Evaluating the Effects of Queue Warning Systems on Freeway Work Zones Using Traffic Simulation Software

by

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ABSTRACT

Work zones are a key component of the rehabilitation efforts that travelers on the U.S. national highway system interact with increasingly every day. Roadway construction, rehabilitation, and maintenance poses a potential risk for the travelling public, whether the work is being done within in the vicinity or on the road itself. To minimize risks and increase safety for both drivers and workers, intelligent transportation systems are now being used to remedy deficiencies of current practices. Rear-end crashes are the most predominant type of collision in work zone environments, mainly due to high-speed vehicles approaching a queue of stopped or slow traffic as a consequence of reduced capacity. The sudden reduction of speed in a freeway environment is not usually expected by the motoring public; therefore, innovative approaches have been applied to warn drivers more effectively. A queue warning system (QWS) is a work zone intelligent transportation system comprised of a set of roadside speed sensors and portable changeable message signs (PCMS). The system possesses a customizable speed threshold to detect queued traffic. This information is then relayed to portable changeable message signs that transmit warning messages to drivers upstream of the work zone. Currently in Alabama, applications of this technology are still in early stages. For the purposes of this research, traffic simulation software is used to investigate the effects of QWS on freeway traffic. A freeway work zone environment with a lane closure is modeled using field-gathered speed data, volumes, and vehicle length. This study proposes a set of six different speed reduction schemes in advance of the lane closure that simulate the QWS effect on three sets of non-compliant driver proportions.

Statistically significant differences are found between some of the proposed speed reduction schemes and between non-compliant driver proportions. From the analysis of results, conclusions are drawn and recommendations for practitioners are made based on speed differential measures for future safety enhancement using QWS application and deployment on freeway work zones.

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LIST OF ABBREVIATIONS

QWS	Queue Warning System
PCMS	Portable Changeable Message Sign
NHS	National Highway System
FHWA	Federal Highway Administration
DOT	Department of Transportation
VMT	Vehicle Miles Traveled
TTC	Temporary Traffic Control
ITS	Intelligent Transportation Systems
ALDOT	Alabama Department of Transportation
SWZ	Smarter Work Zones
EDC-3	Every Day Counts Round 3
TXDOT	Texas Department of Transportation
ADT	Average Daily Traffic
vpd	Vehicles per Day
TTI	Texas Transportation Institute
EOQ	End-of-Queue
ISATe	Enhanced Interchange Safety Analysis Tool
CMF	Crash Modification Factors
IDOT	Illinois Department of Transportation
NB	Northbound
WB	Westbound
mph	Miles per Hour
NCHRP	National Cooperative Highway Research Program
GPS	Geographical Positioning System
HCM	Highway Capacity Manual
FREEWAL-WZ	Freeway Evaluation Tool
LOS	Level of Service
QUEWZ – 98	Queue and User Cost Evaluation of Work Zones

vphpl	Vehicle per Hour per Lane
DSD	Desired Speed Distribution
CDF	Cumulative Distribution Frequency
PV	Passenger Vehicles
HV	Heavy Vehicles
NC-PV	Non-Compliant Passenger Vehicle
NC-HV	Non-Compliant Heavy Vehicle
NCDP	Non-Compliant Driver Proportion
ΔS	Speed Differential
CI	Confidence Interval
ME	Margin of Error
ANOVA	Analysis of Variance

CHAPTER ONE: INTRODUCTION

1.1 BACKGROUND

The core of the national highway system (NHS) consists of 47,575 miles of Interstate Highways, which represents just over 1% of highway mileage, but carries 25% of the nation's traffic (Federal Highway Administration 2013). Most roads in the U.S., 2.94 million miles, are located in rural areas and in 2010; only 64.6% of them were classified as 'Good Ride Quality' condition (Federal Highway Administration 2013).

U.S. highway capacity has been growing very slowly in recent years. From 2000 to 2013, the capacity of the highway system grew only 5.3% while the U.S. population grew 12%, the number of licensed drivers grew 11%, and the highway vehicle miles traveled grew 8.8%. The disparate growth between highway system and driver population suggests that demand is increasing faster than system capacity; therefore, improvements are needed to the current transportation system to meet these needs (Federal Highway Administration 2013). Generally, construction focus has shifted from expansion to maintenance for Departments of Transportation (DOT) across the U.S. in recent years. System preservation better describes the current effort that DOTs are presently undertaking, where the health and serviceability of existing roads are maintained and improved prior to the expansion of the system itself. The continuous and growing rehabilitation initiative leads to the increase of the presence of work zones in the NHS.

Countless highway infrastructure sections constructed during the expansion era, have met or past their useful lives. Consequently, constant construction, rehabilitation, and maintenance activities are tasks DOTs continuously undertake. Highway construction, rehabilitation, and maintenance activities disrupt mobility and increases risk for vehicular crashes. Per the Work Zone Safety and Mobility Fact Sheet, it is estimated that more than 20% of the NHS is under construction and more than 3,000 active work zones are expected during the peak construction season. To put this into perspective, 12 billion vehicle miles travel (VMT) will be through active work zones per year (Federal Highway Administration 2008).

According to the Federal Highway Administration (FHWA) in 2013, 16.1% of the nation's highways are in need of resurfacing or reconstruction. Locally, the state of Alabama has 2,602 miles (9.8%) of roads identified as in need of resurfacing or reconstruction (Federal Highway Administration 2013). This type of maintenance takes place in or within the vicinity of the road being repaired, requiring temporary traffic control (TTC) operations during the active construction, rehabilitation, and maintenance period.

1.2 WORK ZONES

TTC provides and facilitates continuity of the movement of motor vehicles, transit operations and accessibility to property and utilities. The primary function of TTC is to provide for the reasonably safe and effective movement of road users (i.e., motorists, bicyclists, and pedestrians) through and around work zones while reasonably protecting road users, workers, responders to traffic incidents and equipment. Dependent upon the nature of the construction, rehabilitation, or maintenance taking place, these zones present constantly changing conditions that are typically unexpected to the road user (Federal Highway Administration 2009).

A work zone is an area of traffic way where construction, maintenance, or utility work activities are identified by warning signals or indicators (Turner 1999). The presence of work zone conditions translates into the existence of heavy machinery, construction crews, and active work in, or within the vicinity of the roadway for road users. The presence of these features that do not represent typical traffic conditions have great influence on road user behavior and thus, the overall safety of the work zone.

Highway work zones are dangerous for both motorists and construction workers. In 2015, 96,626 crashes were estimated to occur in work zones in the U.S., of which 700 (0.7%) resulted in fatalities; for non-work zone environments the proportion of crashes resulting in fatalities is of 0.5% (Federal Highway Administration 2015), proving that work zone crashes show a greater severity trend when compared to non-work zone crashes. Figure 1-1 shows that fatal injury (K), incapacitating injury (A), and non-evident incapacitating injury (B) crashes constitute greater percentages when work zone conditions are present than when not. While work zone crashes are less common than non-work zone related crashes they tend to be more severe.

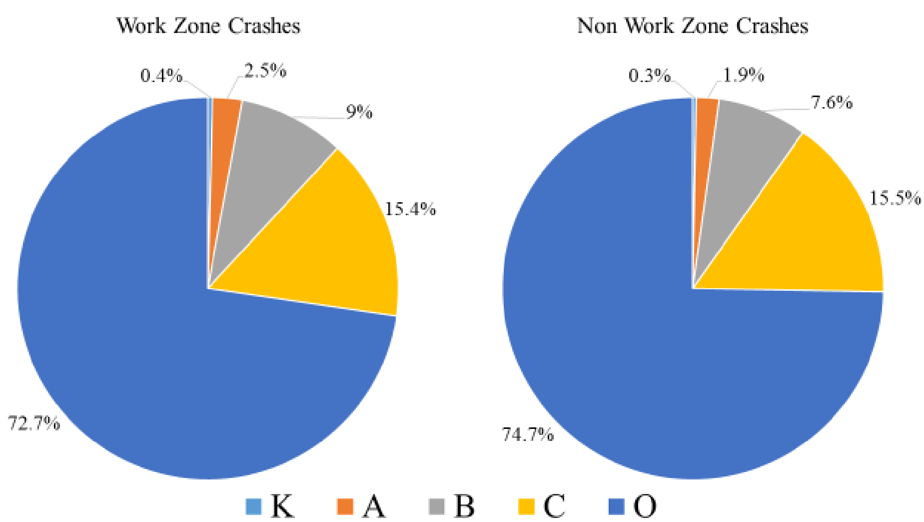


Figure 1-1: Crash Severity Distribution Comparison Between WZ and non WZ crashes (Federal Highway Administration 2014)

In the state of Alabama, 2,630 work zone related crashes were reported annually between 2009 and 2013. An average of 22 crashes per year result in fatalities. However, the actual figures could be larger since only 26% of work zone related crashes are actually reported (ALDOT 2013). More specifically in 2.83% of all reported fatalities that occurred in Alabama in 2015 occurred in work zone environments (National Work Zone Safety Information Clearinghouse 2015). The severity of a crash is closely related to the type of crash itself. The manner of crash is an effective way to classify distribution of crashes. Rear-end crashes tend to occur more frequently under work zone conditions (Federal Highway Administration 2015). When traffic congestion causes queued traffic due to an active work zone or a lane closure, rear-end crashes are more prone to happen than other type of crash. When work zones are present on high-speed facilities such as freeways, unexpected congestion can lead motorists to misjudge their ability to stop or reduce their speed in a safe and timely manner. Studies show rear-end collisions are the most common type of crashes in both conditions when work zones are and are not present (Ullman et al. 2008). Table 1-1 describes type of collision proportion based on lane closure presence.

Table 1-1: Percentage of Crashes by Collision Type during Daytime Conditions (Ullman et al. 2008)

Type of Collision	Active Work with Lane Closure	Active Work without Lane Closure	No Active Work or Lane Closure
Rear-End Crashes	47%	54%	49%
Sideswipe Collisions	14%	15%	15%
Fixed-Object Collisions	20%	10%	16%
All Other Collision	19%	21%	14%

Federal Highway Administration reports that 41% of fatal work zone crashes were rear end collisions, making them the most predominant manner of crash for work zone environments in 2013 (Federal Highway Administration 2015). One way to mitigate severity impacts that work zone environment impose on road users, innovative technology applications can be customized to serve a specific need. Information regarding use of innovative technology and its recommended use can be found in the Intelligent Transportation System Implementation Guide. This document provides direction to agencies regarding procedures, scope, requirements, and associated costs of intelligent transportation systems (ITS). ITS can serve many purposes at once, and also have the ability to help mitigate a very specific but predominant issue such as rear end crashes in work zone environments (Ullman et al. 2014). A Queue Warning System (QWS) is an innovative way to combat safety concerns and rear-end crashes that occur at the end of a queue upstream of a work zone. QWS can be defined as a standalone intelligent system able to warn oncoming traffic regarding of queued conditions upstream of the work zone. A QWS typically consists of non-intrusive speed sensors, several portable changeable message signs (PCMS), and a communication system with an interface based on speed threshold.

1.3 POTENTIAL SMART WORK ZONES IMPLEMENTATION IN ALABAMA

ITSs that make use of QWS as its main component provide an opportunity to directly target rear-end collisions that occur when vehicles reach a queue unexpectedly.

Limited deployments of this type of strategy have been deployed by state DOTs with promising results (e.g. Texas, New Hampshire, Massachusetts, etc.); however, the Alabama Department of Transportation (ALDOT) has yet to formalize deployment guidelines and documents. This study is a step in the development of statewide guidelines of QWS application for freeway work zones; where traffic software simulation helps serve as a pre-deployment tool for safety improvement feasibility.

1.3.1 Motivation

QWS is one of the many approaches available when deploying smarter work zones (SWZ). SWZ is a concept initiated by the FHWA to elevate and improve current work zone practices with innovative technology applications and improved project coordination. The goal encompasses effective traffic management during construction, while maintaining access to local businesses and residences. The Every Day Counts Round 3 (EDC-3) program initiative was established by the FHWA to encourage state DOTs to apply SWZs to use innovative strategies for optimizing work zone safety and mobility. One of FHWA's goals regarding project coordination states that by the end of 2016, at least 25 state DOTs should be incorporating work zone project coordination strategies into agency documentation of business processes. The second goal pertaining to project coordination relates to the work zone implementation strategies estimator.

Currently nine states have incorporated project coordination strategies into standard practice to mitigate work zone impacts and SWZ technology applications have been incorporated as standard practice in eleven states. Eighteen states alongside Washington, D.C. have at least incorporated said coordination strategies and 28 states in addition to Puerto Rico and Washington, D.C. are incorporating technology applications into planning, design, operation, and maintenance practices (Federal Highway Administration 2017).

SWZs have the intent of applying innovative technology to communicate actively and dynamically; with new technology and new communication efforts it would be appropriate to send the right message. The ALDOT applied for a grant through the EDC-3 program and in September 2015, and was selected to receive $100,000 to support procurement and deployment of a QWS.

