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16. Abstract

Lane closures due to highway work zones introduce many challenges to ensuring smooth traffic operations and a safe environment for drivers and workers. In addition, merging has been found to be one of the most stressful aspects of driving and a merge process that is viewed as "unfair" through actions like queue jumping can lead to further unsafe behaviors stemming from "road rage." To address these issues, the work in this project will focus on lane control solutions for intermediate and long-term highway work zones. In order to evaluate network performance, driver behavior, driver operations, and impacts on safety, several tools were used. Using a combination of field observations, microsimulation, and dynamic traffic assignment tools, the main objective was to develop a procedural guide or decision tree for freeway work zone traffic control planning. This procedural guide would then be presented to the Texas Department of Transportation in a pilot training workshop. Using microsimulation software, with a focus on VISSIM, the analysis of different applications of merge concepts through delay and safety is presented in the project. In order to appropriately draw conclusions about and identify trends of different merge concepts from the microsimulation software, early merge, late merge, and signal merge were first explored in a thorough literature review. In addition to delay and queuing analysis completed using VISSIM, the Federal Highway Administration's Surrogate Safety Assessment Model (SSAM) was used to address the effects of implementing signal merge on rear-end and lane-change conflicts. Compiling the data collection, VISSIM microsimulation outputs, and SSAM signal merge safety outputs, general conclusions and decisions were provided.

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Chapter 1. Introduction

1.1 Background

Work zone lane closures on highways create difficulty in providing efficient traffic operations and safe conditions for drivers and workers. Lane closures due to highway work zones reduce available capacity, which increases congestion and poses several issues in maintaining unobstructed traffic operations. Merging at these closures increases weaving, causes queue jumping, and presents the risk of rear-end collisions. Drivers subjected to these stressful conditions may exhibit unsafe behavior that stems from “road rage.” According to a Dallas study by the Texas A&M Transportation Institute, around half the drivers surveyed see merging as the most stressful situation facing drivers. This stress is primarily due to drivers using the closed lane to pass the slower moving traffic in the open lane just to force their way in downstream, otherwise known as *queue jumping* (Walters et al., 2000). Thus, the purpose of this project is to assess delay and safety concerns associated with various work zone conditions and provide recommendations to ensure efficient operations and conditions for both users and workers.

The researchers used a combination of field observations, micro-simulation, and dynamic traffic assignment (DTA) tools to develop a procedural guide for freeway work zone traffic control planning. Key elements of the procedure include determination of hours and days in which traffic demand is less than, equal to, or greater than the proposed work zone capacity. It includes consideration of traffic diversion to paths other than those passing through the work zone. The guide suggests conditions for optimal use of early merge or late merge and provides guidelines for use of signal-controlled merge operations. A schematic version of the procedure is shown in Figure 1.1.

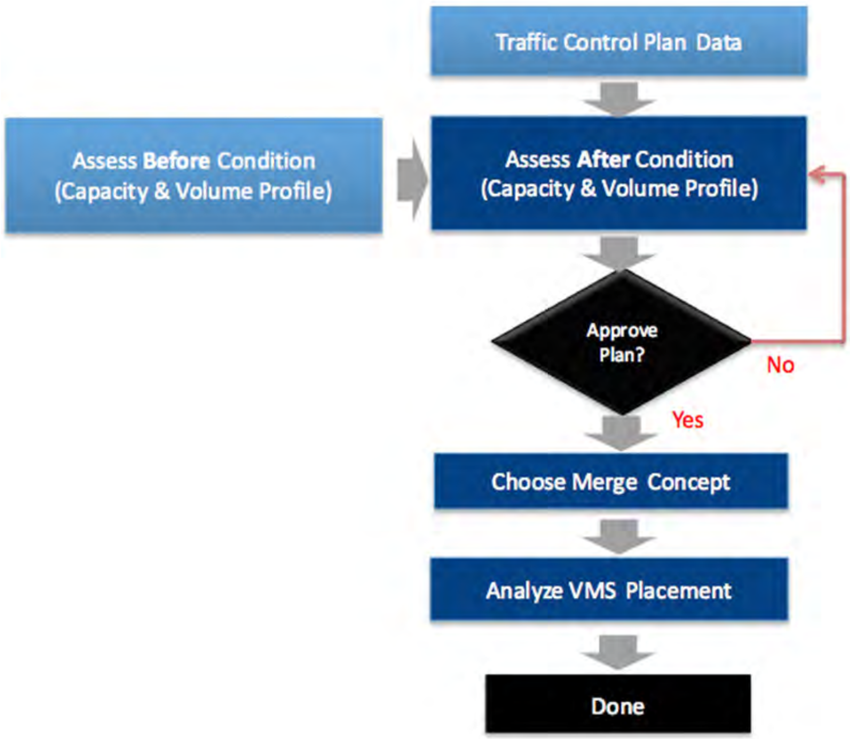


Figure 1.1 Conceptual Guide for Work Zone Traffic Control Planning.

1.2 What Researchers Found

Significant findings include the following:

- If the hours and days of work zone activity can be chosen so that traffic demand does not exceed work zone capacity, an early merge scheme will provide maximum safety and minimum user delay. Under low volume conditions, early merge can enable little or no delay for travelers through the work zone.
- Early merge concepts become highly problematic when traffic demand approaches or exceeds work zone capacity. Under these conditions, incidents of queue jumping, excessive lane changing, and crashes tend to escalate.
- If hours and days of work zone activity must include times in which traffic demand exceeds capacity, late merge concepts are the best option.
- Late merge schemes generally are designed to use all available lane space prior to the work zone for queue storage; therefore, they provide the best available procedure if traffic demand approaches or exceeds work zone capacity.
- For times in which demand exceeds capacity of the work zone, use of the signal-controlled merge offers promise to reduce queue jumping, lane changing, and associated crashes. Suggestions for signal timing for signal-controlled merge processes are provided.

- Ideally, estimation of the work zone traffic demand should be based on counts or at least estimates of traffic volumes prior to work zone installation. Although every work zone is unique, generally the traffic demand after work zone installation will be less than demand before work zone activation—that is, diversion of traffic from the work zone is almost always non-zero.
- Estimation of work zone traffic diversion can best be done through before-after application of a DTA model. The DTA process generally requires a detailed network description and never predicts link volumes that exceed capacity. The DTA process can be expected to yield link traffic volumes, as opposed to the link demands produced by a static assignment process.
- If DTA is not yet available for the work zone location, before-after application of a traditional static traffic assignment (STA) model offers a reasonable second choice. STA has serious limitations as far as realistically representing the process that leads to congestion and increased travel time, but in a before-after comparison of the work zone area, it does provide value. STA assignment models are currently available in all urban and suburban Metropolitan Planning Organization shops.
- If neither DTA nor STA assignment capabilities are available, a rule of thumb of a 15% reduction of before-work-zone traffic volume may be applied to estimate traffic demand during work zone activity.
- A queue length prediction tool was developed for those situations in which traffic demands exceeding work zone capacity are a reality. The tool can be used to estimate where variable message signs or other uniquely critical control devices should be placed.

1.3 What This Means

The procedural guide developed through this study provides a rational approach to work zone traffic control. Application of the guide to urban and suburban projects will provide a basis for reducing user costs and improving both user and worker safety. A workshop with visual aids was developed to present these concepts in an efficient, painless fashion. The workshop materials could be provided as a self-study tool or through a face-to-face training session. In an effort to understand prior applications of work zone traffic control in projects with various departments of transportation (DOTs), a thorough literature review was conducted to inform the presentation of the workshop materials.

Chapter 2. Literature Review

2.1 Lane Control

Lane control techniques facilitate the merging process to reduce highway user stress levels. By guiding the driver at or to a specific point, instances of queue jumping and weaving are lessened, which can increase capacity. These processes are implemented through variations of either early merge or late merge control strategies. Both forms of control can be implemented as either static or dynamic. The static approach employs signs that display a single message at all times and in the same location regardless of traffic conditions. The dynamic approach uses real-time control measures to decide whether to activate additional signage upstream to further inform approaching drivers.

2.1.1 Early Merge Control

Early merge is a strategy that warns drivers in advance of a work zone of an upcoming closed lane. This method allows time for the user to find a gap and complete the merge process ahead of the closure. This technique is found very effective if traffic demand is low compared to capacity. The system breaks down in the face of high demand and fewer gaps (Yang et al., 2009).

A study by Tarko and Venugopal for the Indiana DOT (INDOT) looked at using variable messaging signs to warn drivers to merge ahead of the queue as shown in Figure 2.1 (Tarko et al., 2001). These signs were triggered to flash the message “No Passing When Flashing,” using sensors that activated the next sign upstream of a forming queue. This created a “no passing zone” with enough room for local police to enforce the signage. By merging sooner, aggressive maneuvers were minimized throughout the merge process. This strategy was found useful with low to moderate traffic demands.

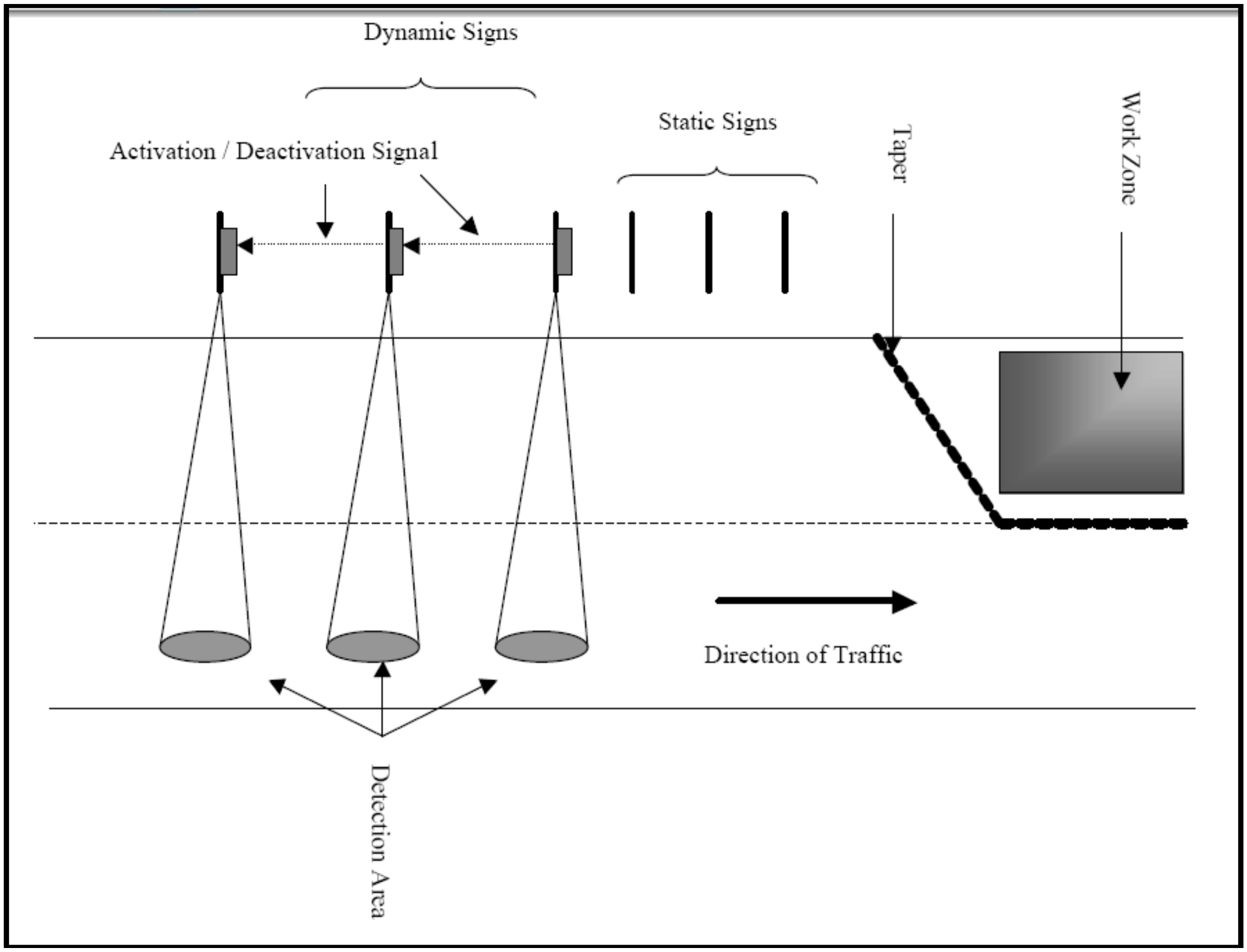


Figure 2.1 Indiana Lane Merge System.

INDOT tested this system on 2-to-1 lane work zones, while in 2004, the Michigan DOT (MDOT) experimented on 3-to-2 lane configurations with approximately the same setup and results (Datta et al., 2004). MDOT also tried this technique in 2000 at five locations along the highway system. To aid in compliance, merging at these locations was enforced by officers and violators risked a $200 fine (Walters et al., 2000).

2.1.2 Late Merge Control

Late merge is a technique that tries to take advantage of the full capacity of the highway approaching a work zone to minimize the length of queue formation. This goal is accomplished by advising drivers to use all available lanes followed by a “take turns” method once at the merge point. The Delft University in the Netherlands described this as the “zipper” method, which means that each driver waits to change lanes until a fixed distance from the lane drop, immediately behind the follower of their original leader. Proper usage of the late merge system can improve throughput significantly while reducing queue length of up to 50% (Walters et al., 2000).

In a comparative study by McCoy and Pesti (McCoy et al., 2001), early merge was noted as being efficient only in low to moderate traffic demands. Once the system approached capacity, significant queues would develop, creating the risk of high-speed drivers encountering stopped queues. Their proposed solution was a dynamic late merge setup where sensors would switch the system from early merge at low volumes to late merge at high volumes. This could be accomplished with real-time sensors that would activate variable message signs (VMS) to inform the driver whether to maintain lanes. One innovative concept was the use of construction placement of sensors in construction barrels, or *smart drums*, to monitor traffic as shown in Figure 2.2. They noted that signs would have to be placed well beyond the anticipated queue length to avoid the aforementioned collision risk. This was also recognized by the Maryland Highway Administration, who found that they had to move their signs three times due to underestimating the queue length. They went on to recommend that warning signs be placed on both sides of the highway to avoid blockage by heavy vehicles and that requiring speed reduction as vehicles approached the merge point smoothed throughput (Kang et al., 2006).

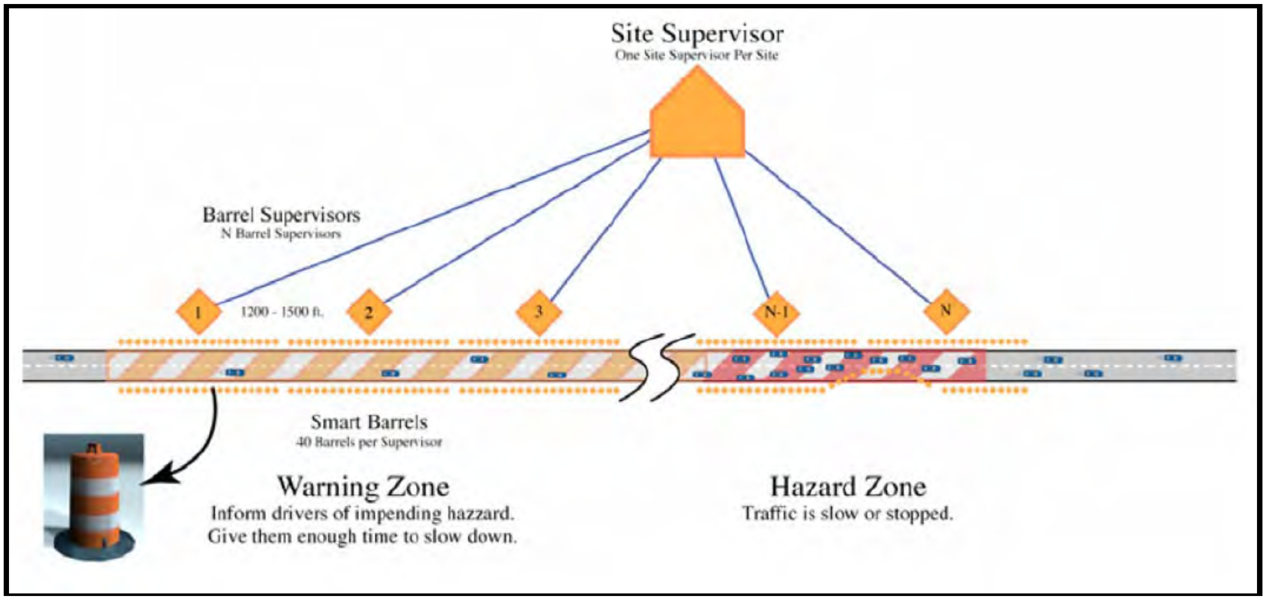


Figure 2.2 Application of Smart Drum.

A study for the Virginia DOT evaluated late merge setups similar to the ones mentioned above but was less significant. A more even lane split developed among users as compared to uncontrolled methods, but the measures of effectiveness were not significantly different. The author mentioned that these variations might be due to site differences, such as driver characteristics, geometry, or even vehicle mix. Meyer ran into a similar challenge when researching work zone behavior for Kansas City. He noticed that drivers would move into the left closed lane even when instructed to remain in the right lane. The site was close to an entrance ramp, which caused drivers to react to outside sources other than sign postings. He also documented that drivers went through a “training” period where they had to become used to the control method before any significant effects manifested. The percentage of drivers following the control doubled from the first week to the second (Meyer, 2004).

2.1.3 Signalized Merge Control

Signalized guidance is a relatively new concept in which traffic signals that are usually located at intersections are placed at work zones to facilitate movement. Signalized control was created to manage merging where sites are heavily congested. The conventional merge scenarios such as early merge and late merge are beneficial as long as traffic volumes remain relatively low. A study conducted in 2009 suggests that early and late merge control methods peak in efficiency at between 700 and 800 vehicles per hour per lane. In contrast, the experimental procedure of lane-based signal merge could handle well above that limit and worked efficiently with high percentages of heavy vehicles (Yang et al., 2009).

A study led by Heng Wei at the University of Cincinnati combined signal control with dynamic late merge. In this study, real-time sensors were used to detect traffic demand. Once the system noted certain control measure limits, a central unit activated upstream signs to warn drivers to maintain their lane position. Once at the merge point, a traffic signal would alternate lanes, allowing users to enter the work zone one at a time. The system was named the Dynamic Merge Metering Traffic Control System (DMM-Tracs) and was noted to work well with cycle lengths of either 60 or 120 seconds, with an optimal length of between 60 and 120 seconds. Lentzakis at the Technical University of Crete in Greece also studied metering effects with signals, but used local metering algorithms (ALINEA) to optimize signal timing (Lentzakis et al., 2008).

All signalized studies above rely heavily on simulation software for testing. No studies were found that showed actual field testing of signalized merge control, although each study reported promising results under heavily congested traffic conditions. Regardless of the roadway demand and lane control measures used, however, a percentage of roadway users will always divert because of the presence of a work zone.

2.2 Traffic Diversion

When drivers approach a work zone, a certain percentage will naturally divert to an alternate path if one is available. With appropriate warning and information provided about alternate paths, an even greater percentage will divert. The question is this: to what extent will drivers divert? The greater the traffic volume that diverts to an alternate route, the greater the congestion alleviation at the work zone.

A certain amount of delay in work zones is typically assumed to be unavoidable and often considered a cost of doing business when roadway improvements are in progress (Lee et al., 2008). Traffic queues when demand exceeds capacity. The queue will continue to grow until demand lessens, usually through drivers seeking alternate routes. When work zones have been in place long enough that traffic reacts to it, the volume tends to drop on the highway system. In this scenario, entrance ramp volumes decrease 20–40% while exit volumes just before the queue increase significantly, thus causing significant queues in a phenomenon known as “exit ramp spill back” (Ullman, 1992; Pesti et al., 2007).

TxDOT has algorithms designed to determine the percentage of vehicles that will naturally detour from the highway system given the presence of a work zone. The software initially used to determine diversion was QUEWZ, a TxDOT program that was generated in 1987. Studies from Ullman and Lee, however, suggest that this model needed to be modified. Both authors stated that traffic will queue to some threshold and remain there until demand decreases. Ullman recommended that diversions should be based at the level where demand exceeds capacity, and that drivers will typically tolerate a delay of 20 minutes before seeking an alternate path (Ullman, 1992). In addition, Lee suggested using an analysis tool called Work Zone Capacity Analysis Tool (WZCAT) to predict delays and queue length. This model is a simple input-output with work zone capacity controlling throughput (Lee et al., 2008).

Ullman later revised his theory and viewed a work zone capacity issue like a permeable pipe. The model used fluid flow analogies to describe driver behavior upstream of a work zone bottleneck. The input of the model is the highway's historic traffic data and it employs a calibration factor to adjust the model to site conditions (Ullman and Dudek, 2003).

With suggestions that several different models could best account for diversion, work zone models thus far have clearly been very site-specific. No one model seems to be transferrable from one location to another. Site conditions play a key role in determining the divergence of traffic and the nature of the queue that forms. Thus, in order to best assess traffic diversion, considering site conditions (like configuration) as well as work zone conditions is important. The following chapter introduces the three sites that were studied for this project, including one Houston site and two Austin sites, in an effort to assess site conditions that could affect diversion and queue formation.

Chapter 3. Methodologies

3.1 Description of Sites

For this task, the research team collected work zone field data from one Houston site (IH 610E at Clinton) and two Austin sites (IH 35 at 51st Street and westbound Oltorf Street at IH 35). In each of these sites a different type of closure was observed. At the Houston site, one highway main lane was closed and no clear alternate routes were available. At the IH 35 Austin site, the lower deck was shut down and vehicles had the option of continuing to the upper deck or exiting onto the frontage road. At the Oltorf Street site, all lanes were closed and a detour was recommended. This chapter summarizes the data collection procedures and analysis at all sites.

As stated in the proposal, the collected data will be used in experiments designed to improve understanding of driver's route choice, safety, and driving behavior, among others. The data can also be used to calibrate future models.

3.2 Data Collection and Results

This section will detail the data collection process and results for each of the three sites.

3.2.1 Houston: IH 610

3.2.1.1 Work Zone Location and Layout

The TranStar website was used to identify an appropriate work zone location for the purpose of this project (at IH 610E and Clinton). Please see Table 3.1 and Figure 3.1 for information about this work zone. Figure 3.2 is a Google Earth picture of the study location.

Table 3.1 Work Zone Information for IH 610 at Houston Site.

Location	IH 610 EAST LOOP Southbound At CLINTON DR to LAWNDALE
Description	Construction
Lanes Affected	1 main lane, left shoulder
Duration	June 8–October 29, 2012

Figure 3.1 Houston Real-Time Traffic Map.

Figure 3.2 Google Earth Image of IH 610 Work Zone Area.

Pictures of the work zone closure and corresponding traffic control signs are presented in Figure 3.3. The work zone layout is presented in Figure 3.4 according to the field survey conducted on August 16, 2012. Four static signs lead to the work zone, including one variable message sign (VMS) and one flash arrow sign. As shown in Figure 3.4, two "road work ahead" signs are located one mile before the closure on both sides of the roadway, one "left lane closed" sign is placed on the left side a half-mile upstream from the closure, and a "left lane closed" sign, also on the left side, is positioned 1000 feet before the site. The VMS, located one mile upstream from the closure, displays a "left lane closed" message. In addition, a flashing arrow is positioned on the left side of the road at the beginning of the closure.

Figure 3.3 Work Zone Traffic Signs.

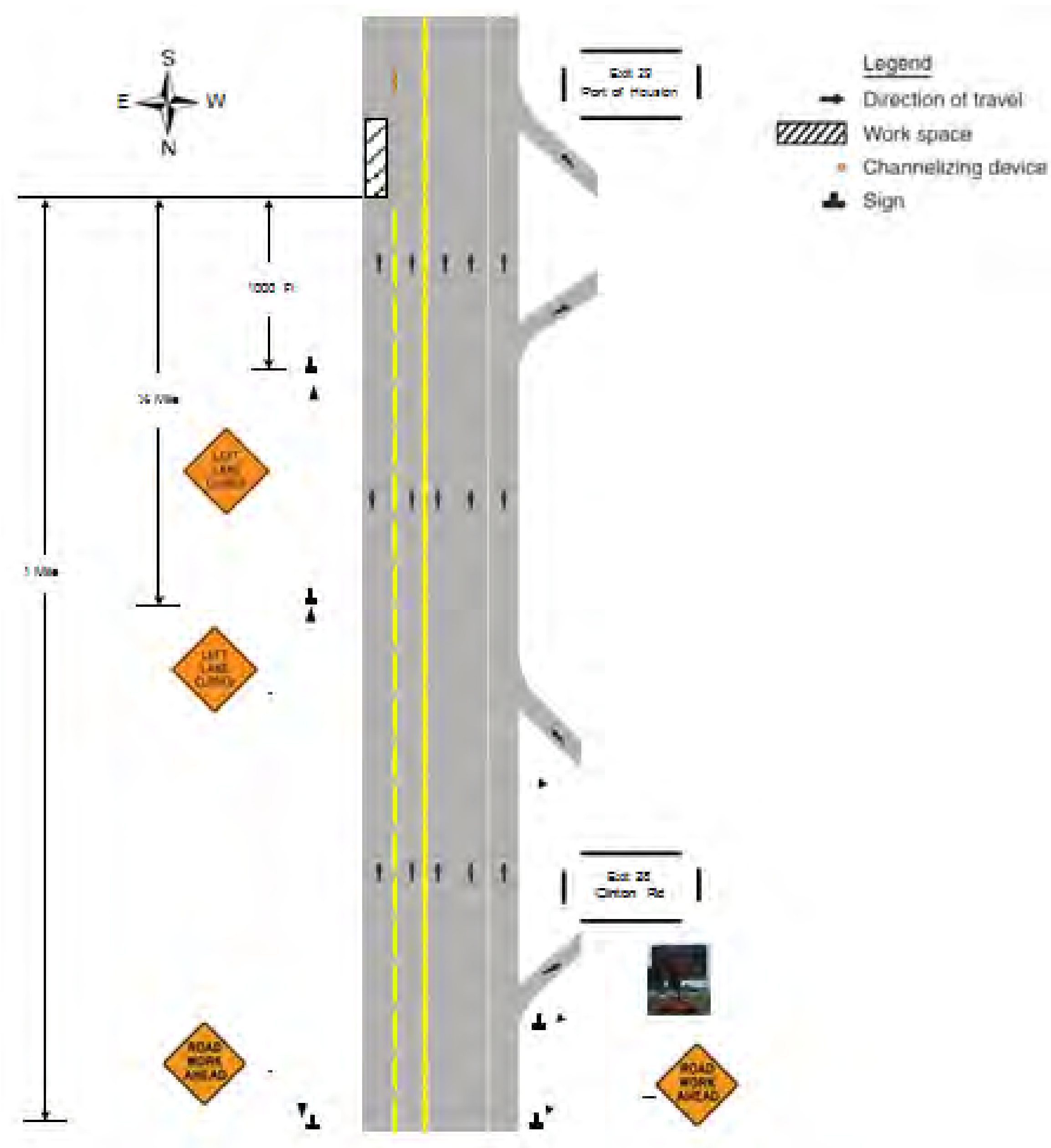


Figure 3.4 Work Zone Layout.

3.2.1.2 Data Collection and Processing

A Houston TranStar real-time surveillance camera was used to record traffic conditions for the study location during the afternoon peak hour between 5:00 p.m. to 6:30 p.m., from June 20 to June 22, 2012. A camera screenshot is shown in Figure 3.5.

Figure 3.5 Screenshot of Houston TranStar Center's Camera View.

The following traffic volume and travel time data were extracted through visual observation. The procedures for processing traffic videos to obtain the above traffic data are presented in the following sections. To record the traffic volume, a scripted program on Microsoft Excel was adopted. The program provided an efficient way to count vehicles passing through the work zone area, by type and lane, at 5-minute intervals. Several buttons were designed for counting different vehicle types like trucks and passenger cars, distinguishing between those on the lane to be closed and those on the remaining lanes.

Software named "Time Machine" was also used to collect travel time information for randomly selected vehicles. Between 7 and 30 vehicles were selected for every 5-minute interval depending on the corresponding traffic volume. To measure travel times, two landmarks were identified in the video recordings: the start and end points. Google Maps was used to define these points for the travel time routes and are shown in Figure 3.6. After that, Time Machine was used to record the travel time of all selected vehicles. Average travel times and speeds per time interval were obtained by post-processing the Time Machine outputs.

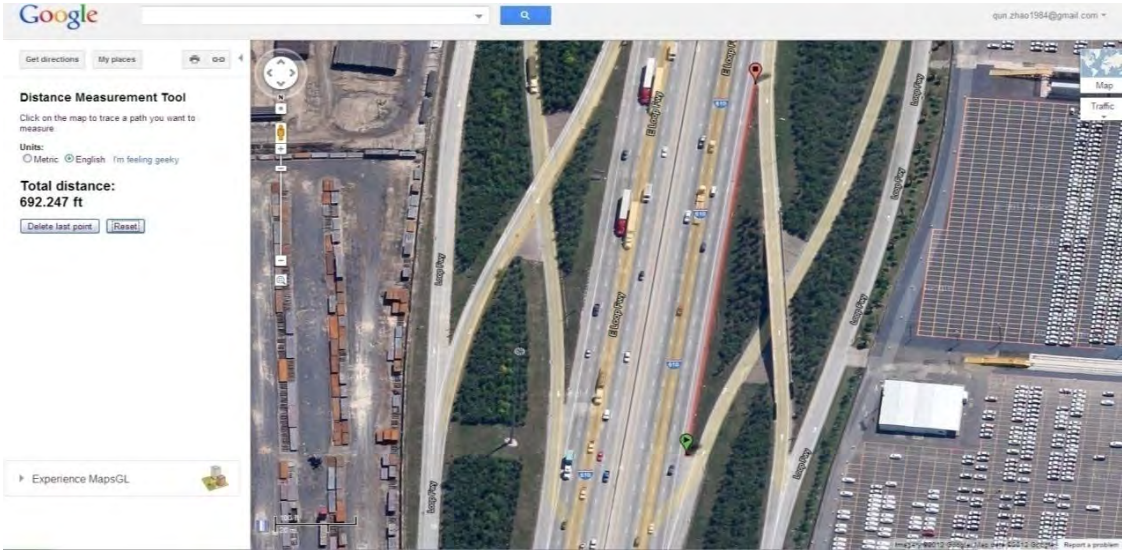


Figure 3.6 Start and End Points for Travel Time Calculations.

The following data were collected at 5-minute intervals for 4.5 hours on each of the 3 days (June 20–22):

- total volume of passenger cars
- total volume of trucks
- volume of passenger cars on lane to be closed
- volume of volume of trucks on lane to be closed
- average travel time average travel speed

Please see Appendix A.1 for the complete dataset.

Figures 3.7–3.10 provide an overview of the observed trends, presenting 15-minute data averaged throughout the analysis period. Figure 3.7 shows the average traffic volume for both passenger cars and trucks. Figure 3.8 displays the same information for late mergers, which are the vehicles that remain on the closure lane after driving past the last work zone warning sign. Figures 3.9 and 3.10 show travel time and speeds, suggesting that 5:45 to 6:00 p.m. is the most congested period.

For relatively low traffic volumes, most vehicles were observed to move away from the lane to be closed before passing the last static "left lane closed" sign. Under more congested conditions, many vehicles changed lanes only after this sign, thus becoming late mergers. Traffic on the lane nearest to the work zone experienced more delay than did the traffic on other lanes. Although these observations and trends are significant to this site, two Austin sites were studied to observe different types of work zone closures.

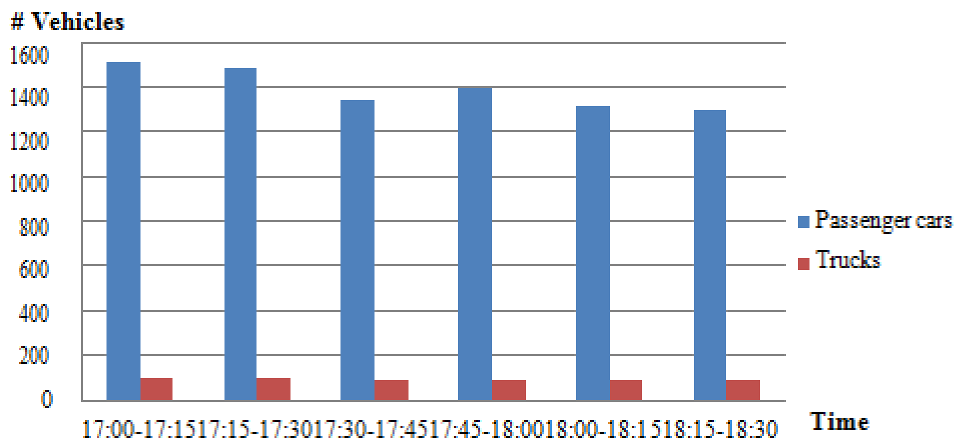


Figure 3.7 Traffic Volume under Work Zone Conditions Averaged across Three Days.

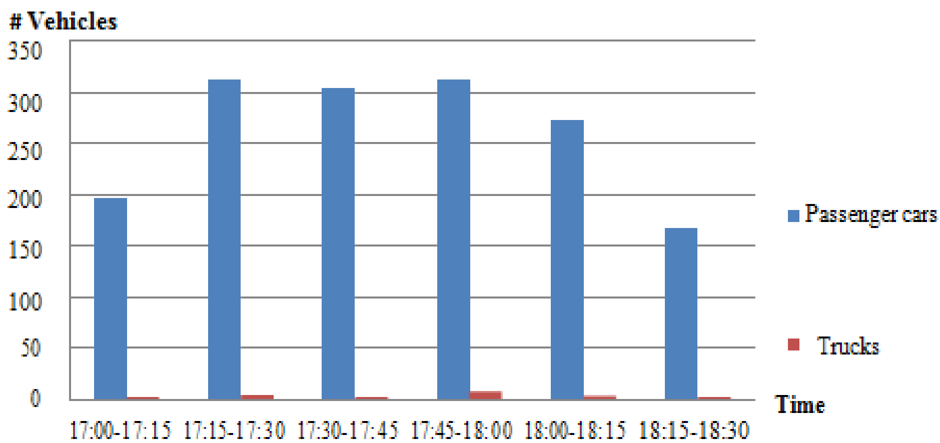


Figure 3.8 Volume of Late Merging Traffic on the Closure Lane under Work Zone Conditions Averaged across 3 Days.

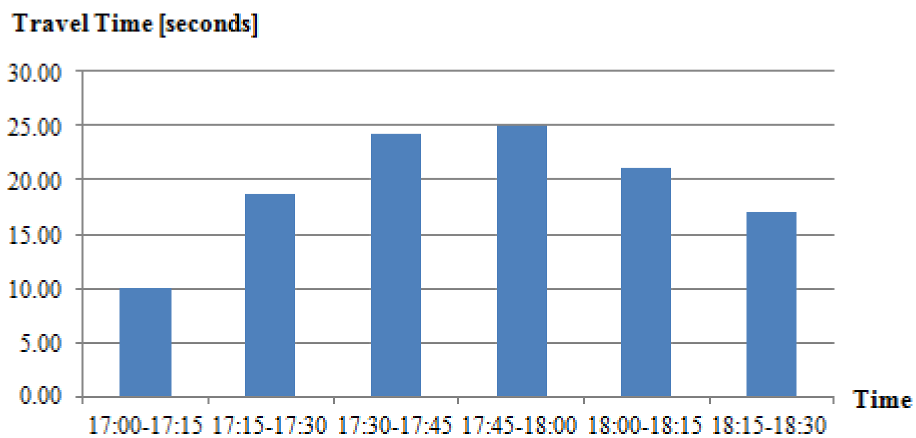


Figure 3.9 Travel Time under Work Zone Conditions Averaged across 3 Days.

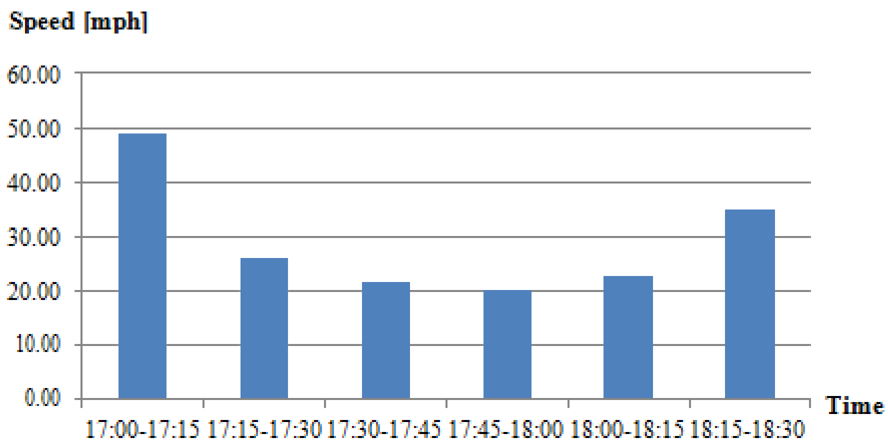


Figure 3.10 Travel Speed under Work Zone Conditions Averaged across 3 Days.

3.2.2 Austin: IH 35

3.2.2.1 Work Zone Location and Layout

Two sites were selected in the Austin area. The first site is located on IH 35 near 51st Street and data was collected on April 23rd and 24th, 2012. During this time the lower deck's left two lanes were closed after 9:00 p.m.

Please refer to Table 3.2 for detailed information about the closure. Figure 3.11 presents a Google Earth image of the analyzed location, where the work zone is indicated using a red line.

Table 3.2 Work Zone Information for IH 35 at Austin Site.

Location	Southbound IH 35 approaching 51st STREET
Description	Construction
Lanes Affected	IH 35 left-side lower deck exit lane closed
Duration	April 23–24, 2012

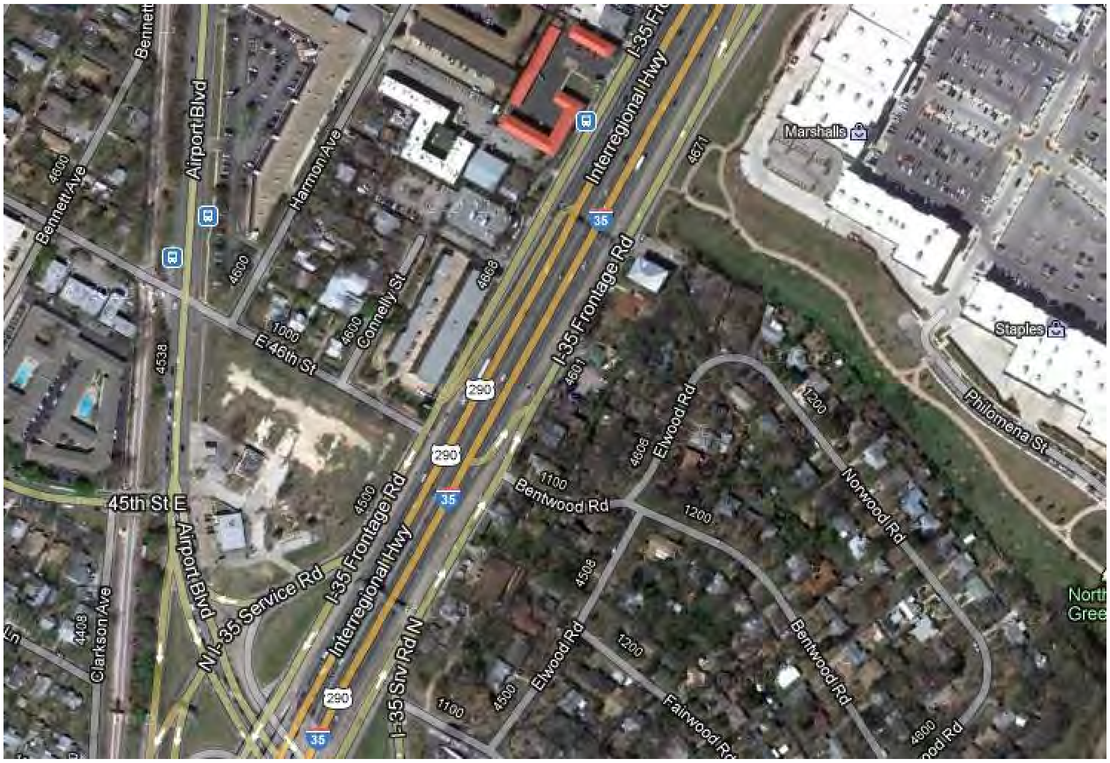


Figure 3.11 Google Earth Image of IH 35 near 51st street, with the Work Zone in Red.

3.2.2.2 Data Collection and Processing

Video data was recorded by TxDOT staff at the Combined Transportation Emergency Communications Center (CTECC). A camera shot is shown in Figure 3.12.

Figure 3.12 Screenshot of CTECC Camera View.

The same software used in the Houston site, Time Machine, was used for collecting traffic volume and travel time data.

