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	42.03	11.19	11.25	42.97	26.09	-	48.06	-	10.14	32.15	-	-	23.57
Throughput	6.1	290.5	16.0	18.8	508.4	-	89.7	-	17.3	8.6	-	-	

Table 5.12 Comparison between Before and After Implementing the Proposed Method

a) Performance Measures before Implementing the Proposed Method (Using the Equivalent Pretimed Signal Timing)

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	35.09	10.52	11.00	30.87	27.67	-	40.21	-	10.88	30.67	-	-	23.14
Throughput	5.1	288.4	16.9	19.2	494.8	-	91.2	-	17.8	8.0	-	-	

b) Performance Measures after Implementing the Proposed Method (Based on the Equivalent Pretimed Signal Timing)

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	31.57	6.20	7.73	31.99	23.55	-	42.59	-	9.06	24.13	-	-	19.67
Throughput	5.2	288.5	17.1	18.3	495.1	-	88.5	-	17.5	8.0	-	-	

5.2.2 Case 2: Lane Closure at the Stop Bar

As discussed in Chapter 3, for this case, the subject approach should first be rechannelized based on the respective demand of the different movements. Based on the channelization results of the lane closure approach, the cases included in this category can be further divided into two conditions: 1) lane closure before the stop bar causes changes in channelization; and 2) after the work zone lane closure before the stop bar, only the number of lanes is reduced, and the channelization does not change.

If the lane closure causes a change in channelization, the phasing pattern should be reconsidered. For configurations with no change in channelization only the green interval for each phase needs to be adjusted according to the corresponding new per lane volume.

Work-zone scenarios at two different intersections are simulated and tested for this case. The first one is a lane closure in the westbound through lane of the intersection of Archer Rd and SW 75th St, and the second is a lane closure in the westbound through lane of the intersection of Archer Rd and SW 63rd Blvd.

Lane Closure in the Westbound of Archer Rd and SW 75th St

A sketch of the configuration of this scenario is shown in Figure 5.3. As shown in the figure, one of the two through lanes is closed during the construction period, resulting in a one-through-lane approach for the westbound vehicles. Volume and queuing data were collected for both the no-work-zone and the work-zone scenarios. Simulation models were then developed and calibrated based on those data and the configuration of the intersection. Since both no-work -zone and work-zone scenarios have relatively low demands, the throughput at the stop bar equals the demand of the approach. Therefore, volume data for the most congested 15 minutes (8:30pm-8:45pm) were used as the demand in the simulation model.

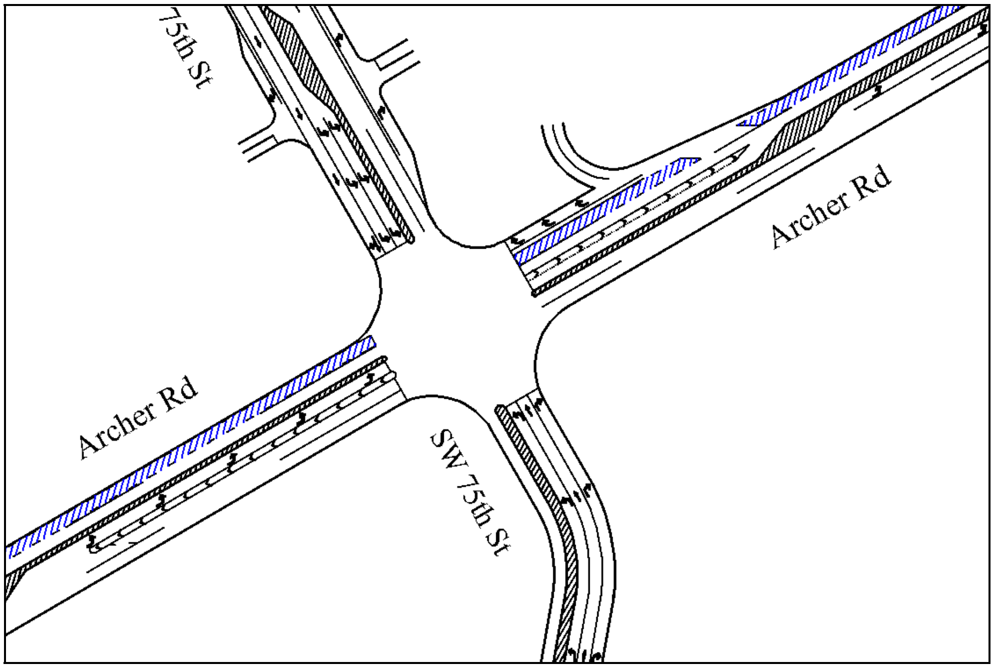


Figure 5.3 Sketch of the Lane Closure in Westbound for the Intersection of Archer Rd and SW 75th St.