1.3.2 TRAFFIC SIMULATION SOFTWARE

The role of traffic analysis tools is to assist transportation professionals and decision makers in evaluating strategies that can better meet their needs. It enables agencies, such as ALDOT, to understand the potential safety and mobility impacts of the project to identify the optimal alternative that will minimize impacts, costs, and time. There are several traffic analysis tools and software available for analysts; however, these tools vary in their scope, capabilities, methodology, input requirements, outputs, and calibration methods.

1.4 RESEARCH OBJECTIVES

The main focus of this research is to evaluate the effectiveness of QWS in a freeway work zone through a traffic simulation tool using field gathered traffic and speed data. The simulation tool used for this study is VISSIM. VISSIM is a microscopic simulation tool used to evaluate and model warning messages in a one directional two-lane freeway segment located on I-59 near Tuscaloosa, AL. This research includes two scenarios based on the simulation of a control and treatment case. Both scenarios will use the same traffic inputs obtained through a data collection plan using pavement-installed data collection equipment. The control case reflects the current conditions of a typical work zone layout on a two-lane facility (one side of a four-lane freeway) with a lane closure and no QWS in place. The treatment case reflects the addition of a QWS and its speed reductive effects to the freeway traffic prior to the work zone under queued conditions. The proposed treatment scenarios tested in the simulation model aim to achieve a better understanding of the speed reductive effects of QWS on freeway traffic under work zone conditions with a lane closure. The objectives of this study are described in the following statements.

1. Identify the need for QWSs application in Alabama freeway work zones.

Work zone crashes tend to be more severe than crashes occurring in non-work zone conditions. Rear-end collisions are the most predominant crash type under work zone circumstances; these typically occur due to an unexpected and abrupt change in travelling speeds. The main goals of a QWS are to monitor slow traffic approaching the work zone effectively and to transmit this information to drivers upstream of a queue in a timely manner. Achieving reduction in speed variability by warning drivers in advance of work zones of downstream traffic conditions can help reduce the frequency and severity of rear-end collisions.

2. Investigate the current availability and state-of-the-practice in QWS; and the safety and mobility effectiveness of previous deployments.

The purpose of this objective is to obtain and summarize existing knowledge regarding the effects of QWS and to determine the availability and characteristics of such systems. This literature review should provide insights on best practices for selection of QWS deployment sites and setups, which can then be considered by ALDOT in selecting locations for QWS freeway deployments.

3. Develop a traffic simulation model to estimate the traffic response when using QWS

To estimate the impacts of QWS on traffic flow parameters, a traffic simulation model will be developed using VISSIM. Using a representative set of field-gathered traffic volumes and related data, the traffic performance of control and treatment scenarios will be evaluated using measures of effectiveness such as speed distributions and travel time.

4. Evaluate the effectiveness of QWS through traffic software simulation

This study simulates queue warning systems as a step to reduce end-of-queue rear-end crashes in work zones. To evaluate the effectiveness of QWS, a traffic simulation model and a variety of speed reduction schemes will be used to measure the effectiveness of the speed reductions due to QWS-like effects prior to a lane closure.

5. Develop recommendations for modelers and practitioners

This task will synthesize the findings of the previous tasks for DOTs to consider in future QWS deployments as well as the results obtained from the QWS simulation model. These recommendations are based on the review of the literature, state of the art QWS practices and applications, and conclusions drawn from the results of the QWS simulation model developed in VISSIM.

1.5 ORGANIZATION OF THESIS

This thesis is organized into five chapters: *Chapter 1: Introduction*, addresses the current need for improvements in work zone safety due to the increase of temporary traffic control presence in freeways and the elevated risk of fatalities and injuries that these conditions represent; as well as the introduction of innovative technology application into current practices to achieve safety improvements. *Chapter 2: Literature Review*, includes the collection of information in form of published articles and guidance documents of several state DOTs is summarized to provide a better understanding of various highway agencies' experiences with QWS. The review addresses characteristics of work zones in which QWS have been deployed, advantages and disadvantages observed in their use, management of the systems, associated costs, and any scientific evaluations that have been conducted. This chapter also includes work zone classification, key concepts, and description of several types of simulation tools. *Chapter 3: Methodology*, provides a description of the model simulation and calibration methods used, as well as the means and process of data collection in a work zone. *Chapter 4: Analysis*, includes the results as well as background and explanation of the statistical analysis and data validation processes. *Chapter 5: Conclusions and*

Recommendations, provides findings obtained from the simulation, which are summarized to contribute to the development of future guidelines to serve ALDOT's purpose of future QWS deployment in freeways.

CHAPTER TWO:
LITERATURE REVIEW

2.1 INTRODUCTION

This chapter begins by identifying the characteristics of work zone crashes. This chapter then briefly details major projects and state of-the-art practices in QWS applications in work zones throughout the U.S. and how safety effects are evaluated. This chapter also describes the most relevant traffic simulation tools that are currently used by researchers with similar goals and approaches, as well as the application of simulation tools for specific freeway work zone conditions.

2.2 CRASHES WITHIN WORK ZONES

A study that examined the location of crashes within Virginia work zones, crash type, and severity concluded that most predominant type of collisions in work zone environments are rear end crashes, and for the distribution of crashes that occur in the advance warning area, 83% are rear end crashes. This study concludes that the causal factor for work zone rear end crashes is strongly correlated to speed; stating that rear end crashes are mainly caused by vehicles driving at different speeds and thus, a higher speed variance (Garber and Zhao 2001).

Another study compared fatal crashes between work zone and non-work zone conditions using several statistical analyses and tools; researchers were able to identify that side swipe and rear end collisions as the most problematic type of crash within work zones and that are the most likely to result in a fatality. Influencing factors that increase propensity of fatal crashes occurring in work zones suggested by this study are an increase in the number of lanes and speed limit. Some suggestions made by the researchers to countermeasure these influencing factors include speed harmonization, increasing the length of advance warning area, and use of dynamic message signs (Silverstein et al. 2014).

2.3 QUEUE WARNING SYSTEM CASE STUDIES

The following case studies highlight the results of QWS deployments that have been documented by state DOTs in detail. These case studies include data collection plans, methodology, deployment, equipment acquired, conclusions and recommendations. These cases have been reported, shared, and included in several guideline documents as examples.

2.3.1 TEXAS CASE STUDY

The Texas Department of Transportation (TxDOT) deployed ITS in work zones on Interstate 35 (I-35), on projects that primarily consisted of widening the facility from four to six lanes. This series of projects spanned 96 miles through a predominantly rural stretch of Central Texas. The average daily traffic (ADT) volume for this section of the interstate varied between 55,000 and 110,000 vehicles per day (vpd) with the percentage of heavy vehicles varying between 35% during daylight and at times exceeded 70% during the night. Although the majority of the work was shielded behind concrete jersey barriers, some lane closures were periodically necessary due to roadway maintenance or traffic switches between lanes. Lane closures were restricted to nighttime between 7:00 PM and 7:00 AM. Driver expectancy, which could correlate to unusual lane configuration or presence work zone equipment, within this corridor was substantially low; therefore, there was a need to provide enhanced warning systems for lane closures during the work zone. Driver expectancy relates to the driver's ability to readily respond to events and situations in a predictable and successful manner (FHWA 1986). The TxDOT and the Texas Transportation Institute (TTI) collaborated to deploy a highly portable work zone traffic control system consisting mainly of speed sensors, portable changeable message signs (PCMS) and portable rumble strip, in an effort to mitigate and reduce end-of-queue rear-end crashes.

The TxDOT deployment configuration consisted of speed sensors placed at the lane closure merging taper and at 0.5, 1.5, and 2.5 miles upstream of the taper; a PCMS was placed at 3.5 miles upstream of the taper, see Figure 2–1 below. When queues were expected, additional sensors were placed at 3.5, 4.5, 5.5 and 6.5 miles upstream of the taper, and an additional PCMS was placed at 7.5 miles upstream of the taper.

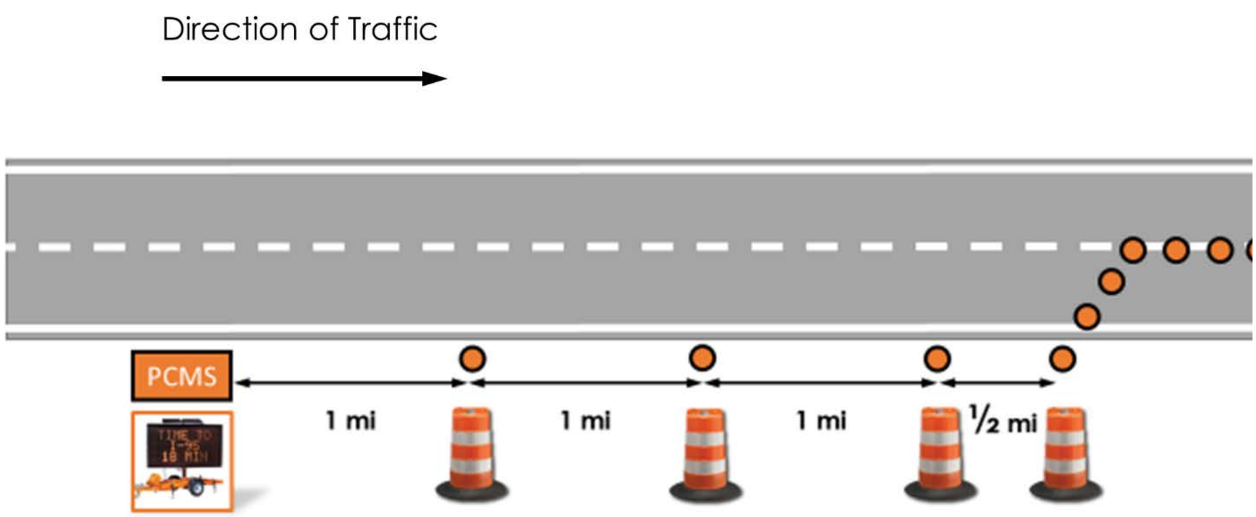


Figure 2–1: TTI's iCone Deployment Configuration Layout (Source: Ullman et al. 2016)

Even though rumble strips were not a mandatory practice in Texas work zones, TxDOT decided to use a new type of temporary rumble strip that could be easily deployed and removed, specially designed for short-term work zone projects. Two sets of rumble strips were placed, the first set consisted of three strips and was placed 0.25 miles upstream of the PCMS; the second set was placed 0.5 miles of the maximum expected queue length for that particular work night. In addition, warning signs for rumble strips were set 700 feet upstream of the first deployed set.

The deployment procedure started with first identifying when and where the nightly lane closures were scheduled; the second step consisted of the contractor submitting an electronic notice of each proposed lane closure including time, direction and location. Next, a queue prediction analysis was used to compare expected traffic demands for the closure to the expected work zone capacity. This prediction analysis was based on expected traffic demands on the planned night of the lane closure to the expected work zone affected capacity. If results indicated that a queue was expected to occur, then the end-of-queue (EOQ) warning system was deployed at that location.

By the time this advance warning system was implemented, over 200 nights of lane closures had already been performed without any EOQ warning system application. The analysis method that TTI used included a control group, corresponding to the nights that EOQ warning systems was not deployed, and a treatment group that corresponded to the nighttime lane closures where the EOQ warning system was deployed. The corridor was divided into a number of homogenous segments for the study; then TTI researchers used the Enhanced Interchange Safety Analysis Tool (ISATe) to estimate the expected number of crashes for each segment. ISATe provides information about the relationship between roadway horizontal geometry, cross-section elements, roadside barriers, and safety; it is based on research that quantified the relationship between various design elements and average crash frequency (Bonneson et al. 2011).

The researchers were able to compare the expected number of crashes per night for each segment to the actual number of crashes per night with both the EOQ and the non-EOQ groups. Table 2-1 summarizes the results of the analysis. The control and the treatment had similar number of nights deployed, with a difference of 10% control night's increase. A difference in 55% more miles for the treatment group and an increase of total number of crashes of 46% when EOQ was not deployed. Summary of results and comparison between control and treatment scenarios can be found in Table 2-1.

Table 2-1: Summary of TxDOT Safety Effects Results (Source: Ullman et al. 2016)

Measured Parameter	Control	Treatment
Total nights of lane closures (where queues were expected) analyzed	234	216
Total miles of lane closure segments analyzed	829	1290
Total number of crashes expected if no work zone present	10.4	10.2
Total number of crashes that actually occurred	19	13

To quantify the effectiveness, a crash modification factor (CMF) was computed using number of expected crashes for both groups; see Table 2-2 for CMF analysis results.

$$CMF_{EOQ} = \frac{\dfrac{TA_{EOQ} \times TE_{no\ EOQ}}{TE_{EOQ} \times TA_{no\ EOQ}}}{\left(1 + \dfrac{1}{TE_{no\ EOQ}} + \dfrac{1}{TE_{EOQ}} + \dfrac{1}{TA_{no\ EOQ}}\right)} \qquad \text{(Eq. 2.1)}$$

Where,

CMF_{EOQ} = Crash modification factor representing the proportional effect of the EOQ deployments on crashes,

TA_{EOQ} = Total crashes actually occurring during nights when an EOQ was deployed,

$TA_{NO\ EOQ}$ = Total crashes actually occurring during nights when no EOQ was deployed,

TE_{EOQ} = Total crashes expected during nights when an EOQ was deployed if no work zone had been present, and

$TE_{NO\ EOQ}$ = Total crashes expected during nights when an EOQ was not deployed if no work zone had been present.