The following data were collected:

- total volume of passenger cars entering the analyzed section
- total volume of trucks entering the analyzed section
- total volume of passenger cars exiting on the ramp
- total volume of trucks exiting on the ramp
- average travel time (see Figure 3.13)
- average travel speed

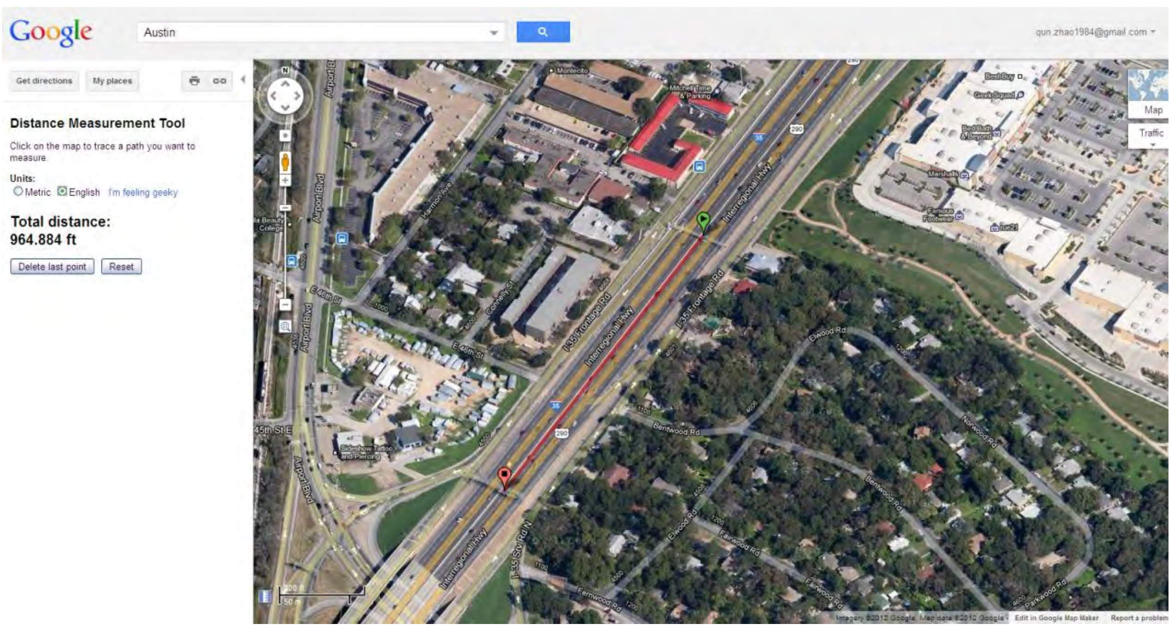


Figure 3.13 Start and End Points for Travel Time Calculations.

Please see Appendix A.2 for a complete dataset of the Austin IH 35 site. The data collected was used to study the diversion rate, defined as 1 minus the ratio of the number of vehicles driving through the work zone area under normal conditions and the corresponding number after the work zone closure is in place.

The percentage of drivers exiting the freeway is used to approximate the diversion rate. The percentage of exiting vehicles is computed as the ratio of vehicles taking the off-ramp immediately upstream from the closure to the total number of vehicles entering the considered section, also referred to as the number of vehicles that remain on the highway plus vehicles that exit. For this site, traffic data was available starting on the first day of the closure, allowing researchers to observe two types of impacts:

- Short-term impacts: these result from the reaction of drivers to the presence of a new work zone, when they do not have enough information to make strategic decisions.

For example, many drivers may initially choose to avoid a new work zone by diverting to alternative routes because they perceive it as a possible cause of delay. However, such decisions may not take into account the possibility of finding worse congestion on alternative routes.

- Long-term impacts: in the long term, drivers have an opportunity to experience both the travel time associated with driving through the work zone and the travel time on alternative routes. Based on their experience, they can make strategic decisions, including major changes to their route. The resulting diversion rate is not only influenced by the work zone area capacity, but also by the local streets' capacity and signal control. The new routing decisions are assumed to lead to a new user equilibrium condition. In this case, the only determination factor on the flow pattern is the travel cost.

Figure 3.14 illustrates the first type of effects. It presents the percentage of exiting vehicles during the first 2 days of road work and compares it to the corresponding value during recurrent or normal conditions.

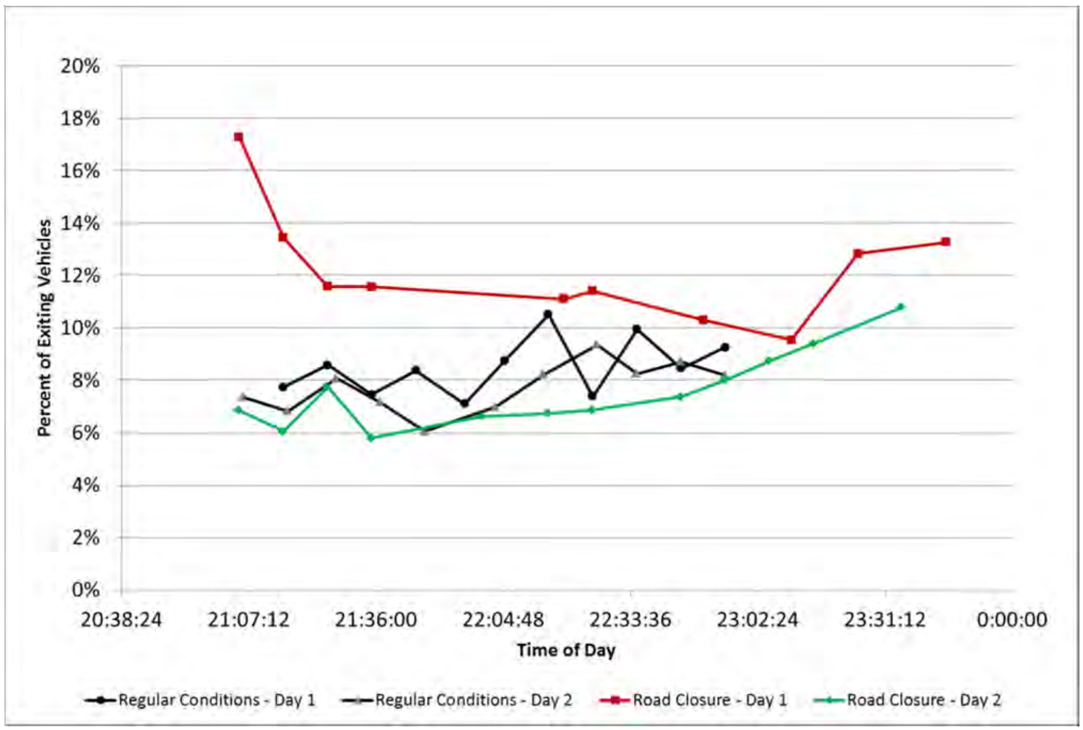


Figure 3.14 Percentage of Exiting Vehicles.

The percentage of exiting vehicles is observed to increase considerably during the first closure day, going back to its original value on the second day. This finding suggests that drivers are likely to initially overreact to the presence of an unexpected closure by exiting the freeway in the hope of avoiding delays. However, as they become more familiar with the actual traffic conditions through the work zone and on alternative routes, some travelers may reassess their decision and return to their original path. It is interesting to notice that during the second day with the work zone in place, the percentage of exiting vehicles is slightly lower than under normal conditions. While this reduction may be a due to regular daily variability, it can also be a consequence of traveler's strategic decision-making process: some drivers normally exiting on the considered ramp may be taking alternative paths to avoid additional delays due to work zone traffic. This theory is supported by the data presented in Figure 3.15, which suggests a small reduction in the total demand through the work zone area. The average diversion rates are presented in Table 3.3, which follow the same trend as the percentage of exiting vehicles and decrease considerably on the second day of road work.

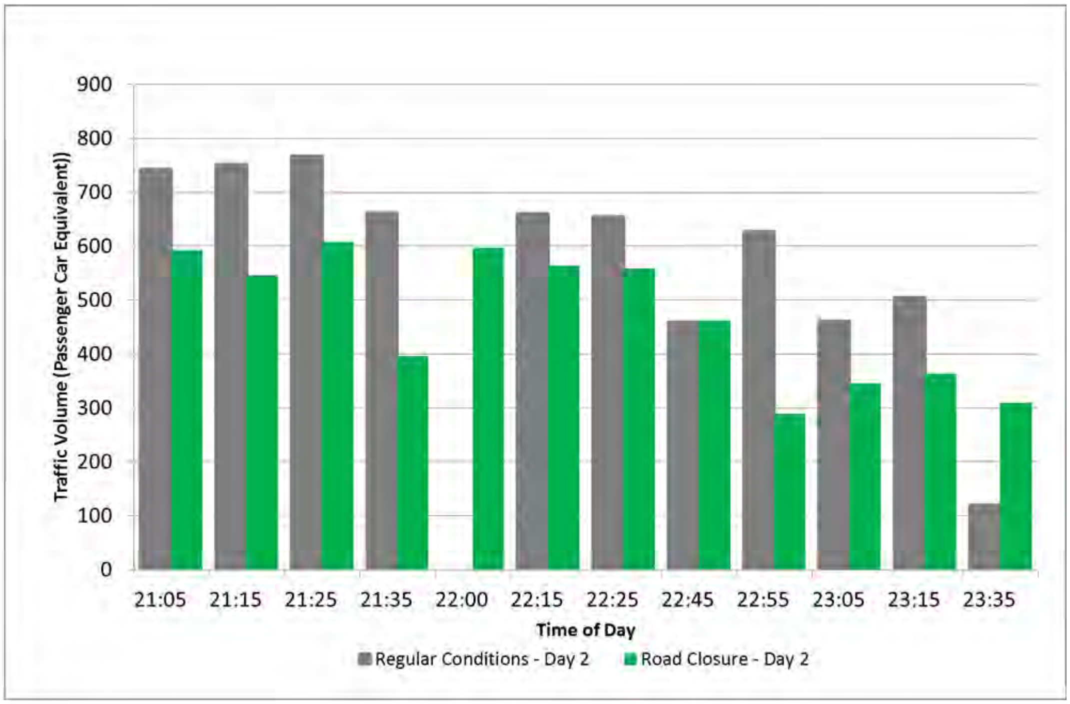


Figure 3.15 Total Traffic Volume into the Work Zone Area.
Note: A truck is assumed to be the equivalent of two passenger cars.

Table 3.3 Average Diversion Rate (9 p.m.–11 p.m.).

	Traffic through Work Zone Area (passenger car equivalent)	Diversion Rate
Regular Conditions (Avg.)	6179	0%
Road Closure - Day 1	2497	60%
Road Closure - Day 2	4609	25%

3.2.3 Austin: Oltorf Street

3.2.3.1 Work Zone Location and Layout

The second site in Austin where data was collected is located at the intersection of westbound (WB) Oltorf Street and IH 35. Oltorf's WB through lane was closed all day during the considered period, and drivers headed in that direction were directed to make a left turn at the intersection. Please refer to Table 3.4 for detailed information about the closure. Figure 3.16 presents Google Earth images of the analyzed location, with the work zone indicated using a red line.

Table 3.4 Work Zone Information for Oltorf Street in Austin Site.

Location	Westbound OLTORF STREET at IH 35
Lanes Affected	Westbound through lane closed
Description	Construction
Duration	July 24–26, 2012

Figure 3.16 Google Earth Image of Oltorf Street near IH 35, with the Work Zone in Red.

3.2.3.2 Data Collection and Processing

Data was collected at the site using video recorded by City of Austin Traffic Management Center staff, as shown by Figure 3.17.

Figure 3.17 Screenshot from City of Austin Traffic Management Center Camera.

For the Oltorf Street site, only traffic volume was extracted. The same software was used for collecting traffic volume as for Houston site. For each turning movement examined, the total volumes of passenger cars and trucks were collected separately in 5-minute intervals.

Please see Appendix A.3.1 and A.3.2 for a complete dataset of the Austin Oltorf Street site.

This site is unique compared to the first two sites in that Oltorf Street was completely closed. Therefore, the goal of the data analysis for this site was to determine if the recommended detour was used as intended. Initial results show that the detour was indeed used as intended. Figure 3.18 depicts the 15-minute volumes of passenger cars headed southbound on the IH 35 frontage road and turning right onto Woodward Street. The turning movement volumes are clearly higher when the work zone is in place.

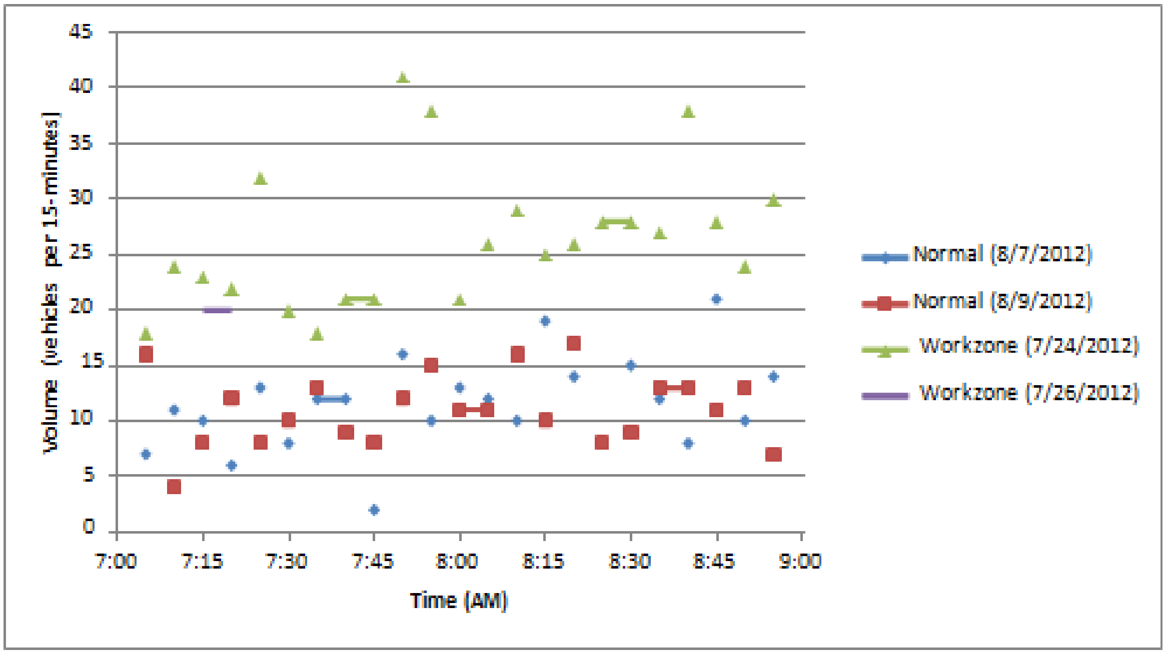


Figure 3.18 Right-Turn Detours from Southbound IH 35 Frontage Road to Woodward Street.

The data collected provides interesting insights into drivers' route choice mechanism and lane-changing behavior around road closures. At the Houston site, drivers were observed to react to work zone warning signs by merging early under relatively uncongested conditions. However, in higher levels of congestion, more drivers adopted a late merge strategy. The Austin site on IH 35 at 51st Street was used to analyze the evolution of driver's route choice process around long-term work zones. Travelers were observed to initially overreact to the presence of the work zone, and eventually return to their original paths for the considered scenario. A diversion rate of 25% was observed in the second day of road work. At the Oltorf Street site, the detour guidance was found to be effective. The collected data will be used to assess the performance of microsimulation and dynamic traffic assignment (DTA) models, and to inform the selection of some modeling parameters. From these three sites, observations were made and data was collected in order to perform simulations to analyze user routes, work zone impact on network performance, and safety, among other topics.

3.3 Design of Simulation Experiments

Experiments conducted in this project include three parts: network-level simulation, microsimulation, and safety modeling. In network-level simulation, DTA models will be used to describe travelers' route changing behavior. The DTA models will take the travel demand as input and will output the number of vehicles flowing through the work zone, allowing for an analytical calculation of the diversion rate. The flow through the work zone will be fed into the microsimulation model where detailed performance measures such as delay, speed, and lane changing will be characterized. These measures will be compared with similar measures obtained from DTA, where available, for calibration purposes. Lastly, the vehicle trajectories output from microsimulation will be input into the safety model where measures of safety (e.g., predicted crash rate) will be obtained. Figure 3.19 illustrates this integrated framework.

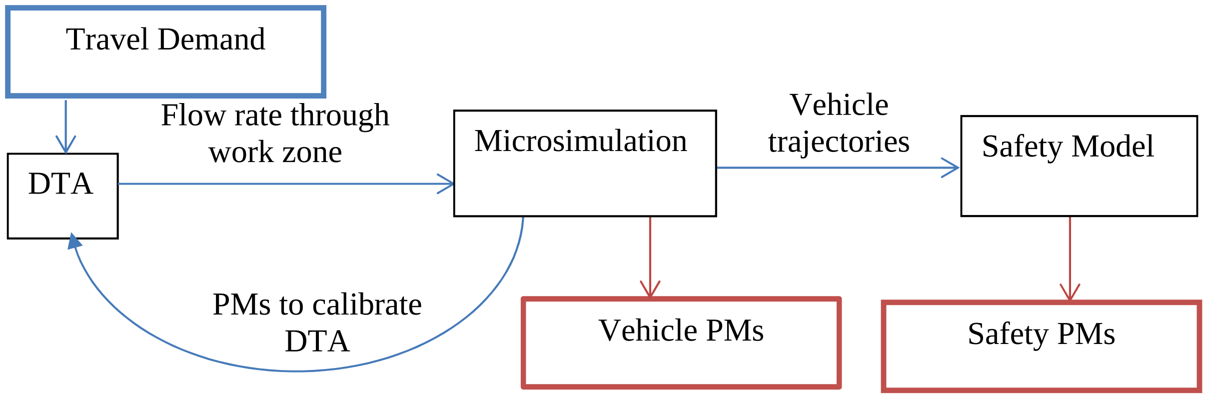


Figure 3.19 Model Integration for Experiment Scenarios.

The scenarios to be tested by the experiments are outlined in the following sections. Since microsimulation models capture a finer level of detail than do the DTA models, the scenarios analyzed by these two model types are presented separately. The scenarios run in DTA will focus on estimating diversion rate, whereas microsimulation will be used for a wider range of objectives. No scenarios are presented specifically for the safety model, but the researchers plan to run the output from each microsimulation scenario into the safety model to evaluate how the safety performance measures varied as different variables are changed.

The goal of the experiments was to gather enough information to construct a decision tree. As stated in the project proposal, the "decision tree process that includes all potentially significant factors will be derived to guarantee inclusion of the many issues and development of an optimal control scheme for each unique work zone." Field data gathered from the work zones will be processed through the microsimulators, which will in turn be used to calibrate the DTA models. The DTA scenarios will be discussed in detail in Section 3.3.1 and the microsimulation scenarios will be discussed in detail in Section 3.3.2.

3.3.1 Dynamic Traffic Assignment Scenarios

3.3.1.1 Framework

The number of vehicles driving through a long-term work zone location is typically lower than the corresponding volume under typical conditions. The former is a result of drivers learning about the presence of the disruption over time and adjusting their routes accordingly, often through diversion. DTA models are capable of capturing the long-term response of drivers to the presence of a work zone, and therefore can provide an estimate of the expected traffic volume in the area of interest.

DTA models can output meaningful predictions of the diversion rate under different work zone scenarios due to the explicit consideration of traffic signals and the capability to adjust the geometric characteristics of links to reflect the presence of disruptions such as work zones, all of which leads to meaningful predictions of the diversion rate under different conditions. However, drivers' behavior, including lane changing, is typically not modeled in the DTA context, and microsimulation approaches provide a better framework to compare the traffic conditions under various work zone management strategies. This is why the DTA model will be used to estimate the diversion rate and the microsimulation model will be used to evaluate how the work zone operates given the level of traffic flowing through it (typical flow minus diverted flow).

While waiting for an appropriate test site to collect field data, tests of the modeling framework were conducted using the shaded subarea shown in Figure 3.20. The Network Modeling Center at UT's Center for Transportation Research has already developed a five-county DTA model of the Austin area consistent with the Capital Area Metropolitan Planning Organization's regional planning model, and the subarea selected is part of this region in Williamson County. As shown in Figure 3.20, the fictitious work zone was modeled on eastbound State Highway 45 on the section between Parmer Lane and FM 620.

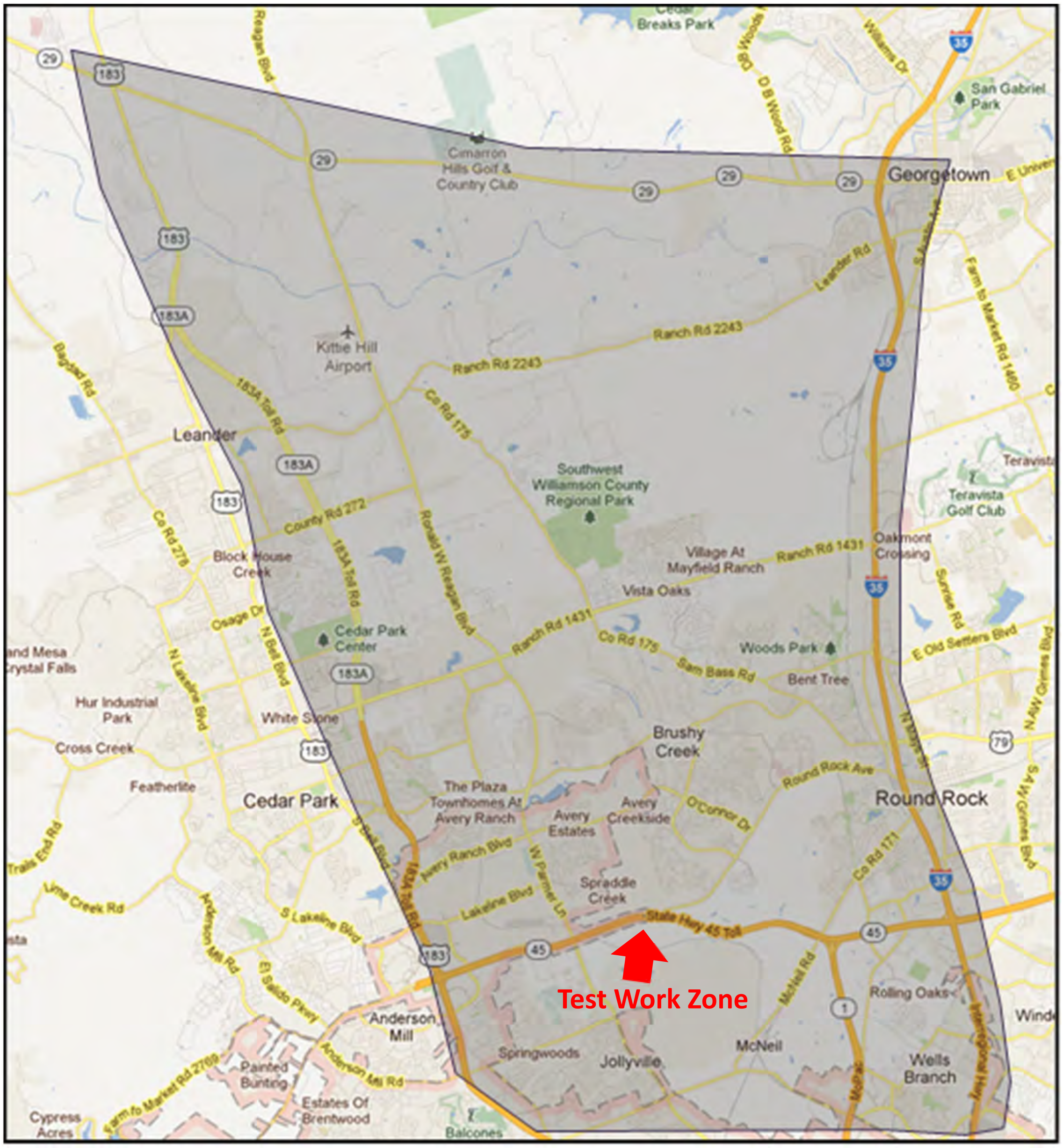


Figure 3.20 Subarea Considered in DTA Experiments.

3.3.1.2 Experiments

The DTA experiments were focused on identifying route changing to avoid the work zone location (diversion rate) under different conditions. Results were compared to the diversion rates estimated from the simpler methodologies recommended by the Federal Highway Administration (FHWA), i.e., "Impact Analysis Tools" (2011). The modeling exercise ultimately led to the development of a decision tree that can be used to predict diversion rates when DTA modeling is not an option due to time constraints.

Table 3.5 displays the set of proposed numerical tests involving DTA modeling. The first parameter to be tested, the ratio of volume to work zone capacity, is the relationship between the "pre-work zone" volume using the link affected by the work zone and the capacity of the lanes that remain open during the road work. As this ratio gets higher the traffic conditions at the work zone worsen and a larger diversion rate is expected. The second and third variables in Table 3.5 aim at capturing the topological network characteristics most likely to affect the diversion decision. Alternative routes, including frontage roads and other alternative streets, provide an opportunity for drivers to take a detour that avoids the work zone altogether. The availability and characteristics of these routes, such as length, speed limit, average traffic conditions, and the presence of traffic signals, are expected to have an impact on the number of drivers that choose to divert. Researchers will identify the minimum requirements for an alternative road to be considered viable—for example, located within one mile of the work zone. All the combinations defined in Table 3.5 may be conducted for one or more of the alternative work zone management strategies considered in this project, and for each of the possible scenarios considered in the microsimulation experiments. Runs were prioritized to ensure the best outcome for the decision tree.

Table 3.5 Proposed DTA Experiments per Microsimulation Scenario.

Ratio of Volume to work zone capacity	Non-frontage alternative streets	Frontage Road
1.5	Yes	Yes
		No
	No	Yes
1	Yes	Yes
		No
	No	Yes
0.8	Yes	Yes
		No
	No	Yes

3.3.2 Microsimulation

3.3.2.1 Framework

Microsimulation models operate on small time increments of one second or less and capture detailed vehicle behavior such as lane-changing. Therefore, the outputs from such models are detailed vehicle-level measures such as delay and queue length over time. In this study, the microsimulation model was a section of roadway containing the work zone. Figure 3.21 shows an example of such a section that was used by the research team for initial testing. The flow rate into the segment was determined using DTA modeling as described in the previous section and the outputs were used as input into a safety model.

While waiting for field data to be collected, the researchers created a 2.8-mile simulated environment to evaluate the modeling of work zones in CORSIM and VISSIM. Within this segment, the speed is gradually reduced from 65 miles per hour (mph) to 45 mph before encountering a signalized control device. Users are guided into a one-mile work zone one lane at a time. Following the lane reduction, traffic has a 750-feet region in which to resume normal operations. This design scheme was compared to the base case with normal conditions and calibrated with field data. Figure 3.21 shows the work zone environment as it would appear for a 4-to-3 lane configuration. Table 3.6 shows the simulation design matrix.

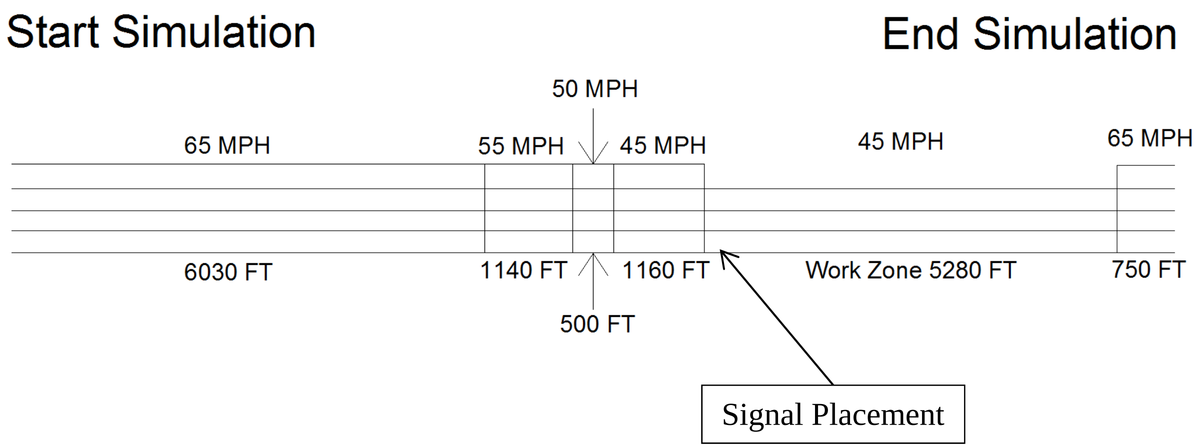


Figure 3.21 Microsimulation Environment.

3.3.2.2 Experiments

A long list of scenarios was identified for the purpose of experimentation and is given in Table 3.6. If patterns emerged among the scenarios, then the researchers condensed the number of scenarios as appropriate. All of the scenarios listed focus on signalized merge control since this has not yet been extensively studied. Other types of merge control may be considered necessary to create the decision tree. For example, static late merge has been shown to have a significant impact on reducing congestion in Pennsylvania and Minnesota.

Table 3.6 Proposed Microsimulation Scenarios.

No.	Lane Change Type	Speed Condition	Cycle Lengths (sec)	Vehicle Mix
1	3 to 1	Normal	30	No Trucks
2				Low HV
3				High HV
4			60	No Trucks
5				Low HV
6				High HV
7			90	No Trucks
8				Low HV
9				High HV
10			Ramp Metering	No Trucks
11				Low HV
12				High HV
13		Reduced	30	No Trucks
14				Low HV
15				High HV
16			60	No Trucks
17				Low HV
18				High HV
19			90	No Trucks
20				Low HV
21				High HV
22			Ramp Metering	No Trucks
23				Low HV
24				High HV

Table 3.6 (cont.) Proposed Microsimulation Scenarios

No.	Lane Change Type	Speed Condition	Cycle Lengths (sec)	Vehicle Mix
25	3 to 2	Normal	30	No Trucks
26				Low HV
27				High HV
28			60	No Trucks
29				Low HV
30				High HV
31			90	No Trucks
32				Low HV
33				High HV
34			Ramp Metering	No Trucks
35				Low HV
36				High HV
37		Reduced	30	No Trucks
38				Low HV
39				High HV
40			60	No Trucks
41				Low HV
42				High HV
43			90	No Trucks
44				Low HV
45				High HV
46			Ramp Metering	No Trucks
47				Low HV
48				High HV

Various work zone lane configurations are possible and the research focused on the most common scenarios as determined by the Project Monitoring Committee (PMC): three lanes reduced to two, and three lanes reduced to one. A "normal" speed and a "reduced" speed scenario were considered to mimic the conditions when a work zone speed is in place and enforced, and when the standard speed limit has not changed. Reduced speed scenarios can also mimic poor lighting and weather conditions.

As mentioned earlier, the scenarios in Table 3.6 reflect the "signalized merge" condition so the cycle lengths, reported in seconds, refer to this signal head installed at the merge point. Initial testing indicates that the optimal cycle length depends on the flow rate. The last parameter listed in the table of scenarios is the mix of vehicles traversing the roadway segment. Trucks take up more roadway capacity than passenger cars and may also require a longer cycle length. The researchers received input from the PMC on the vehicle mix scenarios most appropriate for TxDOT roadways.

Other parameters mentioned in the project proposal were incorporated into the experimental design, including varying the position of the closed lanes, varying the site geometry, and offering detour guidance using static or variable signs placed at different locations.

For each microsimulation scenario, the researchers documented various performance measures, including distributions of speed, travel time and delay, maximum queue length, and total throughput in the simulation time.

Chapter 4. Experiments

4.1 Application of Dynamic Traffic Assignment

Work zones that last multiple days, which are the focus of this project, give drivers a chance to adjust to the new conditions. In this context some drivers may choose to use alternate routes that avoid the work zone area altogether. The actual number of drivers opting to avoid the closure is likely to be a function of the closure type and corresponding lane control measures, as well as the characteristics and availability of alternate routes. In general, neglecting the diversion of vehicles to alternate routes may overstate the anticipated congestion in the work zone area. While most previous research assumes a fixed percentage of drivers will divert, this research showcases the use of dynamic traffic assignment (DTA) to provide a location-specific prediction of the expected diversion rate, leading to more realistic assessment of the impacts of a work zone on the affected highway and nearby network.

As discussed in Chapter 3, traveler diversion to alternative routes may serve to relieve traffic congestion in the vicinity of the work zone, which is expected to improve safety. Conversely, alternate routes may become congested. DTA models may be used to identify areas where congestion may worsen as a result of driver diversion from the work zone area, and to develop traffic management strategies to mitigate such effects.

The VISTA (Visual Interactive System for Transport Algorithms) DTA modeling platform is used for this task. DTA assumes that drivers seek to minimize their travel time (or a more general measure of travel cost). Over time, drivers departing within a specific time learn about the costs on all alternative routes between their origin and destination, and they distribute themselves in such a way that costs are equal on all used routes. This is considered an “equilibrium” condition. The DTA approach has the capability to model corridors or even large regions while explicitly considering the peaking nature of traffic demand and can provide performance measures on a disaggregate level, ranging from 15 minutes to even 6 seconds. Consideration of traffic signals, incidents, and information provision via changeable message signs are all within the capabilities of the DTA model.

DTA provides an excellent tool to analyze work zone impacts from a variety of different perspectives, but some agencies may not yet have access to regional-level DTA models. While the adoption of DTA models is expected to increase in the coming years, planning agencies may need to resort to simpler approaches to estimate driver diversion rates. Other popular methodologies that do not involve modeling are not capable of fully capturing the impact of network connectivity, traffic signal timing plans, and other location-specific characteristics. Thus, modeling-based approaches are always recommended when available.

The remainder of this chapter is organized as follows. Section 4.2 describes the DTA modeling approach and experimental design scenarios. Section 4.3 describes the modeling results and the comparison of diversion rates calculated by different tools. Section 4.4 focuses on outputs from microsimulators like CORSIM and VISSIM.

4.2 DTA Modeling Approach and Scenarios

This study focuses on work zones in an urban network. Under this condition, multiple alternative routes are available upstream of the work zone, and at least one alternative route is available when people reach the work zone link. An example is shown in Figure 4.1. Area A is the upstream area of the work zone and it provides multiple alternative routes to travelers. Travelers switching to other routes on this area affect the performance of the whole network. Area B is the work zone area. When travelers enter area B, one alternative route remains that is the last option to avoid the work zone. The goal is to understand diversion behavior in both areas A and B. Therefore, diversion rates on two levels are defined. Network-level diversion represents how many travelers avoid entering the work zone area choose other arterial streets or expressways. It can describe the change of demand in the work zone area. Another type of diversion rate focuses on the local level or the work zone area. It can be used to find the percentage of travelers who keep their original route through the work zone link when they choose to enter the work zone area. Agencies and contractors can use this type of diversion rate to provide appropriate guidance as to the number of lanes to close.

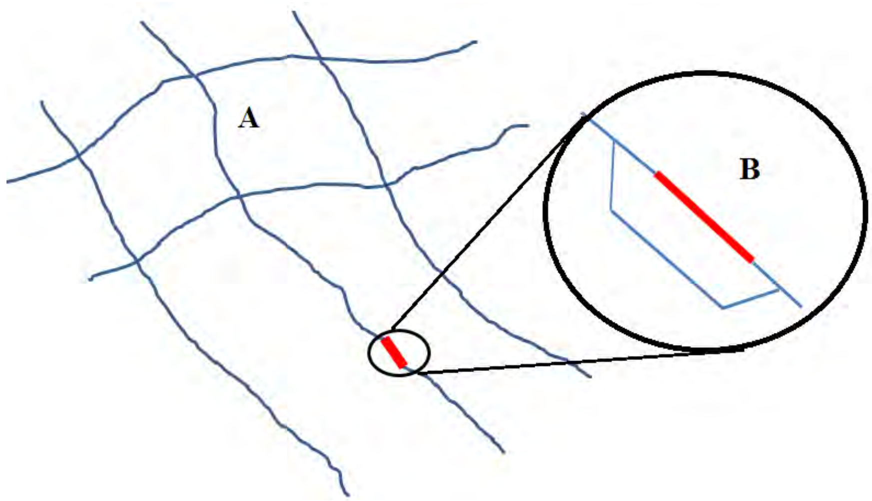


Figure 4.1 Example of a Work Zone in an Urban Network.

This report uses the work zone on IH 35 in Austin as the case study. Figure 4.2 shows the work zone location and the links whose flow may be impacted by the work zone condition. The work zone area lies downstream of the intersection of Airport Blvd and IH 35 southbound in Austin's downtown area. Upstream of the work zone area, the IH 35 lanes are separated into two groups: the lower deck and the upper deck. The work zone is located on the lower deck, which is represented by the red line. During work zone operations, the lower deck work zone area is fully closed.

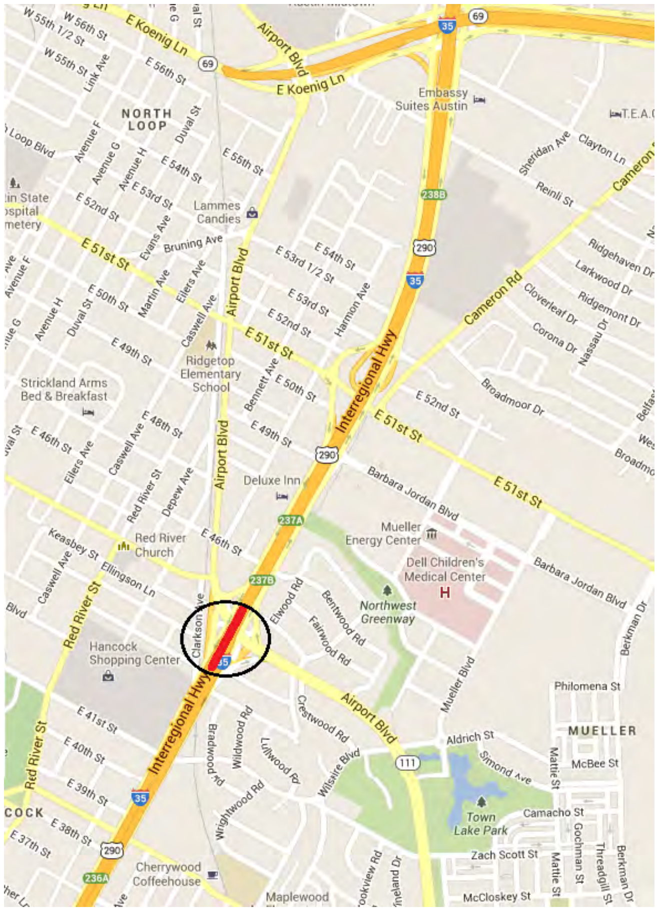


Figure 4.2 Subarea Considered in DTA Experiments.

4.2.1 Modeling Tools

This study uses VISTA as the DTA tool and TransCAD as the static traffic assignment (STA) tool. VISTA is a simulation-based DTA software. It can simulate dynamic user equilibrium based on the cell transmission model (CTM) with extensions for signalized intersections. The CTM was developed by Daganzo. It is a discrete version of the Lighthill-Whitham-Richards hydrodynamic traffic flow model. Each network link is divided into several cells, and the number of vehicles in each cell will be tracked on all iterations, where 6 seconds is used in VISTA. The sending flow of a cell is the number of vehicles that can leave the cell if there is no downstream restriction, and the receiving flow is the number of vehicles that can enter if there is an infinite source. The maximum number of vehicles in each cell and the maximum flow that can be sent from one cell to next are determined from the capacity, density of each network link, and the length of each cell. The most important feature of CTM is that the total number of vehicles in each cell cannot exceed finite limits. Instead, queues will be formed, which is an advantage of CTM over STA.

TransCAD, developed by Caliper Corporation, is a transportation planning software combined with GIS. One function of TransCAD is performing STA. It provides multiple assignment methods, including all or nothing, incremental assignment, capacity restraint, user equilibrium, and system optimum. The Bureau of Public Roads (BPR) function is used to compute link cost, with user-set values of the parameters α and β. The convergence criterion is based on maximum absolute change in link flows between iterations. When maximum absolute change is smaller than the preset threshold, the algorithm will stop. This study chose standard BPR functions to describe link performance and used user equilibrium assignment as the assignment method.

4.2.2 Modeling Approach

Figure 4.3 illustrates the topology of the work zone area links. Link 2 is the work zone link.

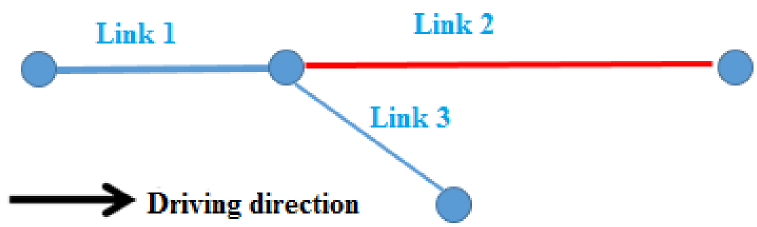


Figure 4.3 Topology of the Work Zone Area Links.

If people choose alternative routes, they will not enter Link 1. Therefore, the change in volume on Link 1 represents the diversion rate on the network level. The change in volume on Links 2 and 3 indicate the diversion rate on the local level.

Equation 4.1 is used to compute network-level diversion:

$$\mathrm{DN_i} = \frac{\mathrm{V_{regular,i}} - \mathrm{V_{workzone,i}}}{\mathrm{V_{regular,i}}} \qquad \text{(Equation 4.1)}$$

where $\mathrm{DN_i}$ is the diversion rate on link i, $\mathrm{V_{regular,i}}$ is the volume on link i under normal conditions, and $\mathrm{V_{workzone,i}}$ is the volume on link i under work zone conditions.

Equation 4.2 is used to compute the diversion rate on the local level:

$$\mathrm{DL_i} = \mathrm{R_{regular,i}} - \mathrm{R_{workzone,i}} \qquad \text{(Equation 4.2)}$$

$\mathrm{DL_i}$ is the diversion rate on Link 2 or Link 3, $\mathrm{R_{regular,i}}$ is the ratio of the volume on link i to the volume on Link 1 under normal conditions, and $\mathrm{R_{workzone,i}}$ is this ratio under work zone conditions.

The process of calculating diversion rates using DTA and STA tools has three steps:

1. Run the DTA or STA model on a network under normal conditions. When the network reaches equilibrium, record the volume on links that relate to the diversion rate.
2. Add the work zone to the appropriate network link(s) and run the models with the same demand again. When the network reaches the equilibrium condition, record the appropriate link volumes.

3. Use Equations 4.1 and 4.2 to compute network-level and local diversion rates.

Figure 4.4 illustrates the topology of a southbound IH 35 work zone. Links 1 and 2 lie upstream of the work zone and Links 3 and 4 denote the lower level and upper deck of IH 35. Link 3 is the work zone and Link 5 is an off-ramp of IH 35. Travelers can use the Link 5 off-ramp to avoid the work zone, choosing the alternative route provided by the frontage road or choose another alternative route. The diversion rate on Link 1 is used to represent network-level diversion rate, while the diversion rate on Link 2 is used to describe the local-level diversion rate. DTA uses time-dependent demand and it has warm-up and clearing periods, which are the periods that begin to load vehicles on the network and let all vehicles leave the network, respectively. So we should calculate diversion rates from DTA based on the stable condition, which excludes warm-up and cooling down periods.