Table 5.13 provides the comparison of the performance measures between the no-work-zone scenario and the work-zone scenario for this intersection. It can be noticed that there are very small changes in the control delay and the throughput after implementing the work zone in the simulation model. The average control delay for the westbound through movement only increases by 1.42 seconds (from 7.59 seconds to 9.01 seconds) and the intersection average control delay increases by 1.38 seconds (from 12.42 seconds to 13.80 seconds). This is because the data were collected during low demand conditions. After implementing the lane closure in the approach, the actuated signal control currently in place can handle the reallocation of green time based on the new per lane volume for each movement. Therefore, there is very small increase in the control delay for each movement and the entire intersection, and there is no need to change the signal timing under the existing demand conditions.

Table 5.13 Comparison between the Before and After Lane Closure Scenarios Implementing Existing Signal Timing

a) Performance Measures before Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	29.69	7.87	-	28.34	7.59	4.29	25.66	35.74	4.82	23.80	18.90	6.93	12.42
Throughput	18.4	54.7	-	24.7	105.3	62.6	1.1	6.6	7.0	43.7	9.7	22.0	

b) Performance Measures after Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	29.31	7.89	-	31.74	9.01	8.64	18.63	31.49	4.08	24.45	20.74	7.45	13.80
Throughput	18.9	55.7	-	24.6	107.3	62.0	1.2	6.5	7.0	42.8	9.2	22.6	

In order to test the proposed method in more congested conditions, the volume at the four approaches were manually increased in the simulation model. The simulation model demand was set to be 2.2 times the original demand to replicate a high demand scenario. The resulting performance measures for the before and after lane closure scenario under this demand condition are shown in Table 5.14. The control delay per vehicle for the westbound approach significantly increased after implementing the lane closure. The weighted average delay for the whole intersection increased by 8.16 seconds (38.0%).

The lane closure in the westbound approach reduced its capacity, however, there is no necessity to rechannelize the movements. The existing phasing pattern can remain, and the green interval for each phase was adjusted according to the corresponding new per lane volume. The proposed timing scheme has longer minimum green time for the westbound through movement and shorter maximum green time for the other movements.

Table 5.15 shows the performance measures after implementing the new signal control scheme. A comparison between this table and Table 5.14-b shows that the performance measures for the eastbound and westbound approach are significantly improved, and the weighted average delay for the entire intersection is decreased by 2.09 seconds.

Table 5.14 Comparison between the Before and After Lane Closure Scenarios using the Increased Volume Data

a) Performance Measures before Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	46.53	12.88	-	64.93	15.33	5.80	55.96	38.70	5.31	34.75	27.77	17.53	21.49
Throughput	41.8	124.7	-	52.9	230.1	138.3	3.2	14.4	15.9	96.5	22.4	50.9	

b) Performance Measures after Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	53.16	13.07	-	97.43	26.32	14.90	49.24	39.63	4.53	37.44	32.78	23.53	29.66
Throughput	39.2	128.6	-	54.2	226.6	136.1	3.3	14.4	15.8	93.6	21.6	49.7	

Table 5.15 Performance Measures After Implementing the Proposed Signal Timing Scheme

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	49.62	11.06	-	96.77	22.25	12.01	57.88	45.95	4.82	37.95	31.41	24.03	27.56
Throughput	40.4	126.5	-	53.1	229.0	138.6	3.1	14.9	15.8	94.1	22.5	50.3	

Lane Closure in the Westbound of Archer Rd and SW 75th St

A sketch of this configuration is shown in Figure 5.4. As shown in the figure, after closing one of the two through lanes, there is only one through lane left, as well as a left turn bay at the intersection for the westbound approach. Volume and queuing data were collected for both no-work-zone and work-zone scenarios at this study site. Simulation models were developed and calibrated based on these data and the configuration of the intersection. For this intersection, both the no-work-zone and work-zone scenarios are for low demand conditions. Therefore, the throughput at the stop bar equals the demand of the approach. Volume data for the most congested 15 minutes (8:00pm-8:30pm) were used as the demand in the simulation model.