Table 2-2: Summary of TxDOT Analysis Results
(Source: Ullman et al. 2016)

Metric	Value
CMF	0.559
Standard Error (CMF)	0.255
Level of Marginal Significance	0.085

Given that the sample size was considerably small and data were collected over less than a year; nevertheless, the results do indicate favorable results regarding crash reduction. TTI researchers also investigated the severity of the crashes happening within these work zones. Table 2-3 summarizes the results of the crash severity analysis.

Table 2-3: Effect of TxDOT EOQ System Deployment
(Source: Ullman et al. 2016)

Crash Type	Control	Treatment
Percentage of Severe Crashes[1]	58%	41%
Percentage of Rear-End Crashes	58%	36%

Note: [1]severe crashes included all fatal and injury crashes

The last step of the analysis was to translate crashes per night into a crash cost savings per night equivalent. The process consisted of estimating the cost of severe crashes and property damage only crashes using the traditional societal crash cost values updated to the 2014 equivalent dollar amount using the consumer price index and present worth method (Council et al. 2005).

The total calculated cost savings during the 216 nights that the EOQ warning system was deployed was of $1.36 million. The savings was due to a reduction in number of crashes severity of the crashes; by dividing the total cost savings into the number of nights of EOQ deployed, an average per night crash cost saving value of $6,313 was obtained. The EOQ system can be obtained for approximately $250,000 and the per-night-costs of the maintenance, labor and deployment of the warning systems can range from $3,700 to $5,000.

The researchers suggested future recommendations including that by using queue prediction and estimation tools, queue formation was forecast for all 216 nights of the study; however, there was no attempt to determine whether the queues actually formed.

2.3.2 ILLINOIS CASE STUDY

The Illinois Department of Transportation (IDOT) employed new queue detection technology on two of their projects; each of them consisted of two interstate highways overlapping, for a length of 4 and 6 miles respectively.

The Interstate 70 (I-70) and Interstate 57 (I-57) interchange was a four-lane facility located in a rural area near central Illinois. These two corridors are concurrent for a distance of 6 miles and in 2011 served 45,000 vpd. Out of the average daily traffic, 45% are represented by trucks during the day and this percentage can increase up to 90% at night. Due to project operations, elimination of shoulders, and lane shifts, temporary lane closures were necessary and affected the capacity of the facility. The rural location of the construction project decreased driver expectancy of unforeseen temporary changes in lane configuration or work zone presence; therefore, there was an urgent need from the IDOT for work zone ITS technology to be implemented.

The I-57 and Interstate 64 (I-64) interchange is a facility where both corridors are concurrent for a length of approximately four miles; the interchange is also located in a predominantly rural area where driver expectancy is considered low. In 2011, the ADT for this section was 39,610 vpd; where 33% of the traffic was represented by heavy vehicles. The improvements for this section included the construction of a new reinforced concrete pavement in both directions and the addition of a third lane in the median. To avoid the need for temporary pavement construction, lane closures on I-57 northbound (NB) and I-64 westbound (WB) were implemented. Lane closures were likely to create queues and increase the possibility of rear-end collisions.

For the I-70/I-57 interchange project, lengths of queues were not calculated, but they were expected to be low and erratic. On the other hand, for the I-57/I-64 project the QuickZone traffic analysis tool provided by the FHWA was used to estimate the length of the queues that would develop due to the lane closures. Based on this analysis tool and the 2011 ADT, IDOT was able to estimate between 3 and 4 miles of accumulated traffic in the NB direction. Regardless of the different work tasks, characteristics, and locations that these two projects had, the requirements to increase work zone safety were of common ground. The general solution requirements for both projects included: the automatic detection of slow or queued traffic beforehand and throughout the work zone, the ability to notify approaching motorists about the queued or slow traffic upstream, and encourage diversion to alternate routes by informing motorists of current delays. The overall objective was to implement an ITS that reduces the frequency and severity of rear-end crashes in slowed/stopped traffic within the vicinity of the work zone. IDOT planned in advance working closely with FHWA and Illinois State Police to incorporate the work zone smart monitoring system description and requirements in the contract document as a bid item. System requirements that were common for both projects included in the "smart traffic monitoring system" bid item, described below:

- Internet-based software to have a real-time system monitored by IDOT and the contractor,
- capability of computing delay and travel times in both directions, and the ability update these times frequently,
- ability to determine an appropriate message to display to motorists based on a predetermined algorithm,

- posting of predetermined messages on PCMS,
- ability to allow operators to override the system to post messages as needed, and
- capability to operate automatically and continuously on a 24/7 basis.

The I-70/I-57 project manager worked closely with several IDOT districts and manufacturers in order to generate a list of system necessities that would satisfy the bid requirements. The solution included:

- 25 PCMS capable of remote control via central computer base station (i.e. Ver Mac®),
- 25 portable traffic sensors linked to the central base station (i.e. Wavetronix®),
- 20 remote video cameras linked to a central station,
- 1 central base station with appropriate software and dedicated communication devices that would link with traffic management system components, and
- a password-protected website accessible to personnel to monitor the project conditions.

Additionally, portable solar-powered trailers and cellular modems were included to ensure communication between devices. Ver-Mac, Inc. provided the PCMS that were deployed 10 to 12 miles upstream of the project in each direction of both I-70 and I-57 covering a total of 76 miles. This was done with intention of notifying motorists far in advance of any congestion or delay advising drivers to seek alternate routes to avoid the work zone. A speed-based and delay threshold as shown in Table 2-4 was determined in order to display warning and informative messages on the PCMS accordingly.

Table 2-4: TxDOT Implemented Thresholds for Warning / Informational Messages (Ullman and Schroeder 2014)

Traffic Logic Condition		Phase I Message	Phase II Message
No congestion detected		NO DELAY TO <I-57 or I-70>	ROADWORK XX MILES AHEAD
Speeds less than 40 mph detected		SLOW SPEEDS AHEAD	PREPARE TO STOP
Significant delays detected (presented on PCMS farther upstream from the congestion)		XX MIN DELAY	NEXT XX MILES
Even more significant delays detected		XX MIN DELAY	CONSIDER ALT ROUTE
Delays exceeding a maximum threshold value detected (i.e. 20 min)	Certain signs	MAJOR DELAYS > 20 MIN	ALT ROUTE EXIT XX
	Other signs	EXPECT MAJOR DELAYS	<Direction> BOUND <I-57 or I-70>

This contract was set to bid a per-month unit cost for the system, to include maintenance, operation, and relocation expenses of the warning system. The initial mobilization was budgeted at $1.5 million and the traffic management operation and maintenance was of $1,800 per month over the project time period of 25 months. A total of $1.545 million was spent on deployment and operational costs.

For the I-57/I-64 interchange project, the IDOT used a followed a different approach that included commercial off-the-shelf technology rather than developing a new system. IDOT included the smart monitoring system requirements and specification in the contract document and the contractor was left to identify the different vendors. The contractor was also in charge of furnishing, installing, maintaining, removing, and programming the various system components to guarantee the functionality of the monitoring system. For this project requirements for volume data or inclement weather condition performance were not included in the contract but rather specified that the devices used had to be crashworthy in accordance with National Cooperative

Highway Research Program (NCHRP) Report 350 or to be made NCHRP 350-compliant for the devices to be placed within 30 feet off the edge of pavement.

The I-57/I-64 project used the iCone® portable traffic monitoring system. This system consists of radar vehicle detection, geographical positioning system (GPS) antenna, cellular and backup satellite communication as well as processing capabilities. The system was deployed covering all four approaches of the work zone, consisting mainly of:

- 32 iCone devices placed on all of the four approaches,
- 15 PCMS, and
- a website portal to monitor the devices and messages displayed.

The setup for this project consisted of placing the sensors 1 mile apart from each other, starting at 3 to 14 miles upstream of the interchange work zone. If the queue extended beyond two miles downstream of the work zone, the PCMS would advise motorists to seek alternate routes. The system was acquired by a subcontractor which was initially budgeted at $1 million, subsequently the system had to be expanded; at an additional cost of $172,000.

Delay computations were based off queue lengths upstream of the work zone and slow/stopped traffic ahead messages were displayed to minimize the probability of rear-end crashes. The speed-based threshold followed the criteria found in Table 2-5.

Table 2-5: TxDOT Speed Based Thresholds
(Ullman and Schroeder 2014)

Criteria to display message		Lowest speed downstream > 45 mph	Lowest speed downstream > 30 to 45 mph	Lowest speed downstream < 30 mph	
PCMS closest to the project	Phase 1 message	ROADWORK NEXT XX MILES	CAUTION SLOWING TRAFFIC	CAUTION STOPPED TRAFFIC	
	Phase 2 message	XX MIN THRU ROADWORK	SLOWING XX MILES AHEAD	STOPPED XX MILES AHEAD	
Criteria to display message		Delay < 5 minutes	Delay 5 to 25 minutes	Delay 26 to 45 minutes	Delay > 45 minutes
PCMS farther upstream from the project	Phase 1 message	ROADWORK NEXT XX MILES	XX MIN DELAY AHEAD	XX MIN DELAY AHEAD	XX MIN DELAY AHEAD
	Phase 2 message	XX MIN THRU ROADWORK	N/A	CONSIDER ALT RTE EXIT XX	FOLLOW ALT RTE EXIT 77

Statistics presented by IDOT show the operational use comparison between the two projects, shown in Table 2-6. PCMS on the I-57/I-64 project were activated for longer periods of time due to a long-term lane closure that significantly affected capacity. This congestion was alleviated by redirecting traffic through exit 77. Field oriented measures of effectiveness for this project, can be observed in Table 2-6 , these consisted on project personnel would driving through the work zone when queues were detected and delay messages were activated; and timed their run to verify the system was functioning properly.

Table 2-6: Operational Data from the I-70/57 & I-57/I-64 projects (Ullman and Schroeder 2014)

Performance Metric	I-70/I-57 Interchange	I-57/I-64 Interchange
Warning Message Activation Frequency	Typically 0 - 6 activations of prepare-to-stop message per month	189 - 264 activations per month for any queue message. 29 - 68 activations per month for extended queues.
Message activation duration	0 - 90 hours per month depending on sign 2 - 254 minutes per occurrence	2 - 510 minutes per occurrence for any queue message 5 - 210 minutes per occurrence of extended queues

Preliminary analyses show the queue detection system after its implementation on I-70/I-57 interchange demonstrated significant safety improvements. Queueing crashes decreased by 14% and crashes involving injuries decreased by 11%, despite a 52% increase in the number of days when temporary lane closures were implemented. Lessons learned for both IDOT projects:

- Camera coverage was useful but not essential for a successful system. For the I-70/I-57 project, traffic cameras were implemented although not required by IDOT. Cameras served as a quick way to monitor and identify traffic issues; however, the lack of cameras on the I-57/I-64 project was not reported as a problem. It is important to highlight that the I-70/I-57 work zone covered significantly more mileage, by adding reduced lane widths and the loss of shoulder this project had higher risks of crashes and delays occurring than its counterpart.

- Calibrate the system to overestimate delays slightly. Overestimation of the delay times showed on the PCMS to the motorist public was perceived better by the travelling public.

- Experience from both systems show that there is an ultimate need to include enough queue warning system detail in the specifications for the vendors to bid competitively, but not so much that excludes them from participating. It is key to identify what data, functions and features need to be implemented to guarantee that the system meets its objectives.

- Adjustments to the systems are likely to be necessary after it is first deployed. Messages posted on the I-70/I-57 project had to be adjusted; the "REDUCE SPEED AHEAD" was replaced with a message showing the actual speeds that the motorists would expect upstream of the work zone. The public showed to be more responsive to this message. Queue lengths for the I-57/I-64 project were underestimated at first. IDOT used ADT volumes and translated them into vehicles per hour in order to input them into QuickZone. The queues extended 11 miles upstream of the work zone and more equipment had to be purchased and installed, so that the queue would not extend beyond the system's limits.

2.4 WORK ZONE CHARACTERISTICS AND MODELING TOOLS

QWS applications on work zones are temporary yet substantially influential traffic control features. Factors associated with modeling a work zone environment include work zone characteristics, transportation management plan strategies, available data archives, agency resources, modeling capabilities, and performance measures. Potential qualitative and meaningful assessment of work zone impact through simulation software relies on the effective integration of data and tools.

2.4.1 WORK ZONE TYPE

Work zone type describes the expected level of traffic impact on drivers due to the presence of work zone conditions. The type of work zone is a strong indicator of the level of resources available to conduct a work zone analysis.

Type I: affects drivers at the metropolitan, regional, and interstate levels over lengthy periods of time. The magnitude of these projects tends to attracts public interest and impact significant amount of road users.

Type II: impacts drivers at the regional and metropolitan level. Similarly, this type of work zone impacts a wide range of road users and tends to attract significant public interest. Cost associated with a Type II operation are considerably moderate to high and the time period attributed are usually prolonged.