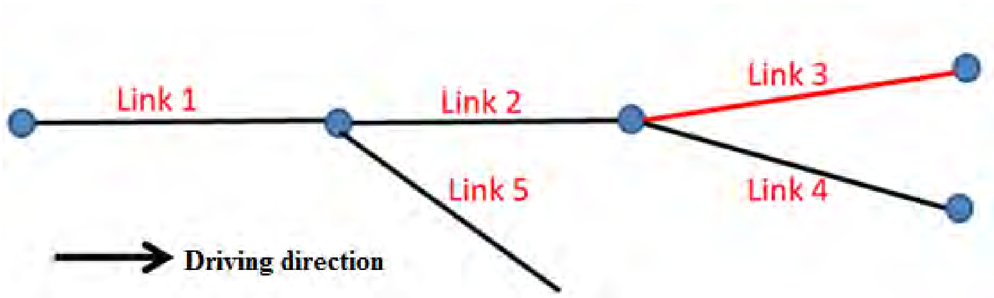


Figure 4.4 Topology of the Work Zone Area.

4.2.3 Scenarios Modeled

The case study is based on two scenarios: the morning peak period from 7 a.m. to 9 a.m. and the off-peak period from 9 p.m. to 6 a.m. The demand profile is shown in Figure 4.5. Only morning peak demand is already provided by the regional network. Off-peak period demand is obtained based on the demand profile and morning peak period demand. This method uses a scale factor representing the ratio of off-peak period demand to morning peak demand, and then uses this factor to convert morning peak demand to the off-peak demand. Equation 4.3 is used to compute this scale factor.

$$f = P_{off\ peak}/P_{morning\ peak} \qquad \text{(Equation 4.3)}$$

where f is the scale factor, $P_{\text{off peak}}$ is the percentage of daily total demand in the off-peak period, and $P_{\text{morning peak}}$ is the percentage of total daily demand in the morning peak.

According to assignment results, the selected stable condition in the morning period is from 8 a.m. to 9 a.m. The volume in the off-peak period has two phases: the congested Phase 1 from 9 p.m. to 12 a.m. and the uncongested Phase 2 from 12 a.m. to 6 a.m. The selected stable condition on Phase 1 is from 9:30 p.m. to 10:30 p.m. and the Phase 2 stable condition is from 2 a.m. to 3 a.m.

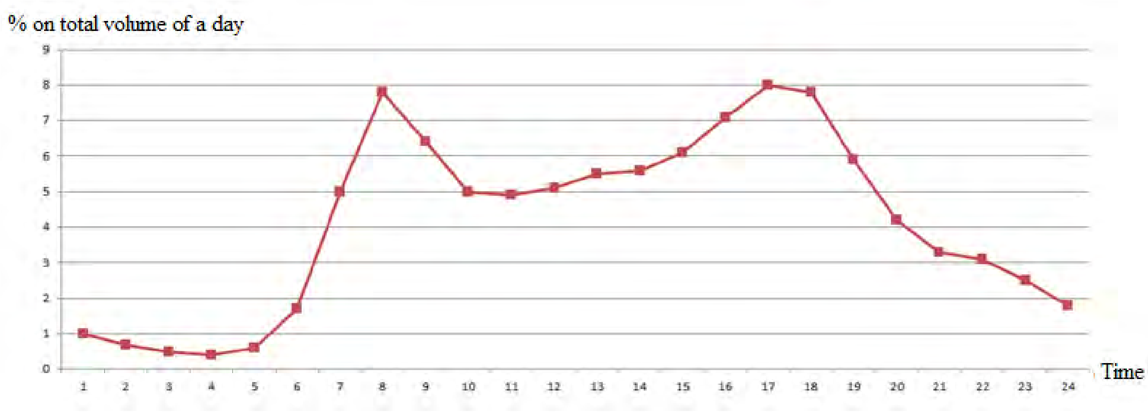


Figure 4.5 Demand Profile of a Day.

4.3 Analysis of Results

4.3.1 Comparison between Dynamic Traffic Assignment and Static Traffic Assignment

Tables 4.1 and 4.3 show the flow assignment results from STA for morning peak and off-peak periods, respectively. Table 4.2 and 4.4 show the results from DTA for morning peak and off-peak periods, respectively.

Table 4.1 Results from STA-TransCAD for Morning Peak Period.

Morning peak average hourly demand by STA				
	Normal		**Work zone**	
Link ID	**Total**	**Per-lane**	**Total**	**Per-lane**
1	18940	4735	14912	3728
2	17296	4324	11160	2790
5	1647	1647	3755	3755

Table 4.2 Results from DTA-VISTA for Morning Peak Period.

Morning peak average hourly demand by DTA				
	Normal		**Work zone**	
Link ID	**Total**	**Per-lane**	**Total**	**Per-lane**
1	6472	1618	3500	875
2	5860	1465	2800	700
5	612	612	697	697

Table 4.3 Results from STA-TransCAD for Off-Peak Period.

Off -peak average hourly demand by STA				
	Normal		Work zone	
Link ID	Total	Per-lane	Total	Per-lane
1	2488	622	2372	593
2	2196	549	1852	463
5	292	292	520	520

Table 4.4 Results from DTA-VISTA for Off-Peak Period.

Off-peak average hourly demand by DTA								
Phase 1					Phase 2			
	Normal		Work zone		Normal		Work zone	
Link ID	total	per lane	total	per lane	total	per lane	total	per lane
1	6616	1654	3958	990	2352	588	2110	528
2	5281	1320	2748	687	2243	561	1490	373
5	1335	1335	1210	1210	109	109	620	620

Based on the flow assignment results, the diversion rates for both scenarios were obtained. Tables 4.5 and 4.6 show the diversion rates on morning peak and off-peak periods, respectively.

Table 4.5 Diversion Comparison during Morning Peak Period.

	Link ID	STA	DTA
Network level	1	21%	46%
Local level	2	16%	11%

Table 4.6 Diversion Comparison during Off-Peak Period.

	Link ID	STA	DTA	
			Phase 1	Phase 2
Network level	1	9%	40%	10%
Local level	2	10%	10%	25%

As Table 4.5 indicates, during the peak hour, the network-level diversion rate from STA is smaller than that from DTA. This is reflected by the comparison to Link 1. Notice that, when a work zone is present, DTA shows that more people will try to avoid entering the work zone area and thus fewer people will enter Link 1. The work zone area has less congestion, so fewer people will switch to alternative routes. Therefore, diversion rates on the local level from DTA are smaller than the ones from STA. Actually, the result from DTA is reasonable: in a long-term work zone scenario, many vehicles will transfer to other routes at the network level instead of passing through work zone area since users know there will be congestion in the work zone area. Meanwhile, because DTA cannot allow link volumes to exceed the capacity of this link, DTA is more likely to assign travelers to other alternative routes than STA will when downstream is congested. So the total number of vehicles that get to the work zone area using DTA will drop dramatically. Table 4.6 reflects a similar trend in network-level diversion. DTA allows more vehicles to use alternative routes during both phases at the network level. Demand in Phase 1 is much higher than in Phase 2, so the work zone area becomes more congested in Phase 1 if all travelers pass through it. For this reason, the diversion rate in Phase 1 is higher than in Phase 2. Diversion rates on the local level from DTA are greater than or equal to STA since more people already choose alternative routes at the network level. During off-peak hours, when there is a work zone on the main lanes, drivers know that they do not need to worry about congestion on local streets. Therefore, most vehicles would prefer to leave the highway and divert to local streets because they know that they can avoid passing through the more "dangerous" work zone area without suffering congestion.

4.3.2 Comparison between Dynamic Traffic Assignment Results and Field Data

The results from DTA are compared with field data using video surveillance provided by TxDOT that shows traffic in the work zone area. Volumes on Link 1, 2, and 5 from 9 p.m. to 11 p.m. on 3 days under regular conditions and the first 2 days under work zone conditions were provided. Link volumes over 2 days with work zone conditions and 1 day with regular conditions are shown in Tables 4.7 and 4.8. Diversion rates on both days are described in Table 4.9.

Table 4.7 Link Volumes under Work Zone Conditions.

	First day of work zone			Second day of work zone		
Time	**Link 1**	**Link 2**	**Link 5**	**Link 1**	**Link 2**	**Link 5**
9:00–10:00	2353	2053	300	2087	1966	121
10:00–11:00	1113	1001	112	2077	1956	121

Table 4.8 Link Volumes under Regular Conditions.

Time	Link 1	Link 2	Link 5
9:00–10:00	4000	3725	275
10:00–11:00	3109	2883	226

Table 4.9 Field Measured Diversion Rates on Both Days.

	First day of work zone		Second day of work zone	
	Network level	**Local level**	**Network level**	**Local level**
Time	**Link 1**	**Link 2**	**Link 1**	**Link 2**
9:00–10:00	41%	6%	37%	-1%
10:00–11:00	64%	3%	33%	-1%

According to the results, diversion rates on both levels of the first day are higher than on the second day. People overreact on the first day of a work zone project. When drivers see the work zone signs, they think the work zone area will be congested. So people are more likely to choose alternative routes to avoid the congestion. On the second day, travelers tend to revert to their usual paths even though they may know that the work zone is present. The volume on Link 2 does not exceed the capacity of this link, so people return to their original routes and the diversion rate on Link 2 is very close to zero.

The diversion behavior on the second day is the same as a long-term work zone project, so we compare the result of the second day with the DTA results. The link volume from 9 p.m. to 11 p.m. and diversion rate provided by DTA are shown in Tables 4.10 and 4.11.

Table 4.10 Link Volumes from 9 p.m. to 11 p.m. Using DTA.

	Regular condition			Work zone condition		
Time	**Link 1**	**Link 2**	**Link 5**	**Link 1**	**Link 2**	**Link 5**
9:00–10:00	5738	5378	360	4655	3071	1584
10:00–11:00	5419	5147	272	4672	3093	1579

Table 4.11 Diversion Rates from 9 p.m. to 11 p.m. Using DTA.

	Network level	Local level
Time	**Link 1**	**Link 2**
9:00–10:00	19%	28%
10:00–11:00	14%	29%

According to the tables, the diversion rate trends by DTA are similar to the real data. Along with decreasing demand, the network-level diversion rate is also lower. Fewer people want to pass through the work zone area and the level of congestion in the work zone area is less. So more people want to keep their original routes and the diversion rate on the network level is smaller. The route choice behavior of the DTA model involves choosing the shortest time path from the origin to the destination. If travel times on an alternate route are less than travel time through the work zone link, people will choose alternative routes rather than pass through the work zone link. Additionally, people will accept a little longer travel time and will keep their original routes when the work zone link presents little or no congestion. These reasons explain why the local-level diversion rates from DTA and field observations do not match exactly.

4.3.3 Diversion Rate Sensitivity Analysis

The diversion rate can be affected by many factors. Demand in work zone areas is one of the most important factors and is easy to quantify. When network demand increases in the work zone area, congestion increases and travelers are then more likely to use alternative routes. Alternately, if the work zone area becomes less congested, more travelers will return to their original routes. This study uses VISTA to simulate traffic conditions on the network under different demand levels to explore the relationship between diversion rate and demand changes.

The simulation process has three steps:

1. Simulate traffic conditions in the work zone area under regular and work zone conditions with the same demand level.
2. Find the total volume on Link 1 and 2 during the simulation period under both conditions and calculate diversion rates based on Equations 4.1 and 4.2.
3. Repeats step 1 and 2 for each demand level.

Diversion behavior is different under different demand levels. Therefore, the traffic conditions from uncongested to congested levels are analyzed. The network reaches the most congested condition during the peak period. Demand in the morning period is considered as the 100% level. The traffic conditions under demand levels from 10% to 90% will also be considered. The diversion rates on both levels under different demand levels are shown in Table 4.12. Scatter diagrams of the diversion rate at the network and local levels are shown in Figures 4.6 and 4.7. Note on Figures 4.6 and 4.7 that the demand Level is the ratio of demand to capacity.

Table 4.12 Diversion Rates with Varying Demand Level.

Demand level	**Network level**	**Local level**
	Link 1	**Link 2**
10%	3.89%	23.51%
20%	5.14%	23.06%
30%	7.15%	27.48%
40%	19.00%	25.25%
50%	27.40%	11.30%
60%	32.80%	10.46%
70%	22.23%	13.69%
80%	20.66%	14.70%
90%	18.17%	13.13%
100%	50.87%	7.04%

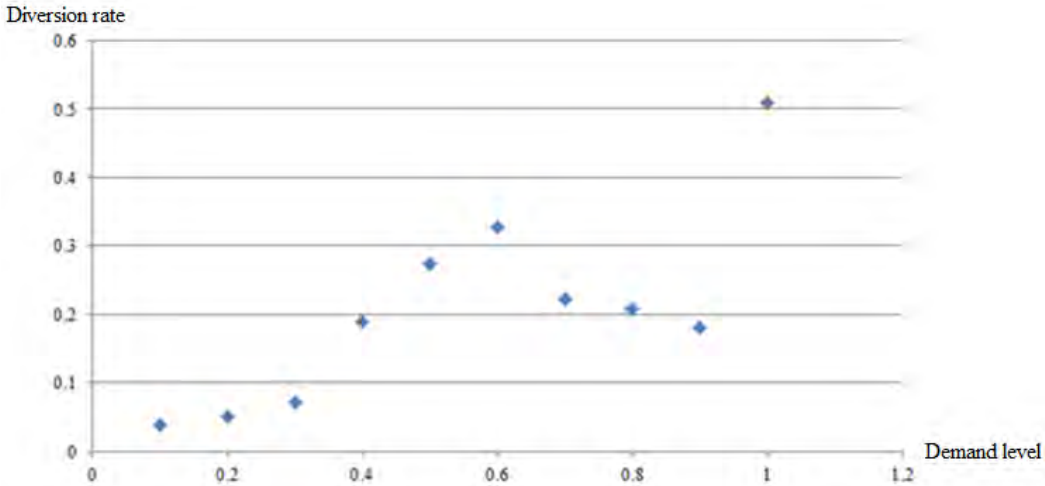


Figure 4.6 Network-Level Diversion Rate versus Demand Level.

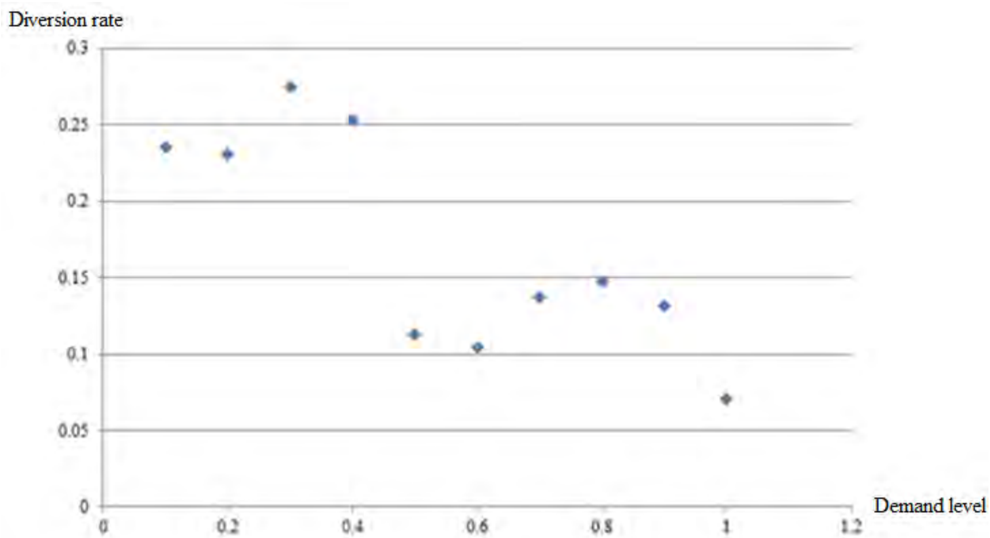


Figure 4.7 Local-Level Diversion Rate versus Demand Level.

According to these results, when demand increases, more people are likely to choose alternative network-level routes. The work zone area becomes more congested as the number of drivers who enter the work zone area increases and a queue may occur. When people see a queue has formed, they will think that the travel time delay may surpass their tolerance and may decide to use alternate routes. In addition, if this work zone is a long-term project, travelers already believe there will be congestion in this area during this period. A different local level diversion rate trend shows a decrease when demand increases. More people do not enter the work zone area, so volume on work zone links and travel time delay will decrease. People are more likely to accept the delay and to return to their original routes. Figure 4.8 shows the scatter diagram of network-level versus the local-level diversion rates.

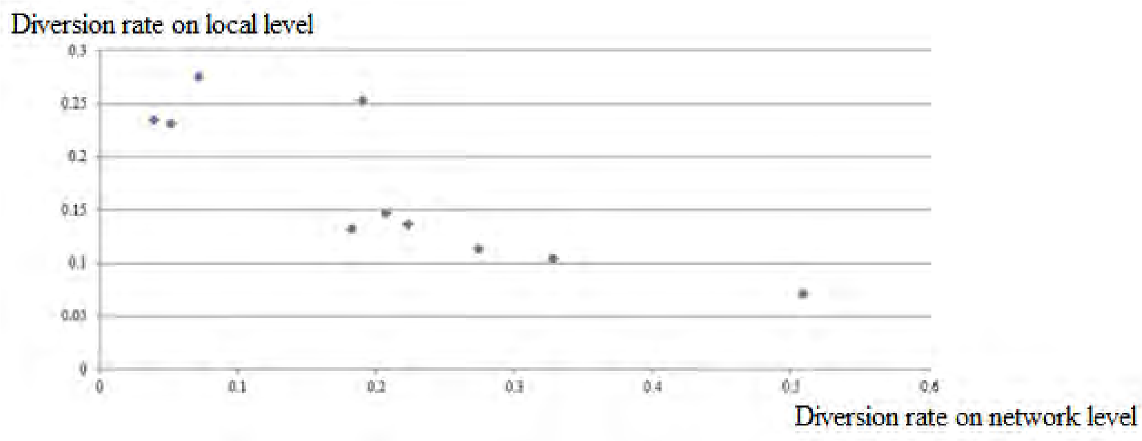


Figure 4.8 Scatter Diagram of Network-Level vs. Local Level Diversion Rate.

Equation 4.4 represents the relationship between network-level diversion rate and demand level. Table 4.13 shows the summary of this model's characteristics and Figure 4.9 shows the model on a scatter diagram.

$$D_{network\ level} = 0.362x + 0.008 \qquad \text{(Equation 4.4)}$$

where x is the demand level expressed as a ratio of demand to capacity.

Table 4.13 The Summary of Equation 4.4 Characteristics.

R	R Square	Adjusted R Square	Std. Error of the Estimate
0.77	0.592	0.541	0.0963367

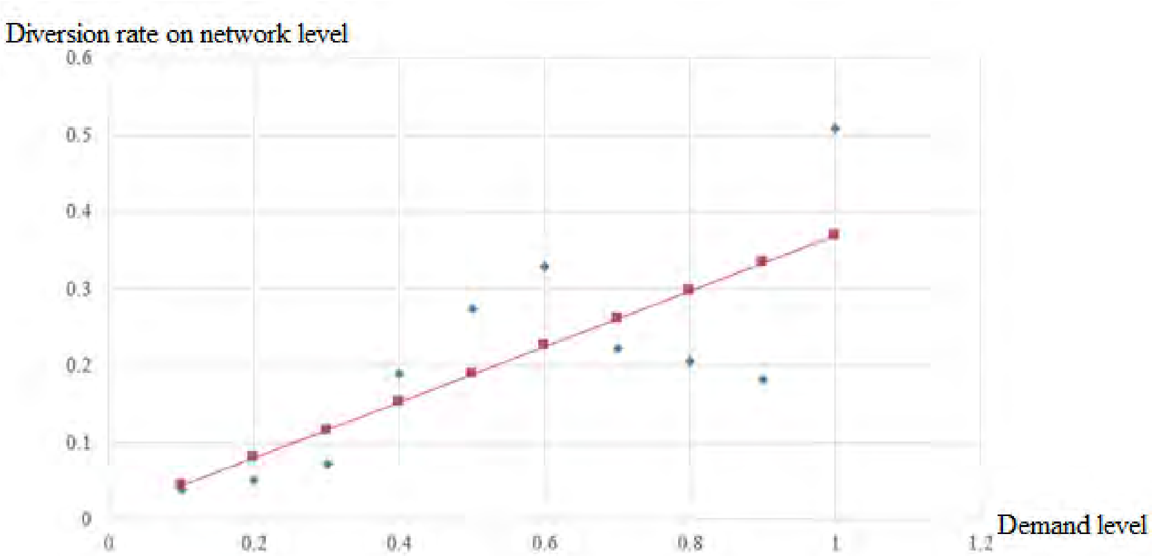


Figure 4.9 Equation 4.4 on a Scatter Diagram.

Equation 4.5 represents a relationship between local level diversion rate and demand level. Table 4.14 shows the summary of this model and Figure 4.10 shows the model on a scatter diagram.

$$D_{local\ level} = -0.192x + 0.275 \qquad \text{(Equation 4.5)}$$

where x is the demand level expressed as a ratio of demand to capacity.

Table 4.14 The Summary of Equation 4.5 Characteristics

R	R Square	Adjusted R Square	Std. Error of the Estimate
0.812	0.659	0.616	0.0444138

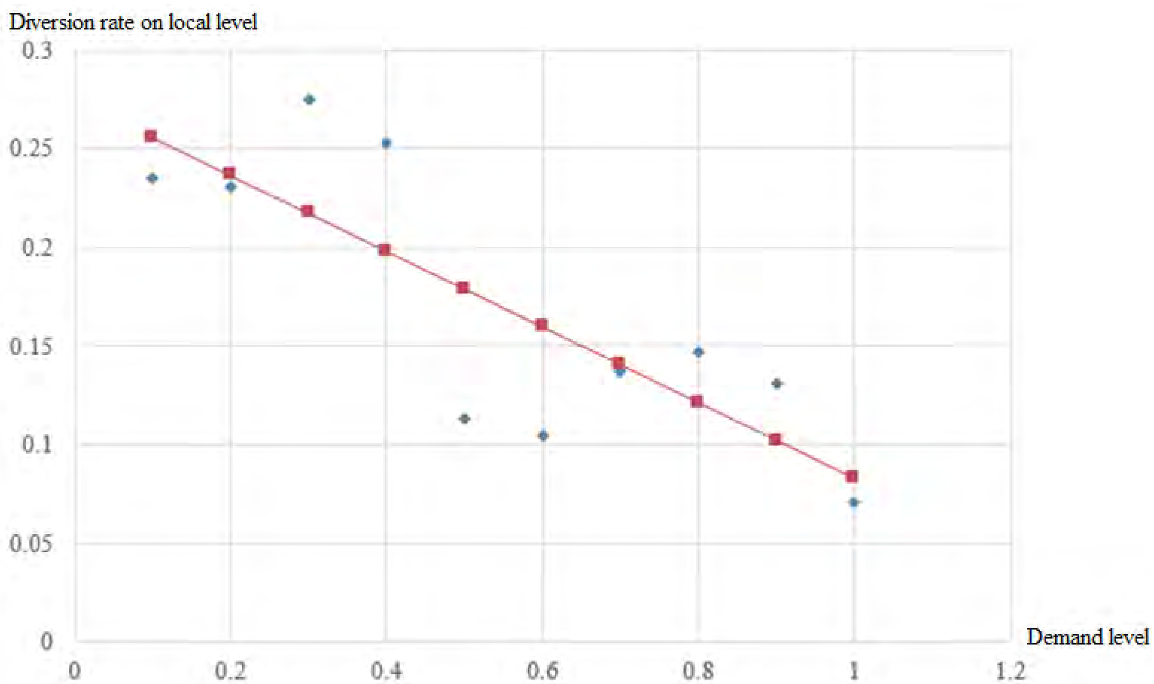


Figure 4.10 Equation 4.5 on a Scatter Diagram.

Equation 4.6 represents the relationship between diversion rate on network and local levels. Table 4.15 shows the summary of this model and Figure 4.11 shows the model on a scatter diagram.

$$D_{local\ level} = -0.424x + 0.258 \qquad \text{(Equation 4.6)}$$

where x is the network-level diversion rate.

Table 4.15 The Summary of the Model 4.6 (Equation 4.6)

R	R Square	Adjusted R Square	Std. Error of the Estimate
0.842	0.709	0.672	0.0410428

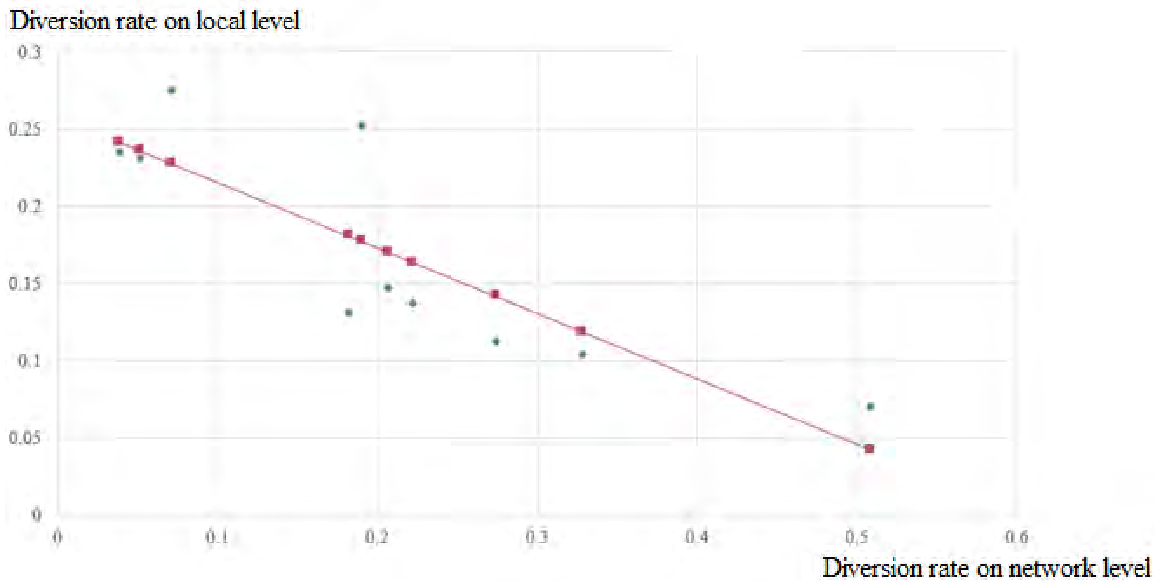


Figure 4.11 The Line of Model 4.6 on a Scatter Diagram.

Based on the results, the R-values of all models are close to or greater than 0.8, the slope of model 4.4 is positive, and the slopes of models 4.5 and 4.6 are negative. A strong linear relationship is established between diversion rates of the network, local, and demand levels.

4.4 Microsimulation

The section focuses on the microsimulation modeling efforts to evaluate innovative control measures such as late merge and Fixed-Cycle Signal Merge Control (FCSMC), which have shown to be more effective for congested corridors. The FCSMC is based on the late merge strategy and was simulated for a work zone using VISSIM and CORSIM software, using a variety of traffic demands. The VISSIM results suggest that FCSMC significantly increases flow throughput at work zones and overall improves traffic operations under saturated conditions. It is important to note, however, that the model is not based on or calibrated to any actual site. Therefore, measures of effectiveness like delay and travel time are deemed appropriate only for the purpose of comparison across scenarios.

4.4.1 Description of Network Configuration

Traditional traffic control included in the Manual on Uniform Traffic Control Devices (MUTCD) includes advance warning signs that guide drivers to merge into open lanes when suitable. Signs are placed on both sides of the roadway 1 mile and ½ mile ahead of the merge taper and inform travelers of lane closures. Closer to the work zone and about 1,500 feet in advance of the taper, lane reduction signs are placed on both sides of the roadway. A flashing arrow panel is usually placed at the beginning of the taper to instruct drivers in the closed lane to merge into the open lane. This traffic control concept works adequately for under-saturated corridors. However, in congested conditions when long queues extend beyond warning signs, it may result in aggressive maneuvers of drivers in closed lanes attempting to merge into the open lanes and, consequently, a further reduction in capacity.

The Fixed Cycle work zone traffic control strategy is embedded in the late merge traffic control. In the late merge concept, travelers are encouraged to use all lanes up to the merge point at the lane closure taper, instead of merging as soon as possible, which is encouraged by traditional and early merge controls. A late merge configuration was developed by the Pennsylvania Department of Transportation (Penn DOT). In this traffic control strategy, approximately 1.5 miles in advance of the lane closure, USE BOTH LANES TO MERGE POINT signs are placed on both sides of the roadway. These signs are followed by conventional ROAD WORK AHEAD and advance lane closed signs. Finally, MERGE HERE TAKE YOUR TURN signs are placed on both sides of the roadway near the beginning of the taper. The primary intent of the late merge configuration developed by Penn DOT is to reduce road rage between early and late mergers by informing drivers that it is permissible for traffic to travel in both lanes to the merge point. The network used in this analysis is similar to that of Penn DOT as shown in Figure 4.12. The beginning section, however, was extended in length to accommodate potential long queues that could form under high traffic demands.

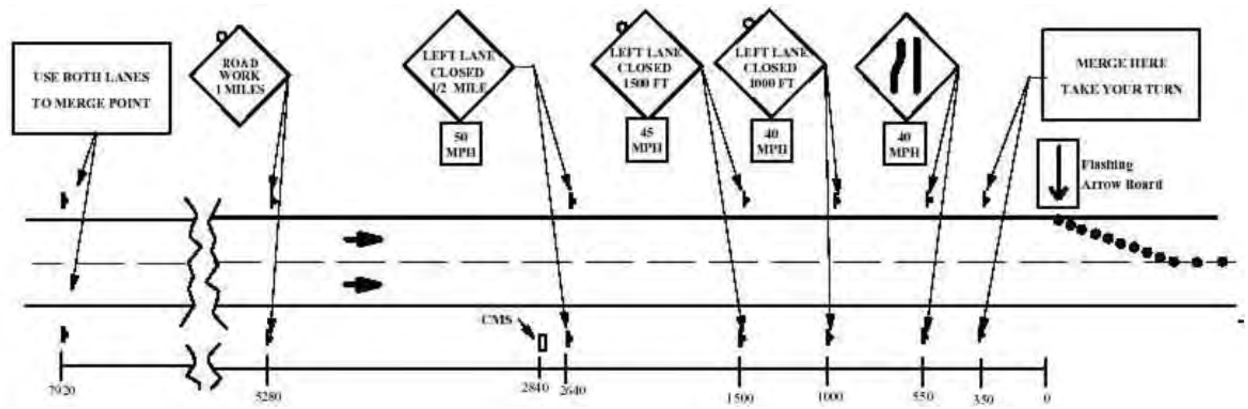


Figure 4.12 Late Merge Traffic Control Plan from Pennsylvania DOT.

The fixed cycle merge metering strategy consists of fixed cycle lengths, including 30, 60, or 120 seconds. These cycle lengths consist of green, amber, and red intervals just like a signal at any intersection. For evaluation purposes, in this study the green interval is equally divided between both lanes as the traffic volumes have been coded as equal in both lanes. However, in actual conditions the traffic volume distribution is not always equal on all lanes. Therefore, the green interval duration could possibly be divided in the same ratio as the lane distribution. The travel speed was assumed 55 mph throughout the study corridor.

4.4.2 Modeling Procedure

Microsimulation analysis was conducted using both CORSIM and VISSIM software. Since this analysis is not based on any actual site, no calibration was involved. However, by adjusting some driving behavior parameters, the models were enhanced to ensure that the simulators mimic realistic driver behaviors observed as a result of late merge and traffic signal implementation. A brief discussion of modeling efforts is presented below.

4.4.2.1 CORSIM

To model signal merge in CORSIM, input flows of 1800, 2000, 2200, and 2400 vehicles per hour per open lane (VPHPL) were used. These parameters were used in 2-to-1, 3-to-1, and 3-to-2 lane configurations. In each scenario the left lane(s) were dropped, leaving the right-most lanes accessible in the work zones. A "base case" simulation was first simulated using late merge control concepts without signalized operations. Lanes were then separated with signals placed one per lane to control merging. Signals ran one at a time for the 2-to-1 and 3-to-1 configurations in 30, 60, 90, 120, 150, and 180-second cycle lengths, divided evenly among the lanes with 4-second ambers. For the 3-to-2 configuration, signalization was maintained with two lanes operating at a time, staggered from each other with equal distribution of green times followed by 4-second ambers. An example of the 3-to-2 signal times are illustrated in Figure 4.13 with a 30-second cycle length. Cycle lengths for the 3-to-2 configuration were also 30- to 180-second cycles in 30-second increments. Desired speeds of 65 mph were maintained for all segments except the work zone segment, which was reduced to 55 mph. Results are the averages of three simulations.

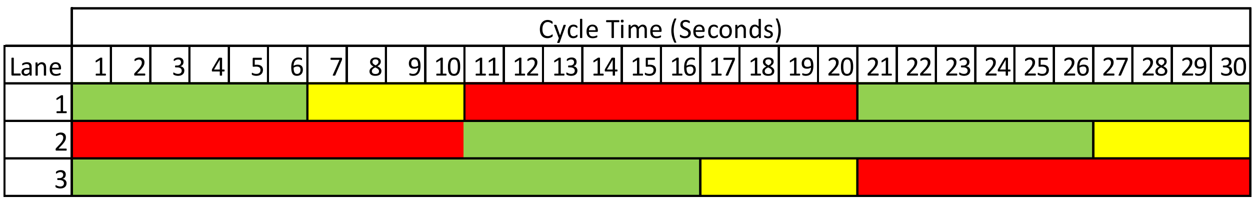


Figure 4.13 Signal Time Allocation for 3-to-2 Lane Configuration with 30-Second Cycle.

4.4.2.2 VISSIM

VISSIM simulation efforts were focused on 2-to-1, 3-to-1, and 3-to-2 configurations, with fixed cycle lengths 30, 60, 90, and 120. In VISSIM, using lane configuration and driving behavior parameters, vehicles are forced to drive up to the merge signal and thus obey the late merge concept. On green, the vehicles can freely move into the open lane as vehicles in the other lane will be stopped for the red signal. The cycle length is split equally between all lanes. For the 3-to-2 configuration, split signal timing was used since traffic in two lanes moves concurrently. Replicate runs of simulation were performed for volumes 1800, 2000, 2200, and 2400 VPHPL.

4.4.3 Results from CORSIM and VISSIM

Results of these analyses for CORSIM and VISSIM simulations are presented below.

4.4.3.1 CORSIM

Several measures of effectiveness were monitored during trial runs, including total travel time, total delay, ratio of input to output flow, and driver behavior. The results gathered for the 3-to-1 lane reduction case are shown in Figures 4.14, 4.15, and 4.16.

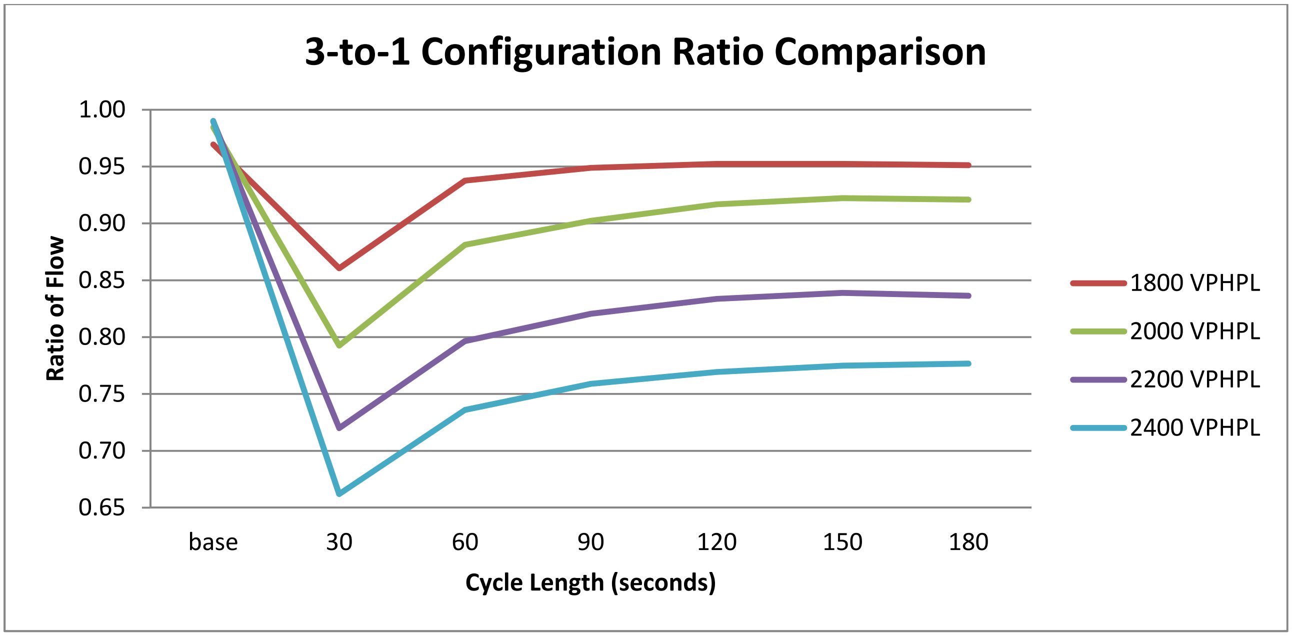


Figure 4.14 Ratios of Flow to Traffic Demand versus Cycle Length for 3-to-1 Lane Reduction.

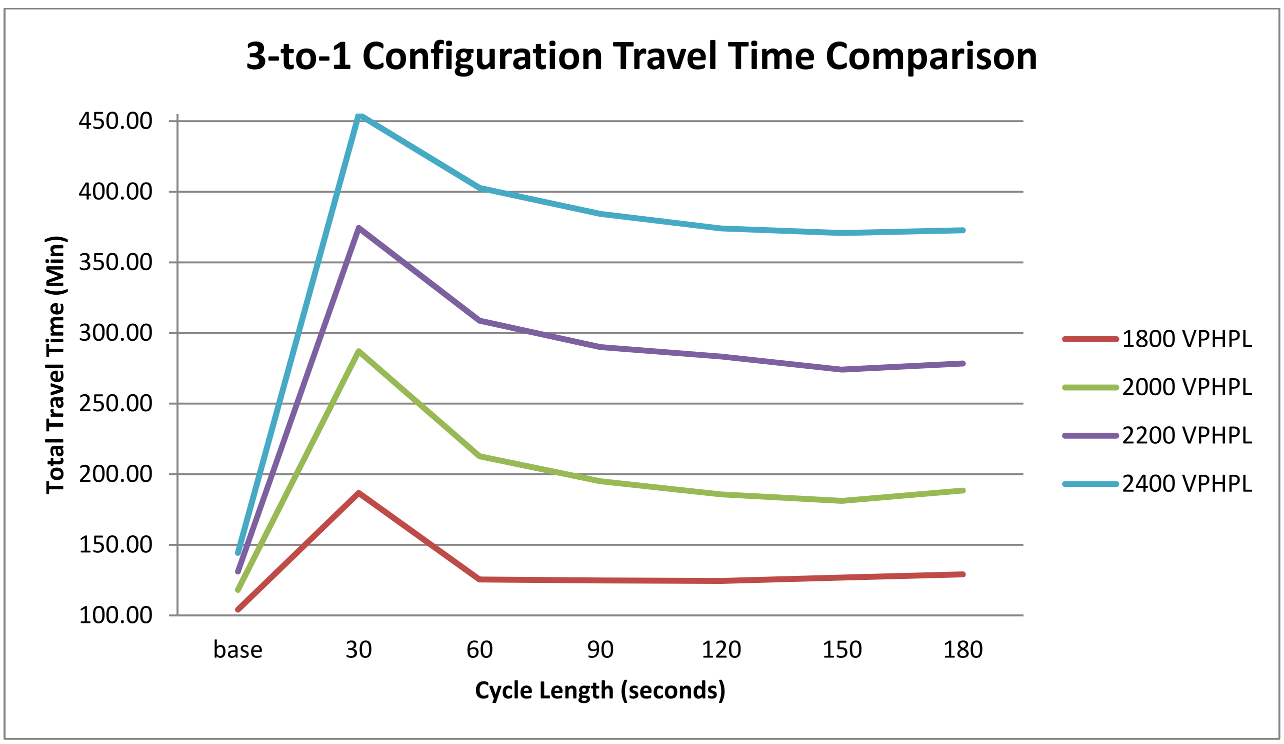


Figure 4.15 Travel Time versus Cycle Length for 3-to-1 Lane Reduction.