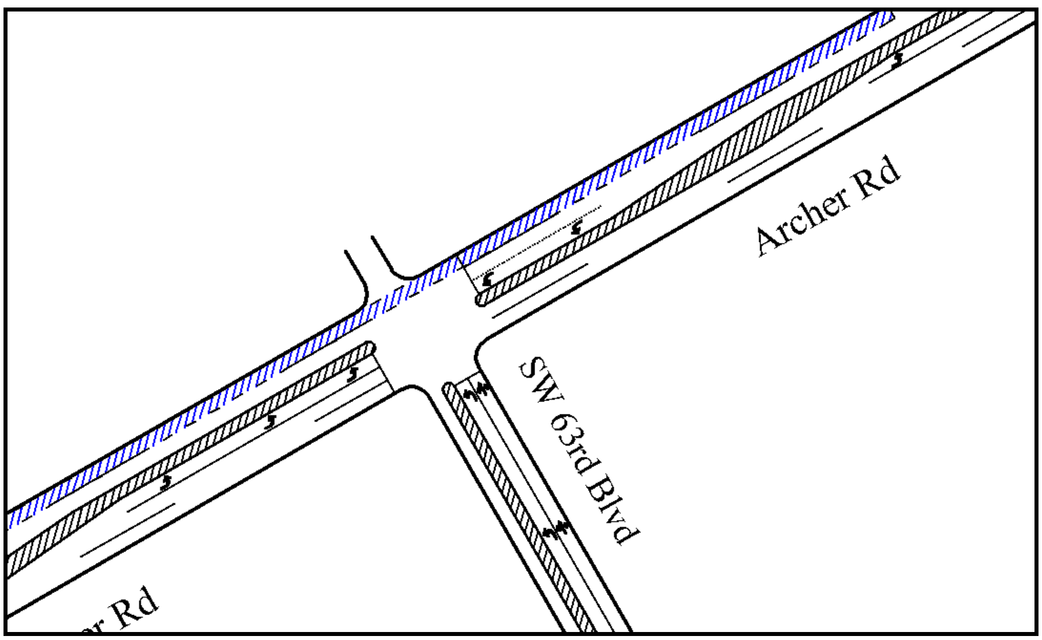


Figure 5.4 Sketch of the Lane Closure in Westbound for the Intersection of Archer Rd and SW 63rd Blvd

Table 5.16 provides the comparison of the performance measures between the no-work-zone scenario and the work-zone scenario for this intersection. Similar to the scenario at the intersection of Archer Rd and SW 75th St., there are very small changes in control delay and throughput after implementing the work zone in the simulation model. The average control delay for the westbound through movement only increases by 2.11 seconds (from 6.00 seconds to 8.11 seconds), but it is still at a very low level. The intersection average control delay increases by

1.38 seconds (from 8.31 seconds to 9.97 seconds) after implementing the lane closure. This is because the data were collected during low demand conditions, and the existing signal timing already has a long maximum green time for the westbound through approach. After implementing the lane closure in the approach, the actuated nature of the existing single timing can provide a long green phase for the high demand in the westbound approach to discharge. The demand for the remaining movements is much lower than the east and westbound through movements, and although for some of the movements delay may increase slightly, there is still a very small increase in the control delay for the entire intersection. For this situation, there is no need to change the signal timing under the existing demand conditions.

Table 5.16 Comparison between the Before and After Lane Closure Scenarios Implementing Existing Signal Timing

a) Performance Measures before Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	25.59	5.52	6.23	23.16	6.00	-	29.01	-	5.43	27.01	-	-	8.31
Throughput	1.4	91.1	5.0	3.5	170.2	-	27.1	-	5.7	2.0	-	-	

b) Performance Measures after Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	20.10	5.70	6.89	34.67	8.11	-	28.85	-	4.58	30.35	-	-	9.97
Throughput	1.7	89.6	5.7	5.8	167.4	-	28.3	-	4.9	2.0	-	-	

In order to test the proposed method in high demand conditions, the volume rates of the four approaches were manually increased in the simulation model. The original demand was doubled in the simulation model and the resulting performance measures for the before and after lane closure scenario are shown in Table 5.17. The per vehicle control delay for all movements in the westbound approach is significantly increased after implementing the lane closure. The weighted average delay for the entire intersection increased by 10.53 seconds (85.3%).

Although the lane closure in the westbound approach reduced the capacity of this approach, there is no need to rechannelize the movements. Therefore, the existing phasing pattern is maintained, and only the green interval for each phase was adjusted according to the corresponding new per lane volume. The proposed timing scheme has longer minimum green time for the westbound through movement and shorter maximum green time for the remaining movements. Table 5.18 shows the performance measures after implementing the new signal control scheme. Compared to Table 5.17-b, the performance measures for the eastbound and westbound approach are significantly improved, and the weighted average delay for the entire intersection is decreased by 7.33 seconds (32.0%).