Type III: affects drivers at the metropolitan or regional level, and drivers are affected at low to moderate levels. These work zones have moderate to low levels of public interest; duration and cost of construction are condensed. Lane closures can be present in Type III work zones, but only to limited periods of time.

Type IV: have limited an effect on driver, minimal public interest, and work zone duration is usually short. Traveler information delivery can be seen as a challenge, since the information might be delivered by the time the project is completed (Hardy and Wunderlich 2009).

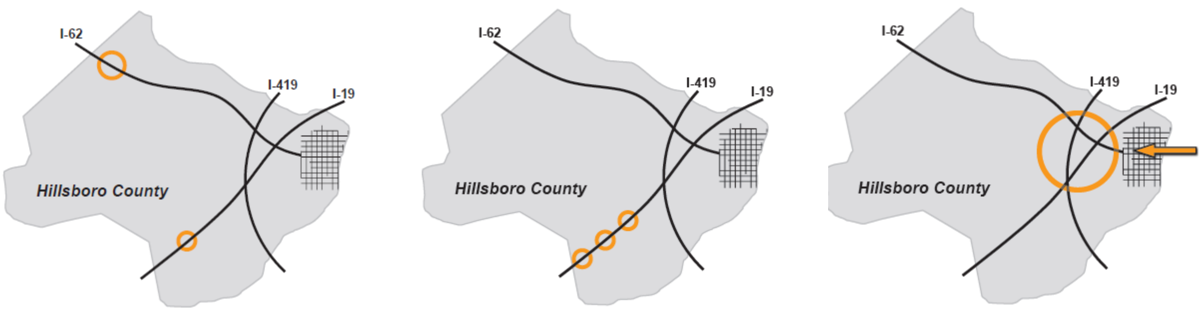
2.4.2 WORK ZONE NETWORK FORM

Network configuration is a factor that must be accounted for when using work zone analysis tools. Network form is a substitute for the entire complexity of the roadway where the work zone impacts will be evaluated; these complexities are independent from FHWA work zone types.

Isolated: network consists of a single work zone that has limited interaction with the surrounding facilities, as seen in Figure 2–2(a). Due to the limited interaction the work zone impacts on the immediate surroundings might be only dependent on the size and duration of the project as well as the traffic demand of the facility.

Pipe: network consists of roadway segments with two or more work zones interacting with each other while including some limited access points and connections with other facilities, as seen in Figure 2–2(b).

Grid: network is a connected, inter-dependent structure with multiple access points and one or more viable alternate routes, as seen in Figure 2–2(c). Grid network examples include urban interstate reconstruction, reconstructions including full roadway closures, signalized arterial roadway reconstruction, and work zones located in urban centers (Hardy and Wunderlich 2009).



(a) Isolated (b) Pipe (c) Grid

Figure 2–2: Types of Work Zone Network Form
(Hardy and Wunderlich 2009)

2.5 SIMULATION MODELS

Traffic and transportation analysis tools have developed immensely over the last few decades with different approaches and focus areas. Each of these types of models have advantages and limitations due to the purpose the tool was designed to fulfill.

Sketch-Planning Tools and Deterministic Tools are described as specialized models designed for specific tasks or applications, such as work zone assessment or ITS implementation analysis. Sketch-Planning tools include a wide range of interfaces such as spreadsheets developed by DOTs designed to address a specific project or simulate specific conditions, to more generalized delay estimation tools like QUEWZ-92 and QuickZone. Most of these tools follow volume-capacity methodologies included in the Highway Capacity Manual (HCM).

Macroscopic simulation models are described in the FHWA Traffic Analysis Toolbox report series as based upon the deterministic relationship of the flow, speed, and density of the traffic stream. A macroscopic model runs on a section-by-section basis instead of tracking individual vehicles by treating traffic flow as an aggregated quantity. Macroscopic models possess the ability to cover large geographical areas; this feature becomes useful when work zone impacts affect areas that are larger than a corridor or region where geographic characteristics must be taken into account to better understand the effects. The main limitation of these models is the simplistic representation of traffic movement which limits the fidelity of the results.

Mesoscopic simulation models are deemed to be the newest generation of traffic modeling simulation software tailored for an intermediate level of analysis. These models provide a greater level of detail when compared to macroscopic models but not as much reliability as microscopic simulation models. Mesoscopic models have the tendency to represent the relative flow of vehicles on network link, but fail to represent individual lanes on the link. When analyzing work zone impacts, these tools have the capability to model both large geographic areas and corridors, as well as diversion routes and signalized intersections. Some limitations of this type of model include its limited ability to model detailed operational strategies, overall model complexity and data requirements necessary for accurate results.

Microscopic simulation models were developed to more accurately depict transportation systems at the individual vehicle level. Microscopic simulation takes into account car-following and lane-changing theories. These models update the position and intentions of individual vehicles every second as they move through a network. Microscopic simulation accounts for vehicular diversity and driving styles that are found in real world traffic conditions. This allows the simulation to account for the manner that driver respond to the presence of other vehicles and traffic control devices. The main limitation of microscopic modeling is the abundant amount of geometric, traffic control and traffic patterns data they usually require (Hardy and Wunderlich 2008).

2.6 TRAFFIC SIMULATION TOOLS WITH WORK ZONE APPLICATIONS

The freeway evaluation tool (FREEWAL-WZ) is a work zone specific traffic simulation tool developed by researchers at the North Carolina State University in 2015. This tool's methodology is based on the sixth edition of the HCM and it is able to evaluate demand changes on ramps and weaving segments in addition to freeways. FREEVAL-WZ is able to evaluate segment level of service (LOS), vehicle delay, travel time, speeds, average queue length, longest queue length, and queue duration when evaluating work zone adjustments along freeways. This tool allows the user to simulate different lane closure configurations (scenarios) at once; while computing the different measures of effectiveness previously mentioned (Trasks et al. 2015).

The Queue and User Cost Evaluation of Work Zones (QUEWZ – 98) is a tool for evaluating freeway work zone lane closures. It was developed in 1993 by TTI using the 1985 HCM estimation procedures for queues and speeds. The latest version is QUEWZ-98, based on the 1985 HCM, and is able to compare traffic flow through a freeway segment with and without a work zone lane closure and is also able to estimate the changes in traffic flow characteristics and road users costs that result from the presence of a work zone (Copeland 1998).

QuickZone: this traffic analysis tool is a spreadsheet-based instrument able to compare traffic impact work zone mitigation strategies. QuickZone is able to estimate user's costs, traffic delays, and potential queues associated with the presence of a work zone. This tool can be used by state and local officials as well as construction, operation, and planning staff to identify the effect that different work zone have on motorist delays and costs (Federal Highway Administration 2017).

VISSIM: is a microscopic simulation tool with the ability to produce detailed outputs featuring down to the second and individual vehicle accuracy. VISSIM's priority rules even allow for a "friendlier" merging driver behavior setting (Gettman and Head 2003). VISSIM allows the user to customize models and default settings to the user's needs; in this case VISSIM does not include a work zone or lane merging "button/function", nonetheless these scenarios' representation can be simply mimicked due to the software's flexibility (PTV Group 2017).

Figure 2–3 has the purpose of illustrating the wide variety of transportation simulation tools, specific uses and capabilities. Ability of model customization plays an important role when selecting the software as well as the specificity of the conditions that are being modeled.

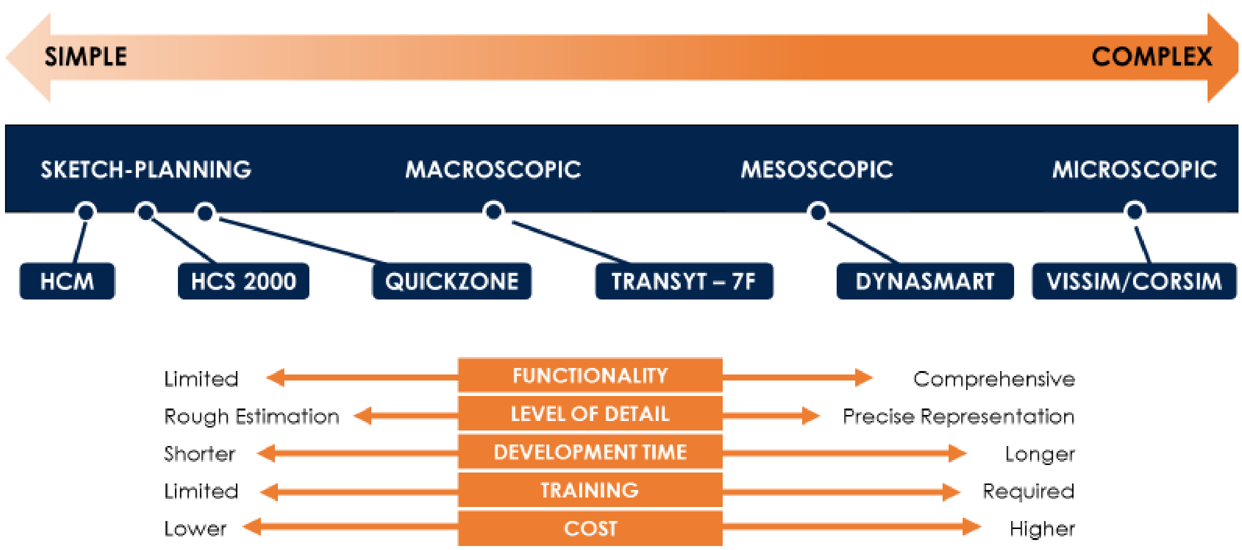


Figure 2–3: Traffic Simulation Software Capabilities Spectrum

2.7 SIMULATION ADVANTAGES

Simulation models prior to full strategic deployment can serve as an advantage for practitioners to better refine the modeling of custom traffic management treatments. More specifically these tools can aid practitioners in the following aspects:

- Improve the decision-making process; these tools are used to measure the impact of suggested changes and strategies, allowing practitioners to evaluate to then prioritize improvements and alternatives.
- Project potential future traffic; simulation software have the capability to project increased traffic volumes in order to analyze and evaluate potential drawbacks in future conditions.
- Evaluate and prioritize planning and operational alternatives; allowing the evaluation of potential impacts of "build" and "no-build" scenarios to better prioritize costs and safety benefits.
- Improve design and evaluation time and costs; traffic simulation tools are readily available and relatively less costly for practitioners than field experiments.
- Operate and manage existing roadway capacity. Several simulation tools are able to provide optimization capabilities under current conditions analysis.
- Monitor performance; these tools are able to monitor efficiency of facilities under current conditions; allowing practitioners and engineers to better document and analyze performance (Dowling et al. 2004).

2.8 USE OF TRAFFIC SIMULATION MODELS IN FREEWAY WORK ZONES

Researchers and practitioners have used VISSIM as a simulation tool in order to evaluate safety effects or test capacity improvements. Studies such as 'Exploring Traffic End Of Queue In A Temporal Spatial Diagram And Applications For Work Zone Safety Analysis', explore the ability of using speed based contour maps to obtain queueing information using VISSIM as a simulation tool (Chou and Nichols 2012).

The purpose of that study was to explore time and space information of queued conditions, and proposes the application of a ready to use safety index to estimate the occurrence of crashes in work zones during the planning stage. The work zone condition in the model was approached by using the function of a ‘Parking event’, length and duration were customized in order to represent a lane closure work zone layout; this approach is basically multiple stationary vehicles parked on a freeway in the interest of modeling the blocked lane. ‘Reduced speed area’ was used to simulate the rubbernecking effect that drivers typically display while driving through a work zone. End of queue conditions were inspected, taking into consideration vehicle trajectories in space and time diagrams speed contours maps where then developed. Researchers used discontinuity of speed contour maps to locate queue formation and then moved to derive and end of queue safety index for evaluating the safety impacts of work zones. A number of characteristics were taken into account to identify queued characteristics using the speed contour maps; average speed, volume, time and the vehicle’s position in space.

This study explored a total 17 of work zone scenarios with three different speed reductions, and five speed difference thresholds for queue determination; resulting in 255 total scenarios when accounting for different volume combinations as well, with the purpose of exploring the best speed threshold values. The results of the experimental iteration pointed towards a 50% speed reduction and a 10 mph speed difference threshold values using a 1,800 vehicle per lane per hour (vplph). These results were used as inputs in the simulation model to obtain time-space based on the results of a macroscopic speed contour map. The same time-space map was obtained through end of queue vehicle projectile trajectories; where it was found that parallel and similar results could be obtained through the two different methods. Proving that it is feasible to use a macroscopic speed contour map as a substitute for a microscopic vehicle trajectory diagram in identifying queue information for work zone management. The experimental design for this study was based on a conceptual network with a two-lane one directional freeway segment with an average speed of 65 mph and with a work zone blocking one lane for 4 hours in a 24-hour simulation period. Combinations of freeway volume inputs and different increments during peak hour periods led to seven volume levels scenarios. Obtaining 840 total simulation runs that include of 24 hours at 7 different volume levels tested with 5 different number random seeds. The results of the different random seeds were then averaged and analyzed. Queue observation space lengths, speed and end of queue safety index were plotted against hours of the day, creating different queue lengths based on different volumes. The rise of a contour line on the y-axis is interpreted as a rise in queue length, speed or safety index. All of these different combinations of queued characteristics versus time aim to identify identifying the best times to schedule work zone active hours depending on traffic volume levels (Chou and Nichols 2012).