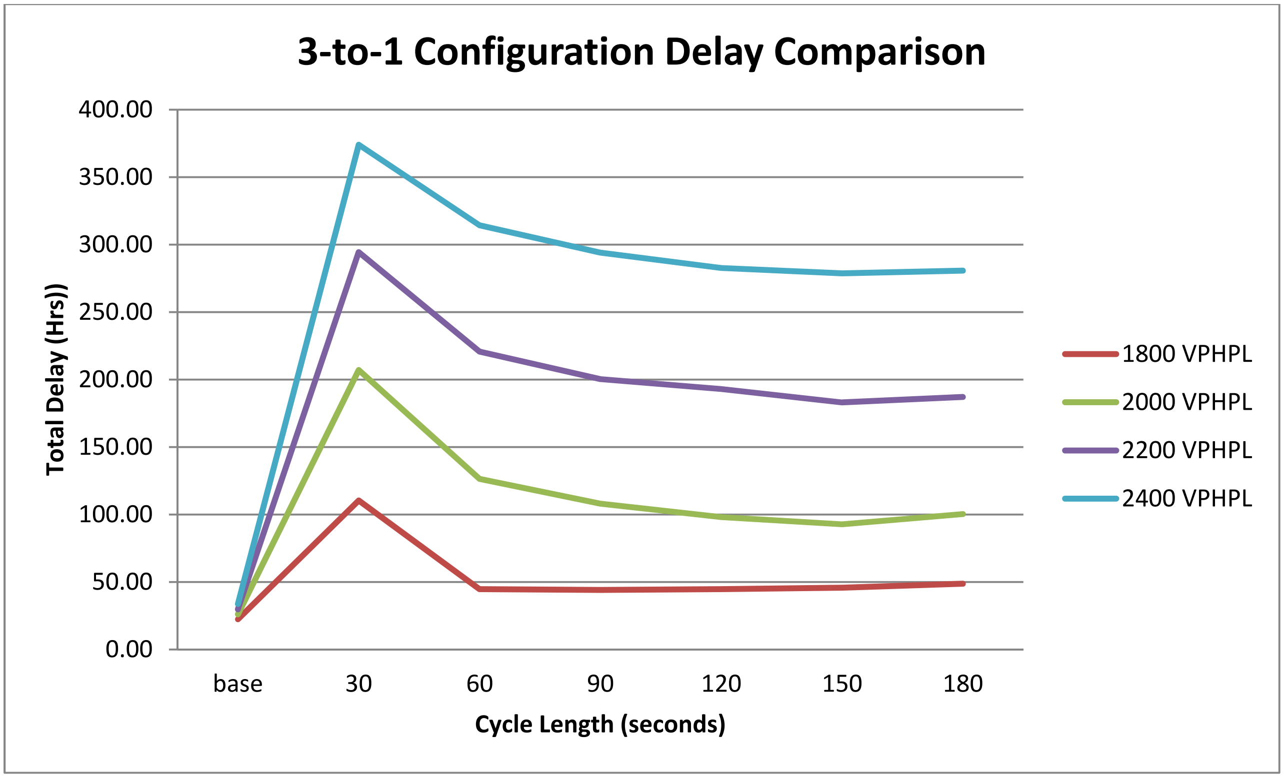


Figure 4.16 Delay versus Cycle Length for 3-to-1 Lane Reduction.

The 3-to-1 lane reduction case showed an optimal cycle length of 150 seconds for inputs of 2000, 2200, and 2400 VPHPL. This cycle allowed the highest throughput, least delay, and least travel time. Optimal results may vary by only a marginal amount. For example, the ratio of flow to demand for 2400 VPHPL varied only 4% between cycles of 60 seconds and 180 seconds. Optimum cycles were determined based upon the best of all three criteria with equal weighting. For example, if an input volume for a configuration had the lowest travel time and lowest delay, but did not have the highest ratio, it would still be considered optimum as it meets two of the best criteria out of three. Table 4.16 outlines the differences among lane reduction cases.

Table 4.16 Signalized Merge Results.

	1800 VPHPL	2000 VPHPL	2200 VPHPL	2400 VPHPL
2 to 1	90 Sec	180 Sec	150 Sec	150 Sec
3 to 1	90 Sec	150 Sec	150 Sec	150 Sec
3 to 2	90 Sec	180 Sec	180 Sec	180 Sec

Tables 4.17 and 4.18 represent the differences in ratios and vehicular delay, respectively. In all setups, throughput was reduced regardless of cycle length. Non-signalized merge control can handle larger through flows, but has areas of significant merge conflict between drivers. While the signalized control has less support for larger volumes, once queuing developed, zero conflicts were noted where passengers were trying to out-merge other drivers, even at the tails of the queue.

Table 4.17 Ratio of Flow Throughput to Demand Flow CORSIM.

	1800 vplph							2000 vplph						
Configuration	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	150 Sec Cycle Length	180 Sec Cycle Length	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	150 Sec Cycle Length	180 Sec Cycle Length
2-to-1	0.97	0.93	0.96	0.96	0.96	0.96	0.96	0.99	0.85	0.89	0.91	0.91	0.92	0.92
3-to-1	0.97	0.86	0.94	0.95	0.95	0.95	0.95	0.98	0.79	0.88	0.90	0.92	0.92	0.92
3-to-2	0.99	0.96	0.98	0.98	0.98	0.98	0.98	0.99	0.87	0.91	0.93	0.93	0.95	0.97

	2200 vplph							2400 v vplph						
Configuration	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	150 Sec Cycle Length	180 Sec Cycle Length	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	150 Sec Cycle Length	180 Sec Cycle Length
2-to-1	0.99	0.72	0.82	0.84	0.84	0.85	0.84	0.99	0.71	0.75	0.77	0.77	0.78	0.77
3-to-1	0.99	0.72	0.80	0.82	0.83	0.84	0.84	0.99	0.62	0.74	0.76	0.77	0.77	0.78
3-to-2	0.99	0.79	0.83	0.84	0.85	0.86	0.90	0.99	0.73	0.76	0.77	0.77	0.79	0.83

Table 4.18 Delay (seconds per vehicle) CORSIM.

	1800 vplph							2000 vplph						
Configuration	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	150 Sec Cycle Length	180 Sec Cycle Length	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	150 Sec Cycle Length	180 Sec Cycle Length
2-to-1	45.3	117.7	77.3	69.3	69.3	76.00	82.70	46.8	276.5	204.0	166.8	163.8	156.00	148.20
3-to-1	44.7	220.7	89.3	88.0	89.3	91.30	97.30	46.8	372.6	227.4	194.4	176.4	166.80	180.60
3-to-2	37.0	99.0	64.0	58.7	60.0	64.70	69.00	40.8	242.4	186.6	151.8	141.0	116.70	92.40

	2200 vplph							2400 v vplph						
Configuration	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	150 Sec Cycle Length	180 Sec Cycle Length	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	150 Sec Cycle Length	180 Sec Cycle Length
2-to-1	49.1	398.6	327.8	304.9	297.8	292.40	299.50	50.5	494.5	423.5	407.5	400.5	389.00	396.50
3-to-1	48.5	481.6	361.1	327.8	315.8	299.50	306.00	50.5	561.0	471.5	441.0	424.0	418.00	421.00
3-to-2	44.5	365.2	319.6	294.3	278.7	276.00	216.30	49.5	428.5	401.3	390.5	390.8	375.50	320.50

4.4.3.2 VISSIM

4.4.3.2.1 Selection of Merge Concepts

In order to model and simulate the merge concepts that would be best applied depending on lane configuration and demand, microsimulation analysis was conducted using VISSIM software to assess performance. VISSIM is a microscopic, time-step, and behavior-based simulation model, developed to model urban traffic. This software can analyze traffic operations under various constraints, including lane configuration, traffic composition, and traffic signals. Due to the variations between the three sites in Austin and Houston, however, this analysis is not based on any specific site and assumes no truck volume. Instead, using the MUTCD from Texas and other states, the 2-to-1, 3-to-2, and 3-to-1 lane configurations were designed with volumes of 1800, 2000, 2200, 2400, and 2600 passenger cars per hour per lane (pcphpl). Specifically, assuming a 12-foot offset and a posted speed of 55 mph, the Texas MUTCD suggests using a minimum desirable taper length of 660 feet (Texas MUTCD, 2012). In a work zone with a 3-to-1 lane closure configuration, the minimum distance between the taper from 3-to-2 lanes and 2-to-1 lane is 1560 feet for speeds less than or equal to 65 mph, as established in various state work zone traffic control guidelines (IDOT, 2013). Thus, these minimum values were used in the simulation to develop thresholds between different merge techniques.

Different lane control techniques like early merge, late merge, and signal merge can help the merging process, while also affecting queue jumping, delay, capacity, and user stress. Both early merge and late merge can be implemented as either static or dynamic. A static merge uses signage that displays a single message in the same location at all times, regardless of traffic conditions. Dynamic merge concepts involve real-time control measures to determine which signage should be used upstream to inform approaching drivers of the upcoming conditions.

4.4.3.2.2 Introduction to Early Merge

The early merge control technique is used to warn drivers in advance of a work zone of an upcoming closed lane(s). The typical layout of the early merge strategy involves lane closure signage 1.5 miles in advance of the transition area, which is depicted in Figure 4.17. The signs are followed by a lane reduction sign about 1,500 feet from the entrance to the transition area (Idewu, 2011).

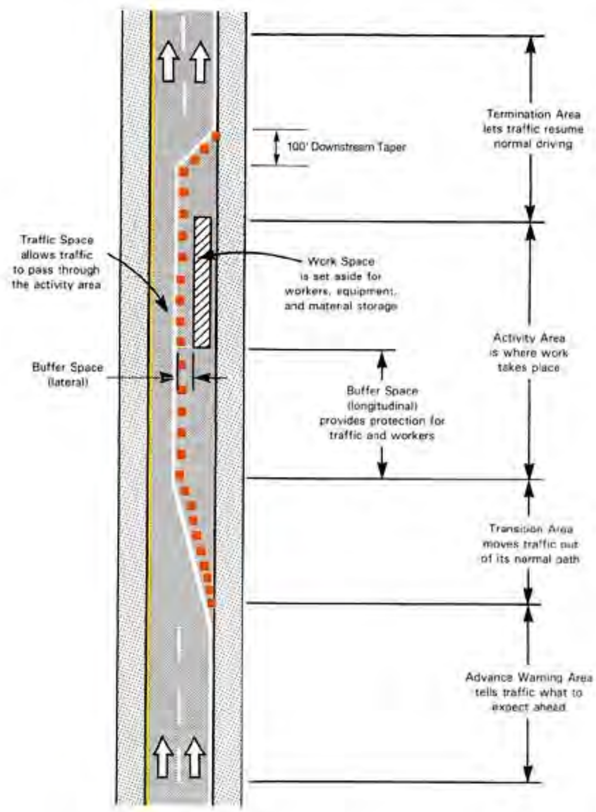


Figure 4.17 Depiction of the Components Parts of a Temporary Work Zone (MUTCD, n.d.).

The early merge technique was modeled in VISSIM using various closure lengths prior to the work zone, up to one-quarter mile. One-quarter mile is assumed to be ample distance for an early merge because the distance should be long enough to prevent the visual distraction of the upstream work zone. If the distance is long enough that users cannot perceive the work zone, then distractions as a result of the work zone are minimized and the merging process can function efficiently. This closure prior to the work zone allows time for the user to find a gap to merge and complete this merging process prior to the lane closure(s). Thus, early merge reduces the capacity of the roadway, but if used properly in conditions with low demand, can minimize queue jumping and user stress. In agreement with this hypothesis is research completed by Yang, which suggests that this technique is very effective if traffic demand is low compared to capacity. However, the system breaks down when the demand is high and vehicles have fewer gaps to merge (Yang, 2009). Therefore, VISSIM is applied to determine the lower demand thresholds that can use the early merge technique for various lane configurations.

4.4.3.2.3 Differing Applications of Early Merge

In order to determine which merge concepts can best be applied based on highway configuration and demand, it is also important to determine the most useful application of the early merge concept. The early merge concept was tested at various closure lengths, including one link, two links, and three links closed prior to the late merge or work zone area. The first two links are each approximately one-sixteenth mile long. Therefore, in the case of early merge with two links closed, the closure length was about one-eighth of a mile or half of the distance of early merge with three links closed. From the VISSIM outputs, as fewer links prior to the work zone are closed, the highway system behaves more like the late merge technique. Thus, three links closed prior to the work zone yields results unlike the late merge technique. In Table 4.19, the ideal merge concept for the 3-to-2 configuration with 2000 pcphpl is the early merge with three links closed technique. Specifically, the average delay time per vehicle, average number of stops per vehicle, average stopped delay per vehicle, average queue length, maximum queue length, and number of stops within the queue are the smallest for early merge with three links closed, when compared to the various early merge closure lengths and late merge. Additionally, the average overall speed and average speed on the link prior to the lane closure is largest for early merge with three links closed, when compared to the various early merge closure lengths and late merge. It is less important that early merge with three links closed is ideal in this situation; rather, these outputs show the trend that early merge with a smaller number of links closed behaves more like the late merge technique. Therefore, early merge with three links closed, referred to in future tables as just early merge, provides a representation that is unique from the late merge concept.

Table 4.19 VISSIM Outputs of Varying Lengths of the Early Merge for 3-to-2 Configuration and 2000 pcphpl.

3 to 2 Configuration: 2000								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge (3 links closed, 1/4 mile)	**3.463**	**0.000333**	**62.534**	**0.003**	**62.739**	**0**	**35.667**	**3**
Early Merge (2 links closed)	3.553	0.001	62.51	0.007	61.091	0.333	49	4.667
Early Merge (1 link closed)	4.323	0.0037	62.32	0.02	51.909	2	114.667	25
Late Merge	16.686	0.033	59.711	0.283	37.649	18.667	203.667	131.667

4.4.3.2.4 Introduction to Late Merge

Unlike early merge, late merge is a technique that encourages all lanes to be used until a specified merging point. Once vehicles reach this point, users in the closed lane(s) merge with those in the open lane(s) in an alternating pattern (Idewu, 2009). Thus, late merge tries to take advantage of the full capacity of the highway approaching the work zone to minimize the queue length. VISSIM is used to test the application of the late merge technique, as one that can be used in cases with low to moderate traffic demand and, therefore, can be used more efficiently than early merge in cases with higher demand. In VISSIM, the late merge technique is applied by keeping all lanes open up until the work zone area and measuring various criteria that show the effectiveness of this technique.

4.4.3.2.5 Introduction to Joint Merge

Although not extensively used or explored, joint merge could present an interesting technique to bridge early and late merge. The joint merge technique uses signage in the advance warning area and channeling devices in the transition zone to help create a balanced distribution of vehicles in each lane (Idewu, 2009). Thus, using various warning signs and a “funnel-shaped” configuration, the joint merge can simultaneously merge two lanes into one more naturally than late merge.

4.4.3.2.6 Introduction to Signalized Merge

Aside from early merge, late merge, and possible future applications of joint merge, the signal merge technique can be used at work zones to facilitate safe, orderly traffic movement. Essentially, signalized control on highways was developed to manage merging when the work zone area is heavily congested. Much like early and late merge, VISSIM is used to determine the threshold for this merge concept and try to numerically assign a value to the term “heavily congested.” Fixed cycle lengths of 30, 60, 90, and 120 seconds are used on 2-to-1, 3-to-2, and 3-to-1 lane configurations. In the case of the 2-to-1 configuration, the cycle lengths were split equally between both lanes. For the 3-to-2 and 3-to-1 configurations, however, equal fractions of green time are provided to the closed lane and the through lanes. Assuming the lane closures occur on the left side of the highway section, the far left lane would close for both the 3-to-1 and 3-to-2 configuration, with an additional lane closure in the 3-to-1 case at a minimum distance between tapers of 1560 feet. In both configurations, split signal timing is used since traffic in two lanes can move concurrently, with green provided to the far left lane to merge, followed by equal green time for the other two through lanes. Thus, the signal merge technique has the capability to minimize queue jumping because of the equal green times for all lanes. Equal green times provided for all lanes, assuming essentially equal queues in all lanes, would likely deter users from queue jumping because there would be minimal space for queue jumping and no benefit to moving into another lane. This equal green time for all lanes is depicted in Figure 4.18. Lanes 1 and 2 are the through lanes that will remain open when merging from 3-to-2 lanes, prior to the merge from 2-to-1 lane. Thus, these two through lanes have the same amount of green time as Lane 3, which is the lane that is merging into the two through lanes before the second lane closure.

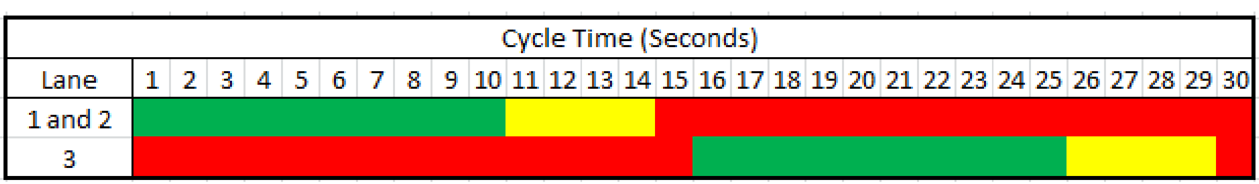


Figure 4.18 Signal Time Allocation for 3-to-1 Lane Configuration with 30-Second Cycle.

4.4.3.2.7 VISSIM Outputs for 2-to-1 Lane Configuration

The first case that was modeled is the 2-to-1 lane configuration with a vehicle demand of 1800 pcphpl (shown in Table 4.20). From the VISSIM outputs, the early merge and late merge techniques were initially compared, using various measures of delay, stops, speed, and queue.

The comparison between these two techniques on all measures shows favorable results for using the early merge technique, as highlighted in yellow. The outputs of four different signalized merge applications of 30-, 60-, 90-, and 120-second cycle lengths were then compared, using measures similar to the early and late merge comparison. However, average speed on the link prior to the closure, average queue length, maximum queue length, and number of stops within the queue did not provide any applicable evidence that is not already shown in the previous measures. Of the four different cycle lengths, 60 seconds is ideal because it minimizes the average delay time per vehicle and average stopped delay per vehicle, while maximizing the average speed. Thus, the ideal signal merge technique is the 60-second cycle length and is highlighted in blue. However, after comparing the early merge outputs to signal merge with 60-second cycle length, early merge would be the ideal approach when comparing all types of merge concepts.

Table 4.20 VISSIM Outputs for 2-to-1 Lane Configuration and 1800 pcphpl.

2 to 1 Configuration: 1800								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**4.752**	**0.013**	**61.517**	**0.183**	**44.447**	**3.333**	**103.333**	**24**
Late Merge	5.766	0.036	61.292	0.46	37.567	14	268.333	71.667
Signal Merge-30 s	198.982	2.236	34.979	9.242				
Signal Merge-60 s	25.782	0.672	56.926	8.224				
Signal Merge-90s	27.466	0.616	56.584	11.382				
Signal Merge-120s	32.749	0.645	55.549	16.167				

The second VISSIM simulation involves a 2-to-1 lane configuration and 2000 pcphpl. The comparisons between early merge and late merge showed that for all measures except maximum queue length, early merge would be ideal for this configuration and user demand. As shown in Table 4.21, the maximum queue length for early merge is 382.7 ft, which is slightly higher than the maximum queue length for late merge of 335 ft. This difference is relatively insignificant because the average queue length for early merge is significantly less than for late merge. Additionally, the average delay time per vehicle for early merge is more than four times less than late merge. Although the optimal signal merge cycle time for three of the four measures is 120 seconds, the output of the fourth measure is an issue. The differences between the 90-second and 120-second cycle lengths in the first three measures are less significant than the difference in the average stopped delay per vehicle. Specifically, the outputs show that vehicles are stopping approximately 5.2 times with 90-second cycles, compared to 1.1 times with 120-second cycles. However, the average stopped delay per vehicle is less for the 90-second cycle length, suggesting that although vehicles stop more with a 90-second cycle length, users are being delayed much less. Thus, the optimal cycle length of those tested for signal merge should be 90 seconds; however, early merge should be preferred to the signal merge with a 90-second cycle length.

Table 4.21 VISSIM Outputs for 2-to-1 Configuration and 2000 pcphpl.

2 to 1 Configuration: 2000								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**9.463**	**0.058**	**60.402**	**1.194**	**29.958**	**30.333**	382.67	**110**
Late Merge	42.731	0.466	54.87	4.64	21.84	78	**335**	299
Signal Merge-30 s	375.207	3.034	24.658	11.45				
Signal Merge-60 s	75.41	1.535	48.348	15.269				
Signal Merge-90s	59.588	5.159	50.799	21.039				
Signal Merge-120s	57.336	1.069	51.16	27.792				

The VISSIM outputs for the 2-to-1 lane configuration with 2200 pcphpl provided different results from the previous two demands and are shown in Table 4.22. The early merge technique yields less average delay time per vehicle, average number of stops per vehicle, and average stopped delay per vehicle, when compared to late merge. Early merge also provides a greater average speed than late merge, but the concern with early merge is queue development. Since these merge concepts are applied just prior to the work zone, average speed on the link prior to the closure is significant because this represents the level of congestion as a result of the merge. For this reason and the queue development when using the early merge concept, late merge is more applicable in this case. The signal merge concept with a 90-second cycle length is optimum for signal merge because it maximizes the average speed, while minimizing the average delay time per vehicle. While the average stopped delay per vehicle is as important as the average delay time per vehicle, the average speed difference between 33.7 mph for a 90-second cycle length and 17.9 mph for a 30 second cycle length is the key reason to use the 90-second cycle length. In this scenario, signal merge with a 90-second cycle length could be selected as the optimum merge concept. However, since late merge significantly minimizes the queue, while essentially having the same average speed and average delay time per vehicle, the late merge concept is optimal. Like the outputs for a 2-to-1 lane configuration and 2200 pcphpl, the 2-to-1 lane configuration and 2400 pcphpl has values that could lead to varying conclusions.

Much like the 2200 pcphpl demand case, the 2400 pcphpl outputs in Table 4.23 lead to the conclusion that late merge is ideal when compared to early merge because of late merge's ability to minimize the queue by taking advantage of all available highway capacity. Although late merge could be interpreted as more appropriate than signal merge with a 90-second cycle length, most of the output values are comparable except for one. The average stopped delay per vehicle for late merge of 223.9 seconds is more than six times greater than the 90-second cycle length signal merge value of 36.6 seconds. For this reason, the ideal merge concept for this case is signal merge with a 90-second cycle length. A similar conclusion can be drawn from Table 4.24 for a demand of 2600 pcphpl. Late merge and signal merge with a 90-second cycle length are also comparable; however, the one distinct difference is the average stopped delay per vehicle is 294.3 seconds for late merge and 38.6 for the signal merge case. Thus, the 2-to-1 lane configuration with a user demand of 2600 pcphpl should ideally use a signal merge with a 90-second cycle length, like the 2400 pcphpl case, because both the outputs show significant differences among measures.

Table 4.22 VISSIM Outputs for 2-to-1 Configuration and 2200 pcphpl.

2 to 1 Configuration: 2200								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	224.967	**3.102**	33.056	62.051	12.208	339.333	515	530.333
Late Merge	268.389	4.137	30.038	89.319	**13.704**	**249.667**	**365**	**411.333**
Signal Merge-30 s	569.251	3.867	17.922	**12.773**				
Signal Merge-60 s	240.116	3.377	31.945	19.483				
Signal Merge-90s	**214.942**	4.021	**33.728**	32.966				
Signal Merge-120s	223.159	4.486	33.138	47.713				

Table 4.23 VISSIM Outputs for 2-to-1 Configuration and 2400 pcphpl.

2 to 1 Configuration: 2400								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	359.349	4.792	**25.132**	195.822	20.687	471.333	515	202
Late Merge	418.524	**4.598**	22.768	233.908	**21.629**	**316.333**	**365**	**142.333**
Signal Merge-30 s	751.98	5.278	14.046	**15.425**				
Signal Merge-60 s	431.7	5.109	22.166	21.122				
Signal Merge-90s	**333.09**	6.498	23.683	36.564				
Signal Merge-120s	405.1	7.602	23.214	56.058				

Table 4.24 VISSIM Outputs for 2-to-1 Configuration and 2600 pcphpl.

2 to 1 Configuration: 2600								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**484.35**	5.76	**20.195**	206.06	21.182	485	515	156.67
Late Merge	560.77	**5.662**	18.004	294.33	**29.852**	**354.33**	**365**	**31.667**
Signal Merge-30 s	882.07	6.425	12.345	**18.233**				
Signal Merge-60 s	594.34	6.393	17.178	21.821				
Signal Merge-90s	572.88	8.105	17.686	38.584				
Signal Merge-120s	573.83	9.175	17.658	56.876				

4.4.3.2.8 VISSIM Outputs for 3-to-2 Lane Configuration

The VISSIM outputs for the 3-to-2 lane configuration and 1800 pcphpl provide support for early merge as the ideal merge concept technique. For each measure, early merge is more efficient in managing the traffic flow than late merge and all forms of signal merge. As shown in Table 4.25, the four measures for signal merge are distributed equally to each of the four cycle lengths. However, the 60-second cycle length would be the optimal signal merge approach of the four cycle lengths—even when it is not the most efficient for a specific measure, no significant increases in delay occur. For example, although the 90-second signal merge has the optimum average delay time per vehicle, the 60-second cycle length is relatively close to the minimum, unlike the 30-second cycle length. Additionally, the average stopped delay per vehicle reaches a minimum value of 20.4 seconds for the 30-second cycle length, but the 60-second cycle length is nearly the same, unlike the 90- and 120-second cycle lengths. Regardless, from the analysis of each measure, early merge would be the most efficient merge technique for this configuration and user demand. The general trends from the VISSIM outputs in the 1800 pcphpl case are applicable to the 2000 pcphpl simulation. In Table 4.26, all measures suggest that early merge is more efficient than late merge. Similar to the approach taken with the 1800 pcphpl demand, even though signal merge with a 60-second cycle length is not the most efficient technique for all measures, it does not deviate far from those values. For instance, even though 15.8 mph is not the optimum average speed, it is close to that ideal value of 16.4 mph for 120-second cycle length. However, the average stopped delay per vehicle for the 120-second cycle length deviates far from the optimum value for the 60-second cycle length. Aside from slight variations, the trends for the 2200 pcphpl demand shown in Table 4.27 correspond directly to the 2000 pcphpl trends. Thus, early merge is overall the most efficient merge concept technique and the 60-second cycle length is the optimum signal merge concept.

Table 4.25 VISSIM Outputs for 3-to-2 Configuration and 1800 pcphpl.

3 to 2 Configuration: 1800								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**3.018**	**0.0003**	**62.669**	**0.002**	**63.423**	**0**	**26.667**	**1.333**
Late Merge	3.296	0.002	62.591	0.0117	53.944	0.667	76	11.667
Signal Merge-30 s	809.46	6.994	13.222	20.371				
Signal Merge-60 s	444.18	5.311	21.728	21.642				
Signal Merge-90s	416.65	6.889	22.786	38.557				
Signal Merge-120s	416.76	7.857	22.798	57.435				

Table 4.26 VISSIM Outputs for 3-to-2 Configuration and 2000 pcphpl.

3 to 2 Configuration: 2000								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**3.463**	**0.0003**	**62.534**	**0.003**	**62.739**	**0**	**35.667**	**3**
Late Merge	16.686	0.033	59.711	0.283	37.649	18.667	203.667	131.667
Signal Merge-30 s	958.77	8.608	11.53	25.027				
Signal Merge-60 s	666.8	7.363	15.797	23.935				
Signal Merge-90s	641.3	9.511	16.284	43.075				
Signal Merge-120s	639.63	10.624	16.358	64.043				

Table 4.27 VISSIM Outputs for 3-to-2 Configuration and 2200 pcphpl.

3 to 2 Configuration: 2200								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**4.07**	**0.001**	**62.362**	**0.008**	**61.756**	**0.667**	**50.333**	**3.667**
Late Merge	139.048	0.349	43.713	6.415	25.759	47.333	279	216.667
Signal Merge-30 s	1048.67	9.602	10.707	26.791				
Signal Merge-60 s	786.523	8.496	13.919	25.545				
Signal Merge-90s	770.464	11.009	14.166	46.937				
Signal Merge-120s	770.45	12.34	14.183	70.382				

The VISSIM outputs for 3-to-2 configuration and 2400 pcphpl demand in Table 4.28 suggests continuation of a trend that was not expected. Although early merge is the optimal merge concept for 1800, 2000, and 2000 pcphpl for the same configuration, the expectation was that early merge is best applied if traffic demand is low compared to capacity. While the VISSIM software does not provide capacity values or the option to explicitly change the lane capacities, the Highway Capacity Manual typically suggests that highway lane capacities range from 1800 to around 2300 pcphpl (MUTCD, n.d.). With a work zone on the highway, it should suggest that the capacity will decrease from the ideal conditions and early merge would be optimal for the lower range of capacity values. However, the VISSIM output suggests that early merge is still optimal when compared to late merge because it moves the traffic more efficiently than late merge in all measures, aside from maximum queue length. One possible reason for this discrepancy is that with higher demand, lanes are more congested. In reality, this congestion increases the likelihood for queue jumping, driving in the closed lane(s), and other dangerous actions. However, VISSIM cannot simulate these actions and could cause discrepancies from previous assumptions. Using similar reasoning to the previous user demands, the 60-second cycle length is the optimal signal merge technique. After comparing the early merge outputs to the signal merge with 60-second cyle lengths, early merge is the ideal merge concept.

Table 4.28 VISSIM Outputs for 3-to-2 Configuration and 2400 pcphpl.

3 to 2 Configuration: 2400								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**222.628**	**0.481**	**35.578**	**1.452**	**17.078**	**53.667**	515	**252**
Late Merge	319.729	0.623	26.975	6.873	13.751	55	**364.667**	293.333
Signal Merge-30 s	1102.3	9.631	10.273	26.031				
Signal Merge-60 s	860.086	9.212	12.969	26.629				
Signal Merge-90s	839.852	11.935	13.234	48.946				
Signal Merge-120s	836.861	13.054	13.298	72.507				

The VISSIM outputs for the 3-to-2 lane configuration with 2600 pcphpl, as shown in Table 4.29, have similar trends to the outputs from the 2400 pcphpl case, at least in the first four measures. One key change is that the values for these four measures are not significantly different. The average delay time per vehicle, average number of stops per vehicle, and average speed are comparable. In addition, the average speed on the link prior to the closure is greater for late merge at 14.7 mph, compared to 13.8 mph for early merge. Furthermore, the maximum queue length is significantly greater for early merge than late merge. For these reasons, late merge is seems more efficient in managing traffic when compared to late merge. Signal merge with a 120-second cycle length is the optimal merge concept because of the ability to keep traffic moving. While the average stopped delay and average delay time per vehicle are not the smallest for signal merge with a 120-second cycle length, the average speed is more than double the speed for any other technique, by maintaining a speed of 49.5 mph.

Table 4.29 VISSIM Outputs for 3-to-2 Configuration and 2600 pcphpl.

3 to 2 Configuration: 2600								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**439.994**	**0.901**	**22.062**	**4.373**	13.871	**60.667**	515	279
Late Merge	458.967	1.354	21.522	22.794	**14.727**	94.333	**365**	**260**
Signal Merge-30 s	1143.25	10.395	9.961	28.867				
Signal Merge-60 s	902.687	9.66	12.476	27.35				
Signal Merge-90s	886.219	12.357	12.687	38.584				
Signal Merge-120s	882.516	13.506	**49.554**	74.709				

4.4.3.2.9 VISSIM Outputs for 3-to-1 Lane Configuration

The VISSIM outputs for a 3-to-1 lane configuration with user demands of 1800, 2000, 2200 pcphpl, as shown in Tables 4.30, 4.31, and 4.32 respectively, provide similar results to each other. In all three simulations, early merge and 60-second cycle length signal merge are the optimum merge techniques because both minimize delay and maximize speed in almost all measures. By comparing early merge to the 60-second cycle length signal merge, early merge provides optimum travel conditions in all measures. Additionally, the 2200 pcphpl user demand provides evidence for early merge because it not only minimizes delay and maximizes speed, but also minimizes queue. Initially, it was concerning that early merge minimized queue better than late merge because late merge typically takes full advantage of roadway capacity. While the queue was minimized in through lanes at the merge, the delay was experienced primarily by the merging lane. This also suggests that the demand is small enough that users were able to find enough gaps at the early merge to prevent queuing. From all the outputs though, early merge and the 60-second cycle length signal merge are ideal and early merge is optimum overall merge concept.

Table 4.30 VISSIM Outputs for 3-to-1 Configuration and 1800 pcphpl.

3 to 1 Configuration: 1800								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**2.175**	**0.0047**	**65.457**	0.0903	**66.449**	0	0	0
Late Merge	2.246	0.0053	65.445	**0.09**	66.272	0	0	0
Signal Merge-30 s	31.122	1.052	58.669	8.139				
Signal Merge-60 s	24.041	0.633	60.228	6.228				
Signal Merge-90s	24.052	0.411	60.203	8.062				
Signal Merge-120s	29.69	0.442	58.979	11.621				

Table 4.31 VISSIM Outputs for 3-to-1 Configuration and 2000 pcphpl.

3 to 1 Configuration: 2000								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**3.022**	**0.008**	**65.223**	0.224	**66.411**	0	0	0
Late Merge	3.364	0.01	65.14	**0.206**	66.253	0	0	0
Signal Merge-30 s	63.04	1.912	52.674	12.852				
Signal Merge-60 s	24.234	0.484	60.133	7.038				
Signal Merge-90s	28.16	0.47	59.3	9.13				
Signal Merge-120s	58.515	0.76	53.948	19.192				

Table 4.32 VISSIM Outputs for 3-to-1 Configuration and 2200 pcphpl.

3 to 1 Configuration: 2200								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**7.47**	**0.033**	**64.08**	**1.983**	**66.336**	**0**	**0**	**0**
Late Merge	65.804	0.823	53.244	15.219	33.321	183.67	705.67	1111
Signal Merge-30 s	213.57	3.495	35.02	19.304				
Signal Merge-60 s	74.872	1.34	52.369	19.692				
Signal Merge-90s	198.1	3.166	36.327	56.039				
Signal Merge-120s	199.3	3.116	36.661	61.486				

The 3-to-1 lane configuration with 2400 pcphpl demand, shown in Table 4.33, yields outputs that could support early or late merge. Although the average speed is greater for early merge, the average stopped delay per vehicle and queue lengths support late merge as the ideal merge concept between early and late merge. Signal merge with 60-second cycle lengths is the optimal signal merge concept because it minimizes the traffic of the work zone in three of the four measures. Additionally, signal merge is ideal when compared to late merge because average delay time per vehicle and average stopped delay per vehicle are minimized and the average speed is essentially the same as late merge. With a user demand of 2600 pcphpl, Table 4.34 shows similar trends to the 2400 pcphpl trends. In addition to minimizing delay, signal merge with a 60-second cycle length in the 2600 pcphpl case allows for a higher average speed of 23.2 mph, when compared to the average speed of late merge of 22.5 mph. For these reasons, signal merge with 60-second cycle lengths is the optimal merge concept for a 3-to-1 lane configuration and 2600 pcphpl.

Table 4.33 VISSIM Outputs for 3-to-1 Configuration and 2400 pcphpl.

3 to 1 Configuration: 2400								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	**189.59**	**2.6487**	**38.047**	62.108	**20.539**	1136	2305	**4057.3**
Late Merge	293.29	3.91	29.505	42.594	5.693	**1036**	**1158**	4689.3
Signal Merge-30 s	413.307	5.343	23.714	**24.031**				
Signal Merge-60 s	262.841	4.315	31.371	49.911				
Signal Merge-90s	364.355	6.705	25.86	75.431				
Signal Merge-120s	396.878	7.418	24.501	88.987				

Table 4.34 VISSIM Outputs for 3-to-1 Configuration and 2600 pcphpl.

3 to 1 Configuration: 2600								
	Avg Delay Time per Vehicle (s)	Avg # of Stops per Vehicle	Avg Speed (mph)	Avg Stopped Delay per Vehicle (s)	Avg Speed on Link Prior to Closure (mph)	Avg Queue Length (ft)	Max Queue Length (ft)	# of Stops within Queue
Early Merge	498.891	8.448	20.38	151.356	**6.121**	1136	2305	**4057.33**
Late Merge	436.494	**6.481**	22.504	58.755	4.994	**1092**	**1163**	4849
Signal Merge-30 s	551.589	7.32	18.826	**30.689**				
Signal Merge-60 s	**424.405**	7.446	**23.229**	57.206				
Signal Merge-90s	522.815	9.609	19.781	84.382				
Signal Merge-120s	563.83	10.052	18.644	97.938				

4.4.3.2.10 VISSIM Merge Concept Conclusion

The decision tree serves as a step-by-step process for construction-related activity at a highway work zone. The merge concepts that provide for ideal management of traffic are supported by the various VISSIM outputs. In general, assumptions about merge concept trends were accurate in that lower demand is best managed by early merge, low to moderate demand by late merge, and high demand by signal merge. Early merge is beneficial in lower demand situations and allows for highway users to merge into gaps prior to the distraction of the work zone. Late merge allows users to utilize all roadway capacity until the actual work zone, while fixed signal merge is best for high demand situations and can significantly reduce lane-change conflicts at work zone closures. Signal merge should also reduce rear-end conflicts for work zones with more than one lane closed. For both safety and minimizing delay in highway work zone situations, short cycle length like 30 seconds should not be used. Thus, Table 4.35 shows the overall optimal merge concept by lane configuration and VISSIM input demands, while Table 4.36 shows the overall optimal merge concept using volume and capacity. Table 4.37 shows the optimal cycle lengths by lane configuration and VISSIM input demands, while Table 4.38 shows the optimal cycle lengths using volume and capacity.

Table 4.35 Overall Optimal Merge Concept Using VISSIM Inputs.

	User Demand				
	1800 pcphpl	2000 pcphpl	2200 pcphpl	2400 pcphpl	2600 pcphpl
2-to-1	EM	EM	LM	SM-90s	SM-90s
3-to-2	EM	EM	EM	EM	SM-120s
3-to-1	EM	EM	EM	SM-60s	SM-60s

Table 4.36 Overall Optimal Merge Concept.

	User Demand				
	V < C	V < C	V = C	V > C	V > C
2-to-1	EM	EM	LM	SM-90s	SM-90s
3-to-2	EM	EM	EM	EM	SM-120s
3-to-1	EM	EM	EM	SM-60s	SM-60s

Table 4.37 Optimal Cycle Lengths for Signal Merge using VISSIM Inputs.

	User Demand				
	1800 pcphpl	2000 pcphpl	2200 pcphpl	2400 pcphpl	2600 pcphpl
2-to-1	60 sec	90 sec	90 sec	90 sec	90 sec
3-to-2	60 sec	60 sec	60 sec	60 sec	120 sec
3-to-1	60 sec	60 sec	60 sec	60 sec	60 sec

Table 4.38 Optimal Cycle Lengths for Signal Merge.

	User Demand				
	V < C	V < C	V = C	V > C	V > C
2-to-1	60 sec	90 sec	90 sec	90 sec	90 sec
3-to-2	60 sec	60 sec	60 sec	60 sec	120 sec
3-to-1	60 sec	60 sec	60 sec	60 sec	60 sec

Based on Greenshield's Traffic Flow Model and traditional belief among practitioners, it is anticipated that longer cycle lengths would maximize throughput flow and reduce delay the most. Therefore, the results presented in this analysis are somewhat inconsistent with the anticipated results. Greenshield's Traffic Flow Model assumes that in each cycle, time headways between consecutive vehicles decrease after the first five vehicles have been served into the green time, due to the phase-change lost time diminishing as a percentage of the cycle. Thus, the common practice is to use longer cycle lengths to increase capacity based on the assumption that saturation flow remains constant once the initial lost time has been accommodated.

In 2008, the FHWA conducted a study to provide guidance on effective strategies to alleviate the traffic congestion at signalized intersections and results were published in a Richard W. Denney Jr. article by the Transportation Research Board. Denney conducted a study to investigate the impacts of long green times and cycles at congested traffic signals. The tested hypothesis was whether headways increase with long green times and whether throughput increases as cycle length increases. The results showed that headways increased with long green times as a result of departing turning vehicles and that this effect could cause a significant increase in overall average approach headways (Denney, 2009). The results also showed that maximum throughput did not increase with longer cycles. With values derived from the field data, increasing the cycle did not increase throughput. In simulation, increasing the cycle length caused a reduction in throughput as a result of increasing the effect of departing turning traffic on the average headway (Denney, 2009).

Two reasons to explain these observations were hypothesized. First, it may be that during the red phase interval, vehicles who intend to turn at the intersection are trapped in a long queue in the through lanes. These vehicles would maneuver from through lanes to the appropriate lanes to turn during the through movement green interval, thus lessening the flow on the through lanes (Denney, 2009). Second, it may be that drivers respond to brake lights of the vehicle in front rather than the green light because their position in the queue is too far to clearly see the signal (Denney, 2009). In such cases, their perception–reaction time may no longer overlap with that of vehicles in front of them as characterized originally by Greenshield's Model. After analyzing the field data, the authors show that headways in lanes adjacent to turning lanes significantly increased and stop line flow reduced when the queue cleared to the upstream end of the turning lane. This finding suggests that maximum throughput is served when green times use the ability to feed the stop line with maximum flow. The simulation results of this study also indicate that larger percentages of green that can be used by flows unaffected by turning traffic causes higher throughput. The paper concluded by suggesting that by keeping the green time down to the point where only the queue to the upstream end of a 500-ft turn lane was served in each cycle, flow at the stop line is maintained close to ideal saturation and the overall throughput does not decrease. It is concluded, therefore, that the common belief that longer cycle lengths can be assumed to result in greater capacity cannot be supported by the behavior at this intersection (Denney, 2009).

The results of this study, although contrary to the traditional belief that throughput increases with longer cycles, can be explained and related to interesting findings of literature review presented above. The two mechanisms that seem to result in traffic behavior inconsistent with Greenshield's Model assumption are connected to the results of this study and are described next.

Although the departing turning vehicles concept does not directly seem relevant to this analysis, it can be translated to movements that cause friction in traffic stream, mainly the numbers of lane changes throughout the network. A major difference between this study network and a traditional intersection is that the conflicting approaches are lanes that run parallel to each other. Therefore, if lane change is not prohibited, drivers are likely to change lanes when they see an emptying queue in the other lane. Since the base FCSMC strategy mandates that drivers stay within their lanes about a mile ahead of the merge point, lane change maneuvers are limited, making it a less effective factor on throughput. Thus, in this case it would be reasonable to assume that while lane change movements have an impact, headways increase with increasing vehicle position primarily due to the lack of clarity with signal lights for some drivers and their considerations of safety distance with front vehicles. Figure 4.19 shows the VISSIM results of headway distribution with respect to vehicle position in queues for the 2-to-1 configuration with a 90-second cycle length at an input demand of 2200 VPHPL and a safety distance reduction factor (SDRF) of 0.4. Figure 4.20 shows the VISSIM results of headway distribution with respect to vehicle position in queues for the 2-to-1 configuration with a 120-second cycle length at an input demand of 2200 pcphpl and a SDRF of 0.4. This parameter means that in lane changing behavior, the safety distance of a driver to change to another lane is 40% of the original distance. This would mean that by reducing this factor, more aggressive lane changing behavior is triggered in the simulation. To evaluate the impact of this parameter on simulation results, sensitivity tests for SDRFs of 0.1 and 0.8 were conducted and headway distributions were analyzed. The results of this evaluation will be presented later in this report. It is also appropriate to perform a sensitivity test of car following behavior parameters in order to evaluate their impact on headway distributions.