Table 5.17 Comparison between the Before and After Lane Closure Scenarios using the Increased Volume Data

a) Performance Measures before Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	30.68	7.68	9.20	31.78	9.58	-	38.83	-	7.10	35.62	-	-	12.35
Throughput	2.4	180.6	11.7	10.1	335.2	-	56.6	-	10.6	5.3	-	-	

b) Performance Measures after Lane Closure

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	35.75	7.18	8.38	60.79	28.62	-	38.73	-	8.02	25.46	-	-	22.88
Throughput	2.1	181.6	10.6	10.6	320.7	-	55.7	-	10.9	5.5	-	-	

Table 5.18 Performance Measures After Implementing the Proposed Signal Timing Scheme

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	32.88	6.00	6.94	50.24	15.62	-	40.36	-	7.13	31.02	-	-	15.55
Throughput	2.4	182.3	10.9	11.2	332.3	-	56.0	-	10.9	5.6	-	-	

5.2.3 Case 3: Lane Closure at Some Distance Downstream from the Subject Intersection

In this case, the work zone area blocks one or more lanes and reduces the capacity in the arterial link between two signalized intersections. The lane closure may generate queues, which may spill back into the intersection if the capacity of the blockage area is too small to serve the high demand. In that case, the upstream vehicles have to slow down to enter the downstream receiving lanes, and therefore, the delay of the approach upstream of the intersection will be increased.

The tested site is the section of Archer Rd that includes two consecutive intersections, the intersection at SW 75th St. and the intersection at SW 63rd Blvd. The intersection of Archer Rd and SW 75th St is located approximately 4200 feet downstream of the intersection of Archer Rd and SW 63rd Blvd. A lane closure was implemented in the simulation 900 feet downstream of the intersection at 63rd Blvd in the westbound approach and extended for 2500 feet. This arterial section originally had two lanes in its westbound approach, and one of them was closed for the work zone. This resulted in a bottleneck in the receiving lanes of the intersection at 63rd Blvd. The field data collection site did not have such a lane closure configuration during the construction period. Therefore, the calibrated model from the no-work-zone scenario was used, and the work zone was implemented in the simulation. The volume data for the most congested 15 minutes (8:00pm-8:15pm) were used as the demand in the simulation model.

Table 5.19 and Table 5.20 present the performance measures of the two intersections for the before and after lane closure scenario, respectively. It can be noticed that there are very small changes in the control delay and throughput after implementing the work zone in the simulation. The average control delay for the intersection at 63rd Blvd only increases by 0.23 seconds (from 9.07 seconds to 9.30 seconds) and the average control delay for the intersection at 75th St. remains almost the same. As for previous cases, this is because the data were collected under low demand conditions, and the capacity of the site can easily accommodate the demand.

In order to test the proposed method in higher demand conditions, the volumes for all approaches were proportionally increased in the simulation model. The simulation demands were set to be 3.1 times the original demands. The resulting performance measures for the before and after lane closure scenario using this demand are shown in Table 5.21 and 5.22, respectively. The control delay of the intersection at 75th Str. was not significantly affected, however for the upstream intersection at 63rd Blvd, the per vehicle control delay of the westbound approach is significantly increased (56.8% for left turning movement and 137.6% for the through and right turning movements) after implementing the lane closure. The weighted average delay for the entire intersection increased by 14.52 seconds (75.7%).

The animation of the simulation model shows that the vehicles in the westbound approach of the upstream intersection were greatly delayed by the slowly moving queues generated by the bottleneck when entering the downstream receiving lane. The long signal cycle at this intersection made this situation even worse. The reduced capacity in the downstream receiving lane could not handle the amount of vehicles discharged from the upstream intersection during the long green interval. The proposed method was used to improve the traffic operation under this condition.

First, the hourly demand from the upstream intersection (1446 veh/h) was compared with the capacity of the lane closure area (1800 veh/h) and the capacity of the downstream intersection (2127 veh/h). Both constraints in step 1 were satisfied. For step 2, since the demands of the upstream left-turning and right-turning movements are very low, the proposed method was only used for the upstream through movement. The average per phase demand is 61 veh/phase which is greater than $C_{closure}^{p}$ (30 veh/phase). Therefore, Equation 24 was used to calculate the proposed green time, which was found to be 45 sec. The green times for the remaining phases were shortened proportionally. The proposed signal timing was then implemented in the simulation model.

Table 5.23 shows the performance measures after implementing the proposed signal timing scheme for the intersection at 63rd Blvd. For the intersection at 75th St., the performance measures did not change significantly, but for the upstream intersection at 63rd Blvd, the performance measures are already comparable to the before lane closure scenario (19.64 sec vs. 19.19 sec for the intersection average control delay). The weighted average delay for the entire intersection is decreased by 14.06 seconds (41.7%) compared to the lane closure scenario using the existing signal timing plan.