Similar simulation model configuration can be found in other study where reduced speed areas and parking lot decisions where introduced in order to simulate lane blockage as well as the rubbernecking effect on drivers. This simulation study relied was more focused on incident management strategies for freeway operations, nonetheless valuable simulation configurations were learned from these approaches (Chou and Miller-Hooks 2011).

In another study, researchers used the traffic simulation software CORSIM to explore and evaluate the operational impacts of different merge control strategies in freeway work zone environments. This study utilizes incidents within CORSIM to simulate a lane blockage, as well as the simulation of a rubbernecking effect on 20% of its drivers to simulate merge control behavior. This study proposes capacity-derived measures, such as density, in order to evaluate and determine best use of capacity in freeway work zones (Ramadan and Sisiopiku 2016).

2.9 SUMMARY

Information provided by case studies illustrate current practices applied and recommendations made by Texas and Illinois DOTs. Documentation of advantages and limitations of trial QWS deployments help serve ALDOT in developing standard drawings and set forth the implementation of new standardize practices. This would also allow ALDOT to provide beneficial and detailed information regarding QWS deployment to vendors for a more efficient understanding of available software and equipment requirements.

Simulation tools described in this section help illustrate the wide spectrum of complexity and numerous capabilities of simulation software. Traffic simulation studies included in this section describe different approaches and guidelines to model specific freeway work zone environments; these also provide insightful information in regards to calibration and driver behavior features that can be applied to better replicate field conditions.

CHAPTER THREE: METHODOLOGY

3.1 INTRODUCTION

This chapter is intended to describe the logical processes that lead to the results, conclusions, and recommendations. This chapter describes the data collection measurements and plan, quality control, and screening process of field-gathered data, traffic simulation model establishment and calibration, determination of number of repetitions for validation. This chapter also includes a set of proposed treatment scenarios that simulate the effect of a QWS under current traffic conditions.

3.2 DATA COLLECTION PLAN

The data collection plan was based on the sensor layout for a QWS deployed and evaluated in Texas (Ullman et al. 2016). TTI's researchers deployed QWS roadside sensors at 0.5, 1.5, 2.5 and 3.5 miles upstream of the work zone lane closure on I-35 in Texas. In the field data collection effort for the current study, sensors were placed at the same distances upstream of the lane closure. The equipment used for this research acts only as a data collector but not as a part of a deployed QWS, meaning that there is no communication between PCMSs and the speed data collectors. The sensor type used in this study was the NC-350, which is a portable traffic analyzer manufactured by M.H. Corbin LLC, and is able to collect single lane speed and vehicle length data continuously for up to 21 days.

Figure 3–1: NC-350 Portable Traffic Analyzer
(MH Corbin LLC 2016)

The NC-350, shown in Figure 3–1, uses vehicle magnetic imaging technology to detect and collect vehicle count and speeds. For the purpose of this research and in order to collect data on both lanes, eight data collectors were deployed upstream of the lane closure taper, as shown in Figure 3–2. Four pairs of NC-350 data collectors were deployed every mile until half mile prior to the lane closure. In addition to replicating TTI's data collection layout, a ninth collector was deployed after the lane closure taper in order to collect data in the open lane. Identification numbers that pertain to a specific data collector are shown in Figure 3–2, for the purpose of brevity the following definitions are introduced and used for the remaining of this study: data collectors at the 3.5 mile location upstream from the work zone are referred to as sensors 100 and 96 for right and left lanes respectively, data collectors located at 0.5 mile upstream from the work zone are referred to as sensors 103 and 99 for right and left lane respectively, and the data collector located 300 feet after the lane closure on the right lane will be referred to as sensor 104.

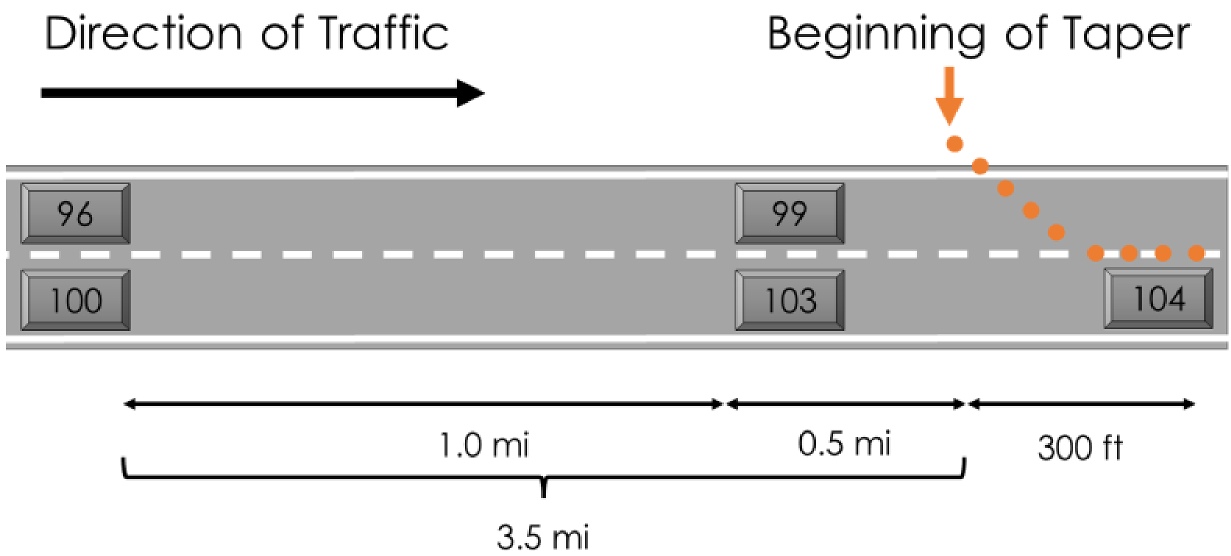
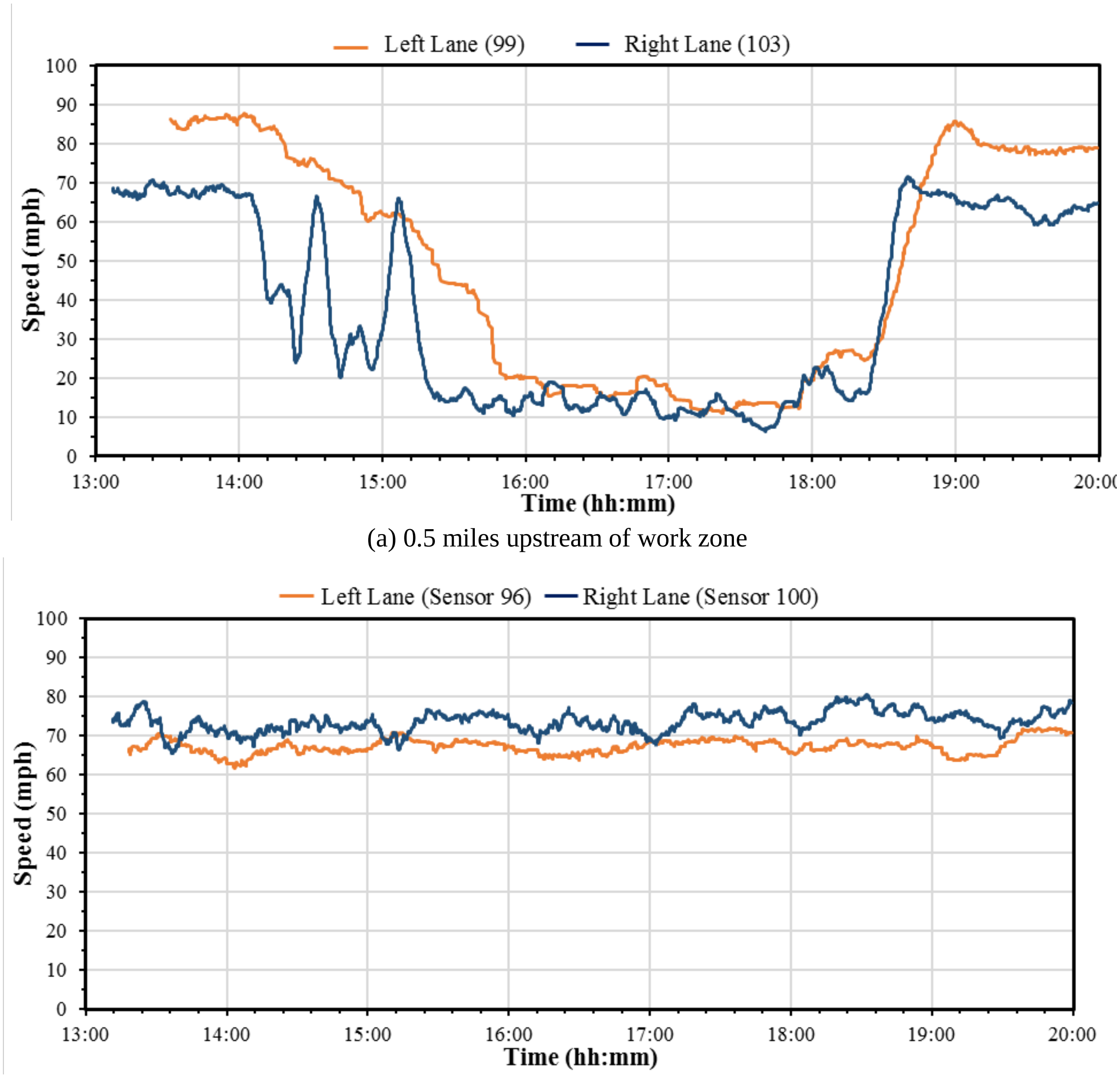


Figure 3–2: Data Collection Layout of Sensor Deployment

Data collected from the first set of NC-350s encountered by traffic, 96 and 100, were used to determine the inputs of the traffic simulation model; this included vehicle speed distribution, lengths and volumes. The traffic simulation model used for the purposes of this research was VISSIM, due to its customization capabilities and flexibility to model specific case conditions such as lane closures. Speed distribution data collected by sensors 99 and 103, located at half mile prior to the lane closure, was used for model calibration purposes. In addition to the calibration process, speed distribution data collected by sensor 104 with the purpose of controlling the throughput traffic speeds to simulate queued conditions. The following section describes the data screening process applied to data from sensors 96, 99, 100 and 103.

3.3 DATA SCREENING

The data used in this study was collected on October 3rd, 2016 for a time window of seven hours. The time window analyzed starts at 1 pm and ends at 8 pm, a graphical description of vehicle speeds 0.5 miles upstream of the work zone taper can be observed in Figure 3–3 (a). Speed behavior, collected by sensors 99 and 103, describe pre-and post-queue formation speed behavior which for simulation modeling purposes, including one hour prior to queue formation and after queue dissipation is a common practice.



(a) 0.5 miles upstream of work zone

(b) 3.5 miles upstream of work zone

Figure 3–3: Speed Vs. Time Upstream of Lane Closure.

Sensors 96 and 100, located 3.5 miles prior to the lane closure on I-59; collected speed and vehicle lengths that are expected in freeway traffic, speeds generally within the range of 60 mph to 80 mph prior to the lane closure. The purpose of including a thorough data screening and quality control method is to obtain the most accurate representation of real freeway traffic under work zone conditions to be introduced into VISSIM. Sensors 96 and 100 depict high speeds and no indication of queue spillover at the 3.5 mile mark prior to the lane closure, shown in Figure 3–3 (b). Speed behavior at the 3.5 mile location for the right lane is noticeably higher than those collected in the left lane; however this behavior is overturned at the 0.5 mile location having higher speed behavior in the left lane when compared to the right lane. Speed records collected at sensors 99 and 103 after 2 pm (shown as 14:00 in the figure) start to drastically decline, indicating that vehicles are slowing down due to the effect of the lane closure on freeway capacity and resulting in queue formation.

Data collected on October 3rd, 2016 was determined to be representative, as the remainder of the raw data collected on the following days was compared and found to have similar speed, volume and heavy vehicle presence (25%) trends. The seven-hour analysis period was then fragmented into 5-minute interval bins, resulting in 84 time intervals.

Original traffic volumes for the four sensors corresponding to the 1:00 PM – 8:00 PM time window are included in Table 3-1. Sensors in the right lane of the roadway (100 and 103) show significantly greater volumes than those in the left lane (96 and 99).

Table 3-1: Original Traffic Volumes Collected from 1:00 PM – 8:00 PM Period on 10/03/2017

Sensor	Left Lane (96)	Right Lane (100)	Left Lane (99)	Right Lane (103)
Volume	2,084	2,981	1,422	4,340

Is expected for the right travel lane to be more occupied in freeway traffic than the left lane, since vehicles are supposed to drive on the left lane for passing maneuvers only. Another factor that could influence this split is the presence of a merging ramp half mile prior to the first set of sensors. The on-ramp at Exit 68 (Joe Mallisham Parkway) is located on the right side of the freeway and its location is believed to contribute to the right lane higher vehicular volumes; sensors 96 and 100 were 3.8 miles after the ramp.