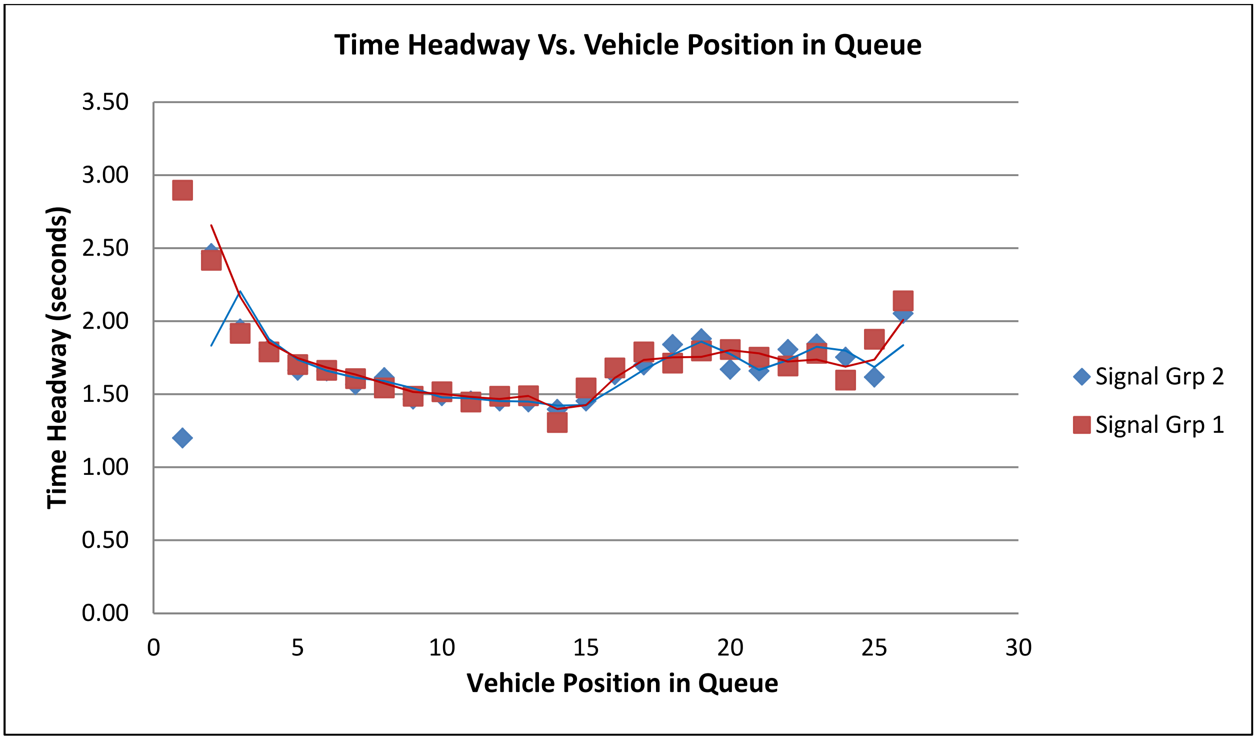


Figure 4.19 Time Headway versus Vehicle Position in Queue for 90-Second Cycle Length.

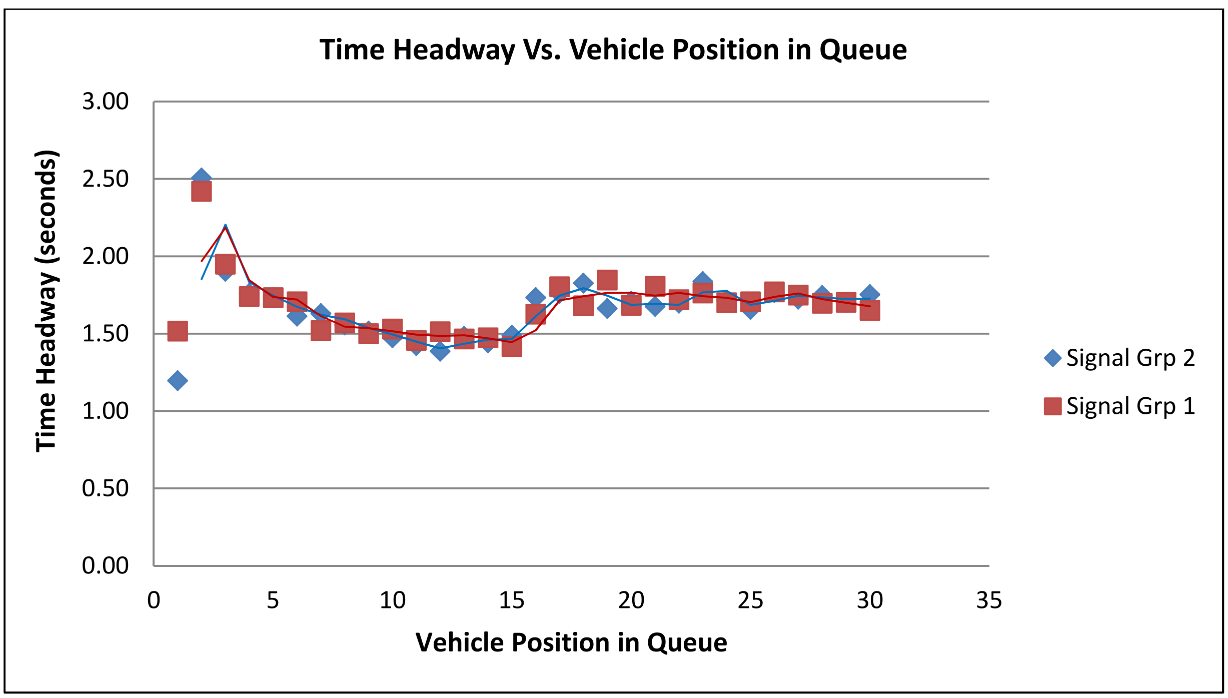


Figure 4.20 Time Headway versus Vehicle Position in Queue for 120-Second Cycle Length.

As shown in Figures 4.19 and 4.20, after the first five vehicles are served for both cycle lengths, headways decrease to about 1.5 seconds for vehicles up to 15th position in the queue and then increase to approximately 1.75. This mechanism can explain our observations for the 2-to-1, 3-

to-1, and 3-to-2 configurations. In the 2-to-1 and 3-to-1 configurations, the lane volume distributions are more uneven compared with the 3-to-2 configuration, meaning that the queue lengths can grow large with long cycle lengths and would thus result in reduced throughput and increased delays.

Allowing lane changes can be expected to both improve and worsen traffic conditions, depending on the start point of lane change prohibition. Allowing lane changing provides drivers with the opportunity to use available capacity, but can also increase traffic stream friction. Additionally, traffic flow throughput can be optimized with managing queue lengths through appropriate selection of the lane change start point. These tests were performed for the 2-to-1 configuration. First, a set of runs were conducted and lane change was allowed throughout the network. In the second set of runs, lane change was prohibited about half a mile from the merge point compared with the base scenario of prohibiting lane change one-mile ahead of merge point. The results of this analysis are presented in Tables 4.39 and 4.40. Table 4.40 shows the relationship between queue length, ratio of throughput flow and demand flow, and total number of lane changes for the half mile start point of the lane change.

Table 4.39 Sensitivity Test for Start of Lane Change Prohibition—Ratio of Throughput Flow to Demand Flow.

	1800 vplph					2000 vplph				
Configuration	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length
Lane change prohibition at one mile prior to merge point	0.91	1.01	1.00	1.00	1.00	0.89	0.95	1.00	1.01	1.00
Lane change prohibition at half a mile prior to merge point		0.85	1.00	1.00	1.00		0.95	1.00	1.01	1.00
No lane change prohibition		0.86	0.89	0.89	0.91		0.78	0.80	0.80	0.82

	2200 vplph					2400 vplph				
Configuration	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	Baseline	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length
Lane change prohibition at one mile prior to merge point	0.83	0.87	0.96	0.95	0.96			0.88	0.88	0.88
Lane change prohibition at half a mile prior to merge point		0.87	0.96	0.96	0.96			0.88	0.87	0.88
No lane change prohibition		0.70	0.73	0.73	0.74			0.67	0.66	0.68

Note: Pink, green, and purple highlights indicate optimal cycles for each scenario.

Table 4.40 Sensitivity Test for Start of Lane Change Prohibition—Delay (Seconds per Vehicle).

	1800 vplph				2000 vplph			
Configuration	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length
Lane change prohibition at one mile prior to merge point	33.0	21.5	26.1	31.4	242.8	31.5	40.9	44.6
Lane change prohibition at half a mile prior to merge point	574.1	21.6	26.4	31.9	246.4	33.9	41.5	45.5
No lane change prohibition	511.2	443.2	443.0	363.0	749.8	602.2	685.4	582.1

	2200 vplph				2400 vplph			
Configuration	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length	30 Sec Cycle Length	60 Sec Cycle Length	90 Sec Cycle Length	120 Sec Cycle Length
Lane change prohibition at one mile prior to merge point	414.0	214.6	236.9	229.3		355.7	364.4	362.0
Lane change prohibition at half a mile prior to merge point	502.6	214.3	233.4	236.5		443.8	470.8	458.4
No lane change prohibition	845.8	780.1	770.7	754.3		836.6	839.9	818.7

Note: Pink, green, and purple highlights indicate optimal cycles for each scenario.

As shown in Tables 4.39 and 4.40, the results of moving the lane change start point to a half mile from the merge point is similar to the base case where this point was a mile from the merge point. These results can be interpreted to indicate that the queue downstream of the lane change point is more likely to clear within the green time of the 60-second cycle.

After analyzing the impact of a work zone on network performance and determining the appropriate lane control technique for various situations, it is paramount to consider safety in the analysis and merge concept decisions that were made. Ultimately, the major operational concerns with work zone lane closures are delay and minimizing delay in a way that provides safe travel conditions.

Chapter 5. Analysis

5.1 Introduction to Safety Analysis

The purpose of this section is to analyze traffic safety performance in highway work zone areas, with and without Fixed-Cycle Signal Merge Control (FCSMC), under various traffic and geometric conditions. For this task, a two-stage, simulation-based approach was used. In the first stage, micro-simulation models were developed and calibrated based on field data to generate vehicle trajectories. In the second stage, the FHWA's Surrogate Safety Assessment Model (SSAM) was employed to identify potential conflicts under different conditions. In this task, two types of work zone scenarios were tested: 1) a set of hypothesized work zone scenarios and 2) a set of real-world work zone scenarios in Houston where we conducted field studies. The results of this study showed that, in most cases, the Fixed Cycle Work Zone Traffic Signal Control (FCWZTSC) strategy can significantly reduce conflicts caused by work zone construction, especially lane-change conflicts. However, FCWZTSC is not suggested when the traffic volume is relatively light and the use of very short signal cycle lengths (30 seconds or less) is also not recommended.

The MUTCD provides basic guidelines for traffic control devices in work zone areas, including placing "Road Work Ahead" signs, flash yellow arrows, etc. It also suggests the location where these signs should be placed. Although MUTCD traffic control strategies work well in most work zone areas, in some conditions, especially when the traffic volume is high, traditional work zone traffic control strategies cannot effectively control work zone congestion and result in extremely long queues and problematic driving behaviors, such as queue jumping. As a result, innovative traffic control strategies were developed to reduce congestion and crash potential in work zones. The FCWZTSC is an example of an innovative strategy. In the following sections, the FCWZTSC strategy is compared to traditional traffic control strategies like early and late merge to evaluate the safety impacts of FCWZTSC.

5.1.1 Introduction to Concepts in SSAM

To supplement the existing studies, simulation studies were performed by the researchers using VISSIM in conjunction with Siemens SSAM, which was developed by the FHWA.

A traffic conflict modification factor (TCMF) was developed in this study. Similar to the CMF presented in the American Association of State Highway and Transportation Officials Highway Safety Manual (AASHTO HSM), TCMF factors were provided for estimating the expected changes of traffic conflict frequency after implementing specific geometric changes associated with an auxiliary lane. The TCMF was calculated as follows:

$$\text{TCMF} = \frac{\text{Traffic Conflict Frequency after Treatment}}{\text{Traffic Conflict Frequency before Treatment}} \times 100\% \qquad \text{(Equation 5.1)}$$

A TCMF with a value less than 1.0 means the treatment can potentially reduce the occurrence of traffic conflicts and improve the safety performance; a TCMF with a value greater than 1.0

indicates the treatment can potentially increase the occurrence of traffic conflicts and compromise safety performance.

The traditional way of assessing safety impacts is to analyze historical crash data at the study sites. Recognizing the fact that crashes are rare events and subject to randomness inherent to small numbers, the crashes are normally observed over a relatively long period, such as 1–6 years. This process is relatively slow to reveal the need for remediation, and also not applicable to conduct safety assessment for design of roadways that have not been built or operational strategies that have not been applied in the field.

An available alternative to assess safety impacts of roadway designs is to use microscopic traffic simulation models to obtain useful safety surrogate measures that can reflect their safety impacts. A typical procedure for applying such methods begins with development of microscopic traffic simulation scenarios characterizing the roadway designs to be examined. Then, together with operational measures, safety surrogate measures, which can be derived from the results of the microscopic traffic simulation, are computed, extracted, and analyzed to estimate the conflict frequency and the safety risk. This process is depicted in Figure 5.1. In this task, the SSAM developed by Siemens was used for assessing the safety impacts of various design options. Directly processing vehicle trajectory data obtained from the results of microscopic traffic simulation enables researchers to estimate traffic conflict frequency.

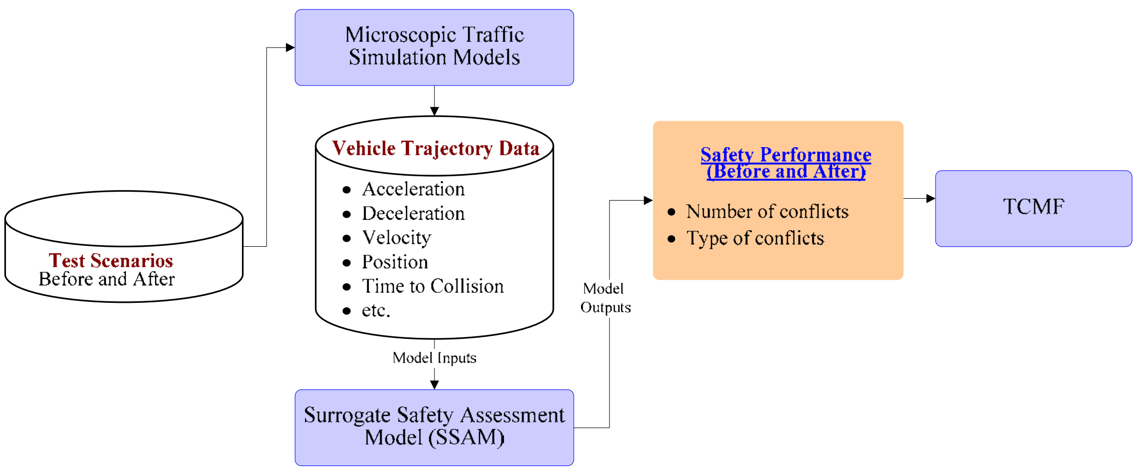


Figure 5.1 Method of Estimating Traffic Conflict Frequency.

5.2 Scenario Design and Experimental Results—Stage One: VISSIM Model

Various scenarios were designed to calculate TCMF under different conditions, including traffic volume, number of lanes, number of closed lane(s), and cycle lengths. The scenarios included two parts: 1) a set of hypothesized work zone scenarios, and 2) a set of real-world field-studied work zone scenarios in Houston.

5.2.1 Hypothesized Work Zone Scenarios

According to the VISSIM simulation experiments conducted in Task 5, four different levels of traffic demand were tested: 1800 vehicles per hour per lane (VPHPL), 2000 VPHPL, 2200 VPHPL, and 2400 VPHPL. Four different cycle lengths were selected: 30 seconds, 60 seconds, 90 seconds, and 120 seconds. In addition, three different types of roadway closure were designed:

- Two-lane highway, one lane closed,
- Three-lane highway, one lane closed,
- Three-lane highway, two lanes closed.

Therefore, for scenarios with FCWZTSC, 48 scenarios (4 volume levels × 4 cycle lengths × 3 roadway closure types) were created; for the baseline scenarios without FCWZTSC, 12 scenarios (4 volume levels × 3 roadway closure types) were created. Please refer to Figure 5.2 for the layouts of hypothesized work zone scenarios.

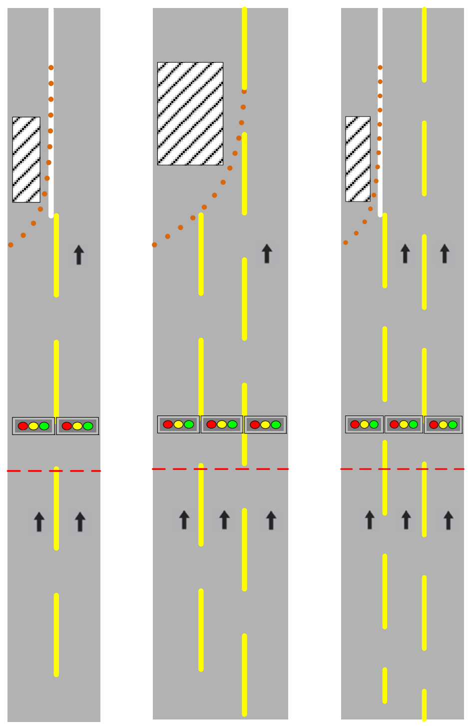


Figure 5.2 Layouts of Hypothesized Work Zone Scenarios.

5.2.2 Real-World Work Zone Scenarios

The second part of the simulation experiment is based on a real-world work zone in Houston. The data collected, including volumes and average travel times, were used to build and calibrate the model. Figure 5.3 shows the work zone layout.

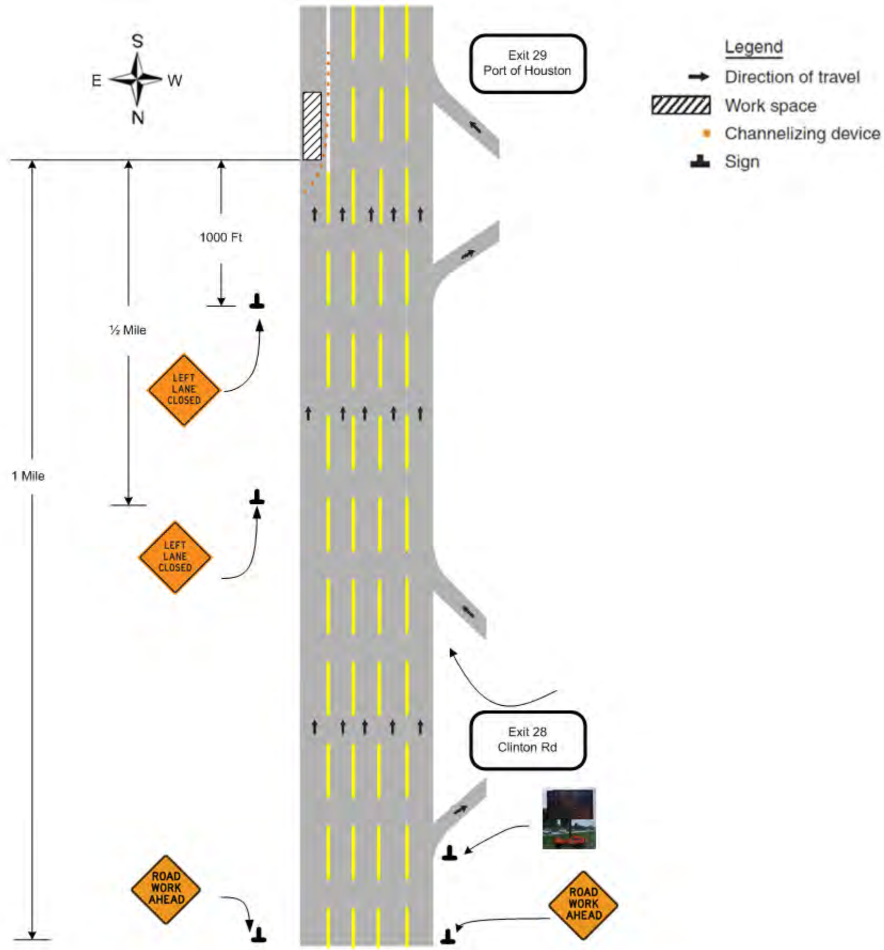


Figure 5.3 Work Zone Layout of Selected Houston Site.

The authors selected a real Houston work zone scenario to simulate different traffic conditions. The five-lane highway had one lane closure and four different traffic volume levels: 1500, 1600, 1800, and 2000 VPHPL. Note that 1500 VPHPL is the actual average hourly traffic volume at this location from 5:00 p.m. to 6:30 p.m., and 1800 VPHPL is the peak 5-minute traffic flow at this work zone multiplied 12. In addition, the four signal cycle lengths are the same as the hypothesized model, which are 30 seconds, 60 seconds, 90 seconds, and 120 seconds. Besides the real-world work zone layout (five lanes with one lane closed), another work zone layout was also tested, which is four lanes with one lane closed. For this case, three different levels of traffic volume were tested, which are 1600, 1800, and 2000 VPHPL. Finally, two different types of roadway closure were designed:

- Five-lane highway, one lane closed,
- Four-lane highway, one lane closed.

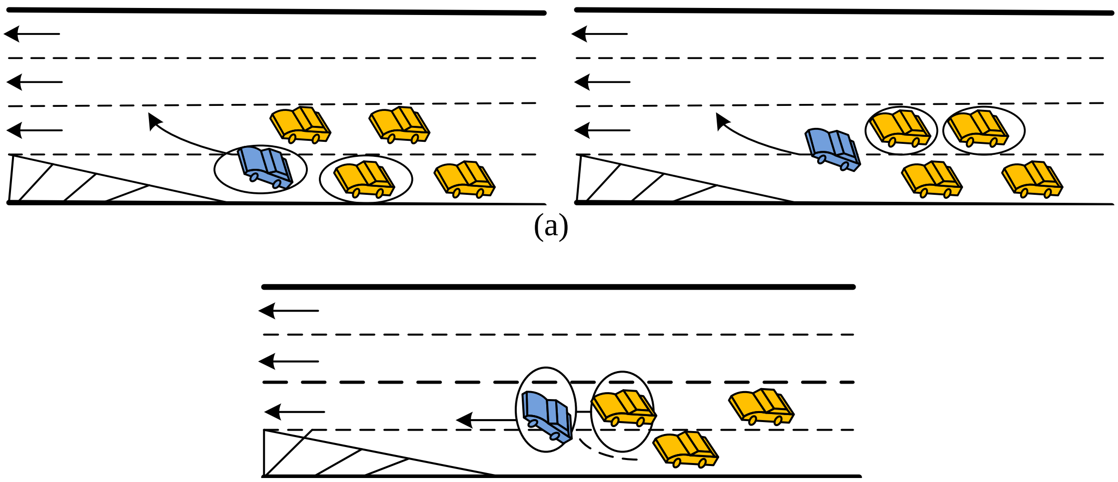
Therefore, 28 scenarios (4 volume levels × 4 cycle lengths + 3 volume levels × 4 cycle lengths) were created for scenarios with FCWZTSC; for the baseline scenarios without FCWZTSC, 7 scenarios (4 volume levels + 3 volume levels) were created.

5.3 Stage Two: SSAM Model Traffic Conflicts

In all experiments, the simulation of each sub-scenario covered 90 simulation minutes, and was conducted with 10 or 20 different random seeds. Each run generated one vehicle trajectory file, which was then input to SSAM for processing. SSAM produced estimates of traffic conflicts for each scenario.

5.3.1 Conflicts Related to Work Zone Closure

Two types of conflicts are highly related to work zone closure: “rear-end conflicts” and “lane-change conflicts.” Figure 5.4 (a) illustrates two instances of rear-end conflicts, and (b) shows a typical lane-change conflict.



(a)

(b)

Figure 5.4 Conflicts Related to Work Zone Closure.

5.4 SSAM Outputs

5.4.1 Outputs for 2-to-1 Lane Configuration

The first modeled geometric design was the 2-to-1 lane configuration with four different volume levels, including 1800 pcphpl or VPHPL, 2000, 2200, and 2400 VPHPL. For the signalized lane control strategy, four different cycle lengths were tested, including 30, 60, 90, and 120 seconds. Figures 5.5 and 5.6 are the lane-change and rear-end conflicts comparison results.

Figure 5.5 demonstrates that in all conditions except 1800 VPHPL, implementing the FCWZTSC strategy could significantly reduce lane-change conflicts. Since there is minimal traffic congestion at the work zone merge point, vehicles can easily pass the merge point without conflict. Thus, the traditional traffic control strategy works adequately. Under light traffic demands, use of the FCWZTSC strategy will increase vehicle stops and cause more traffic conflicts. In addition, the 30-second cycle length causes the most conflicts and is not recommended.

Figure 5.6 shows that implementation of the signalized merge control strategy (FCWZSC) increases rear-end conflicts for all volume conditions, especially for shorter cycle lengths. This finding is reasonable because use of FCWZSC will cause more vehicle stops when the cycle length is short, which increases the chance of rear-end conflicts. See Appendix B.1 and B.2 for lane-change and rear-end conflict look-up tables for a 2-to-1 lane configuration, showing before and after implementation of FCWZSC and the TCMF.

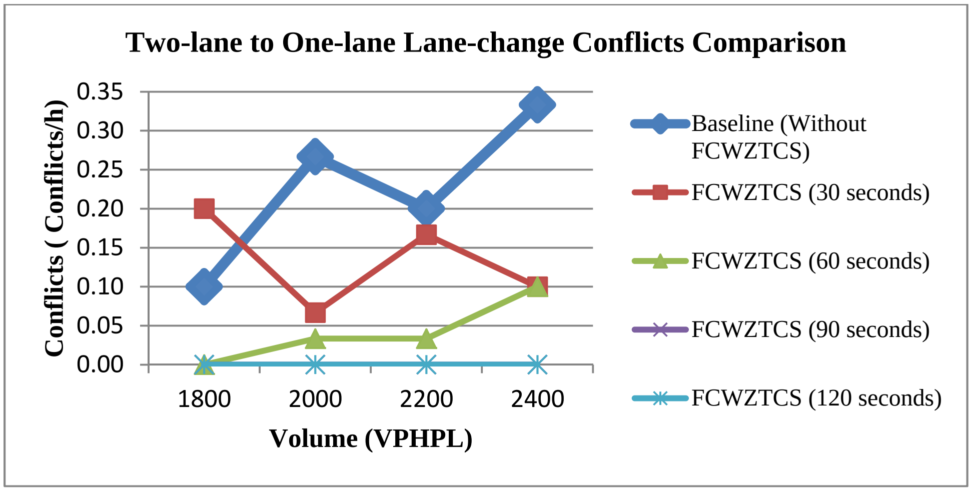


Figure 5.5 Lane-change Conflicts versus Cycle Length for 2-to-1 Lane Configuration.

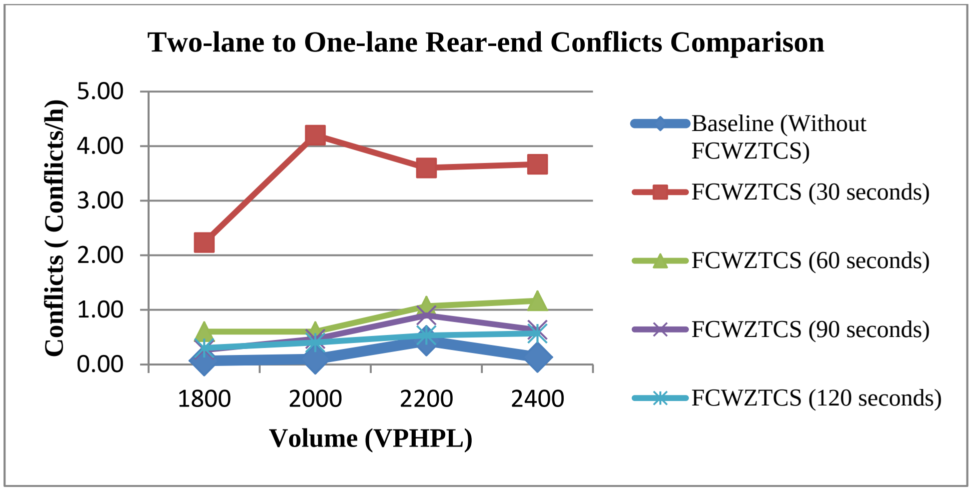


Figure 5.6 Rear-end Conflicts versus Cycle Length for 2-to-1 Lane Configuration.

5.4.2 Outputs for 3-to-2 Lane Configuration

Figures 5.7 and 5.8 show the lane-change and rear-end conflict comparison results for the 3-to-2 lane configuration. For light traffic demands like 1800 or 2000 VPHPL, the MUTCD lane control strategy works well because it has the least lane-change and rear-end conflicts. With the increase of volume, more traffic conflicts occur due to the congested traffic condition, thus highlighting the benefits of FCWZSC. See Appendix B.3 and B.4 for lane-change and rear-end conflict look-up tables for a 3-to-2 lane configuration, showing before and after implementation of FCWZSC and the TCMF.

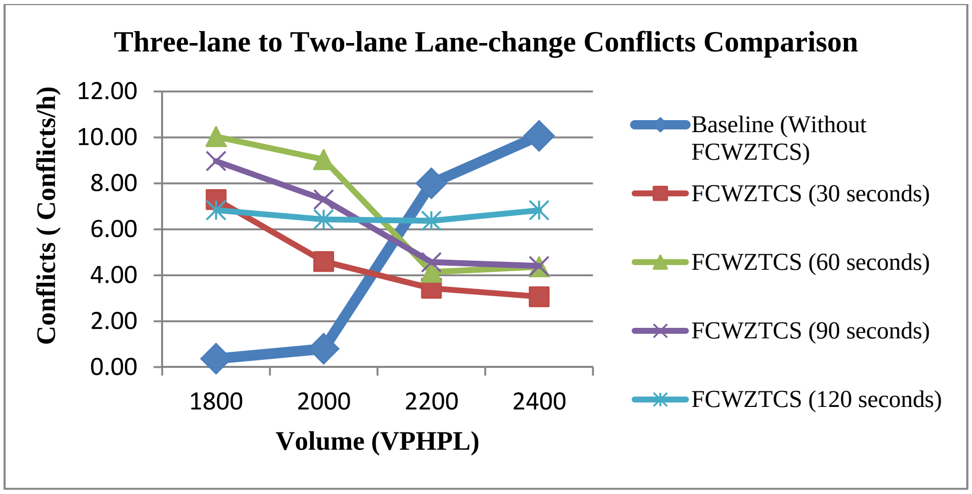


Figure 5.7 Lane-change Conflicts versus Cycle Length for 3-to-2 Lane Configuration.

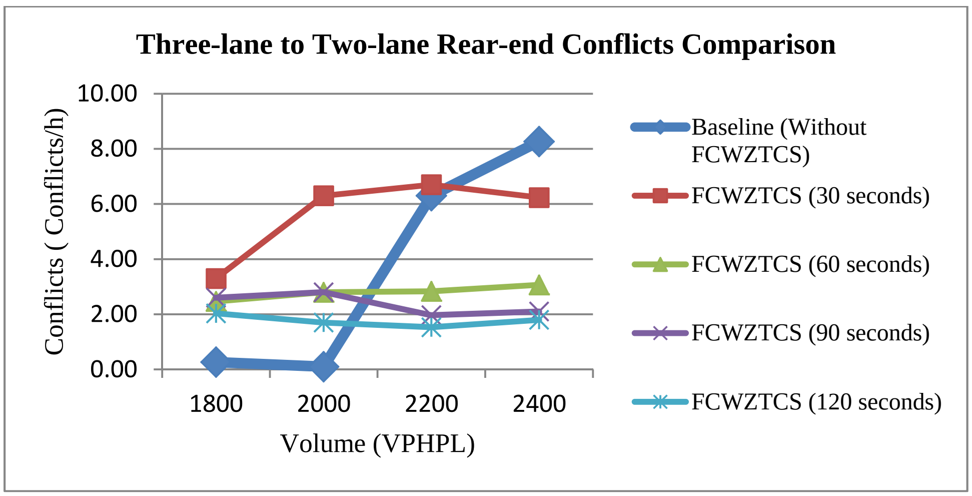


Figure 5.8 Rear-end Conflicts versus Cycle Length for 3-to-2 Lane Configuration.

5.4.3 Outputs for 3-to-1 Lane Configuration

Figures 5.9 and 5.10 show the lane-change and rear-end conflicts comparison results for the 3-to-1 lane configuration. Figure 5.9 demonstrates that FCWZTSC can significantly reduce lane-change conflicts, especially when the traffic demand is high.

Similar to the other two lane configurations, Figure 5.10 shows that when traffic demand is lighter (as represented here by cases of 1800, 2000, or 2200 VPHPL), the FCWZTSC does not reduce rear-end conflicts. When traffic demand reaches 2400 VPHPL, FCWZTSC starts to work well and reduces rear-end conflicts. See Appendix B.5 and B.6 for lane-change and rear-end conflict look-up tables for a 3-to-1 lane configuration, showing before and after implementation of FCWZSC and the TCMF.

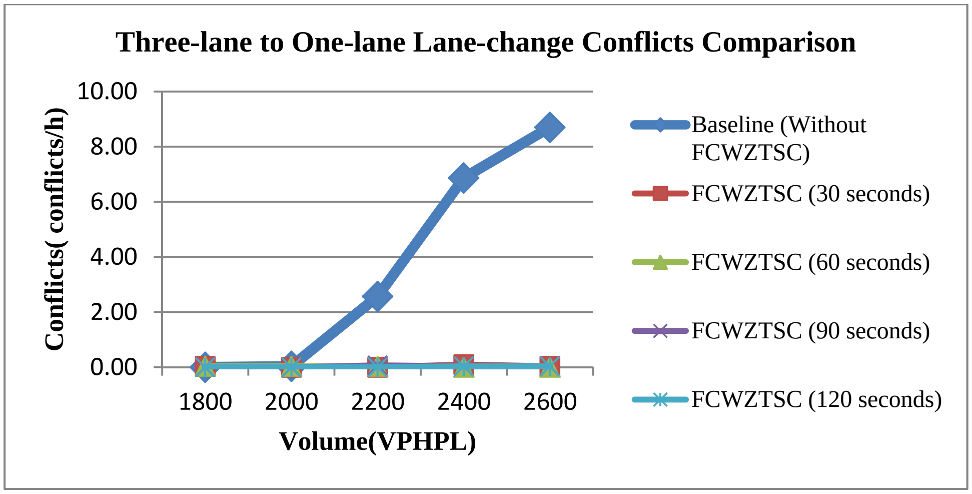


Figure 5.9 Lane-change Conflicts versus Cycle Length for 3-to-1 Lane Configuration.

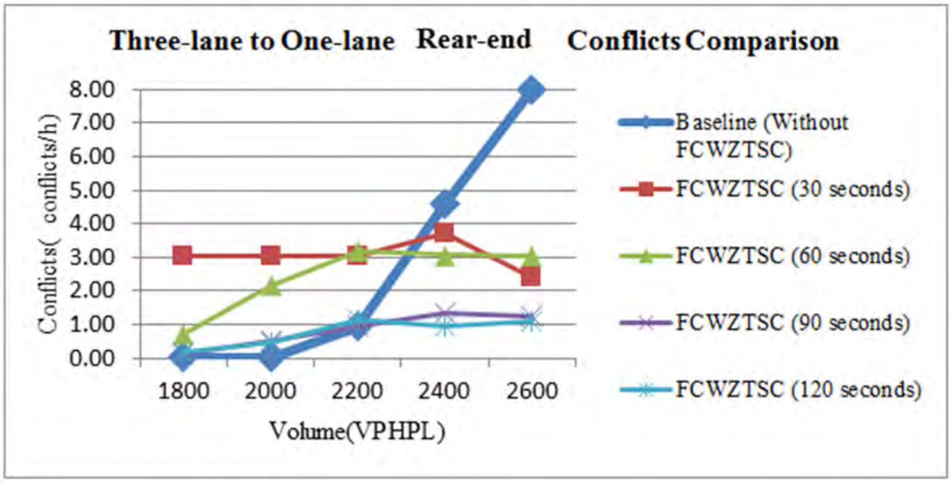


Figure 5.10 Rear-end Conflicts versus Cycle Length for 3-to-1 Lane Configuration.

Chapter 6. Recommendations and Conclusions

The first section of this chapter describes the recommended procedure to evaluate and select a traffic management plans for long-term work zones. The process is presented as a decision tree, and was developed based on the analysis of field data, literature, and simulation experiments. Additionally, Section 6.1.7 presents a simple Excel-based tool that may be used to estimate the length of the queues formed upstream from a work zone. Section 6.2 summarizes the pilot training workshop that was performed for TxDOT to explain and provide examples for the steps represented in the decision tree or procedure. The slides presented at this pilot training workshop are shown in Appendix E. Section 6.3 summarizes the main outcomes of the work conducted for this project and suggests further research directions.

6.1 Work-Zone Traffic Management Plan Evaluation Process

The proposed decision tree provides a step-by-step procedure to evaluate traffic management plans for long-term work zones. The approach takes into account the number of lanes that must be closed, evaluates the suggested times of day to close work zone lanes based on historical demand data, and provides guidance regarding the best type of lane management or merge concept that can be applied. The decision tree also considers the diversion of traffic to alternative roads, which is highly dependent on location-specific factors such as traffic demand, roadway capacity, traffic composition, traffic variability, physical roadway configuration, and available alternative network paths. A flow chart representation of the procedure developed for this project is presented in Appendix C. The following sections describe each of the steps included in the decision tree, as well as the suggested approach to estimate queue lengths.

6.1.1 Obtain Traffic Control Plan Data

The first input to the procedure is the proposed traffic control plan data. In the traffic control plan, the lane configuration of the work zone should be determined. In addition, the hours that work can be completed at the work zone should be negotiated with the contractor. Lastly, the per-lane capacity through the work zone should be estimated, in an effort to determine which hours have a minimum volume-to-capacity (V/C) ratio, such that the ratio is less than one. The capacity of the work zone section can be determined using a general rule of thumb of 1,800 passenger cars per hour per lane (pcphpl). Although the 2000 Highway Capacity Manual (HCM) suggests that under ideal conditions a multi-lane freeway may have a capacity of more than 2,000 pcphpl, the use of 1,800 pcphpl is a reasonable approximation of capacity loss due to typical work zone changes to driver behavior (Texas MUTCD, 2012). As a result of merging between lanes and the likelihood of drivers being distracted by work zone signage, equipment, and workers, lane capacities through work zones are typically less than ideal values. Thus, the presence of the work zone constitutes a condition that is not “ideal” and should reduce the per-lane capacity to a value less than the HCM ideal capacity. However, since the 1800 number should be considered a very liberal “rule of thumb," experience or empirically measured values for specific locations and situations should be used instead, if available.

6.1.2 Assess "Before" Conditions

The next step in the procedure is to assess traffic conditions before the work zone is in place. The first step required to assess the before-work-zone conditions is to collect data. Ideally, data should be collected 24 hours per day, 7 days a week, during a representative week of the year. Obtaining a complete set of data is often difficult and it may be necessary to work from information obtained for a single day of the week, or even to extrapolate peak-hour data. Further, in some cases, data collection may not be feasible; under such circumstances, TxDOT can contact the corresponding metropolitan planning organization (MPO) and request an estimation of traffic volumes at the desired location based on the corresponding regional model. The collected or synthetized data is used to generate a plot such as the one depicted in Figure 6.1 that displays volume in vehicles per hour, as opposed to hours of the day. This plot is useful to understand the nature of the travel demand through the work zone area. For example, Figure 6.1 shows similar volume counts on Monday through Thursday as on Friday, although some data points are slightly higher on Friday. The Saturday and Sunday data shows a greater volume in the early morning hours than during the week, but after 4 a.m., the traffic counts are significantly less on weekends.

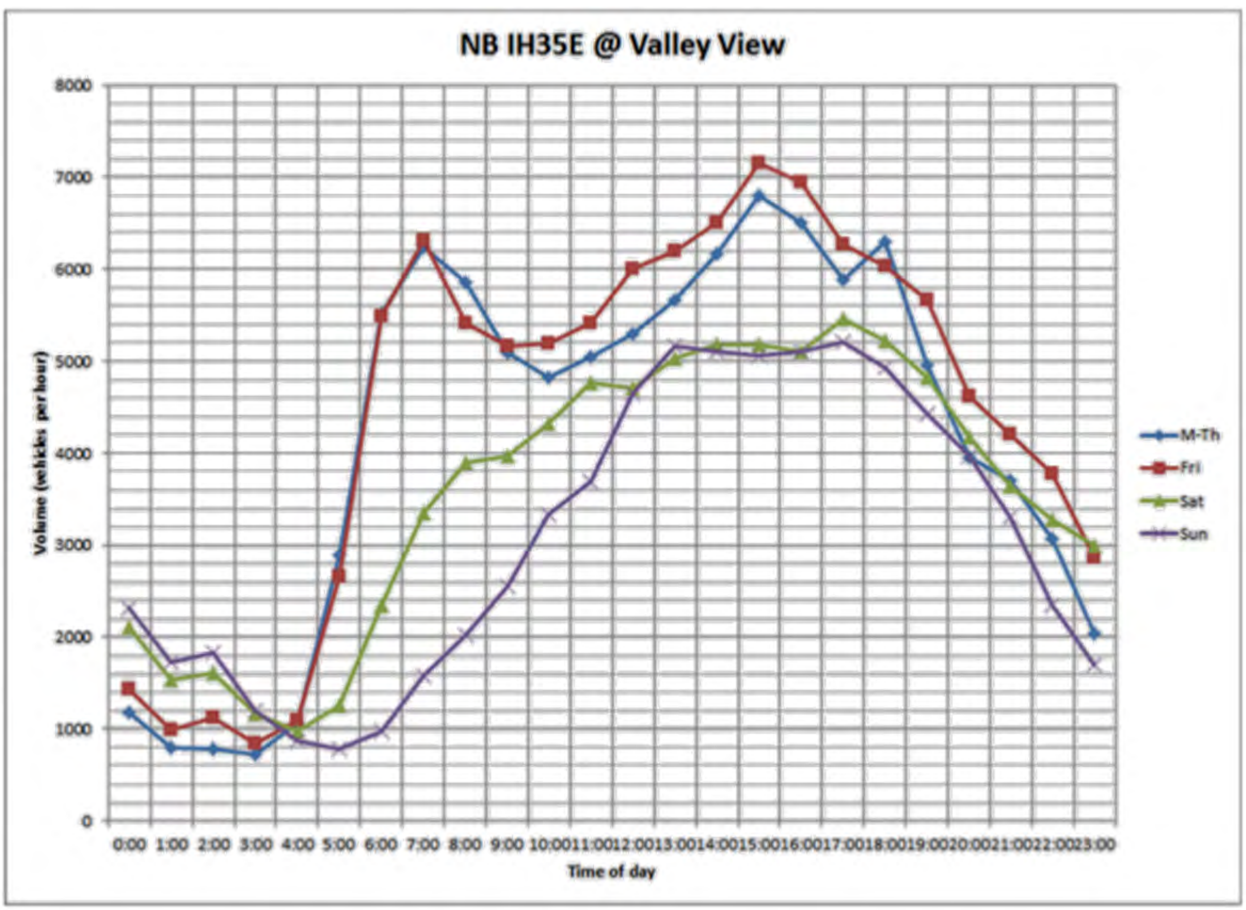


Figure 6.1 Example Count Data from I-35E near Dallas, Texas.