Table 5.19 Performance Measures for the Before Lane Closure Scenario Using Existing Signal Timing

a) Performance Measures for the Intersection at 63rd Blvd

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	22.46	6.90	7.69	26.02	6.75	-	27.92	-	4.79	26.09	-	-	9.07
Throughput	1.9	89.1	6.4	3.4	169.4	-	26.7	-	5.6	2.0	-	-	

b) Performance Measures for the Intersection at 75th St.

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	27.61	5.75	-	30.34	8.83	5.35	16.05	32.90	4.47	23.08	18.14	6.62	11.96
Throughput	19.7	45.7	-	11.2	132.0	55.9	0.6	5.8	6.5	45.8	9.4	17.0	

Table 5.20 Performance Measures for the After Lane Closure Scenario Using Existing Signal Timing

a) Performance Measures for the Intersection at 63rd Blvd

	EB			WB			NB			SB			Average Delay
	LT	**TH**	**RT**	**LT**	**TH**	**RT**	**LT**	**TH**	**RT**	**LT**	**TH**	**RT**	
Control Delay per Vehicle	12.51	7.43	5.71	25.82	6.89	-	27.82	-	6.03	25.87	-	-	9.30
Throughput	2.2	87.1	6.5	3.5	169.4	-	27.7	-	5.3	2.0	-	-	

b) Performance Measures for the Intersection at 75th St.

	EB			WB			NB			SB			Average Delay
	LT	**TH**	**RT**	**LT**	**TH**	**RT**	**LT**	**TH**	**RT**	**LT**	**TH**	**RT**	
Control Delay per Vehicle	27.90	5.33	-	29.47	8.50	5.70	22.37	30.59	4.00	23.58	17.77	7.73	11.92
Throughput	21.4	44.2	-	10.4	126.9	61.1	0.8	5.7	6.5	46.2	8.3	16.5	

Table 5.21 Performance Measures for the Before Lane Closure Scenario Using Increased Volume Data

a) Performance Measures for the Intersection at 63rd Blvd

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	41.19	13.19	11.21	49.03	18.28	-	39.07	-	7.99	29.32	-	-	19.19
Throughput	8.7	282.4	20.4	16.6	519.5	-	87.6	-	16.7	8.4	-	-	

b) Performance Measures for the Intersection at 75th St.

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	108.10	14.55	-	78.06	24.69	14.71	61.91	46.83	6.73	40.20	26.65	22.32	30.10
Throughput	55.6	144.0	-	37.7	386.0	172.6	2.3	18.0	21.7	145.8	28.0	51.4	

Table 5.22 Performance Measures for the After Lane Closure Scenario Using Increased Volume Data

a) Performance Measures for the Intersection at 63rd Blvd

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	62.66	14.55	16.38	76.86	43.43	-	39.61	-	11.92	33.54	-	-	33.70
Throughput	6.3	289.6	21.9	15.6	506.4	-	86.1	-	16.7	8.2	-	-	

b) Performance Measures for the Intersection at 75th St.

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	107.25	15.23	-	53.45	28.65	11.50	61.98	43.35	8.66	39.65	27.43	22.91	30.13
Throughput	57.2	148.1	-	30.7	381.9	169.2	2.6	17.9	21.6	145.3	27.0	51.1	

Table 5.23 Performance Measures after Implementing the Proposed Signal Timing Scheme for the Intersection at 63rd Blvd

a) Performance Measures for the Intersection at 63rd Blvd

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	37.17	9.99	11.68	38.02	21.32	-	39.04	-	13.02	28.24	-	-	19.64
Throughput	6.3	286.3	19.8	17.0	518.7	-	85.9	-	16.7	8.0	-	-	

b) Performance Measures for the Intersection at 75th St.

	EB			WB			NB			SB			Average Delay
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
Control Delay per Vehicle	101.38	15.51	-	57.02	27.35	11.17	50.77	45.14	9.78	38.73	26.14	21.62	29.03
Throughput	54.1	145.2	-	32.3	388.3	174.2	2.7	18.5	21.4	147.4	28.8	52.0	

6 CONCLUSIONS AND RECOMMENDATIONS

This research has developed guidelines for the development of signal control plans at various work zone configurations along arterial streets. Three work zone cases are distinguished:

- *Case 1: Lane Closure Before the Intersection*. In this case, the work zone area blocks one or more lanes upstream of the intersection, and there is some distance from the work zone to the stop bar. In this case it is important that the green given to the approach does not significantly exceed the demand that is able to pass through the work zone. Otherwise, portions of the green time given to that approach would be wasted. The key issue for signal timing under this case is to efficiently use the available storage area (i.e., the area between the downstream end of the work zone and the signalized approach stop bar) so that the capacity of the approach is optimally used.
- *Case 2: Lane Closure at the Stop Bar.* This case can be further divided into two subcases: lane closure at the stop bar that causes changes in the type of the remaining lanes and lane closure at the stop bar that reduces the number of lanes but does not change the remaining channelization. When developing a signal timing plan for this case, the subject approach should first be considered for rechannelization based on the respective demand of the different movements. The next step is to retime the traffic signal to optimize the intersection operations based on the new channelization and per lane demand.
- *Case 3: Lane Closure at Some Distance Downstream from the Subject Intersection.* In this case, the work zone area will block some lanes in the middle of an arterial link between two intersections. There are three key parameters in this case: demand from the upstream intersection ($D_{upstream}$), capacity of the lane closure area ($C_{closure}$) and capacity of the downstream intersection ($C_{downstream}$). The values of the three parameters determine at which area congested conditions may occur and which signal timing would need to be modified.

Detailed guidelines were developed to optimize signal control around each of the work zone cases described above. A combination of field data and simulation was used to evaluate these guidelines and document their effectiveness under different demand conditions. Generally signal retiming around work zones is warranted only when the work zone is expected to significantly impact operations and increase delay. This occurs when demand is high, approaching or exceeding capacity. If that is not the case, the existing signalization plan should be retained. The following paragraphs summarize the guideliness for each of the three cases:

<u>Signal control optimization for Case 1:</u>

a) Calculate the maximum green interval, which should be set equal to the time required to clear the queue between the stop bar and the downstream of the work zone:

$$G_{\max} = \frac{D}{V_L} \times S_h + L_T - Y - AR \tag{1}$$

where

G_{max}: Maximum green time for the approach with lane closure upstream of the intersection

D: Length of the section between the stop bar and the downstream end of the work zone (ft)

V_L: Average space occupied by a queued vehicle (ft)

S_h: Saturation time headway (sec/veh)

L_T: Total lost time per phase (sec)

The lost time is:

$$L_T = l_1 + l_2 \tag{2}$$

l_1: Start-up lost time per phase (sec)

l_2: Clearance lost time per phase (sec)

Y, AR: Length of the yellow and all red intervals, respectively (sec)

This green time in a cycle may not be long enough to meet the high demand of the subject approach. In that case, the required total green time can be split into two short intervals to ensure that each phase is not longer than the suggested maximum green, and the phase of another movement in between allows the queue to build up before the repeated green is given.

b) Calculate the throughput of the approach before and after splitting the phase using the following equations:

$$N_{before} = \frac{G_1 - l_1}{H_s} \times N + \frac{G_2 + Y + AR - l_2}{H_s} \times N_r \quad (3)$$

$$N_{after} = \frac{G_1 + G_2 + Y + AR - 2(l_1 + l_2)}{H_s} \times N = \frac{G_1 + G_2 + Y + AR - 2L_T}{H_s} \times N \quad (4)$$

where

N_{before}: Number of vehicles the subject approach can serve per cycle without splitting the green

N_{after}: Number of vehicles the subject approach can serve per cycle after splitting the green

H_s: saturation time headway (sec/veh)

N: Number of lanes of the subject approach at stop bar

N_r: Reduced number of lanes of the lane closure section

L_T: Total lost time per phase (sec)

l_1: Start-up lost time per phase (sec)

l_2: Clearance lost time per phase (sec)

G_1: Time period from the beginning of green to the time when vehicles stored in the branched section have cleared.

The relationship between G_1 and G_{max} is:

$$G_1 = G_{\max} + Y + AR - l_2 \quad (5)$$

G_2: The remaining green time for the subject phase.

G_2 is also:

$G_2 = G - G_1$ (6)

where

G: The actual green time before splitting the green.

c) Compare the before and after throughput and select the signal timing that results in the higher throughput.

Signal control optimization for Case 2:

This lane closure case can be further divided into two subcases: a) lane closure at the stop bar causes change in the channelization type of the remaining lanes; b) lane closure at the stop bar reduces the overall number of lanes, but there are no changes in channelization.

For the situation in which the lane closure causes changes in channelization, the phasing pattern along with the timing should be reconsidered. The HCM 2000, among others, provides detailed instructions for developing a phase plan and estimating the basic signal timing parameters. For developing a phase plan, the general guideline is that a simple two-phase control should be used unless conditions dictate the need for additional phases. For the configurations that do not result in changes in channelization, only the green interval for each phase needs to be adjusted according to the corresponding new per lane volume.