Figure 3–4: Location of Data Collectors 96 and 100. (Google 2017)

3.3.1 Phase One: Threshold Value Data Screening

Unprocessed data records showed speed and length records that are deemed erroneous and unrealistic. Speeds over 100 mph were not believed to be possible given the geometry of the site as well as the presence of active work zone components such as heavy machinery and crews at work. Another indication that supports the elimination of speeds over 100 mph was the fact that taking a contextual perspective when analyzing the data, speeds over 100 mph would follow and be followed by speeds well below 100 mph. Original data showed time headways to be too small for such a speed differential to be possible.

The first step towards the data screening process was to eliminate outliers. Lengths of vehicles over 100 feet were determined to be unrealistic given that according to FHWA, the average length of the longest vehicle classification is of 73.1 feet with a standard deviation of 4.37 feet (Jessberger 2011). For the purposes of this research, vehicles exceeding 100 feet in length were removed.

For all four data collectors (96, 99, 100 and 103) values of speed and length that were above the 100 mph or 100 feet value were prematurely discarded due to the geometric conditions of the site and surrounding data points; these are likely erroneous readings by the data collectors. This first stage of screening was responsible for eliminating the following percentages of records per sensor.

Table 3-2: Percentage of Records Discarded Due to Values Over 100 feet and 100 mph

Sensor	**Left Lane (96)**	**Right Lane (100)**	**Left Lane (99)**	**Right Lane (103)**
Volume	2,029	2,832	1,349	4,199
Percent Reduction	2.64%	5.00%	5.13%	3.25%

Vehicle length data was used in order to classify passenger vehicles (PV) and heavy vehicles (HV). Data from sensors 96 and 100 were used to develop a frequency distribution based on vehicle length in order to determine the length that would govern these two classifications. When plotted, two distributions were observed. The first curve would correspond to the PV classification with a peak center at 16 and 22 feet for left lane and right lane respectively. The second curve peaked at the 83 feet for the right lane and 64 feet for the left lane. The length chosen to be the decisive parameter between is shown in Figure 3–5, where a frequency distribution was graphed and the lowest frequency between the peaks was found to be approximately 53 feet. Data points with a length of less than 53 feet were classified as a PV and any data point with a length over 53 feet, was considered as a HV.

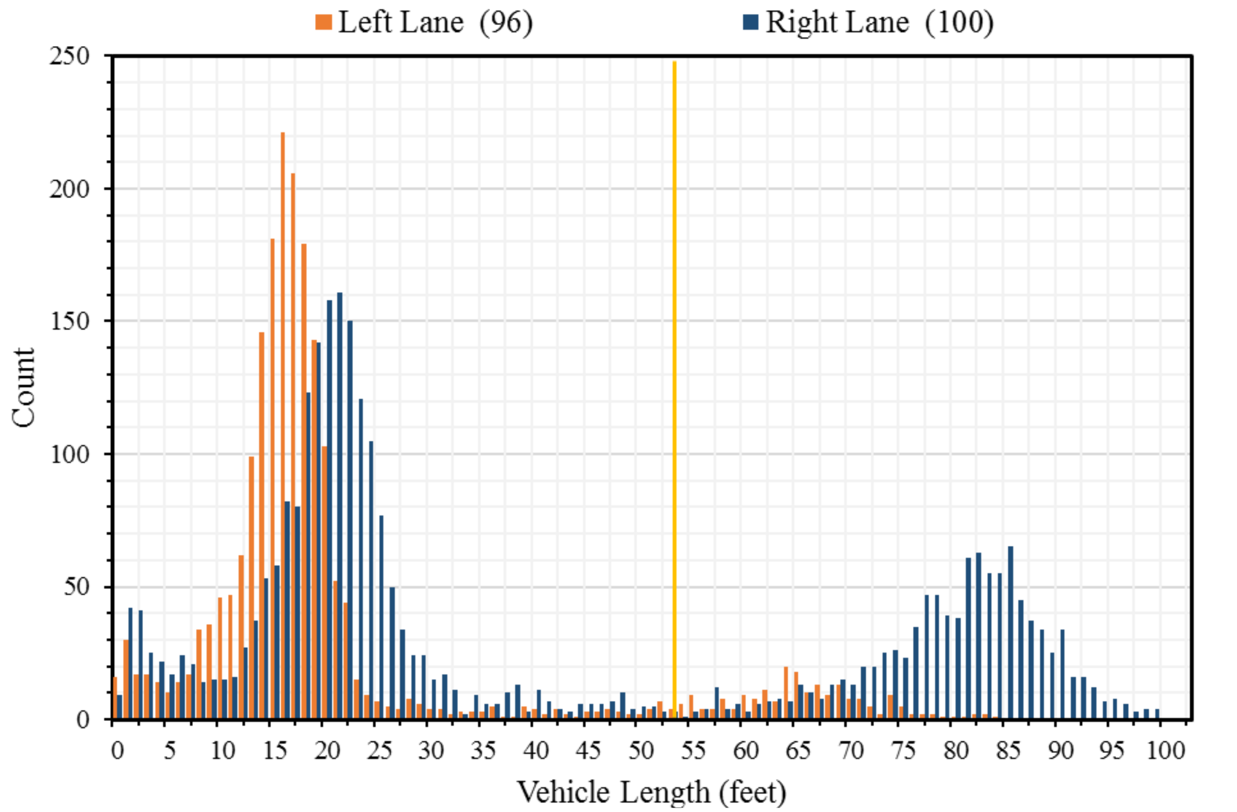


Figure 3–5: Vehicle Length Frequency Distribution (Sensors 96 & 100).

The purpose of developing at least two vehicle classifications was to study speed distributions for PV and HV separately. This would allow maximizing the capabilities of VISSIM to simulate realistic conditions in which heavy vehicle drivers have a different driving behavior than the rest traveling public. Heavy vehicles comprise 24.5% and passenger vehicles represent 75.5% of the traffic during the analysis time window.

Given that average length of passenger vehicles fall well below 53 feet, vehicular records that are within the 45 to 55 feet length range represent only 2% of the data. It is believed that modifying the value of the length threshold would have not impacted substantially the final results obtained from the simulation model. It is important to notice that the FHWA does not use established values of length to serve vehicle classification, number of axles and axle spacing remains the indicative factor for classification. The sensor used in this study was unable to detect axle number or axle spacing, just as pressurized pneumatic tubes would; it was only capable of measuring vehicle speeds.

3.3.2 Phase Two: Statistical Quality Control

The second step of the data screening process was shaped by a quality control method based on standard deviation. The seven hours selected for analysis were then divided into 5-minute intervals resulting in 84 time bins for each sensor. All vehicle records were categorized as PV or HV based on their vehicle length, each category was analyzed separately for each sensor. Speed average and standard deviations were calculated for the 5-min intervals independently; the purpose of this process was to eliminate outliers that would not be within the two-standard deviation distance from the 5-min interval mean.

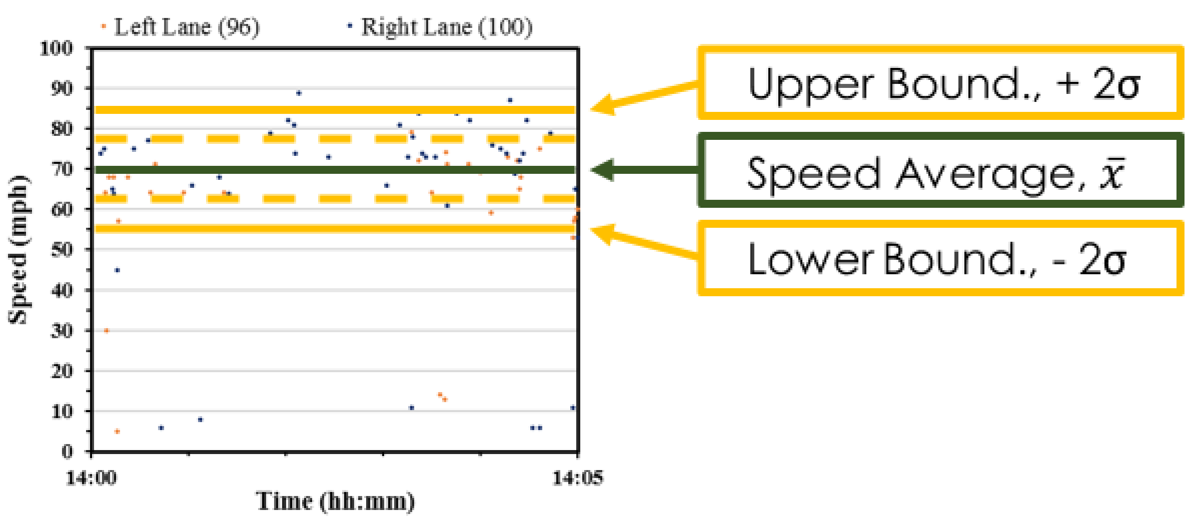


Figure 3–6: Graphical Sample of Methodology Used in Phase Two.

Figure 3–6 shows the upper and lower boundaries based of statistical measures used in for 5-min bins. Standard deviation serves as an indication of the dispersion of the speed records within the interval. Given that the speeds approximately follow a normal distribution, the two standard deviation distance would contain about 95% of the observations and discard outliers. The first two-standard deviation test was found to discard 987 records; percentages of records eliminated are shown in Table 3-3.

Table 3-3: Percentage of Records Discarded After Data Screening

Sensor	**INPUT (3.5 mi)**		**CALIBRATION (0.5 mi)**	
	Left Lane (96)	**Right Lane (100)**	**Left Lane (99)**	**Right Lane (103)**
Volume Records	1,887	2,677	1,253	4,023
Phase One[1]	7.00%	5.47%	7.12%	4.19%
Phase Two	9.45%	10.20%	11.88%	7.30%
Final Volume Records	1,708	2,404	-	-

1. Percentage of records discarded after the vehicles with speeds over 100 mph or lengths with over 100 feet were removed

New 5-min interval based averages were calculated after outliers were discarded; these would then serve as a component of the VISSIM inputs. The two-step screening process managed to discard a total of 9.12%.

3.4 VISSIM MODEL PREPARATION – CONTROL CASE

The creation of the VISSIM model requires both geometric and traffic inputs as components. This section is intended to cover the speed distribution and volume inputs that belong to the 1 pm – 8 pm analysis window. This section also includes the geometric layout, since VISSIM is not currently capable of simulating work zone conditions or a lane closure with the touch of a button; the software has to be ‘tricked’ and manually introduced to a lane drop as well as queued conditions.

3.4.1 VOLUME INPUTS AND DESIRED SPEED DISTRIBUTION

Volume inputs were obtained using the screened data from sensors 96 and 100. Two sets of vehicle compositions were developed, PV and HV. In order to introduce volumes and vehicle composition in VISSIM, four aspects have to be determined: Vehicle Type, Relative Flow, Volume, and Desired Speed Distribution.

'Vehicle Type' refers to a set of vehicles with the same technical driving characteristics. Passenger vehicles in VISSIM are also referred as vehicle type "100: Car" and heavy vehicles are known as "200: HGV"; these vehicle types were used for the control scenario. VISSIM allows the user to customize and develop a certain vehicle type with a driving behavior or speed distribution different from defaults. For the purposes of the control case, no customized vehicle types were used in this scenario.

'Relative Flow' describes the share of each vehicle type in the traffic stream. For the purposes of the control scenario, the PV and HV relative flow proportional volumes were introduced as 75.5% and 24.5% respectively. These values are a product of the screened data obtained from the quality control process.

'Volume' is described as the number of vehicles per hour. In VISSIM, a single volume input at the beginning of the simulation period would periodically discharge input vehicles every hour at random pace. Since the data shows the volume records to change significantly due to queue formation, 5-min time intervals were created in VISSIM, in order to control volume input and to better simulate volume inputs. These volumes were obtained from the unprocessed data from 96 and 100; regardless of erroneous records of speed and vehicle length, the deployed sensors still recorded a vehicle. Another important aspect to consider when handling input of volumes in VISSIM is the unit in which the input is being interpreted as. Since volumes were obtained in 300 second intervals, each input had to be multiplied by a factor of 12 given that there are 12 five-minute intervals in one hour. To better illustrate this process Table 3-4 describes the translation process for the first hour of simulation; the volumes for the six hours remaining followed the same process and can be found in Appendix C. Volume inputs were lane specific, therefore the resulting factored volumes of sensors 96 and 100 were introduced as left and right lane volumes respectively.

Table 3-4: Sample Volume and VISSIM Inputs from 1:00 PM - 2:00 PM

Time Interval	Original Volume (96)	Volume vph (96)	Original Volume (100)	Volume vph (100)
13:00 - 13:05	36	432	42	504
13:05 - 13:10	23	276	42	504
13:10 - 13:15	24	288	37	444
13:15 - 13:20	26	312	30	360
13:20 - 13:25	17	204	37	444
13:25 - 13:30	30	360	41	492
13:30 - 13:35	28	336	36	432
13:35 - 13:40	30	360	47	564
13:40 - 13:45	38	456	39	468
13:45 - 13:50	26	312	47	564
13:50 - 13:55	21	252	40	480
13:55 - 14:00	32	384	40	480

Desired speed distribution (DSD) is defined in VISSIM using a cumulative distribution frequency (CDF) curve. To achieve curvature, intermediate points can be added, the points included are shown in red in Figure 3–8. Since vehicles were analyzed separately according to a length parameter that would designate them as PV or HV, two DSD were created in VISSIM. As shown in Figure 3–5, screened speed data for both input sensors, 96 and 100, were combined and a HV speed CDF was produced. The same process was followed with the PV data.