The second step in the process of assessing "before" conditions involves analyzing hourly V/C conditions. A table such as the one shown in Table 6.1 may be used to determine which hours of the day would be best for highway work zone construction. In Table 6.1, period A represents weekday peak conditions, when the full capacity of the corridor is needed. Therefore, between the hours of 6 a.m. and 8 p.m. on Monday through Friday, work zone lane closure(s) cannot take place because the full roadway capacity is needed. Periods B, C, and D are weekend peak, off-peak, and night respectively, and require less than full capacity. Thus, these times would be the only options for highway work zone lane closure(s), but TxDOT would have to determine which of these hours would be best for roadway operations and for the contractor.

Table 6.1 Hourly Volume-to-Capacity Conditions for I-35E Example Site near Dallas, Texas.

	Northbound IH 35E			
	Sunday	Monday-Thursday	Friday	Saturday
0:00	D	D	D	D
1:00	D	D	D	D
2:00	D	D	D	D
3:00	D	D	D	D
4:00	D	D	D	D
5:00	D	D	D	D
6:00	D	A	A	D
7:00	D	A	A	C
8:00	D	A	A	C
9:00	D	A	A	B
10:00	D	A	A	B
11:00	B	A	A	B
12:00	B	A	A	B
13:00	B	A	A	B
14:00	B	A	A	B
15:00	B	A	A	B
16:00	B	A	A	B
17:00	B	A	A	B
18:00	B	A	A	B
19:00	B	A	A	B
20:00	B	A	A	C
21:00	D	C	C	C
22:00	D	D	D	D
23:00	D	D	D	D

A	Weeday peak
B	Weekend peak
C	Off-peak
D	Night

6.1.3 Assess "After" Conditions

The third step in the procedure is assessing the highway conditions after the work zone is in place. With the presence of the work zone on the highway, some amount of diversion is expected based on the availability of alternative routes and the congestion level on such roads. To estimate the highway conditions and volumes after the work zone is in place, diversion estimation is necessary as long as at least one alternative route is available. Prediction of the change in traffic demand at a work zone site due to work zone capacity changes is a network issue. If the network surrounding the work zone provides feasible alternative paths, diversion to those paths is reasonable. The best procedure for estimating traffic diversion from a work zone to alternative paths is using a network model. If available, a straightforward before-work-zone to after-work-zone comparison of link volumes can provide an estimate of diversion. Two rather different traffic assignment tools are available for this process.

MPOs in urban areas maintain a network model that could be a handy tool for assessing diversion. Traditionally, MPOs have used a static traffic assignment (STA) procedure as the fourth step in the four-step demand estimating process. STA models require minimal detail to describe the subject network and minimal computational resources to produce assigned link volumes. However, they frequently predict link traffic demands that are greater than link capacity (demand > capacity); since all highways are not actually included in the typical coded network, STA models may not be able to provide detailed evaluations of specific links or routes.

A few MPOs are now implementing dynamic traffic assignment (DTA) tools to replace the traditional STA processes. The DTA process generally requires a detailed network description and never predicts link volumes that exceed capacity. The DTA process can be expected to yield link traffic volumes as opposed to link demands produced by a STA process.

The most accurate method to determine work zone traffic diversion rates is the application of a DTA to produce the before-work-zone and after-work-zone link volumes. Refer to Chapter 4.2 for more on the DTA modeling approach and scenarios. DTA analysis results can be used to evaluate individual travel time and cost measures, as well as system-wide network measures (Chiu et al., 2010). For these reasons, DTA is increasingly being adopted by planning organizations across the United States because it is better suited for use in forecasting. The DTA approach in this case involves running the model, ideally a 24-hour model, with and without the planned work zone to compare before and after traffic volumes. If the 24-hour model is not available, a model for the proposed period of analysis will be developed. While the DTA approach uses iterative algorithmic procedures to describe individual routes, traditional STA uses volumes on a link directly from the loading of the origin-destination matrix (Chiu et al., 2010). Using STA, the travel times on each link are summed together to determine the route travel time. Thus, STA has serious limitations as far as realistically representing the process that leads to congestion and increased travel time (Chiu et al., 2010).

While application of DTA would provide the best possible assessment of diversion around a work zone, a before-after comparison of link volumes from a static assignment process would likely be the next best procedure. A STA tool is generally available in every Texas metropolitan area.

If neither DTA nor STA are available, research about urban freeway short-term lane closures from South Dakota State University suggests that up to 15% of traffic will divert without advanced user information during time intervals when 1,000 pcphpl is exceeded, which is called *natural diversion* (Qin et al., 2010). Examples of observed diversion rates and are shown in Appendix D.

6.1.4 Approve Traffic Control Plan

In order to determine whether to approve the traffic control plan, the hours and days requested by the contractor should be compared to the hours that are deemed useful with a V/C ratio less than one. If the needs cannot be met, the traffic control plan—and the work zone lane configuration in particular—must be adjusted.

If STA is used for the diversion computation, the final value of demand/capacity (D/C) may be greater than one. If D/C is less than one (thus, the demand is less than capacity) for fewer daily hours than requested by the contractor, the lane configuration of the work zone section and the number of hours requested by the contractor must be readdressed before continuing through the rest of the decision tree. If D/C is less than one for more hours than the number of hours requested by the contractor, the contractor could be provided the number of hours requested for work zone activity.

If DTA is used to compute diversion, the final volume through the work zone will not exceed the capacity. However, the time for drivers to traverse the work zone may become longer compared to the before case. In order to generate a V/C ratio consistent with the rest of this analysis, it is proposed to compute V/C as the total demand through the work zone during the entire simulation period and the corresponding capacity for the number of hours during which demand is loaded into the network.

6.1.5 Choose Merge Concept

As first introduced in Chapter 2.1, early merge, late merge, and signal merge all provide benefits in certain scenarios. Early merge is hypothesized as preferred with conditions where the V/C ratio is less than one and thus can yield less queuing and lower user costs. If demand approaches capacity and queuing is expected, late merge is hypothesized to be preferred. Both early and late merge can be implemented as static or dynamic, such that dynamic early merge can switch to early merge based on traffic conditions. A merge concept that is set up static uses signage that displays a single message in the same location at all times, regardless of traffic conditions. Conversely, a dynamic merge concept refers to real-time control measures that are used to determine which signage should be used upstream to inform drivers of upcoming conditions.

In order to determine the appropriate merge concept in the decision tree or procedure, various conclusions were made after using VISSIM (described in Section 4.4.3.2). The early merge concept works best with low volume conditions and becomes highly problematic when traffic demand approaches or exceeds work zone capacity. Under these conditions with higher demand, incidents of queue jumping, excessive lane changing and crashes tend to escalate. In cases where hours and days of work zone activity must include times when the demand exceeds capacity, late merge and signal merge are the best options. Late merge schemes generally are designed to use all available lane space prior to the work zone for queue storage; therefore, they provide the best available procedure if traffic demand approaches or exceeds work zone capacity. In addition, for times in which demand exceeds capacity of the work zone, use of the signal-controlled merge offers the potential to reduce queue jumping, lane changing, and associated crashes.

6.1.6 Select Sign Placement

Using the Texas MUTCD or the TxDOT standards sheet, determine the placement of signage and variable message signs in the work zone depending on the merge concept that is selected. Once the sign placement is finished, the decision-making process is complete. However, depending on the ratio of demand to capacity at the work zone area, queues may form. Further, if the number of vehicles exiting the freeway upstream from the work zone increases, additional weaving conflicts may lead to longer queues than expected. The following section describes a simple Excel-based tool that can be used to estimate queue lengths under a variety of scenarios. While queue length estimation is beyond the scope of this project, the proposed tool is described in order to showcase the importance of considering traffic conditions realistically in order to effectively deploy variable message signs (VMS).

6.1.7 Perform Excel-Based Queue Length Estimation Procedure

An Excel-based methodology has been developed to estimate queue length based on expected flow through the work zone area and nearby exit ramps, and proposed capacity reductions. The methodology may be used to estimate appropriate location of VMS, if these are to be used. It can also be used to reassess VMS deployment decisions based on traffic data obtained after the work zone is in place, such as exit ramp volumes. Further, if real-time data becomes available, the framework could be extended to support real-time traffic management.

The following sections briefly describe the theory underlying the proposed methodology, describe its utilization, and present some examples of its results.

6.1.7.1 Methodology

This work uses Yperman's Link Transmission Model (LTM) to estimate the length of queues due to freeway closures based on prevailing conditions (Yperman, 2007). In an LTM model, homogenous roadway segments are represented by links connected by nodes. The solution of an LTM model involves computing a number of state variables at every timestep Δt. Every link a is characterized by its length L_a, free flow speed u_a^f, capacity q_a^{max}, jam density k_a^{jam}, and backwards wave speed w_a. Cumulative counts, or the number of vehicles that have passed a point at any time *t*, are tracked at the upstream and downstream ends of link as $N_a^{\uparrow}(t)$ and $N_a^{\downarrow}(t)$, respectively. Receiving flow, $R_a(t)$, the maximum flow that could enter a link in the timestep starting at time *t*, is limited by capacity when uncongested, and by density when congestion propagates to the upstream end of the link:

$$R_a(t) = \min\left\{q_a^{\max}\Delta t, k_a^{\text{jam}} L_a + N_a^{\uparrow}\left(t - \frac{L_a}{w_a} + \Delta t\right) - N_a^{\uparrow}(t)\right\} \qquad \text{(Equation 6.1)}$$

Similarly, sending flow, $S_a(t)$, the maximum flow that could exit a link in the timestep starting at t, is limited by capacity and by the number of vehicles that reach the downstream end of the link:

$$S_a(t) = \min\left\{q_a^{max}\Delta t, N_a^{\uparrow}\left(t - \frac{L_a}{u_a^{\mathrm{f}}} + \Delta t\right) - N_a^{\downarrow}(t)\right\} \qquad \text{(Equation 6.2)}$$

Flow per timestep is generally the minimum of sending and receiving flows, with additional restrictions placed at intersections. Simple intersection models include diverges, where one incoming link is connected to several outgoing links, and merges, where several incoming links are connected to one outgoing link. Freeway models of exit ramps and on-ramps map directly to LTM diverge and merge concepts.

The simplest diverges model in LTM involves using fixed proportions to define the desired flow split at a node, and defining the final sending volumes by taking into account the downstream links' maximum receiving flows $R_a(t)$. Such a model is implemented in the software tool described below, which computes the position of the queue tail in a freeway segment using the outputs of the LTM model and Kinematic Waves theory. Further technical details are available from the authors of this report.

6.1.7.2 Excel Implementation

The freeway segment in LTM is represented as a series of links and diverge nodes. Aside from link length, free flow speed, and capacity, each segment is characterized by the backward propagation wave speed and the jam density.

The software tool developed by the researchers allows the user to define link characteristics for a freeway segment with up to three exit ramps. Further extensions will incorporate entry ramps as well. Figure 6.2 exemplifies the input spreadsheet, which aside from the freeway segment characteristics includes travel demand. The latter is specified by defining the input flows at any desired aggregation interval (15 minutes in this example) as well as the fraction of vehicles that exits in each available ramp. When the values in yellow are changed by the user, the plot on the right changes to reflect the position of the queue tail with respect to the work zone location as a function of time.

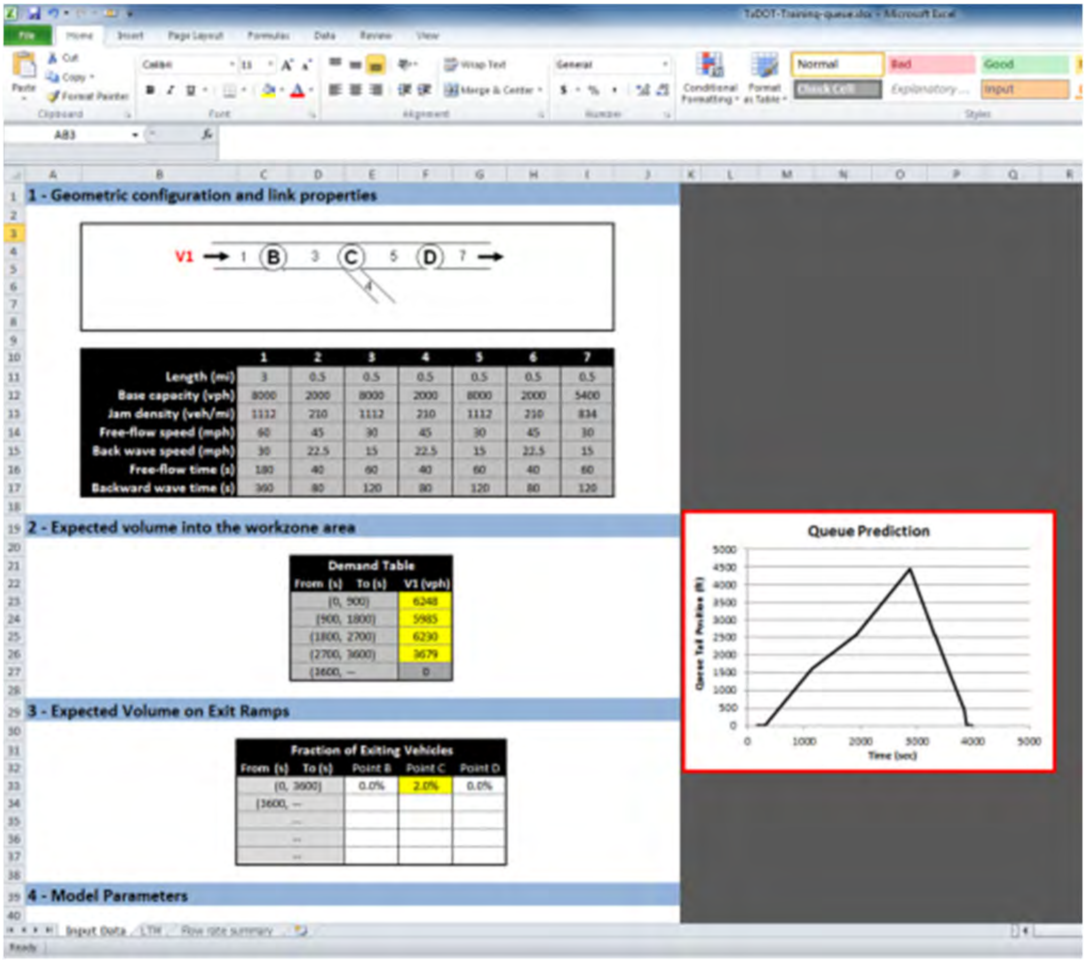


Figure 6.2 Excel-based Queue Estimation Tool.

6.1.7.3 Experimental Results

In order to assess the effectiveness of LTM as a queue tail position prediction tool, a hypothetical freeway section is modeled in both a Microsoft Excel-based LTM experimental platform and the PTV-Vision VISSIM microsimulator (Fellendorf, 1994). Microsimulation results are used as a substitute for field data, which are not readily available. Although further research will be conducted in order to contrast LTM queue length predictions to real data, the use of microsimulation in the present work allows for a variety of sensitivity tests that would not be otherwise possible.

Figure 6.3 presents our considered freeway segment, consisting of four continuous mainline roadway segments, referred to as *mainline links*, that are intersected by one-lane exit ramps. While the upstream freeway links have three lanes, Link 7 is reduced to two lanes.

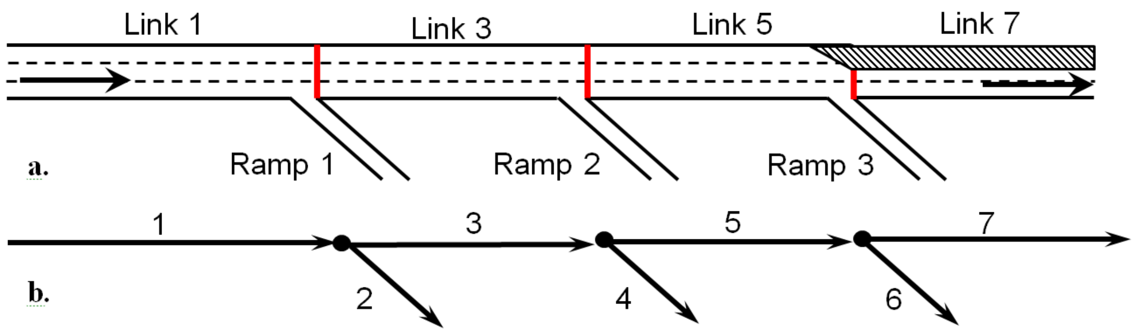


Figure 6.3 Freeway Corridor Network (a) and Corresponding Node/Link Model (b).

In our freeway model, Link 1 is 1.0 mi long, and the remaining links are 0.5 mi long. The free flow speed is assumed to be 60 mph in the mainline segments and 45 mph in exit ramps, which are not preceded by deceleration lanes. We represent a lane closure in Link 7 by regarding the entire length of Link 7 as having two lanes. Different scenarios involving various demand patterns through the segment are considered (see Table 6.2). Further details about the numerical experiments may be requested from authors of this report.

Table 6.2 Demand Scenarios.

ID	Geometry Type	Demand	Ramp 1	Ramp 2	Ramp 3	$(q_a/q_a^{\max})$>1
U1a	Simple (no merge/diverge)	5500 vph	10%	10%	Closed	M
U1			10%	25%	Closed	
U1c			10%	40%	Closed	A
U2a		6000 vph	10%	10%	Closed	MZ
U2			10%	25%	Closed	A M
U2c			10%	40%	Closed	A C
U3a		6500 vph	10%	10%	Closed	A MZ
U3			10%	25%	Closed	A M
U3c			10%	40%	Closed	A C
D1a	Compounded (merge/diverge)	5500 vph	Closed	10%	10%	M
D1			Closed	10%	25%	
D1c			Closed	10%	40%	B
D2a		6000 vph	Closed	10%	10%	MZ
D2			Closed	10%	25%	B M
D2c			Closed	10%	40%	B D
D3a		6500 vph	Closed	10%	10%	B MZ
D3			Closed	10%	25%	B M
D3c			Closed	10%	40%	B D

***A**: Link 4 diverge influence area, **B**: Link 6 diverge influence area (solely Section 3.3.1 analysis), **C**: Link 4 diverge, **D**: Link 6 diverge, **M**: the lane closure merge influence area (solely Section 3.3.2 analysis), and **Z**: the downstream mainline link.

In scenarios with the “Simple” geometry type, Ramp 3 is closed, and no interaction occurs between exiting vehicles and those merging into the two lanes that remain open on Link 7. The “Compounded” cases include a 10% exit volume on Ramp 3 that causes merge and diverge maneuvers to overlap in the same freeway region. This scenario is expected to lead to more pronounced congestion.

Figure 6.4 presents the evolution of the queue tail position over time, as predicted by both of the considered modeling approaches for the “Simple” scenarios. For the purpose of this work, the queue position is measured in feet from the tail of Link 7.

As the demand and ramp exit fractions are varied, the evolution of the queue tail position and corresponding estimation accuracy are largely dependent upon dominant bottleneck locations.

In the least congested cases (Scenarios U1and U1b), a single queue forms at the merge location. The evolution of the queue tail position rarely extends beyond Ramp 3. For scenario U1b, the LTM approach does not predict a queue formation. The corresponding microsimulation results present considerable “noise,” produced by very small queues that form and disappear erratically. Such queues, observed across most low-volume scenarios, are likely to be the result of random vehicle interactions that cannot be captured by the mesoscopic model. Their impact in terms of traffic operations is expected to be minimal, so this is not considered a serious limitation of the proposed framework.

In the remaining scenarios (U1c, U2b, U2c, U3a, U3b, and U3c), the queue formation consistently begins at Ramp 2, given that $q_a/q_a^{\max} > 1$ at such location. For scenario U3a, LTM correctly captures the early formation of a queue at Ramp 2 followed by the arrival of a shockwave originating on Ramp 3 due to the higher volume of vehicles that do not exit the freeway. In Scenarios U2b, U3b, and U1c, the LTM model overestimates the queue tail position throughout the simulation. In these cases $q_a/q_a^{\max} > 1$ due to heuristic adjustments to the value of q_{max} that may require further research. A similar observation is valid for scenarios U2c and U3c, where the queue build-up is reasonably estimated given that the flow is controlled by the downstream capacity, but the queue dissipation is not tracked properly.

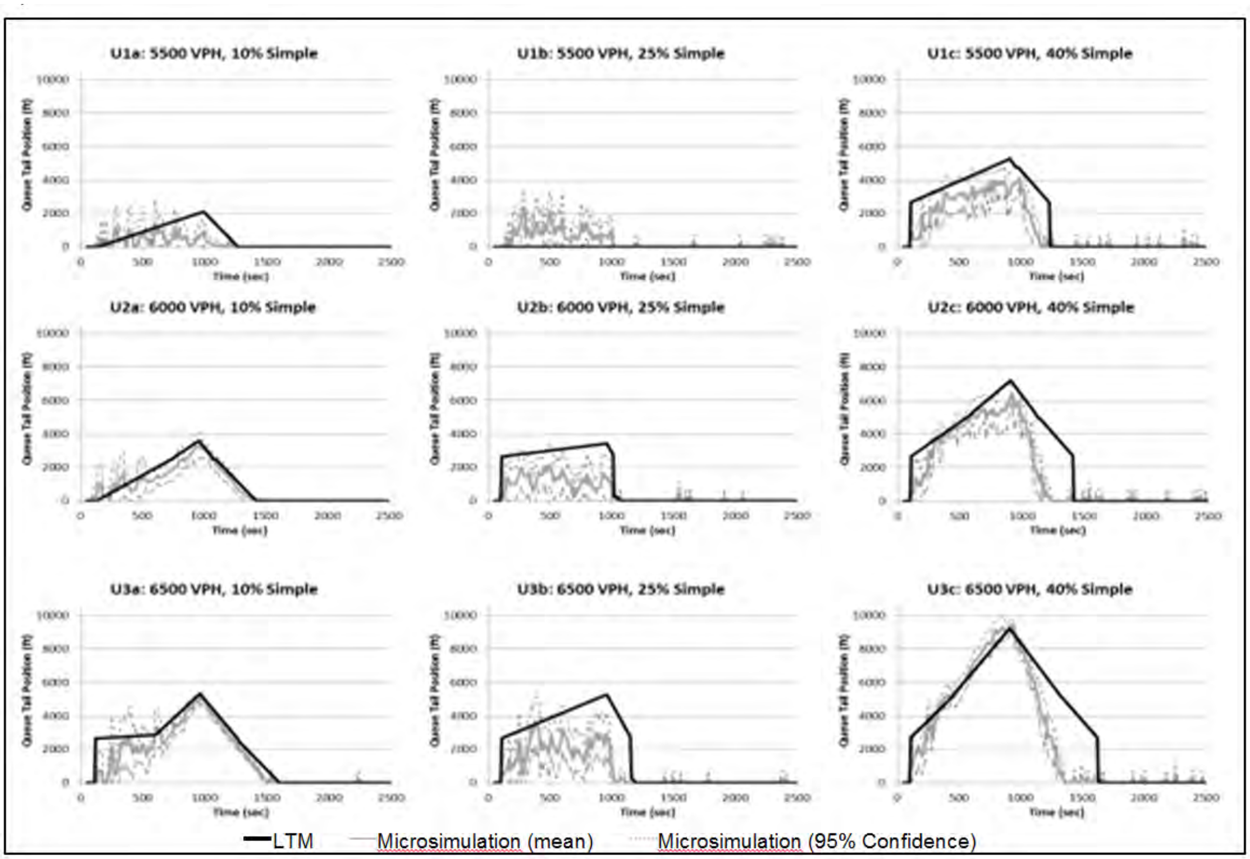


Figure 6.4 Queue Tail Position as a Function of Time for Scenarios with "Simple" Geometry.

The results presented in this section are very encouraging; the queue formation and dissipation patterns obtained from the LTM-based framework are remarkably similar to the ones observed in the microsimulation experiments. The corresponding root mean squared errors values are below 0.1 miles in most cases. Further, the proposed LTM-based methodology requires a quarter-second to run in a mainstream 3.30-GHz, 4-GB Pentium Core i3 desktop computer. This is a sizeable improvement over the time of a microsimulation cycle, which is about 25 minutes for a set of 12 runs.

6.2 Pilot Training Workshop

The concept of a decision tree or procedural framework is to provide a step-by-step procedure for making decisions. Specifically, a decision tree is applied to assist with making choices involving the number of lanes that will be closed in a work zone, acceptable times of day to have work zone lane closures based on historical demand data, and the best type of lane management or merge concept that can be applied. The purpose of this decision tree is to consider all variable roadway factors to create an optimal procedure for construction-related work zone traffic control plan decisions.

This decision tree developed through TxDOT Project 0-6704 was used by the research team to conduct a pilot training workshop for TxDOT personnel members who develop and review traffic control plans. Thus, the main purpose of this workshop was to introduce these personnel members to the developed decision tree. The pilot training workshop was held on Tuesday, August 6, at the TxDOT offices in Austin, Texas. The workshop consisted of two parts, including a presentation of approximately 1 hour and an example worksheet with calculations. The presentation introduced the decision tree and its component parts, while mentioning key issues that need further discussion. The slides are in the Appendix E. Among those key issues discussed in the presentation were the 24-hour demand concept, merge concepts, networks and diversion, and a queue length prediction tool. The worksheet example that composed the second portion of the pilot training workshop consisted of five steps that are typically followed in the decision tree or procedural framework. The first two steps addressed initial boxes within the procedural framework, including assessing the current conditions by determining traffic volume and roadway capacity, as well as assessing the work zone conditions by determining roadway capacity through the work zone and the predicted traffic volume. The next determination made in that example was whether the traffic control plans needed to be revisited, by adjusting the number of lanes proposed for closure during each hour of the day and reassessing work zone conditions until the traffic control plan was satisfactory. Once completed, the merge concept and sign placement were selected by the personnel members as the final part of the pilot training workshop. After completing this worksheet, a brief wrap-up was given.

6.3 Conclusions

Traffic modeling through simulation is a vital tool for transportation research. Simulators allow for a window into the real world that can be calibrated to match field conditions. Studies mentioned above (Yang, 2009;Wei, 2010; Lentzakis, 2008; Pesti, 2007) used simulators to analyze variable lane configurations, divergence, and differing control methods. By using these programs the programmer can maintain consistent parameters and gain accurate comparisons through different scenarios.

VISSIM was the primary simulator of choice for the above evaluations. This behavior-based traffic simulator can optimize complex technical systems while being calibrated to real-world situations.

The data collected for this task provides interesting insights into drivers' route choice mechanism and lane-changing behavior around road closures. At the Houston site, drivers were observed to react to work zone warning signs by merging early under relatively uncongested conditions. However, under higher levels of congestion, more drivers adopted a late merge strategy. The Austin site on IH 35 at 51st Street was used to analyze the evolution of driver's route choice process around long-term work zones. Travelers were observed to initially overreact to the presence of the work zone, and eventually return to their original paths for the considered scenario. A diversion rate of 25% was observed in the second day of road work. At the Oltorf Road site, we found the detour guidance to be effective. The collected data will be used to assess the performance of microsimulation and DTA models, and to inform the selection of some modeling parameters.

In this study, we investigated the safety impacts of the use of the Fixed Cycle Work Zone Traffic Signal Control (FCWZTSC) strategy at highway work zones under various traffic and geometric conditions. Instead of actual crash rates, traffic conflicts derived from the microscopic traffic simulation results were used as safety surrogates. Traffic simulation models were developed and calibrated based on the field data. Based on the results of the traffic conflict analysis, the following conclusions can be drawn regarding FCWZTSC

(1) FCWZTSC can significantly reduce lane change conflicts at work zone closures.

(2) FCWZTSC may increase rear end conflicts at work zones with only one lane closed.

(3) FCWZTSC is not recommended for low traffic volumes (1800 vehicles per hour per lane or less).

(4) FCWZTSC should only be attempted with cycle lengths longer than 40 seconds.

(5) For work zones with more than one lane closed, use of FCWZTSC will result in a significant reduction of both lane-change and rear-end conflicts.

The decision tree frames a step-by-step process for construction-related activity at a highway work zone. The merge concepts that provide for ideal management of traffic are supported by the various VISSIM outputs. In general, assumptions about merge concept trends were accurate in that lower demand is best managed by early merge, low to moderate demand by late merge, and high demand by signal merge. Early merge is beneficial in lower demand situations and allows highway users to merge into gaps prior to the distraction of the work zone. Late merge allows users to utilize all roadway capacity until the actual work zone, while fixed signal merge is best for situations where demand exceeds capacity and can significantly reduce lane-change conflicts at work zone closures. Signal merge can also reduce rear-end conflicts for work zones with more than one lane closed.

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Appendix A. Field Data

Appendix A.1 Houston Site (IH 610 at Clinton) Data from June 20 to June 22.

Date	Time	Duration (minutes)	Total Volume		Closed Lane		Distance (ft)	Average Travel Time (seconds)	Sample Size for Average Travel Time	Average Speed (mph)
			Passenger Car	Truck	Passenger Car	Truck				
6/20/2012	17:00-17:05	5:00	460	49	63	1	692.247	8.136	27	58.01
6/20/2012	17:05-17:10	5:00	490	29	50	1	692.247	8.062	30	58.55
6/20/2012	17:10-17:15	5:00	565	34	83	3	692.247	11.366	20	41.53
6/20/2012	17:15-17:20	5:00	556	44	110	3	692.247	15.030	16	31.40
6/20/2012	17:20-17:25	5:00	574	36	110	4	692.247	16.465	11	28.67
6/20/2012	17:25-17:30	5:00	523	36	91	2	692.247	17.681	15	26.70
6/20/2012	17:30-17:35	5:00	495	29	110	1	692.247	19.377	14	24.36
6/20/2012	17:35-17:40	5:00	468	31	97	2	692.247	18.056	15	26.14
6/20/2012	17:40-17:45	5:00	447	33	103	2	692.247	18.320	14	25.76
6/20/2012	17:45-17:50	5:00	446	32	102	6	692.247	18.961	14	24.89
6/20/2012	17:50-17:55	5:00	456	27	102	2	692.247	23.259	10	20.29
6/20/2012	17:55-18:00	5:00	427	25	119	2	692.247	18.416	15	25.63
6/20/2012	18:00-18:05	5:00	458	32	106	1	692.247	18.518	14	25.49
6/20/2012	18:05-18:10	5:00	404	38	102	3	692.247	19.843	14	23.79
6/20/2012	18:10-18:15	5:00	437	38	84	1	692.247	16.395	16	28.79
6/20/2012	18:15-18:20	5:00	452	33	44	5	692.247	17.203	15	27.44
6/20/2012	18:20-18:25	5:00	432	26	40	2	692.247	8.002	30	58.99
6/20/2012	18:25-18:30	5:00	410	27	33	2	692.247	7.766	30	60.78
6/21/2012	17:00-17:05	5:00	493	32	50	0	692.247	12.380	19	38.13
6/21/2012	17:05-17:10	5:00	509	37	39	0	692.247	11.347	19	41.60
6/21/2012	17:10-17:15	5:00	549	28	85	0	692.247	10.511	21	44.91
6/21/2012	17:15-17:20	5:00	476	35	106	0	692.247	13.463	16	35.06
6/21/2012	17:20-17:25	5:00	527	20	110	0	692.247	21.123	13	22.35
6/21/2012	17:25-17:30	5:00	530	31	94	2	692.247	21.538	11	21.91
6/21/2012	17:30-17:35	5:00	635	38	107	0	692.247	40.143	7	11.76

Date	Time	Duration (minutes)	Total Volume		Closed Lane		Distance (ft)	Average Travel Time (seconds)	Sample Size for Average Travel Time	Average Speed (mph)
			Passenger Car	Truck	Passenger Car	Truck				
6/21/2012	17:35-17:40	5:00	475	38	104	0	692.247	29.396	9	16.06
6/21/2012	17:40-17:45	5:00	394	23	100	0	692.247	36.197	8	13.04
6/21/2012	17:45-17:50	5:00	496	34	107	0	692.247	41.258	7	11.44
6/21/2012	17:50-17:55	5:00	547	24	112	0	692.247	29.136	7	16.20
6/21/2012	17:55-18:00	5:00	503	41	105	4	692.247	30.407	9	15.52
6/21/2012	18:00-18:05	5:00	407	23	85	2	692.247	25.887	10	18.23
6/21/2012	18:05-18:10	5:00	556	29	112	4	692.247	25.642	10	18.41
6/21/2012	18:10-18:15	5:00	518	22	103	0	692.247	21.053	11	22.42
6/21/2012	18:15-18:20	5:00	507	35	113	0	692.247	25.947	9	18.19
6/21/2012	18:20-18:25	5:00	432	35	79	1	692.247	25.636	10	18.41
6/21/2012	18:25-18:30	5:00	393	19	60	0	692.247	27.005	8	17.48
6/22/2012	17:00-17:05	5:00	446	29	72	1	692.247	7.881	30	59.89
6/22/2012	17:05-17:10	5:00	517	40	56	1	692.247	8.093	22	58.32
6/22/2012	17:10-17:15	5:00	494	27	93	3	692.247	12.476	22	37.83
6/22/2012	17:15-17:20	5:00	434	28	87	2	692.247	19.677	11	23.99
6/22/2012	17:20-17:25	5:00	435	36	117	1	692.247	22.233	11	21.23
6/22/2012	17:25-17:30	5:00	406	24	111	2	692.247	21.516	11	21.94
6/22/2012	17:30-17:35	5:00	370	15	94	3	692.247	20.014	12	23.58
6/22/2012	17:35-17:40	5:00	333	29	105	1	692.247	16.695	14	28.27
6/22/2012	17:40-17:45	5:00	408	24	95	0	692.247	19.966	12	23.64
6/22/2012	17:45-17:50	5:00	401	20	87	4	692.247	20.045	13	23.55
6/22/2012	17:50-17:55	5:00	436	31	96	3	692.247	20.778	12	22.72
6/22/2012	17:55-18:00	5:00	475	37	105	4	692.247	22.808	8	20.69
6/22/2012	18:00-18:05	5:00	369	30	84	0	692.247	19.019	12	24.82
6/22/2012	18:05-18:10	5:00	401	22	81	0	692.247	20.550	11	22.97
6/22/2012	18:10-18:15	5:00	409	28	61	1	692.247	23.124	10	20.41
6/22/2012	18:15-18:20	5:00	425	29	51	0	692.247	19.202	12	24.58
6/22/2012	18:20-18:25	5:00	392	29	49	0	692.247	13.007	17	36.29
6/22/2012	18:25-18:30	5:00	449	38	36	0	692.247	8.935	21	52.83

Appendix A.2 Austin Site 1 (IH 35 near 51st street) Data from April 23 to April 27.

Date	Time	Duration (minutes)	Inflow		Outflow to Exit		Distance (ft)	Average Travel Time (seconds)	Sample Size for Average Travel Time	Average Speed (mph)
			Passenger Car	Truck	Passenger Car	Truck				
4/23/2012	21:01:01-21:11:27	10:26	457	35	79	0	964.884	14.074	29	46.75
4/23/2012	21:13:59-21:22:23	8:24	387	34	52	0	964.884	13.665	27	48.14
4/23/2012	21:22:33-21:32:04	9:31	449	30	52	1	964.884	14.841	28	44.33
4/23/2012	21:32:13-21:40:06	7:53	337	36	39	2	964.884	12.886	29	51.06
4/23/2012	22:18:15-22:18:52	0:37	18	4	2	0	964.884	10.671	2	61.65
4/23/2012	22:19:27-22:29:52	10:25	412	33	47	0	964.884	13.101	30	50.22
4/23/2012	22:45:42-22:56:07	10:25	330	38	34	1	964.884	12.994	30	50.63
4/23/2012	23:03:34-23:13:59	10:25	283	33	27	0	964.884	13.185	30	49.90
4/23/2012	23:19:18-23:29:43	10:25	226	37	29	0	964.884	12.308	29	53.45
4/23/2012	23:41:28-23:51:35	10:07	211	41	28	0	964.884	12.060	30	54.55
4/24/2012	20:51:10-20:59:23	8:13	NA	NA	32	0	964.884	13.264	27	49.60
4/24/2012	20:59:40-21:10:06	10:26	482	55	33	0	964.884	23.334	21	28.19
4/24/2012	21:10:17-21:20:30	10:13	463	41	28	0	964.884	38.996	13	16.87
4/24/2012	21:20:42-21:31:00	10:18	503	52	39	1	964.884	36.308	14	18.12
4/24/2012	21:31:13-21:37:12	5:59	345	25	20	0	964.884	33.480	9	19.65
4/24/2012	21:51:39-22:02:05	10:26	483	57	32	1	964.884	15.131	30	43.48
4/24/2012	22:07:35-22:18:00	10:25	460	52	31	0	964.884	15.508	30	42.42
4/24/2012	22:20:21-22:30:47	10:26	422	68	29	0	964.884	20.108	25	32.72
4/24/2012	22:41:03-22:50:51	9:48	366	48	27	1	964.884	20.894	18	31.49
4/24/2012	22:51:10-23:01:07	9:57	213	38	34	0	964.884	21.645	21	30.39
4/24/2012	23:01:21-23:10:32	9:11	263	41	23	0	964.884	18.130	23	36.29
4/24/2012	23:10:47-23:21:13	10:26	266	48	25	0	964.884	15.693	28	41.92

Date	Time	Duration (minutes)	Inflow		Outflow to Exit		Distance (ft)	Average Travel Time (seconds)	Sample Size for Average Travel Time	Average Speed (mph)
			Passenger	Truck	Passenger	Truck				
4/24/2012	23:28:03-23:38:29	10:26	213	48	23	0	964.884	14.197	30	46.34
4/25/2012	20:51:27-21:00:08	8:41	536	40	35	0	964.884	10.923	30	60.23
4/25/2012	21:00:16-21:10:34	10:18	670	46	55	0	964.884	10.718	30	61.38
4/25/2012	21:10:44-21:20:21	9:37	594	45	46	1	964.884	11.095	30	59.30
4/25/2012	21:20:32-21:30:36	10:04	594	46	51	0	964.884	11.023	30	59.68
4/25/2012	21:30:46-21:39:32	8:46	523	41	39	1	964.884	10.813	30	60.84
4/25/2012	21:39:46-21:50:12	10:26	549	41	46	1	964.884	10.828	30	60.76
4/25/2012	21:52:20-22:00:46	8:26	450	44	32	0	964.884	10.828	30	60.76
4/25/2012	22:00:58-22:10:38	9:40	469	42	41	1	964.884	10.855	30	60.61
4/25/2012	22:10:50-22:20:29	9:39	447	47	47	0	964.884	10.888	30	60.42
4/25/2012	22:20:39-22:30:39	10:00	460	50	34	0	964.884	10.441	30	63.01
4/25/2012	22:30:51-22:40:22	9:31	372	55	37	1	964.884	10.680	30	61.60
4/25/2012	22:40:32-22:50:22	9:50	390	57	33	0	964.884	10.490	30	62.72
4/25/2012	22:50:32-23:00:58	10:26	367	61	34	0	964.884	10.713	30	61.41
4/26/2012	20:50:40-21:01:04	10:20	661	50	58	0	1087.65	13.685	30	54.19
4/26/2012	21:01:19-21:11:39	10:20	639	52	47	0	1087.65	13.802	30	53.73
4/26/2012	21:11:56-21:21:55	9:59	674	39	46	0	1087.65	14.147	30	52.42
4/26/2012	21:22:06-21:32:28	10:22	693	37	56	0	1087.65	13.387	30	55.40
4/26/2012	21:32:39-21:42:14	9:35	572	45	41	1	1087.65	12.064	30	61.47
4/26/2012	21:42:30-21:52:55	10:25	628	33	38	0	1087.65	12.957	30	57.24
4/26/2012	21:58:08-22:08:33	10:25	588	36	41	0	1087.65	12.489	30	59.38
4/26/2012	22:09:32-22:19:57	10:25	571	42	47	0	1087.65	12.782	30	58.02

Date	Time	Duration (minutes)	Inflow		Outflow to Exit		Distance (ft)	Average Travel Time (seconds)	Sample Size for Average Travel Time	Average Speed (mph)
			Passenger	Truck	Passenger	Truck				
4/26/2012	22:22:05-22:29:49	7:44	374	43	35	0	1087.65	13.953	28	53.15
4/26/2012	22:29:59-22:40:50	10:25	533	47	44	0	1087.65	13.807	30	53.71
4/26/2012	22:41:14-22:49:50	8:36	379	41	33	0	1087.65	12.712	30	58.34
4/26/2012	22:49:59-23:00:10	10:11	403	51	33	1	1087.65	13.240	30	56.01
4/27/2012	20:11:29-20:13:00	1:31	106	7	NA	NA	NA	NA	NA	NA
4/27/2012	20:50:30-21:00:56	10:26	NA	NA	NA	NA	NA	NA	NA	NA
4/27/2012	21:02:30-21:12:55	10:25	NA	NA	NA	NA	NA	NA	NA	NA
4/27/2012	21:15:34-21:25:51	10:17	NA	NA	NA	NA	NA	NA	NA	NA
4/27/2012	21:26:04-21:36:29	10:25	NA	NA	NA	NA	NA	NA	NA	NA
4/27/2012	21:36:41-21:47:07	10:26	841	37	58	1	964.884	11.414	30	57.64
4/27/2012	21:48:51-21:58:43	9:52	755	33	44	0	964.884	10.911	30	60.30
4/27/2012	21:58:52-22:09:18	10:26	739	36	58	0	964.884	10.910	30	60.30
4/27/2012	22:11:42-22:22:08	10:26	708	28	61	1	964.884	10.709	30	61.43
4/27/2012	22:24:24-22:34:38	10:14	682	36	39	0	964.884	11.213	30	58.67
4/27/2012	22:34:49-22:40:56	6:07	384	25	17	0	964.884	11.047	30	59.55
4/27/2012	22:41:09-22:51:35	10:26	666	46	43	1	964.884	11.281	30	58.32

Appendix A.3.1 Austin Site 2 (IH 35 near 51st street) Data from July 24 and 26, August 7 and 9 for Woodward at IH 35.