Signal control optimization for Case 3:

a) Compare $D_{upstream,}$ $C_{closure}$ and $C_{downstream}$ on an hourly basis to ensure that all the vehicles from the upstream intersection can be served. The values of the three parameters have to satisfy the following constraints:

1) $D^h_{upstream} \le C^h_{closure}$;

2) $D^h_{upstream} \le C^h_{downstream}$.

where

$D^h_{upstream}$: hourly demand from the upstream intersection (veh/h), it consists of the demand of the three movements at the upstream intersection;

$C^h_{closure}$: hourly capacity of the lane closure area (veh/h);

$C^h_{downstream}$: demand from the upstream intersection (veh/h).

If $D^h_{upstream} > C^h_{closure}$, the capacity of the lane closure area cannot meet the demand, resulting in queues building up and spilling back into the upstream intersection. In this case, a smaller *g/C* ratio of the corresponding upstream phases should be implemented to reduce the number of vehicles that enter the work zone area. The maximum *g/C* ratio can be calculated using the following equation.

$$(g/C)_{\max,upstream} = (g/C)_{existing,upstream} \times \frac{C^h_{closure}}{D^h_{upstream}} \tag{15}$$

$(g/C)_{max,\ upstream}$: maximum g/C ratio of the upstream intersection;

$(g/C)_{existing,\ upstream}$: existing g/C ratio of the upstream intersection;

Other variables are as previously defined.

If the *g/C* ratio cannot be reduced to the value calculated from the equation above, the analyst should consult the third step of the method. Alternatively, the construction work should be rescheduled for another time period when $D^h_{uptream}$, is less than $C^h_{closure}$.

If $D^h_{upstream} > C^h_{downstream}$, then not all vehicles can be served at the downstream intersection. Queues will keep building and finally spill back into the upstream intersection. There are two ways to address this problem. The first one is to reduce the upstream demand by using a smaller *g/C* ratio of the corresponding upstream phases. The second way is to increase the downstream capacity by increasing the *g/C* ratio of the downstream intersection.

If the first method is used, the maximum *g/C* ratio of the upstream intersection can be calculated using the following equation.

$$(g/C)_{\max,upstream} = (g/C)_{existing,upstream} \times \frac{C^h_{downstream}}{D^h_{upstream}} \tag{16}$$

All variables are as previously defined.

If the upstream demand cannot be reduced, increasing the downstream capacity is another way to satisfy the constraints. Capacity of the downstream intersection can be calculated by the following equation.

$$C^h_{downstream} = s_{downstream} \times g/C \tag{17}$$

where

$S_{downstream}$: saturation flow rate of the studied approach at downstream intersection (veh/h);

g : length of the green interval (sec);

C: cycle length of the downstream intersection (sec).

Therefore, the minimum g/C ratio of the downstream intersection should be:

$$(g / C)_{\min,downstream} = D^{h}_{upstream} / s_{downstream} \tag{18}$$

$(g/C)_{min,\ downstream}$: minimum g/C ratio of the downstream intersection;
All other variables as previously defined.

If the g/C ratio cannot be adjusted to the value calculated from Equation 16 or Equation 18, the analyst should consult the third step of the method. Alternatively, it is suggested that the construction work should be rescheduled for a less congested time period when $D^{h}_{upstream}$ is less than $C^{h}_{downstream}$.

b) Compare $D_{upstream}$, $C_{closure}$ and $C_{downstream}$ on a per phase basis to avoid the occurrence of the spillback queues on a per phase basis. The analysis procedure discussed below should be conducted for each of the three upstream phases (left-turning, through, and right-turning).

The three parameters analyzed in this step are the per phase demand from the upstream intersection ($D^{p}_{upstream}$, veh/phase), per phase capacity of the lane closure area ($C^{p}_{closure}$, veh/phase) and the number of upstream vehicles that can be absorbed by the downstream intersection per phase ($N^{p}_{downstream}$, veh/phase). The per phase capacity of the lane closure area, which represents the number of vehicles that can be discharged from the lane closure area during each phase, can be calculated using the following equation:

$$C^{p}_{closure} = DR_{sat} \times g_{upstream} \tag{19}$$

where

DR_{sat}: saturation discharge rate of the lane closure area (veh/sec);

$g_{upstream}$: green time of the studied phase at the upstream intersection(sec).

The parameter $N^{p}_{downstream}$ represents the number of upstream vehicles that can be discharged by the downstream intersection without a stop at the downstream stop bar:

$$N^{p}_{downstream} = P_{discharge} \times D^{p}_{upstream} \qquad (20)$$

where

$P_{discharge}$: percentage of upstream vehicles that can be discharged by the downstream intersection without a stop at the downstream stop bar;

$D^{p}_{upstream}$: per phase demand from the upstream intersection (veh/phase).