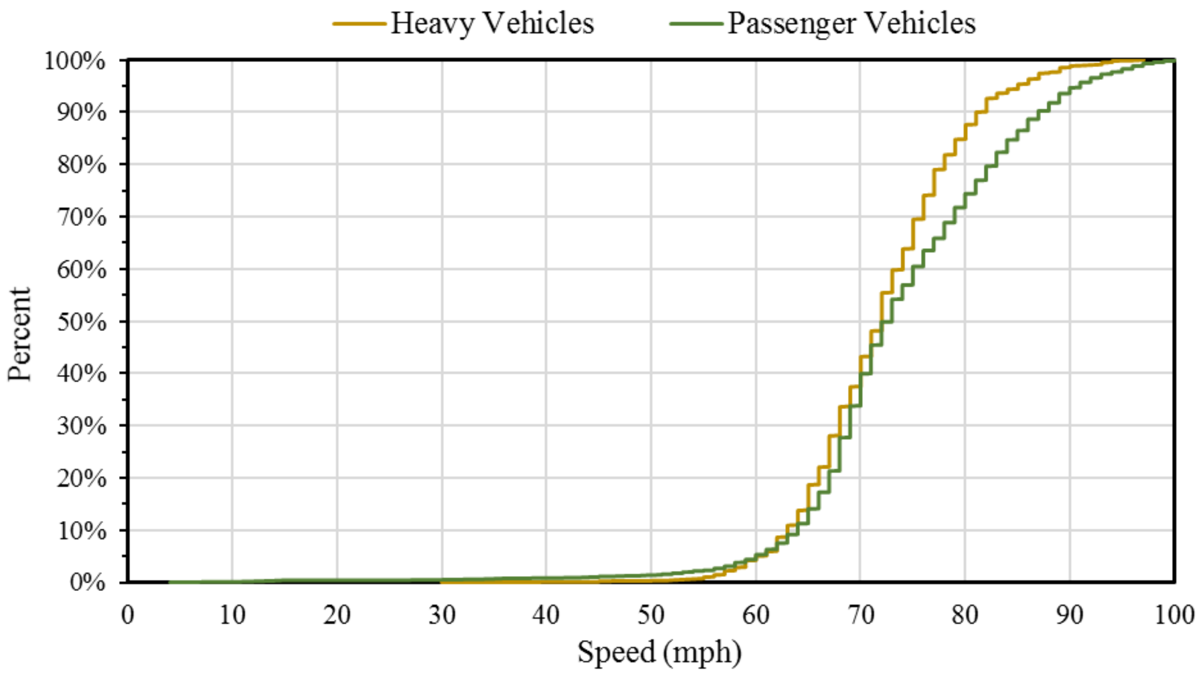
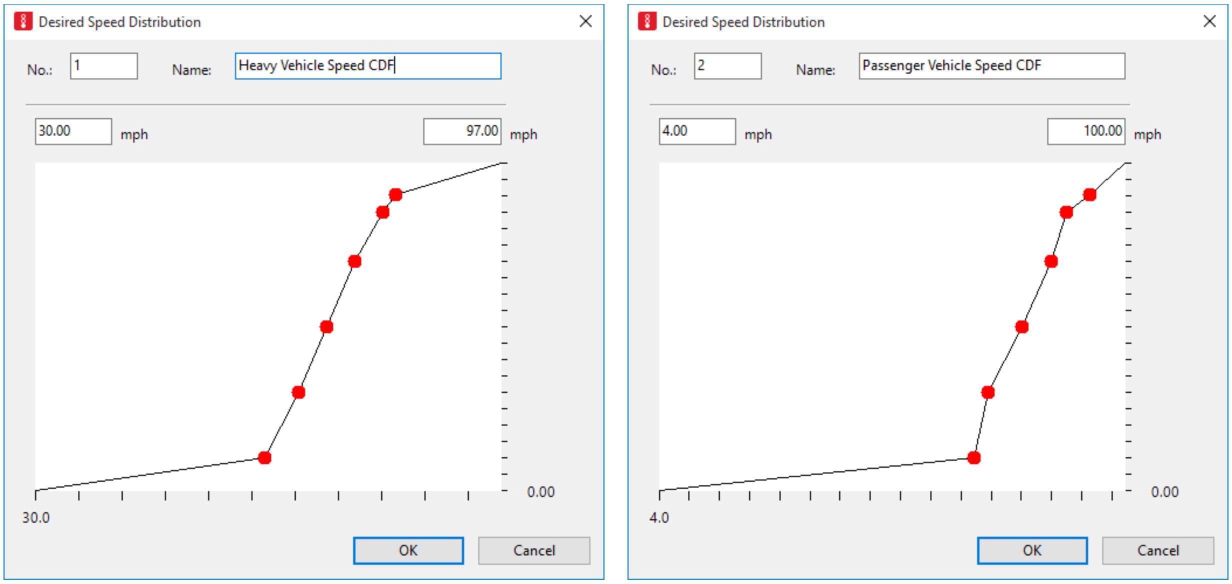


Figure 3–7: Cumulative Speed Distribution.

Speed distribution points for the CDF were taken at the following percentiles: 0th, 10th, 30th, 50th, 70th, 85th, 90th and 100th, speed values can be observed in Table 3-5. The DSD for HV and the seven points defining the cumulative distribution can be observed in Figure 3–8(a).

Table 3-5: Cumulative Speed Distributions Inputs

Percentile	Speed (mph)					
	Sensor 96		Sensor 100		Compiled	
	PV	HV	PV	HV	PV	HV
10%	61	60	69	66	64	63
30%	67	63	75	70	69	68
50%	69	66	80	74	73	72
70%	71	69	85	77	79	76
85%	75	72	89	80	88	80
90%	77	74	91	82	87	82



(a) heavy vehicle (b) passenger vehicle

Figure 3–8: Cumulative Speed Distribution Frequency, VISSIM Interface.

The same process was applied to the data corresponding to the passenger vehicle classification, observed in Figure 3–8(b). Desired speed distribution between HV and PV followed generally the same trend; one major difference was the minimum speed of the HV being at 30 mph and 4 mph for the PV.

3.5 MODEL GEOMETRY AND WORK ZONE SET UP

The first step in creating the geometry of the model was to locate in VISSIM the actual site with the use of embedded maps. After the analysis segment of I-59 was located, a 3.5-mile two lane long link was created. At the beginning of the link, input volumes and static route decisions were placed and at the end of the 3.5-mile long link, the beginning of the work zone lane closure can be found.

'Static Vehicle Routes' are defined as the ability for the user to specify a routing decision for a link and one or multiple destination links. By using this tool, the user is able to customize the routing decisions and use of lanes and connectors throughout the model.

VISSIM's ability to customize case specific conditions are valuable since the software does not readily contain a work zone package; for the purposes of this project the lane closure conditions had to be manually arranged. Prior to the lane drop the two open lane link spans for 3.5 miles where the geometry begins. At the merging point, a new one-lane link is introduced in the right lane since this lane remained open for the analysis period. Connector links, shown in Figure 3–9, where arranged to connect each of the right and left lanes opened in the two-lane link separately with the active lane after the closure took place.

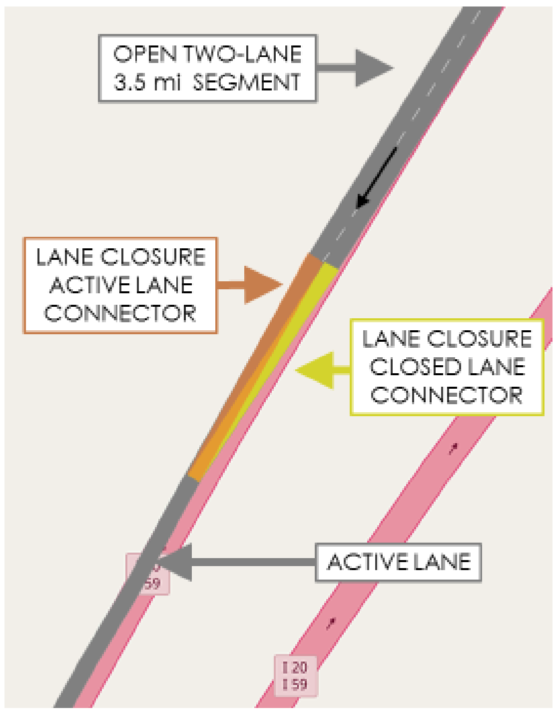


Figure 3–9: VISSIM Lane Closure Geometric Configuration.

Two sets of static routes had to be used in this case, to assure that both of the lane merging connectors were being used. Both static routes started at the beginning of the 3.5-mile long link and both ended at the active lane after the lane closure had taken place. As observed in in Figure 3–10, the difference lays in the lane merging connector link use; static route decision can only function by following a chain of links, but since the merging segment is described by two separate and parallel connectors, two static routes have to be placed.

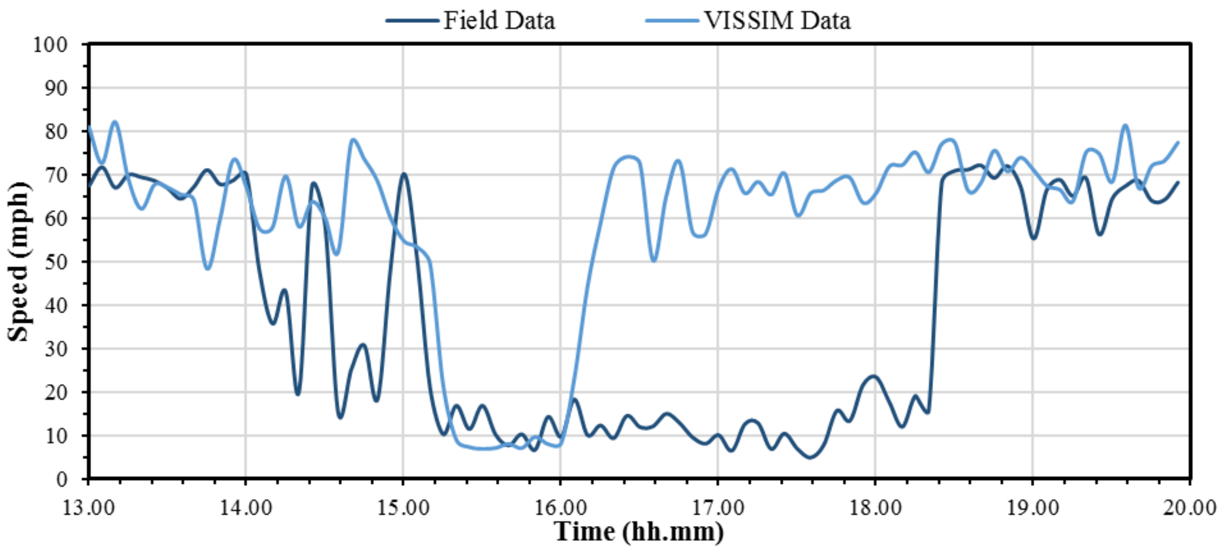
(a) Right Lane (b) Left Lane

Figure 3–10: Static Route Decision – VISSIM Merging Segment.

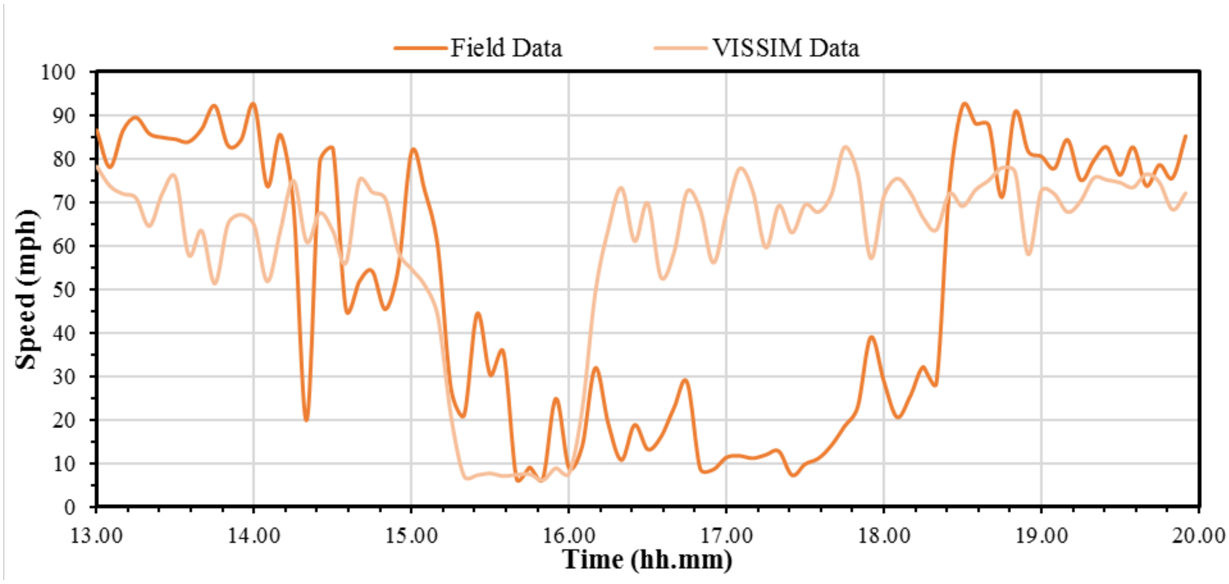
'Data Collection Points' can be used for monitoring the simulated number of vehicles, the attributes being evaluated for the purpose of this project is average speed for every 300 seconds interval. A pair of data collection points was placed at 0.5-mile before the beginning of the lane closure taper, where sensors 99 and 103 were located; this would allow to better measure the effects of the lane closure and keep record of the speed reduction due to queued conditions.