Date	Work Zone (Y or N)	Time	Duration (minutes)	Southbound Right Turn		Westbound Through	
				Passenger Car	Truck	Passenger Car	Truck
8/7/2012	N	07:00-07:05	5:00	12	0	9	0
8/7/2012	N	07:05-07:10	5:00	7	1	15	2
8/7/2012	N	07:10-07:15	5:00	11	0	7	0
8/7/2012	N	07:15-07:20	5:00	10	0	13	0
8/7/2012	N	07:20-07:25	5:00	6	0	17	1
8/7/2012	N	07:25-07:30	5:00	13	0	15	1
8/7/2012	N	07:30-07:35	5:00	8	0	14	0
8/7/2012	N	07:35-07:40	5:00	12	0	17	0
8/7/2012	N	07:40-07:45	5:00	12	0	14	0
8/7/2012	N	07:45-07:50	5:00	2	1	9	0
8/7/2012	N	07:50-07:55	5:00	16	0	25	0
8/7/2012	N	07:55-08:00	5:00	10	0	9	0
8/7/2012	N	08:00-08:05	5:00	13	0	14	0
8/7/2012	N	08:05-08:10	5:00	12	0	27	0
8/7/2012	N	08:10-08:15	5:00	10	0	25	0
8/7/2012	N	08:15-08:20	5:00	19	0	5	0
8/7/2012	N	08:20-08:25	5:00	14	0	20	0
8/7/2012	N	08:25-08:30	5:00	8	0	8	0
8/7/2012	N	08:30-08:35	5:00	15	0	13	2
8/7/2012	N	08:35-08:40	5:00	12	0	11	0
8/7/2012	N	08:40-08:45	5:00	8	0	20	0
8/7/2012	N	08:45-08:50	5:00	21	0	15	0
8/7/2012	N	08:50-08:55	5:00	10	0	9	0
8/7/2012	N	08:55-09:00	5:00	14	0	13	0
8/9/2012	N	07:00-07:05	5:00	5	1	11	0
8/9/2012	N	07:05-07:10	5:00	16	0	7	0
8/9/2012	N	07:10-07:15	5:00	4	0	11	0
8/9/2012	N	07:15-07:20	5:00	8	0	13	1
8/9/2012	N	07:20-07:25	5:00	12	0	16	3
8/9/2012	N	07:25-07:30	5:00	8	0	23	1
8/9/2012	N	07:30-07:35	5:00	10	0	17	1
8/9/2012	N	07:35-07:40	5:00	13	0	13	0
8/9/2012	N	07:40-07:45	5:00	9	0	10	0
8/9/2012	N	07:45-07:50	5:00	8	0	11	0
8/9/2012	N	07:50-07:55	5:00	12	0	25	0
8/9/2012	N	07:55-08:00	5:00	15	0	20	0
8/9/2012	N	08:00-08:05	5:00	11	1	22	1
8/9/2012	N	08:05-08:10	5:00	11	0	13	1
8/9/2012	N	08:10-08:15	5:00	16	0	15	0
8/9/2012	N	08:15-08:20	5:00	10	0	8	1

Date	Work Zone (Y or N)	Time	Duration (minutes)	Southbound Right Turn		Westbound Through	
				Passenger Car	Truck	Passen-ger Car	Truck
8/9/2012	N	08:20-08:25	5:00	17	0	14	1
8/9/2012	N	08:25-08:30	5:00	8	0	10	0
8/9/2012	N	08:30-08:35	5:00	9	0	16	1
8/9/2012	N	08:35-08:40	5:00	13	0	11	1
8/9/2012	N	08:40-08:45	5:00	13	1	11	0
8/9/2012	N	08:45-08:50	5:00	11	0	8	0
8/9/2012	N	08:50-08:55	5:00	13	0	10	0
8/9/2012	N	08:55-09:00	5:00	7	0	12	0
8/9/2012	N	16:00-16:05	5:00	11	0	9	0
8/9/2012	N	16:05-16:10	5:00	10	0	17	0
8/9/2012	N	16:10-16:15	5:00	4	0	11	0
8/9/2012	N	16:15-16:20	5:00	4	0	13	0
8/9/2012	N	16:20-16:25	5:00	6	0	14	0
8/9/2012	N	16:25-16:30	5:00	10	0	18	0
8/9/2012	N	16:30-16:35	5:00	11	0	10	0
8/9/2012	N	16:35-16:40	5:00	15	0	17	0
8/9/2012	N	16:40-16:45	5:00	11	0	18	0
8/9/2012	N	16:45-16:50	5:00	13	0	16	0
8/9/2012	N	16:50-16:55	5:00	2	0	11	0
8/9/2012	N	16:55-17:00	5:00	18	0	17	0
8/9/2012	N	17:00-17:05	5:00	9	0	21	1
8/9/2012	N	17:05-17:10	5:00	17	0	21	0
8/9/2012	N	17:10-17:15	5:00	10	0	20	0
8/9/2012	N	17:15-17:20	5:00	9	0	15	0
8/9/2012	N	17:20-17:25	5:00	6	0	16	1
8/9/2012	N	17:25-17:30	5:00	11	0	16	1
8/9/2012	N	17:30-17:35	5:00	8	0	6	0
8/9/2012	N	17:35-17:40	5:00	16	0	18	0
8/9/2012	N	17:40-17:45	5:00	9	0	18	0
8/9/2012	N	17:45-17:50	5:00	7	0	19	0
8/9/2012	N	17:50-17:55	5:00	13	2	11	0
8/9/2012	N	17:55-18:00	5:00	8	0	18	0
7/24/2012	Y	07:00-07:05	5:00	20	0	15	0
7/24/2012	Y	07:05-07:10	5:00	18	1	10	1
7/24/2012	Y	07:10-07:15	5:00	24	2	7	0
7/24/2012	Y	07:15-07:20	5:00	23	2	17	3
7/24/2012	Y	07:20-07:25	5:00	22	2	21	1
7/24/2012	Y	07:25-07:30	5:00	32	0	20	0
7/24/2012	Y	07:30-07:35	5:00	20	0	20	0
7/24/2012	Y	07:35-07:40	5:00	18	3	18	0
7/24/2012	Y	07:40-07:45	5:00	21	1	6	1

Date	Work Zone (Y or N)	Time	Duration (minutes)	Southbound Right		Westbound Through	
				Passenger Car	Truck	Passenger Car	Truck
7/24/2012	Y	07:45-07:50	5:00	21	2	18	0
7/24/2012	Y	07:50-07:55	5:00	41	3	27	1
7/24/2012	Y	07:55-08:00	5:00	38	1	14	0
7/24/2012	Y	08:00-08:05	5:00	21	3	16	3
7/24/2012	Y	08:05-08:10	5:00	26	0	21	0
7/24/2012	Y	08:10-08:15	5:00	29	4	15	1
7/24/2012	Y	08:15-08:20	5:00	25	1	12	1
7/24/2012	Y	08:20-08:25	5:00	26	4	14	0
7/24/2012	Y	08:25-08:30	5:00	28	1	8	2
7/24/2012	Y	08:30-08:35	5:00	28	2	12	1
7/24/2012	Y	08:35-08:40	5:00	27	1	11	1
7/24/2012	Y	08:40-08:45	5:00	38	0	8	0
7/24/2012	Y	08:45-08:50	5:00	28	3	9	0
7/24/2012	Y	08:50-08:55	5:00	24	2	14	1
7/24/2012	Y	08:55-09:00	5:00	30	0	13	1
7/24/2012	Y	16:00-16:05	5:00	36	1	11	0
7/24/2012	Y	16:05-16:10	5:00	31	0	17	0
7/24/2012	Y	16:10-16:15	5:00	27	1	21	0
7/24/2012	Y	16:15-16:20	5:00	21	0	13	0
7/24/2012	Y	16:20-16:25	5:00	28	1	19	1
7/24/2012	Y	16:25-16:30	5:00	15	0	13	1
7/24/2012	Y	16:30-16:35	5:00	21	1	15	0
7/24/2012	Y	16:35-16:40	5:00	24	0	16	0
7/24/2012	Y	16:40-16:45	5:00	25	0	15	1
7/24/2012	Y	16:45-16:50	5:00	28	2	15	0
7/24/2012	Y	16:50-16:55	5:00	19	1	23	1
7/24/2012	Y	16:55-17:00	5:00	18	0	17	0
7/24/2012	Y	17:00-17:05	5:00	22	1	9	1
7/24/2012	Y	17:05-17:10	5:00	28	1	21	1
7/24/2012	Y	17:10-17:15	5:00	35	0	24	0
7/24/2012	Y	17:15-17:20	5:00	18	0	13	0
7/24/2012	Y	17:20-17:25	5:00	19	1	12	0
7/24/2012	Y	17:25-17:30	5:00	28	1	18	0
7/24/2012	Y	17:30-17:35	5:00	29	1	11	0
7/24/2012	Y	17:35-17:40	5:00	25	0	13	0
7/24/2012	Y	17:40-17:45	5:00	21	0	17	0
7/24/2012	Y	17:45-17:50	5:00	24	0	17	0
7/24/2012	Y	17:50-17:55	5:00	25	1	13	0
7/24/2012	Y	17:55-18:00	5:00	27	1	17	1
7/26/2012	Y	07:00-07:05	5:00	19	0	11	0
7/26/2012	Y	07:05-07:10	5:00	12	3	14	0

Date	Work Zone (Y or N)	Time	Duration (minutes)	Southbound Right Turn		Westbound Through	
				Passenger Car	Truck	Passenger Car	Truck
7/26/2012	Y	07:10-07:15	5:00	18	1	10	1
7/26/2012	Y	07:15-07:20	5:00	20	0	16	0
7/26/2012	Y	07:20-07:25	5:00	20	1	22	1
7/26/2012	Y	07:25-07:30	5:00	16	0	14	0
7/26/2012	Y	07:30-07:35	5:00	20	1	22	0
7/26/2012	Y	07:35-07:40	5:00	24	1	10	2
7/26/2012	Y	07:40-07:45	5:00	25	5	14	0
7/26/2012	Y	07:45-07:50	5:00	21	0	19	0
7/26/2012	Y	07:50-07:55	5:00	30	2	24	0
7/26/2012	Y	07:55-08:00	5:00	26	2	25	1
7/26/2012	Y	08:00-08:05	5:00	28	1	12	0
7/26/2012	Y	08:05-08:10	5:00	24	2	23	1
7/26/2012	Y	08:10-08:15	5:00	32	1	20	1
7/26/2012	Y	08:15-08:20	5:00	21	0	18	2
7/26/2012	Y	08:20-08:25	5:00	25	1	23	1
7/26/2012	Y	08:25-08:30	5:00	30	1	11	2
7/26/2012	Y	08:30-08:35	5:00	27	2	18	1
7/26/2012	Y	08:35-08:40	5:00	42	6	15	0
7/26/2012	Y	08:40-08:45	5:00	39	1	17	0
7/26/2012	Y	08:45-08:50	5:00	29	2	12	1
7/26/2012	Y	08:50-08:55	5:00	23	2	12	0
7/26/2012	Y	08:55-09:00	5:00	35	2	6	2

Appendix A.3.2 Austin Site 2 Oltorf at IH 35 (Normal Conditions).

Date	Time	Duration (minutes)	Westbound Left Turn		Westbound Through		Southbound Left Turn		Southbound Through		Southbound Right Turn	
			Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
8/7/2012	07:00-07:05	5:00	35	1	22	1	20	0	3	0	11	1
8/7/2012	07:05-07:10	5:00	44	0	24	0	25	0	7	0	20	1
8/7/2012	07:10-07:15	5:00	41	0	17	0	16	0	10	0	17	0
8/7/2012	07:15-07:20	5:00	38	0	23	4	18	0	7	0	13	0
8/7/2012	07:20-07:25	5:00	28	1	19	2	29	0	13	1	14	1
8/7/2012	07:25-07:30	5:00	65	0	37	2	15	0	5	0	11	2
8/7/2012	07:30-07:35	5:00	51	1	23	1	17	0	9	0	14	0
8/7/2012	07:35-07:40	5:00	37	1	24	1	31	0	17	2	19	0
8/7/2012	07:40-07:45	5:00	52	2	37	0	14	1	12	0	14	0
8/7/2012	07:45-07:50	5:00	56	1	31	0	18	0	5	1	15	2
8/7/2012	07:50-07:55	5:00	44	0	37	2	39	0	19	0	14	0
8/7/2012	07:55-08:00	5:00	55	0	35	1	23	0	12	0	29	1
8/7/2012	08:00-08:05	5:00	62	0	44	0	22	0	7	0	18	0
8/7/2012	08:05-08:10	5:00	35	1	26	1	18	0	15	0	10	0
8/7/2012	08:10-08:15	5:00	47	0	33	3	26	0	7	0	25	0
8/7/2012	08:15-08:20	5:00	42	0	34	2	30	0	11	0	13	0
8/7/2012	08:20-08:25	5:00	35	1	21	1	21	0	11	0	16	2
8/7/2012	08:25-08:30	5:00	45	0	23	0	15	0	11	0	22	1
8/7/2012	08:30-08:35	5:00	46	0	41	1	19	0	6	1	18	3
8/7/2012	08:35-08:40	5:00	41	0	28	0	16	0	10	2	25	1
8/7/2012	08:40-08:45	5:00	44	0	28	0	29	0	12	0	23	1
8/7/2012	08:45-08:50	5:00	61	0	28	1	20	0	13	0	19	0

Date	Time	Duration (minutes)	Westbound Left Turn		Westbound Through		Southbound Left Turn		Southbound Through		Southbound Right Turn	
			Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
8/7/2012	08:50-08:55	5:00	48	0	43	2	17	0	10	0	21	1
8/7/2012	08:55-09:00	5:00	53	0	30	0	23	0	12	2	25	0
8/7/2012	16:00-16:05	5:00	37	0	28	1	24	0	11	0	15	0
8/7/2012	16:05-16:10	5:00	44	2	28	1	21	1	22	0	15	0
8/7/2012	16:10-16:15	5:00	38	0	45	0	22	0	32	0	24	0
8/7/2012	16:15-16:20	5:00	43	0	27	2	20	0	20	0	22	1
8/7/2012	16:20-16:25	5:00	43	0	30	0	23	0	27	0	16	0
8/7/2012	16:25-16:30	5:00	30	0	37	0	17	0	22	1	11	0
8/7/2012	16:30-16:35	5:00	37	1	22	0	37	0	27	0	14	0
8/7/2012	16:35-16:40	5:00	43	0	28	1	25	0	27	0	12	0
8/7/2012	16:40-16:45	5:00	36	0	35	1	17	0	38	0	11	0
8/7/2012	16:45-16:50	5:00	48	0	30	0	20	0	30	0	15	0
8/7/2012	16:50-16:55	5:00	40	0	22	1	20	0	34	0	13	0
8/7/2012	16:55-17:00	5:00	49	1	29	1	24	0	19	0	22	0
8/7/2012	17:00-17:05	5:00	52	0	32	1	23	0	19	0	13	0
8/7/2012	17:05-17:10	5:00	47	0	35	0	39	1	35	1	12	0
8/7/2012	17:10-17:15	5:00	39	0	23	0	6	0	13	0	16	1
8/7/2012	17:15-17:20	5:00	46	0	31	1	30	0	29	1	10	0
8/7/2012	17:20-17:25	5:00	53	3	34	0	16	1	21	0	9	0
8/7/2012	17:25-17:30	5:00	42	0	25	0	26	0	38	0	13	1
8/7/2012	17:30-17:35	5:00	37	1	25	0	24	0	25	1	15	0
8/7/2012	17:35-17:40	5:00	39	0	31	3	27	0	20	1	13	1
8/7/2012	17:40-17:45	5:00	44	0	44	0	15	0	38	0	12	0

Date	Time	Duration (minutes)	Westbound Left Turn		Westbound Through		Southbound Left Turn		Southbound Through		Southbound Right Turn	
			Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
8/7/2012	17:45-17:50	5:00	43	1	38	0	17	0	21	0	19	0
8/7/2012	17:50-17:55	5:00	44	0	36	0	28	0	28	0	17	0
8/7/2012	17:55-18:00	5:00	35	0	35	0	18	0	26	0	15	0
8/9/2012	07:00-07:05	5:00	41	0	17	0	19	0	6	0	15	1
8/9/2012	07:05-07:10	5:00	41	1	22	2	29	1	8	0	9	0
8/9/2012	07:10-07:15	5:00	50	0	23	0	24	0	4	0	15	1
8/9/2012	07:15-07:20	5:00	46	0	15	1	15	1	3	0	20	2
8/9/2012	07:20-07:25	5:00	31	1	24	1	22	0	13	0	9	1
8/9/2012	07:25-07:30	5:00	52	1	40	0	21	0	8	0	11	0
8/9/2012	07:30-07:35	5:00	39	1	29	0	24	0	8	0	17	1
8/9/2012	07:35-07:40	5:00	41	2	30	1	26	0	16	0	22	1
8/9/2012	07:40-07:45	5:00	54	0	36	2	22	0	8	0	24	0
8/9/2012	07:45-07:50	5:00	61	1	45	0	19	0	12	0	14	0
8/9/2012	07:50-07:55	5:00	51	2	34	2	31	0	8	1	21	2
8/9/2012	07:55-08:00	5:00	52	2	40	0	25	1	11	1	22	2
8/9/2012	08:00-08:05	5:00	47	1	44	0	21	1	11	0	18	0
8/9/2012	08:05-08:10	5:00	35	1	44	3	25	1	5	0	17	1
8/9/2012	08:10-08:15	5:00	49	0	29	0	33	0	15	1	32	0
8/9/2012	08:15-08:20	5:00	57	2	30	1	17	0	12	0	10	1
8/9/2012	08:20-08:25	5:00	34	0	27	0	15	0	9	0	22	1
8/9/2012	08:25-08:30	5:00	46	1	31	3	37	0	15	0	18	2
8/9/2012	08:30-08:35	5:00	54	0	35	2	23	1	10	1	15	1
8/9/2012	08:35-08:40	5:00	31	3	48	0	20	0	15	0	15	0

Date	Time	Duration (minutes)	Westbound Left Turn		Westbound Through		Southbound Left Turn		Southbound Through		Southbound Right Turn	
			Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
8/9/2012	08:40-08:45	5:00	38	1	28	0	29	0	9	0	17	1
8/9/2012	08:45-08:50	5:00	53	3	26	2	15	1	7	0	30	1
8/9/2012	08:50-08:55	5:00	57	0	41	0	15	0	5	0	20	2
8/9/2012	08:55-09:00	5:00	43	2	25	1	22	0	9	0	17	1
8/9/2012	04:00-04:05	5:00	53	1	27	2	19	1	28	0	14	0
8/9/2012	04:05-04:10	5:00	47	1	24	1	25	1	25	0	7	0
8/9/2012	04:10-04:15	5:00	43	0	29	0	21	0	27	0	13	0
8/9/2012	04:15-04:20	5:00	37	0	34	1	19	0	25	0	20	0
8/9/2012	04:20-04:25	5:00	47	2	21	0	27	0	18	1	14	0
8/9/2012	04:25-04:30	5:00	39	0	22	0	22	0	28	0	16	0
8/9/2012	04:30-04:35	5:00	36	1	26	1	18	0	24	1	13	0
8/9/2012	04:35-04:40	5:00	48	1	35	1	23	0	27	0	8	0
8/9/2012	04:40-04:45	5:00	51	0	33	0	27	0	25	1	8	0
8/9/2012	04:45-04:50	5:00	45	1	38	0	15	0	32	2	19	0
8/9/2012	04:50-04:55	5:00	47	1	30	2	13	0	30	0	20	0
8/9/2012	04:55-05:00	5:00	58	1	34	0	11	0	29	0	6	0
8/9/2012	17:00-17:05	5:00	34	0	30	1	25	0	30	1	15	1
8/9/2012	17:05-17:10	5:00	46	0	40	0	24	0	25	0	14	0
8/9/2012	17:10-17:15	5:00	52	1	27	0	22	0	34	1	8	0
8/9/2012	17:15-17:20	5:00	50	0	26	1	25	0	35	0	14	1
8/9/2012	17:20-17:25	5:00	55	1	32	0	21	0	31	0	22	0
8/9/2012	17:25-17:30	5:00	49	0	34	0	23	0	29	1	14	0
8/9/2012	17:30-17:35	5:00	66	0	29	1	23	0	32	0	12	0

Date	Time	Duration (minutes)	Westbound Left Turn		Westbound Through		Southbound Left Turn		Southbound Through		Southbound Right Turn	
			Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
8/9/2012	17:35-17:40	5:00	52	0	34	0	22	0	30	1	14	0
8/9/2012	17:40-17:45	5:00	53	0	25	0	16	1	32	2	12	0
8/9/2012	17:45-17:50	5:00	42	1	31	2	27	0	30	1	11	0
8/9/2012	17:50-17:55	5:00	48	2	43	0	23	1	29	0	15	0
8/9/2012	17:55-18:00	5:00	44	0	30	0	19	0	28	2	15	0

Appendix A.3.3 Austin Site 2 Oltorf at IH 35 (Work Zone Conditions).

Date	Time	Duration (minutes)	Westbound Left Turn (Left-Most Lane)		Westbound Left Turn (Right-Most Lane)		Southbound Through		Southbound Left Turn	
			Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
7/24/2012	07:00-07:05	5:00	51	0	7	0	26	0	28	4
7/24/2012	07:05-07:10	5:00	45	2	1	0	30	0	10	1
7/24/2012	07:10-07:15	5:00	45	0	2	0	19	0	17	1
7/24/2012	07:15-07:20	5:00	47	1	2	0	39	0	27	0
7/24/2012	07:20-07:25	5:00	74	1	11	1	26	0	17	0
7/24/2012	07:25-07:30	5:00	51	0	7	0	21	0	21	1
7/24/2012	07:30-07:35	5:00	26	1	1	0	22	1	23	1
7/24/2012	07:35-07:40	5:00	51	1	5	1	27	0	32	1
7/24/2012	07:40-07:45	5:00	72	1	4	0	20	0	18	2
7/24/2012	07:45-07:50	5:00	64	1	4	0	19	1	21	1
7/24/2012	07:50-07:55	5:00	68	3	13	1	37	0	37	1
7/24/2012	07:55-08:00	5:00	67	4	23	2	23	0	20	0
7/24/2012	08:00-08:05	5:00	71	0	17	1	21	0	21	0
7/24/2012	08:05-08:10	5:00	46	3	3	0	26	0	34	0
7/24/2012	08:10-08:15	5:00	61	1	8	1	19	0	18	1
7/24/2012	08:15-08:20	5:00	47	2	4	2	19	0	19	2
7/24/2012	08:20-08:25	5:00	48	0	17	1	29	0	28	3
7/24/2012	08:25-08:30	5:00	51	3	12	1	20	1	32	0
7/24/2012	08:30-08:35	5:00	65	2	9	1	17	0	22	3
7/24/2012	08:35-08:40	5:00	49	0	9	0	11	1	19	0
7/24/2012	08:40-08:45	5:00	56	0	2	0	19	0	18	0
7/24/2012	08:45-08:50	5:00	43	1	5	0	19	0	24	2
7/24/2012	08:50-08:55	5:00	39	1	12	0	16	0	25	1
7/24/2012	08:55-09:00	5:00	59	0	22	2	37	1	24	0
7/24/2012	16:00-16:05	5:00	34	1	5	0	26	0	29	1

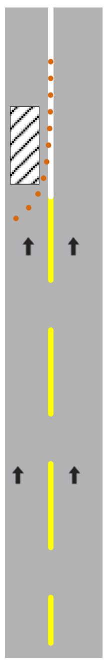
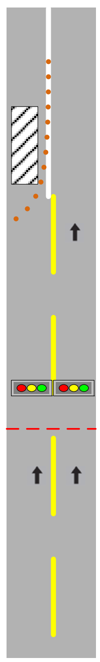
Date	Time	Duration (minutes)	Westbound Left Turn (Left-Most Lane)		Westbound Left Turn (Right-Most Lane)		Southbound Through		Southbound Left Turn	
			Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
7/24/2012	16:05-16:10	5:00	58	0	8	0	17	0	31	0
7/24/2012	16:10-16:15	5:00	56	0	7	0	26	0	34	0
7/24/2012	16:15-16:20	5:00	53	1	3	0	22	0	40	1
7/24/2012	16:20-16:25	5:00	50	2	5	1	19	0	42	0
7/24/2012	16:25-16:30	5:00	50	2	4	0	23	0	31	2
7/24/2012	16:30-16:35	5:00	50	0	7	2	18	0	42	0
7/24/2012	16:35-16:40	5:00	49	1	2	0	23	0	36	1
7/24/2012	16:40-16:45	5:00	52	1	18	0	20	0	40	0
7/24/2012	16:45-16:50	5:00	55	1	9	1	25	0	39	0
7/24/2012	16:50-16:55	5:00	58	0	19	0	12	0	41	0
7/24/2012	16:55-17:00	5:00	32	2	7	0	22	1	43	0
7/24/2012	17:00-17:05	5:00	51	2	7	2	23	0	33	0
7/24/2012	17:05-17:10	5:00	56	1	6	0	28	1	41	0
7/24/2012	17:10-17:15	5:00	62	0	5	0	28	0	41	0
7/24/2012	17:15-17:20	5:00	46	1	1	0	26	0	45	0
7/24/2012	17:20-17:25	5:00	50	0	7	2	19	2	29	0
7/24/2012	17:25-17:30	5:00	61	0	7	0	23	1	45	0
7/24/2012	17:30-17:35	5:00	52	0	6	1	22	0	42	0
7/24/2012	17:35-17:40	5:00	58	1	6	1	23	0	37	0
7/24/2012	17:40-17:45	5:00	69	1	6	0	26	0	38	0
7/24/2012	17:45-17:50	5:00	52	1	10	0	26	0	38	0
7/24/2012	17:50-17:55	5:00	24	2	0	0	10	0	21	0
7/24/2012	17:55-18:00	5:00	46	1	13	1	14	0	21	0
7/26/2012	07:00-07:05	5:00	42	1	3	0	18	0	28	0
7/26/2012	07:05-07:10	5:00	45	3	2	0	18	3	23	0
7/26/2012	07:10-07:15	5:00	41	1	7	0	14	2	21	1

Date	Time	Duration (minutes)	Westbound Left Turn (Left-Most Lane)		Westbound Left Turn (Right-Most Lane)		Southbound Through		Southbound Left Turn	
			Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
7/26/2012	07:15-07:20	5:00	52	1	13	0	22	0	18	0
7/26/2012	07:20-07:25	5:00	54	0	5	1	14	0	28	3
7/26/2012	07:25-07:30	5:00	55	1	0	0	17	2	20	0
7/26/2012	07:30-07:35	5:00	38	1	3	0	16	0	22	0
7/26/2012	07:35-07:40	5:00	51	0	6	1	28	1	34	1
7/26/2012	07:40-07:45	5:00	68	1	11	0	13	2	18	0
7/26/2012	07:45-07:50	5:00	65	0	4	1	14	2	10	0
7/26/2012	07:50-07:55	5:00	51	2	9	0	31	1	32	0
7/26/2012	07:55-08:00	5:00	78	1	17	0	24	0	23	0
7/26/2012	08:00-08:05	5:00	62	2	9	0	19	2	17	0
7/26/2012	08:05-08:10	5:00	59	1	9	1	23	2	23	0
7/26/2012	08:10-08:15	5:00	61	1	10	0	27	0	21	2
7/26/2012	08:15-08:20	5:00	59	3	7	1	24	0	23	0
7/26/2012	08:20-08:25	5:00	46	0	8	0	20	2	22	0
7/26/2012	08:25-08:30	5:00	45	0	9	0	33	0	0	1
7/26/2012	08:30-08:35	5:00	51	0	12	1	24	0	26	1
7/26/2012	08:35-08:40	5:00	47	0	9	0	28	1	16	0
7/26/2012	08:40-08:45	5:00	51	0	1	0	32	3	19	1
7/26/2012	08:45-08:50	5:00	58	1	2	0	16	1	23	0
7/26/2012	08:50-08:55	5:00	54	1	9	0	25	1	16	0
7/26/2012	08:55-09:00	5:00	50	2	8	0	32	0	18	2
7/26/2012	16:00-16:05	5:00	45	1	8	0	36	0	22	0
7/26/2012	16:05-16:10	5:00	52	0	7	0	32	0	21	0
7/26/2012	16:10-16:15	5:00	59	0	8	0	28	2	19	0
7/26/2012	16:15-16:20	5:00	45	0	7	0	31	0	20	1
7/26/2012	16:20-16:25	5:00	55	0	12	1	43	0	20	1

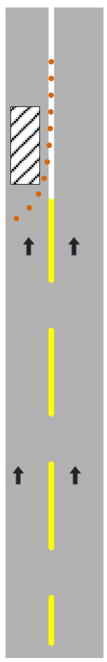
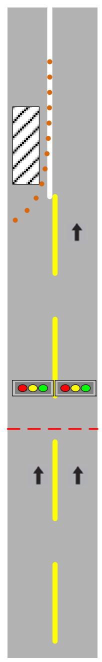
Date	Time	Duration (minutes)	Westbound Left Turn (Left-Most Lane)		Westbound Left Turn (Right-Most Lane)		Southbound Through		Southbound Left Turn	
			Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
7/26/2012	16:25-16:30	5:00	51	1	8	0	27	0	17	0
7/26/2012	16:30-16:35	5:00	47	3	14	1	33	0	20	0
7/26/2012	16:35-16:40	5:00	42	0	7	0	33	0	20	0
7/26/2012	16:40-16:45	5:00	48	0	3	0	42	0	20	0
7/26/2012	16:45-16:50	5:00	48	0	7	1	38	2	23	0
7/26/2012	16:50-16:55	5:00	51	1	5	0	42	0	22	0
7/26/2012	16:55-17:00	5:00	46	2	8	0	35	0	19	0
7/26/2012	17:00-17:05	5:00	51	1	10	1	43	0	30	0
7/26/2012	17:05-17:10	5:00	62	0	10	0	39	1	22	0
7/26/2012	17:10-17:15	5:00	46	0	5	0	42	1	25	0
7/26/2012	17:15-17:20	5:00	50	0	7	0	47	0	20	0
7/26/2012	17:20-17:25	5:00	55	0	6	1	43	2	21	1
7/26/2012	17:25-17:30	5:00	52	1	11	0	43	0	20	0
7/26/2012	17:30-17:35	5:00	51	1	5	1	37	0	29	0
7/26/2012	17:35-17:40	5:00	58	1	10	0	39	2	24	1
7/26/2012	17:40-17:45	5:00	47	0	3	0	39	1	30	1
7/26/2012	17:45-17:50	5:00	46	2	6	0	41	2	27	0
7/26/2012	17:50-17:55	5:00	56	1	17	1	40	0	14	1
7/26/2012	17:55-18:00	5:00	60	1	14	3	36	0	18	0

Appendix B. Conflict Identification for Safety Analyses

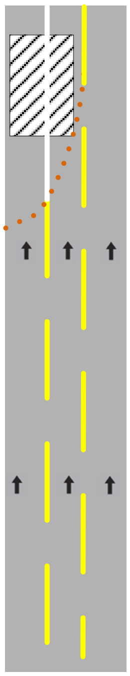
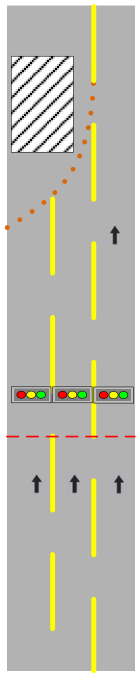
Appendix B.1 Lane-Change Conflict Look-Up Table for Highway Work Zone Closures (Two-Lane Highway with One Lane Closed)

<table>
<tr><th>Before Implementation of Signal(s)</th><th>After Implementation of Signal(s)</th><th>Cycle Length(s)</th><th>Traffic Volume</th><th>Before (Conflicts/h)</th><th>After (Conflicts/h)</th><th>TCMF</th></tr>
<tr><td rowspan="16"></td><td rowspan="16"></td><td rowspan="4">Before: N/A
After: 30</td><td>1800</td><td>0.10</td><td>0.20</td><td>2.00</td></tr>
<tr><td>2000</td><td>0.27</td><td>0.07</td><td>0.26</td></tr>
<tr><td>2200</td><td>0.20</td><td>0.17</td><td>0.85</td></tr>
<tr><td>2400</td><td>0.33</td><td>0.10</td><td>0.30</td></tr>
<tr><td rowspan="4">Before: N/A
After: 60</td><td>1800</td><td>0.10</td><td>0.00</td><td>0.00</td></tr>
<tr><td>2000</td><td>0.27</td><td>0.03</td><td>0.11</td></tr>
<tr><td>2200</td><td>0.20</td><td>0.03</td><td>0.15</td></tr>
<tr><td>2400</td><td>0.33</td><td>0.10</td><td>0.30</td></tr>
<tr><td rowspan="4">Before: N/A
After: 90</td><td>1800</td><td>0.10</td><td>0.00</td><td>0.00</td></tr>
<tr><td>2000</td><td>0.27</td><td>0.00</td><td>0.00</td></tr>
<tr><td>2200</td><td>0.20</td><td>0.00</td><td>0.00</td></tr>
<tr><td>2400</td><td>0.33</td><td>0.00</td><td>0.00</td></tr>
<tr><td rowspan="4">Before: N/A
After: 120</td><td>1800</td><td>0.10</td><td>0.00</td><td>0.00</td></tr>
<tr><td>2000</td><td>0.27</td><td>0.00</td><td>0.00</td></tr>
<tr><td>2200</td><td>0.20</td><td>0.00</td><td>0.00</td></tr>
<tr><td>2400</td><td>0.33</td><td>0.00</td><td>0.00</td></tr>
</table>

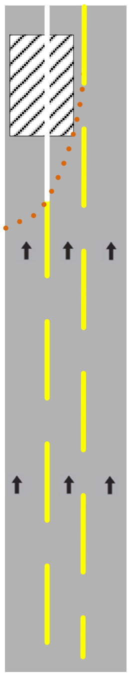
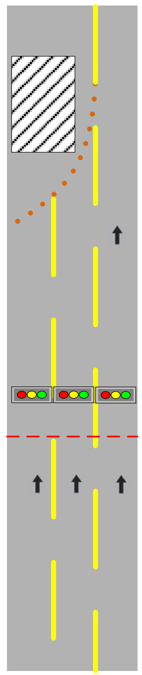
Appendix B.2 Rear-End Conflict Look-Up Table for Highway Work Zone Closures (Two-Lane Highway with One Lane Closed)

Before Implementation of Signal(s)	After Implementation of Signal(s)	Cycle Length(s)	Traffic Volume	Before (Conflicts/h)	After (Conflicts/h)	TCMF
		Before: N/A After: 30	1800	0.07	2.23	31.86
			2000	0.10	4.20	42.00
			2200	0.43	3.60	8.37
			2400	0.13	3.67	28.23
		Before: N/A After: 60	1800	0.07	0.60	8.57
			2000	0.10	0.60	6.00
			2200	0.43	1.07	2.49
			2400	0.13	1.17	9.00
		Before: N/A After: 90	1800	0.07	0.27	3.86
			2000	0.10	0.47	4.70
			2200	0.43	0.90	2.09
			2400	0.13	0.63	4.85
		Before: N/A After: 120	1800	0.07	0.30	4.29
			2000	0.10	0.40	4.00
			2200	0.43	0.53	1.23
			2400	0.13	0.57	4.38

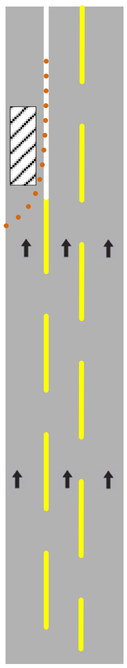
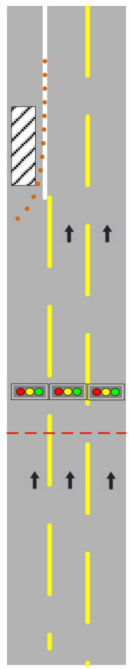
Appendix B.3 Lane-Change Conflict Look-Up Table for Highway Work Zone Closures (Three-Lane Highway with Two Lanes Closed)

Before Implementation of Signal(s)	After Implementation of Signal(s)	Cycle Length(s)	Traffic Volume	Before (Conflicts/h)	After (Conflicts/h)	TCMF
		Before: N/A After: 30	1800	22.50	0.03	0.00
			2000	31.53	0.00	0.00
			2200	33.27	0.00	0.00
			2400	24.93	0.10	0.00
		Before: N/A After: 60	1800	22.50	0.03	0.00
			2000	31.53	0.03	0.00
			2200	33.27	0.03	0.00
			2400	24.93	0.10	0.00
		Before: N/A After: 90	1800	22.50	0.00	0.00
			2000	31.53	0.00	0.00
			2200	33.27	0.07	0.00
			2400	24.93	0.03	0.00
		Before: N/A After: 120	1800	22.50	0.00	0.00
			2000	31.53	0.00	0.00
			2200	33.27	0.00	0.00
			2400	24.93	0.03	0.00

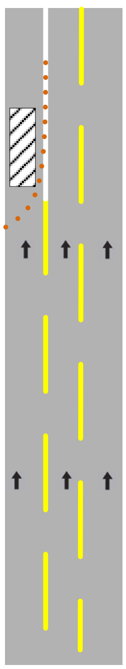
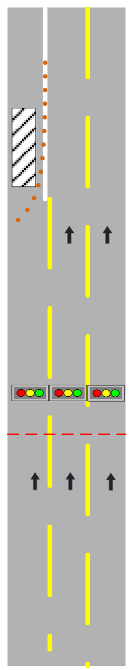
Appendix B.4 Rear-End Conflict Look-Up Table for Highway Work Zone Closures (Three-Lane Highway with Two Lanes Closed)

Before Implementation of Signal(s)	After Implementation of Signal(s)	Cycle Length(s)	Traffic Volume	Before (Conflicts/h)	After (Conflicts/h)	TCMF
		Before: N/A After: 30	1800	32.10	3.03	0.09
			2000	28.40	3.03	0.11
			2200	27.17	3.03	0.11
			2400	19.03	3.80	0.20
		Before: N/A After: 60	1800	32.10	0.70	0.02
			2000	28.40	2.17	0.08
			2200	27.17	3.20	0.12
			2400	19.03	3.07	0.16
		Before: N/A After: 90	1800	32.10	0.13	0.00
			2000	28.40	0.53	0.02
			2200	27.17	0.97	0.04
			2400	19.03	1.37	0.07
		Before: N/A After: 120	1800	32.10	0.20	0.01
			2000	28.40	0.13	0.00
			2200	27.17	1.17	0.04
			2400	19.03	0.93	0.05

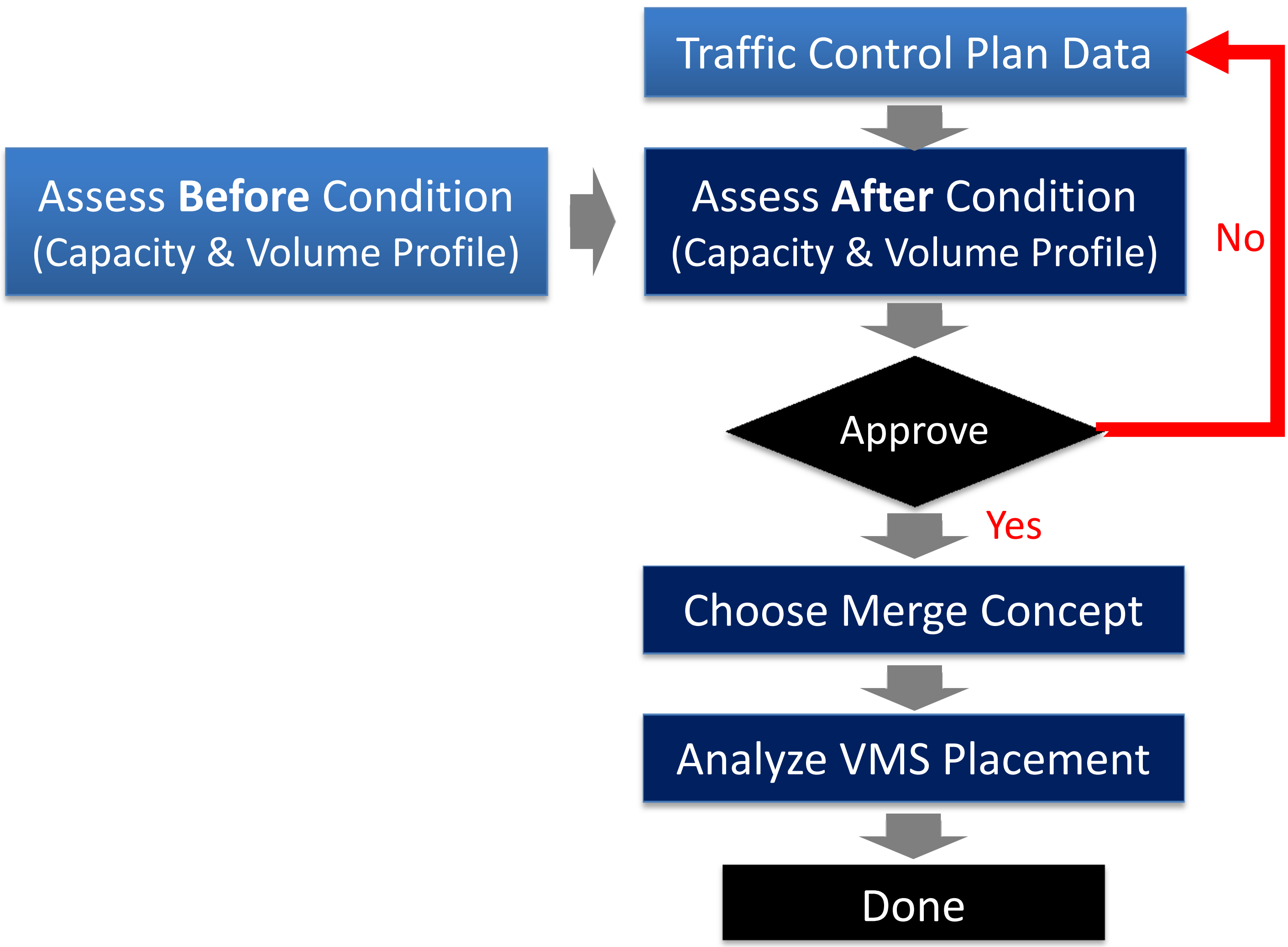
Appendix B.5 Lane-Change Conflict Look-Up Table for Highway Work Zone Closures (Three-Lane Highway with One Lane Closed)