The calculation for the parameter $P_{discharge}$ depends on the coordination of the two intersections. If the two intersections are coordinated, $P_{discharge}$ can be calculated using Equation 21.

$$P_{discharge} = \frac{\left(\min(g_{upstream}, g_{downstream}) - \max(offset - \frac{L}{v_{avg}}, 0)\right) \times s_{downstream}}{g_{upstream} \times s_{upstream}} \qquad (21)$$

where

$g_{upstream}$: green time of the subject phase at the upstream intersection (sec);

$g_{downstream}$: green time of the corresponding downstream phase (sec);

offset: offset of the downstream intersection relative to the upstream intersection (sec);

L : distance from the upstream stop bar to the end of downstream queue (ft);

v_{avg} : vehicles' average speed (ft/sec);

$S_{downstream}$: saturation flow rate of the subject approach at downstream intersection (veh/sec);

$S_{upstream}$: saturation flow rate of the subject approach at upstream intersection (veh/sec).

If the two intersections are not coordinated, we assume no upstream vehicle can be discharged by the downstream intersection without a stop and $P_{discharge}$ equals to 0.

In order to avoid the occurrence of spillback queues, the following constraints should be satisfied for the parameters calculated using the equations above.

1) If $D^p_{upstream} \le C^p_{closure}$, $D^p_{upstream} \le N^p_{downstream} + C_{storage,downstream}$;

2) If $D^p_{upstream} > C^p_{closure}$, $D^p_{upstream} \le C^p_{closure} + C_{storage,closure}$.

where

$C_{storage,downstream}$:storage capacity of the link between the two intersections (veh);
$C_{storage,closure}$: storage capacity of the section upstream of the lane closure area (veh);
Other variables as previously defined.

If $D^p_{upstream} \le C^p_{closure}$, all the vehicles from the upstream intersection can be absorbed by the lane blockage section. The lane closure does not have much impact on traffic. $D^p_{upstream}$ should be compared with the summation of $N^p_{downstream}$ and the link storage capacity ($C_{storage,downstream}$). If $D^p_{upstream} > N^p_{downstream} + C_{storage,downstream}$, queues at the downstream stop bar will spill back into the upstream intersection. In this case, the offset needs to be adjusted to increase $N^p_{downstream}$. Alternatively, $D^p_{upstream}$ should be reduced by reducing the cycle length. However, since $D^p_{upstream} \le C^p_{closure}$, the demand is relatively low in this case. Also, the arterial links are usually long enough to store the vehicles, therefore, this situation is not very common.

If $D^p_{upstream} > C^p_{closure}$, there are more vehicles than that can be discharged from the work zone area in one phase. In this situation, queues will form at the lane closure area. In order to avoid spillback, $D^p_{upstream}$ should not exceed the summation of $C^p_{closure}$ and $C_{storage,closure}$ (storage capacity of the section upstream of the lane closure area). Using the calculation for $D^p_{upstream}$ and $C^p_{closure}$, the constraint can be written as follows:

$$\frac{D^h_{upstream}}{3600 / C_{upstream}} \leq DR_{sat} \times g_{upstream} + C_{storage,closure} \tag{22}$$

where

$D^h_{upstream}$: hourly upstream demand for the subject phase (veh/h);

$C_{upstream}$: cycle length of the upstream intersection (sec).

Other variables as previously defined.

The cycle length can be written as:

$$C = \frac{g}{g / C} \tag{23}$$

Using the above equation of cycle length, Equation 22 can be written as:

$$g_{upstream} \leq \frac{C_{storage,closure}}{\left(\frac{D^h_{upstream}}{3600 \times (g / C)_{upstream}} - DR_{sat} \right)} \tag{24}$$

All variables as previously defined.

In equation 24, if we assume that the g/C ratio does not change, only the value of $g_{upstream}$ is unknown. Therefore, if $D^p_{upstream} > C^p_{closure} + C_{storage,closure}$, the phase length of the upstream intersection should be reduced based on Equation 24.

c) Conduct further analysis for cases when the conditions in the previous steps cannot be met. The steps discussed above involve basic analysis for a general case when a lane closure is installed along an arterial link between two signalized intersections. The proposed methodology can accommodate some of the spillback that may occur. In cases when the recommended modifications cannot adequately address the expected operational conditions, it is possible that a full signal optimization would be able to address those. A full signal optimization can adjust the offset between intersection so that the queues can be better managed. However, in cases of severe congestion, it may be that there are no signalization improvements that can alleviate congestion when the work zone is installed. In those cases, it may be preferable to schedule the work zone for another time period, if feasible.

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