Before Implementation of Signal(s)	After Implementation of Signal(s)	Cycle Length(s)	Traffic Volume	Before (Conflicts/h)	After (Conflicts/h)	TCMF
		Before: N/A After: 30	1800	0.37	7.30	19.73
			2000	0.80	4.60	5.75
			2200	8.00	3.43	0.43
			2400	10.07	3.07	0.30
		Before: N/A After: 60	1800	0.37	10.03	27.11
			2000	0.80	9.03	11.29
			2200	8.00	4.13	0.52
			2400	10.07	4.37	0.43
		Before: N/A After: 90	1800	0.37	8.97	24.24
			2000	0.80	7.30	9.13
			2200	8.00	4.57	0.57
			2400	10.07	4.40	0.44
		Before: N/A After: 120	1800	0.37	6.83	18.46
			2000	0.80	6.43	8.04
			2200	8.00	6.37	0.80
			2400	10.07	6.83	0.68

Appendix B.6 Rear-End Conflict Look-Up Table for Highway work Zone Closures (Three-Lane Highway with One Lane Closed)

Before Implementation of Signal(s)	After Implementation of Signal(s)	Cycle Length(s)	Traffic Volume	Before (Conflicts/h)	After (Conflicts/h)	TCMF
		Before: N/A After: 30	1800	0.27	3.30	12.22
			2000	0.10	6.30	63.00
			2200	6.30	6.70	1.06
			2400	8.27	6.23	0.75
		Before: N/A After: 60	1800	0.27	2.47	9.15
			2000	0.10	2.80	28.00
			2200	6.30	2.83	0.45
			2400	8.27	3.07	0.37
		Before: N/A After: 90	1800	0.27	2.60	9.63
			2000	0.10	2.80	28.00
			2200	6.30	1.97	0.31
			2400	8.27	2.10	0.25
		Before: N/A After: 120	1800	0.27	2.03	7.52
			2000	0.10	1.70	17.00
			2200	6.30	1.53	0.24
			2400	8.27	1.80	0.22

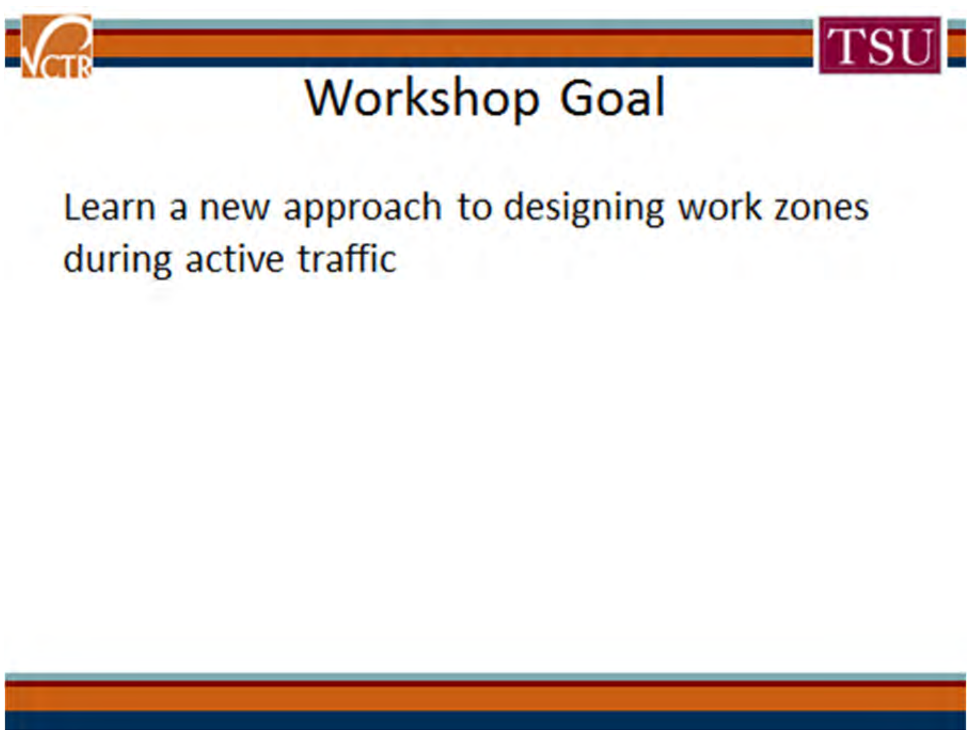
Appendix C. Decision Tree or Procedure for Construction-Related Activities at Highway Work Zones



Appendix D. Examples of Actual Work Zone Traffic Diversion Rates in the United States (Song et al., 2008)

Location	Facility	Work zone	Diversion ratio	Information	Diverted route	Source
Nebraska	I-80	Two lanes closed; Two-lane, two-way operation on the other side	8-11% (peak period)	CMS	One alternative route	McCoy and Pesti (2001)
Racine, Wisconsin	I-94	12miles One lane closure on two lanes each direction	10% (peak period)	CMS with travel time estimation	Yes, known to all regular drivers; runs in parallel	Horowitz et al. (2003)
Rocky Mount, North Carolina	I-95	1.25-2.5 miles	10.9-20.2% (peak period)	Smart Work Zone system	One alternative route	Bushman, et al. (2004)
Santa Clarita, California	I-5	1.3 miles, one lane closure on three lanes each direction	3-20% (average)	Automated work zone information system (AWIS)	One alternative route	Chu et al. (2005)
San Bernardino, California	I-15	4.5 km, closed half of eight lanes; two by three lane configuration on the left half	17-18% (peak hour)	AWIS coupled with multi-faceted proactive public outreach	I-10 and I-215	Lee and Kim (2006)

Appendix E. Pilot Training Workshop Slides



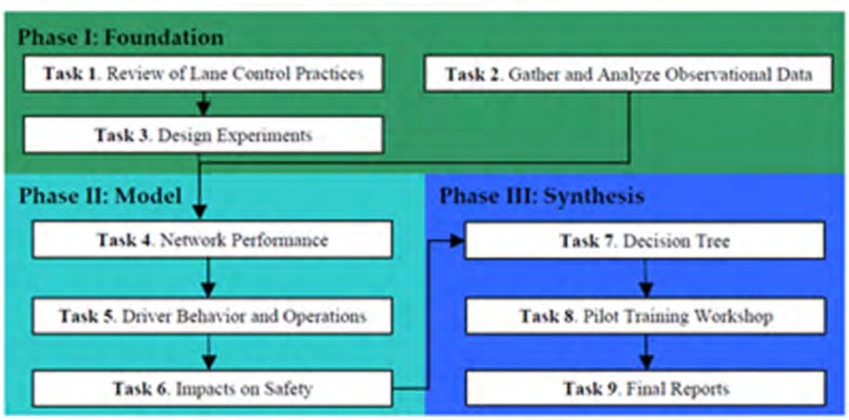
Agenda

- 0-6704 Tasks
- Decision Tree
- Merging at a Work Zone
- Measuring Traffic Diversion
- Queue Length Prediction tool

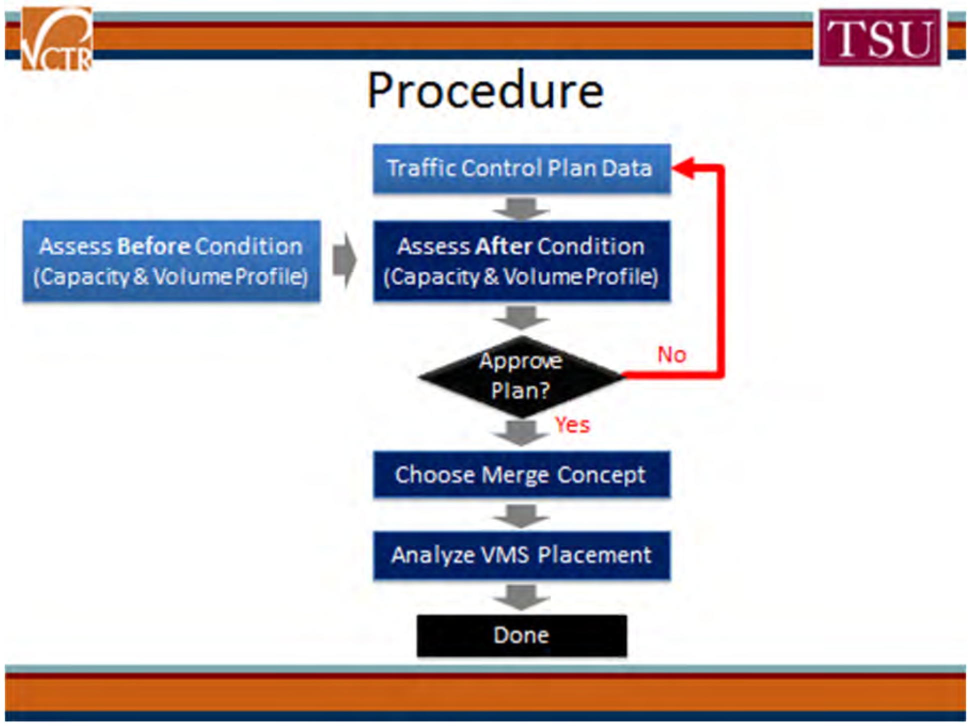
BREAK

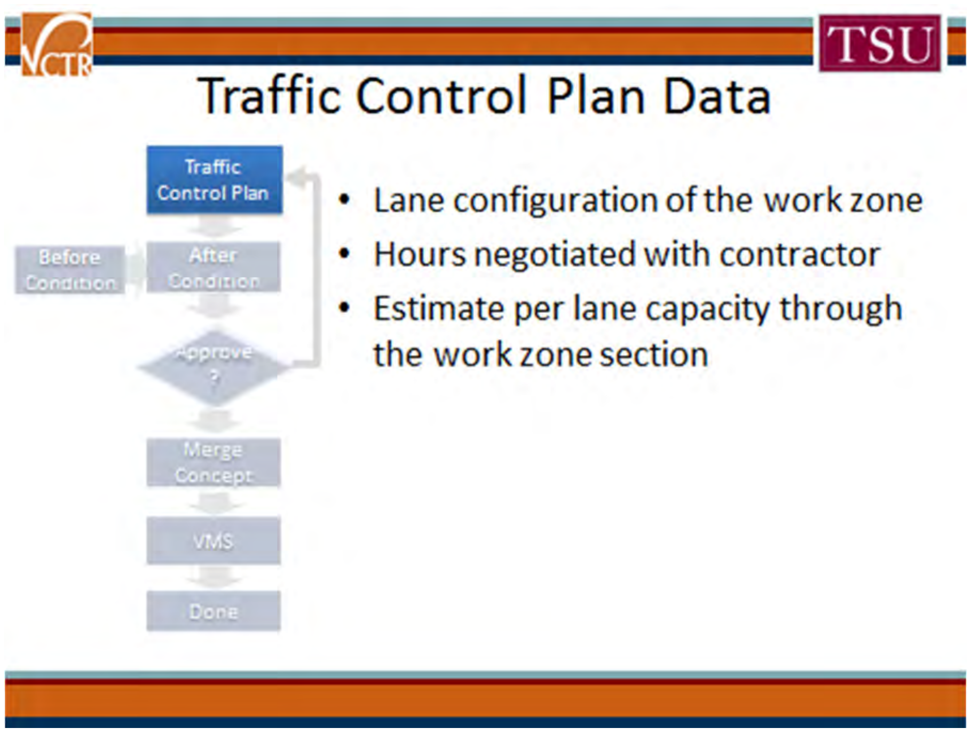
- Interactive Handout
- Wrap-up

Task Outline

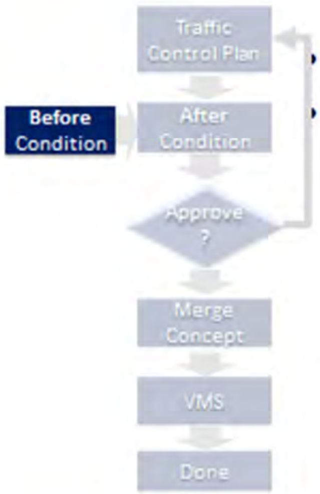


Schematic showing the logical relationships between tasks and phases


VCTR
TSU
Procedure
Traffic Control Plan Data
Assess Before Condition
(Capacity & Volume Profile)
Assess After Condition
(Capacity & Volume Profile)
Approve
Plan?
No
Yes
Choose Merge Concept
Analyze VMS Placement
Done


VCTR
TSU
Traffic Control Plan Data
Traffic
Control Plan
Before
Condition
After
Condition
Approve
?
Merge
Concept
VMS
Done
• Lane configuration of the work zone
• Hours negotiated with contractor
• Estimate per lane capacity through the work zone section

Assess Before Conditions

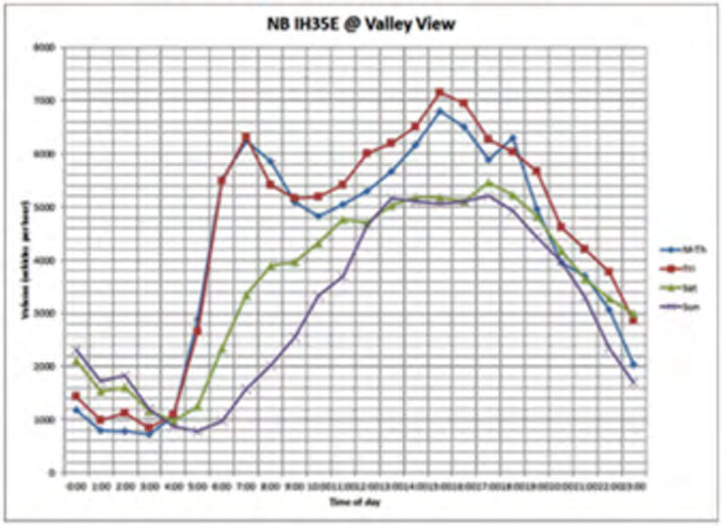


- Data Collection
- Obtain counts for
  - 7 days of week
  - 24 hours

24-hour Demand Data

- One data point for day
- Peak hour data
- Hourly data
- No data
  - Contact Metropolitan Planning Organization

Example Count Data



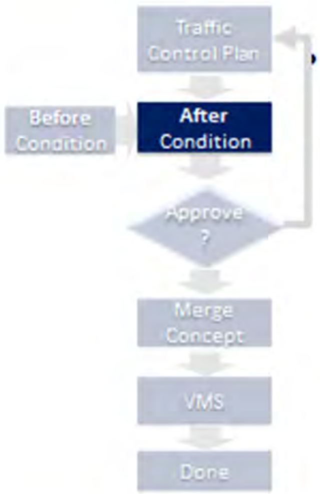
Example Count Data

- Period A, represents peak conditions, **full capacity of the corridor is necessary**
- Periods B, C, and D, are weekend peak, off-peak, and night respectively, and require **less than full capacity**

	Northbound IH 35E			
	Sunday	Monday-Thursday	Friday	Saturday
0:00	D	D	D	D
1:00	D	D	D	D
2:00	D	D	D	D
3:00	D	D	D	D
4:00	D	D	D	D
5:00	D	D	D	D
6:00	D	A	A	D
7:00	D	A	A	C
8:00	D	A	A	C
9:00	D	A	A	B
10:00	D	A	A	B
11:00	B	A	A	B
12:00	B	A	A	B
13:00	B	A	A	B
14:00	B	A	A	B
15:00	B	A	A	B
16:00	B	A	A	B
17:00	B	A	A	B
18:00	B	A	A	B
19:00	B	A	A	B
20:00	B	A	A	C
21:00	D	C	C	C
22:00	D	D	D	D
23:00	D	D	D	D

A	Weekday peak
B	Weekend peak
C	Off-peak
D	Night

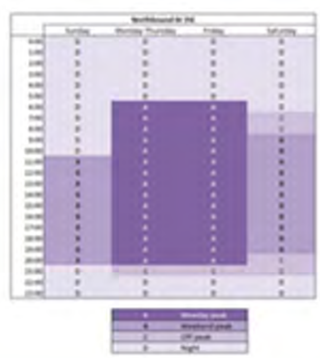
Assess After Conditions



- **Diversion Estimation, after demand is usually less than before demand**
  - If available, use Dynamic Traffic Assignment (DTA)
  - If DTA is not available, use Static Traffic Assignment (STA)
  - If neither DTA or STA is available, *assume 15% diversion*

Approve Traffic Control Plan

- Identify days and hours available for work
- Find hours and days when demand to capacity ratio (D/C) less than 1

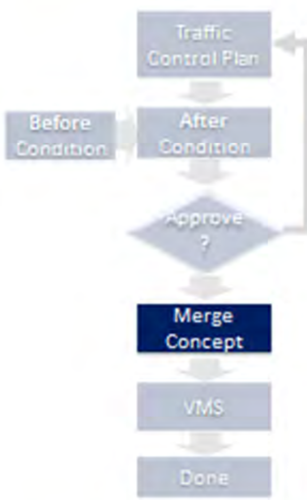


(a)

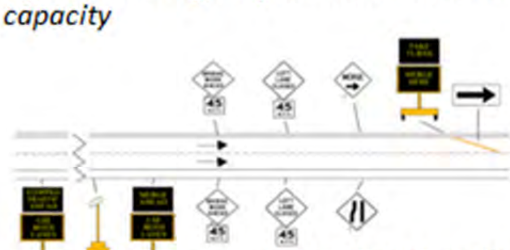
Approve Traffic Control Plan

- Compare days/hours to needs/requests made by the contractor
- If needs cannot be met, adjust configuration . .

Choose Merge Concept

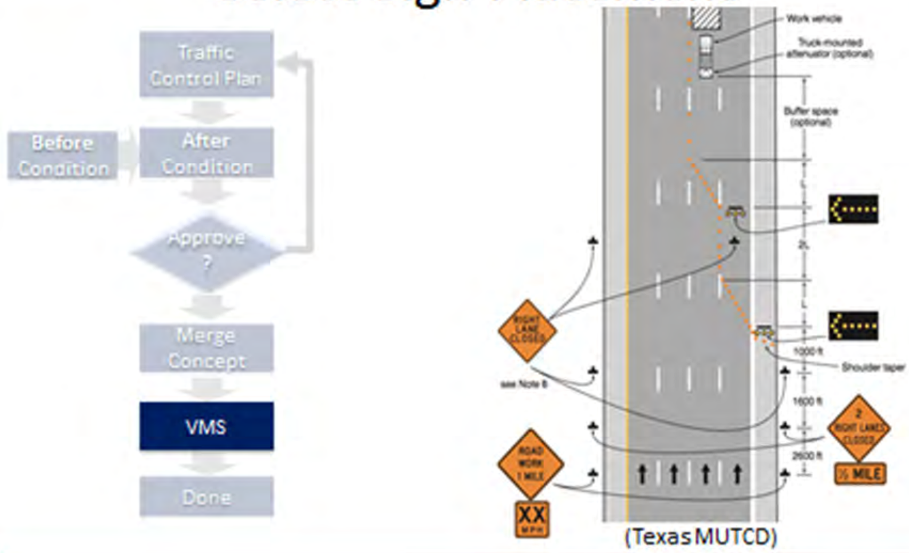


- Early Merge . .*preferred* . .*less queuing* . .*less user cost*
  - Static or Dynamic
- Late Merge . .*if Demand approaches capacity* . . *queuing expected*
  - Static or Dynamic
- Signalized Merge . .*if Demand exceeds capacity*



Dynamic Late Merge (ITE)

Select Sign Placement



(Texas MUTCD)

Issues in the Decision Tree

- Merge Concepts
- Networks and Diversion
- Queue Length Prediction tool

Introduction to Merge Concepts

- Three families of merge concepts are available:
  - Early Merge
    - Ideal for low demand (V/C < 1)
  - Late Merge
    - Ideal for low to moderate demand (V/C = 1)
  - Signal Merge
    - Ideal for heavy demand (D/C > 1)

Merge Concepts (1 of 3)

- Early Merge
  - Assumed effective with low traffic demand (V/C < 1)
    - Queue Length Prediction tool
  - Implemented as either static or dynamic
  - Used to warn drivers in advance of work zone of upcoming closed lane(s)
    - Allows time for users to find gaps to merge and complete merging prior to closure
    - Reduces roadway capacity
  - Early merge guidance given at varying distance up to ¼ mile, assuming ample merge distance before work zone

Merge Concepts (2 of 3)

- Late Merge
  - Assumed effective with moderate traffic demand (V/C approximately 1.0)
  - Implemented as either static or dynamic
  - Encourages all lanes to be used until specified merging point
    - Once vehicles reach point, users in closed lane(s) merge in an alternating pattern
    - Takes advantage of full capacity of highway to store queue
  - Safety Implications

Merge Concepts (3 of 3)

- Signalized Merge
  - Assumed effective with heavily congested traffic demand (V/C > 1)
    - All vehicles treated equally by giving equal fractions of green times for all open lanes
      - Green provided to left lane to merge followed by equal green time for one or two through lanes (depending on lane configuration)
      - Used to minimize queue jumping and driving in closed lanes, since there is no priority for any lanes
  - Simulations run for 30s, 60s, 90s, 120s cycle lengths

Introduction to VISSIM

- VISSIM microsimulation used to determine optimal merge concept techniques
  - Simulation run for 2-to-1, 3-to-2, and 3-to-1 lane configuration
  - Varying per lane demand on remaining open lanes through the work zone of 1800, 2000, 2200, 2400, and 2600 pcphpl
- Texas MUTCD suggests minimum desirable taper length of 660 ft and minimum distance between tapers of 1560 ft for speeds less than or equal to 65 mph

Optimal Merge Concepts

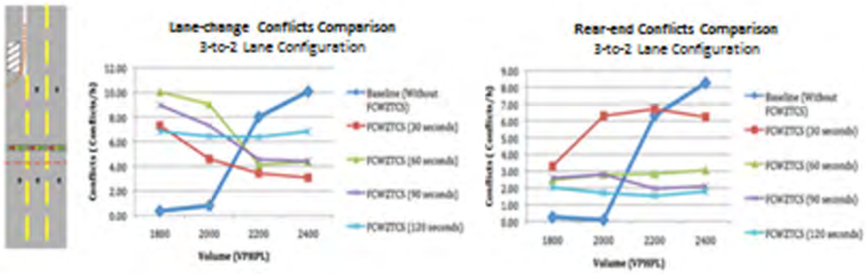
	User Demand				
	1800 pcphpl	2000 pcphpl	2200 pcphpl	2400 pcphpl	2600 pcphpl
2-to-1	EM	EM	LM	SM-90s	SM-90s
3-to-2	EM	EM	EM	EM	SM-120s
3-to-1	EM	EM	EM	SM-60s	SM-60s

Optimal Merge Concept Techniques based on Lane Configuration and Demand

	User Demand				
	1800 pcphpl	2000 pcphpl	2200 pcphpl	2400 pcphpl	2600 pcphpl
2-to-1	60 sec	90 sec	90 sec	90 sec	90 sec
3-to-2	60 sec	60 sec	60 sec	60 sec	120 sec
3-to-1	60 sec	60 sec	60 sec	60 sec	60 sec

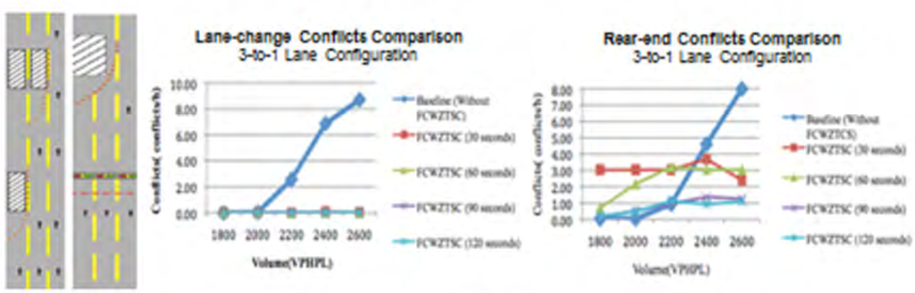
Optimal Signalized Merge by Cycle Length

SSAM Result: 3-to-2 lane configuration



- For low traffic demand, such as 1800 pcphpl or 2000 pcphpl, the MUTCD lane control strategy works well and has the least lane-change and rear-end conflicts.
- For heavy traffic condition, Fixed Cycle Work Zone Traffic Signal Control (FCWZTSC) can reduce both lane-change and rear-end conflicts.

SSAM Result: 3-to-1 lane configuration



- FCWZTSC can significantly reduce lane-change conflicts, especially when the traffic volume is high.
- When the traffic demand is light, such as 1800 pcphpl, 2000 pcphpl or 2200 pcphpl, the implementation of FCWZTSC does not reduce rear-end conflicts. When the traffic volume reaches to 2400 pcphpl, FCWZTSC starts to work well and reduce rear-end conflicts.

4-Step Transportation Planning Process

- Generate a model capable of replicating recurrent traffic conditions
- Compare alternative network improvements in future years
  - Run the assignment using different network configurations & assess network performance
- Operations: assess network-level impacts of proposed changes

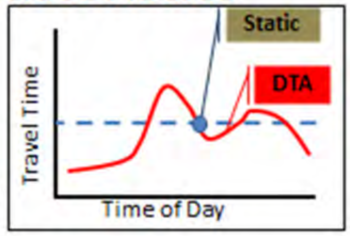
Recap: Assignment in the Planning Process

Given
- Trip matrix
- Transportation Network & Traffic Control

Find
- Paths chosen by drivers under recurring
- Corresponding network performance

DTA Distinctive Characteristics

Drivers make different choices depending on their departure time



Captures the evolution of traffic conditions during the analyzed period

- Static models assume AVERAGE conditions

Simulation-based approach used to compute travel time (instead of BPR formula)

- Explicit modeling of signals and other traffic control

VISTA: A SIMULATION-BASED DTA SOFTWARE

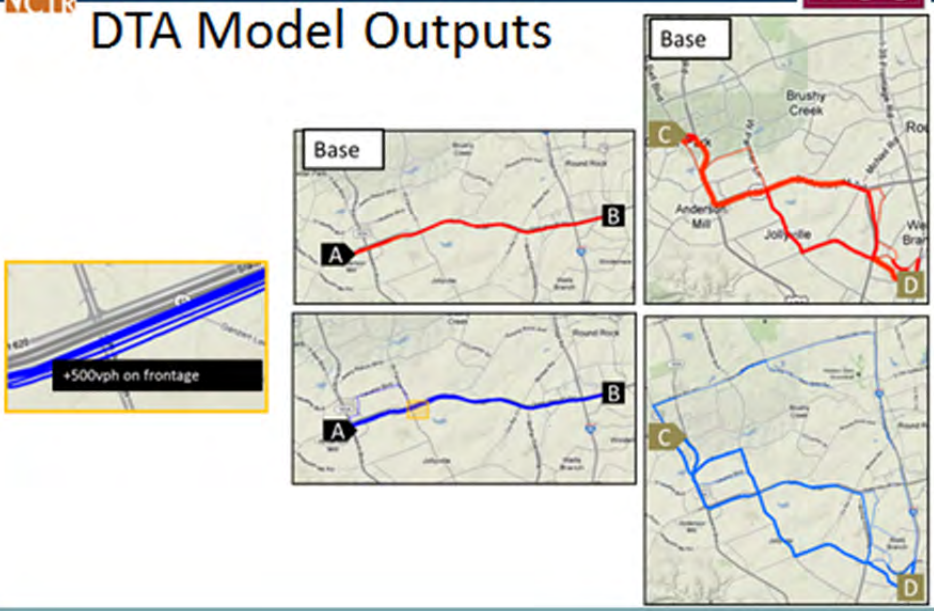
- Simulation based DTA software
  - Uses a simulation technique that can model VERY LARGE regions efficiently
  - Traffic simulator can be used as a stand-alone module to support network operation
- Web-based implementation
  - Does not require purchasing hardware
  - Potential to be run in parallel in many servers
  - Data Warehousing
- Algorithms & methodologies supported by research
  - Equilibrium-based methodology
  - Can be continuously customized & improved

DTA Model Outputs

Vehicles spatio-temporal trajectories

- Link volumes per time of day
- Travel time along corridors per time of day
- Sufficient detail to show network effects of small work zones
- Turning movements and approximate delay at intersections
- Inputs for sub-network & micro-simulation analysis

DTA Model Outputs



Traffic Diversion

- Usefulness of diversion rate

Network level diversion rate:

describe how the work zone affects the whole network

determine the demand at work zone when active

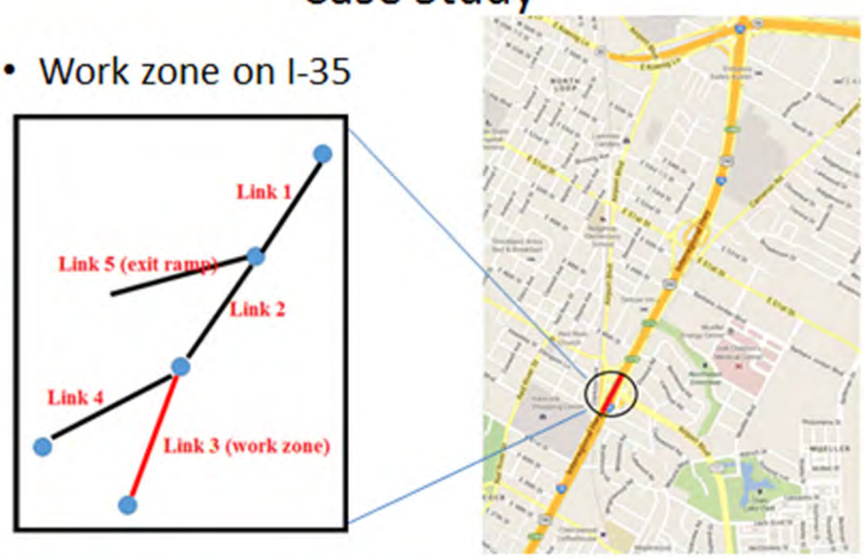
Local level diversion rate:

determine what types of guidance should be provided

how many lanes can be closed

Case Study

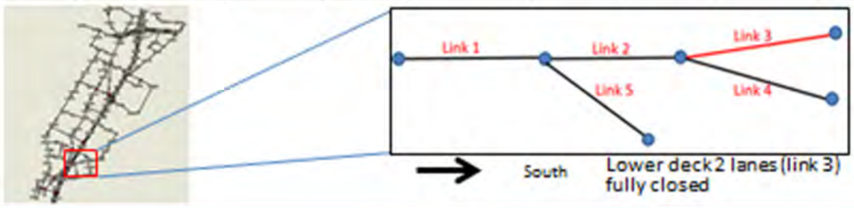
- Work zone on I-35



Demand Comparison between DTA and STA

Morning Peak Average Hourly Demand by STA				
	Normal		Work zone	
Link ID	Total	per lane	Total	per lane
1	**18940**	4735	14912	3728
2	**17296**	4324	11160	2790
5	**1647**	1647	3755	3755

Morning Peak Average Hourly Demand by DTA				
	Normal		Work zone	
Link ID	Total	per lane	total	per lane
1	6472	1618	3500	875
2	5860	1465	2800	700
5	612	612	697	697



Traffic Diversion

- Morning peak period (7am-9am)

	Link ID	STA	DTA
Network level	1	21%	**46%**
Local level	2	**16%**	11%

- DTA has higher network diversion rate
- DTA has smaller local diversion rate

ESTIMATING QUEUE LENGTHS

Queues in the proximity of work zones may form for several reasons

- Capacity through work zone lower than demand
- Merge/diverge maneuvers
- Other behavioral factors

Queue length estimates can inform traffic management decisions

- Placement of VMS
  - Far enough behind queue so that drivers have options
  - Close enough to work zone encourage compliance
- Detour information
  - Avoid secondary queues due to excessive diversion

METHODS TO ESTIMATE QUEUE LENGTHS

Simple macroscopic models (QUEWZ, etc.)

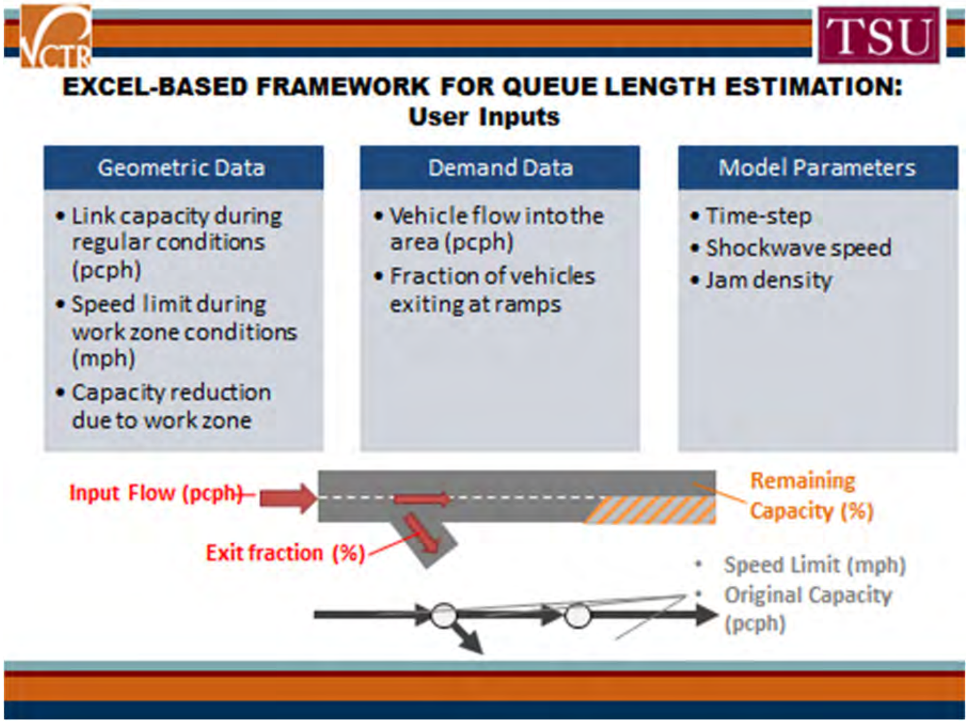
- Not detailed enough for traffic management

Detailed microscopic models (VISSIM,CORSIM)

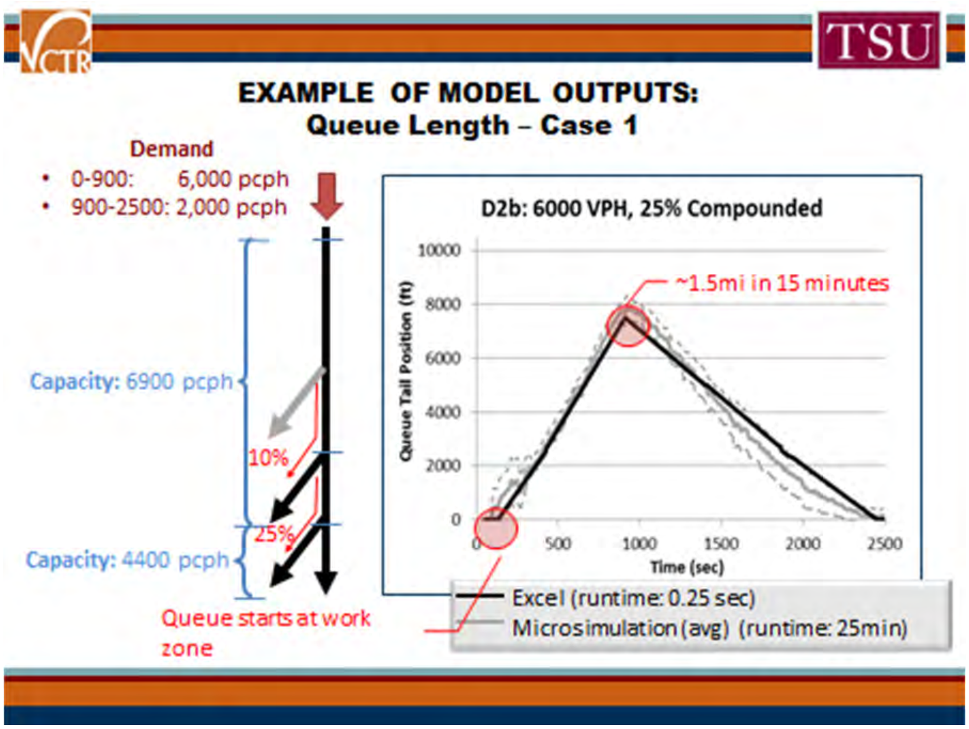
- Computationally intensive and time consuming coding

Medium resolution models (e.g. Link Transmission Model - LTM)

- Can be implemented in excel for freeway segments with complex geometry
- Capture queue formation and dissipation with enough accuracy
- Run VERY fast
  - Can be used for real time traffic management
  - City-wide implementations are feasible (integrated with planning models)

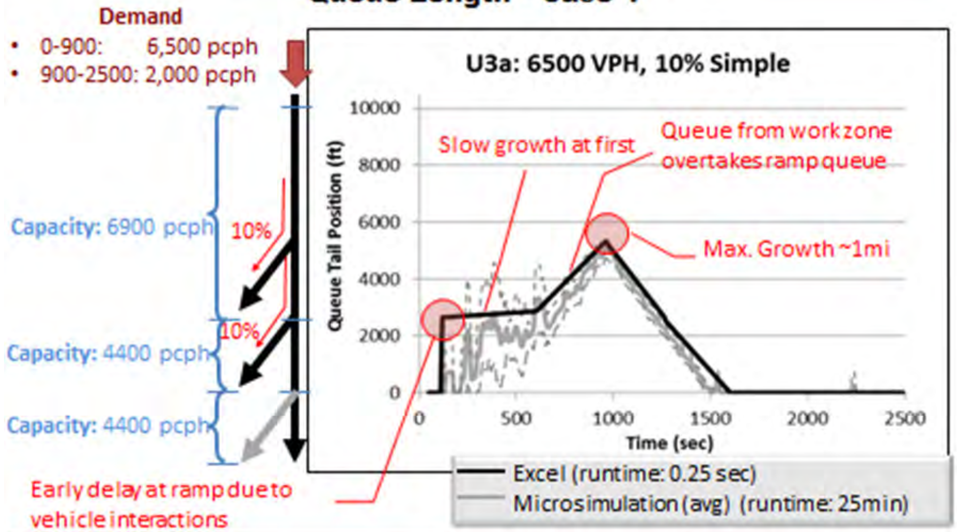


TSU
EXCEL-BASED FRAMEWORK FOR QUEUE LENGTH ESTIMATION:
User Inputs
Geometric Data
• Link capacity during regular conditions (pcph)
• Speed limit during work zone conditions (mph)
• Capacity reduction due to work zone
Demand Data
• Vehicle flow into the area (pcph)
• Fraction of vehicles exiting at ramps
Model Parameters
• Time-step
• Shockwave speed
• Jam density
Input Flow (pcph)
Remaining Capacity (%)
Exit fraction (%)
Speed Limit (mph)
Original Capacity (pcph)



TSU
EXAMPLE OF MODEL OUTPUTS:
Queue Length – Case 1
Demand
• 0-900: 6,000 pcph
• 900-2500: 2,000 pcph
Capacity: 6900 pcph
10%
25%
Capacity: 4400 pcph
Queue starts at work zone
D2b: 6000 VPH, 25% Compounded
~1.5mi in 15 minutes
Queue Tail Position (ft)
10000
8000
6000
4000
2000
0
500
1000
1500
2000
2500
Time (sec)
Excel (runtime: 0.25 sec)
Microsimulation (avg) (runtime: 25min)

EXAMPLE OF MODEL OUTPUTS: Queue Length – Case 1



EXCEL-BASED FRAMEWORK FOR QUEUE LENGTH ESTIMATION: User Inputs Detail: Demand Data

Vehicle flow into the area (pcph)

- Based on real data or estimates
- May be changed to test impact of diversion
- May have different values during analyzed period

Fraction of vehicles exiting at ramps

- Based on real data or estimate
- Can be changed to test impacts of proposed detours

EXCEL-BASED FRAMEWORK FOR QUEUE LENGTH ESTIMATION: User Inputs Detail: Model Parameters & Capacity Reduction

Model Parameters

- Time-step: smaller than the free flow travel time in any link.
- Shockwave speed: Empirically set to half the speed limit / 30mph. May be calibrated.
- Jam density (pcpmpl): maximum number of vehicles that can fit in one mile of freeway. Empirically set to 315vpmpl. Can be calibrated.

Questions and Suggestions?