3.5.1 Disparity between Field Data and Model Results

After having introduced the major inputs, simulation run trials were conducted in order to test the model. When evaluating the speed averages at the 0.5-mile location prior to the lane closure, no significant queues or speed reductions were observed. Queued conditions in this case refers to the significant decrease in average speeds between 5 mph and 20 mph. Speed range for the trial set of runs varied from 65 mph to 85 mph, meaning that the vehicles were following the speed distribution that was assigned to them but the queued conditions were not present for the same amount of time in which the data specified. Field data shows and VISSIM model results show similar speed variations for the first hour and the last hour and a half of the study period. Nonetheless, queued conditions were not being represented for as long as the field data showed. Results obtained from the first attempt at the simulation model at the 0.5 mile location can be observed in the Figure 3–11 compared to the screened real data from each of the sensors. Queued conditions in the field began to appear approximately at 2:15 pm and started to dissipate around 6:25 pm. In the first VISSIM simulation model, queued conditions appear at 3:15 pm and dissipate less than an hour later. These observations are true and common for both right and left lanes. These preliminary results set forth the first stepping-stone towards a calibration process using this set of conditions as baseline conditions prior to calibration.



(a) right lane data: field vs. simulation



(b) left lane data: field vs. simulation

Figure 3–11: Set of VISSIM Model Run Results Prior to Calibration at 0.5 mi.

3.6 CALIBRATION

This section describes the calibration process; it is based on the field gathered data from sensors 99 and 103 located 0.5 mile prior to the lane closure. This section aims to illustrate the iterative process of calibrating the traffic simulation model to the field data within a specific freeway work zone with lane closure conditions.

3.6.1 PLACEMENT OF WORK ZONE 'PACERS'

It was determined that in order to recreate "queued conditions", a form of speed distribution had to control the speeds within the active lane of the work zone. Sensor 104 was deployed approximately 300 feet after the lane closure had already taken place. Data from this sensor underwent the same quality control and screening method as sensors 96, 99, 100 and 103. Data from sensor 104 was not divided into 5-minute bins; instead, 15-minute bins were used to determine distributions that would 'pace' oncoming traffic. Initially speed distributions of 60-min bins were used to reduce traffic to simulate queued conditions, nonetheless the timing of the queue formation and dissipation was not accurate when compared to the field data. A total of 28 (four intervals per hour and seven hours of analysis) speed cumulative distributions were calculated from the raw data and then introduced using the 'Desired Speed Decisions' function in VISSIM at the beginning of the active lane (immediately after the merging segment).

Desired speed decisions can be placed anywhere on a link and the user is able to customize a starting time and a set duration for vehicles to follow a specific speed distribution. The 'work zone speed' distribution was placed immediately before the beginning of the merging taper that closes the left lane. Figure 3–12 shows the 28 speed CDF bins that simulate queued conditions similar to those observed in the field. These desired speed decision thresholds that were put in place to better simulate work zone queued conditions will be referred to as pacers for the purposes of this project.

No	Name	LowerBound	UpperBound
1	TRUCKS COMPILED	30.00	97.00
2	CARS COMPILED	4.00	100.00
1300	13:00 - 13:15	15.00	71.00
1315	13:15 - 13:30	17.00	71.00
1330	13:30 - 13:45	16.00	72.00
1345	13:45 - 14:00	12.00	71.00
1400	14:00 - 14:15	2.00	46.00
1415	14:15 - 14:30	3.00	46.00
1430	14:30 - 14:45	2.00	47.00
1445	14:45 - 15:00	2.00	41.00
1500	15:00 - 15:15	1.00	36.00
1515	15:15 - 15:30	1.00	35.00
1530	15:30 - 15:45	1.00	36.00
1545	15:45 - 16:00	1.00	34.00
1600	16:00 - 16:15	1.00	34.00
1615	16:15 - 16:30	1.00	40.00
1630	16:30 - 16:45	2.00	33.00
1645	16:45 - 17:00	1.00	39.00
1700	17:00 - 17:15	1.00	42.00
1715	17:15 - 17:30	1.00	33.00
1730	17:30 - 17:45	1.00	35.00
1745	17:45 - 18:00	1.00	42.00
1800	18:00 - 18:15	7.00	63.00
1815	18:15 - 18:30	6.00	64.00
1830	18:30 - 18:45	6.00	65.00
1845	18:45 - 19:00	7.00	66.00
1900	19:00 - 19:15	30.00	67.00
1915	19:15 - 19:30	30.00	67.00
1930	19:30 - 19:45	26.00	62.00
1945	19:45 - 20:00	31.00	69.00

Figure 3–12: VISSIM Interface Describing Pacer Speed Ranges.

The inputs for these speed distributions followed the same process as the original inputs that control the general speeds for PVs and HVs respectively. Figure 3–13 shows 15-min intervals on the x-axis and speed in miles per hour on the y-axis. The colors depict a different percentile corresponding to that time bin to better illustrate the upper and lower boundaries of each percentile. As an example, looking at the first interval 13:00 – 13:15, the legend corresponding to the 10^{th} percentile indicate that the first 10% of the speeds can be found within the 15 – 39 mph range. The legend corresponding to 30^{th} percentile for the same time interval indicate that the second 20% of the speeds can be found between the range of 39 – 43 mph for the first time interval. The 50th percentile ranges from 43 – 47 mph and this represents the third 20% of the speed distribution.

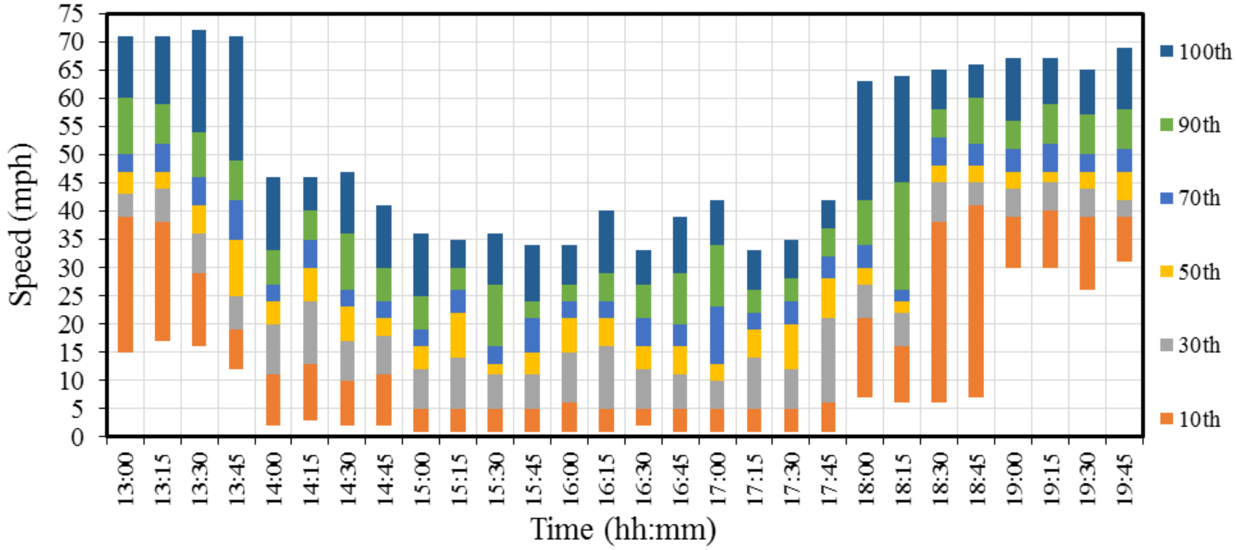
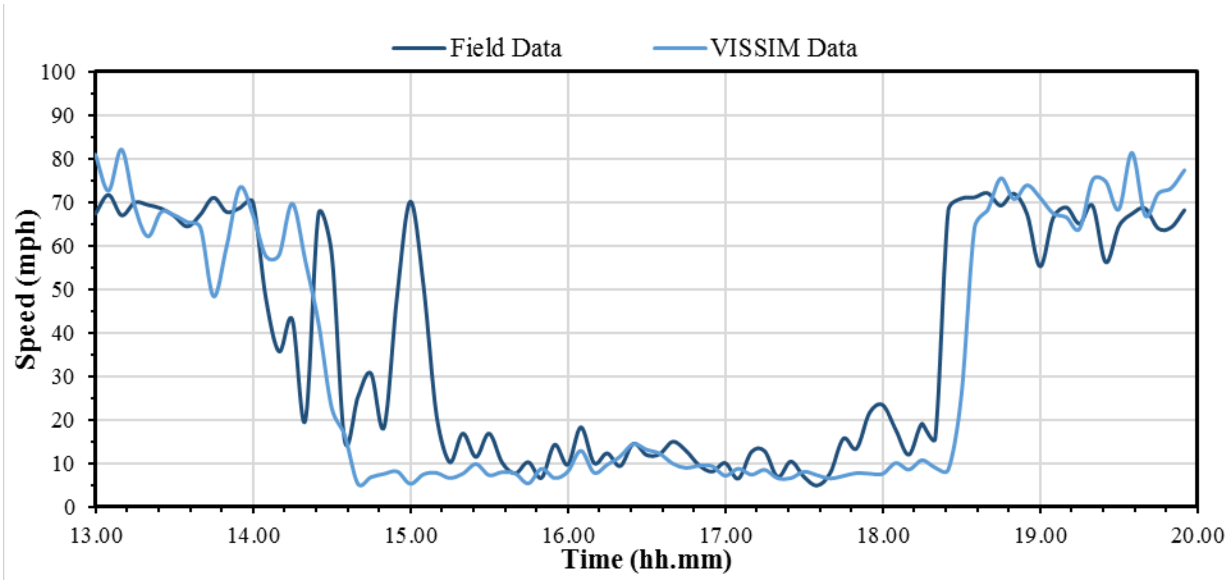
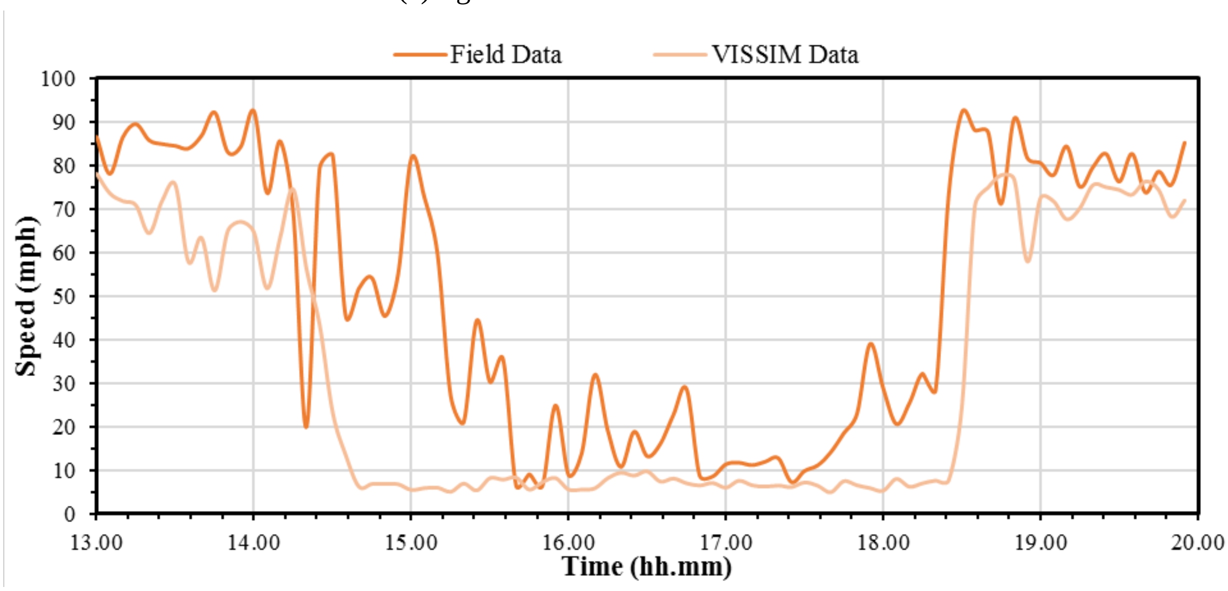


Figure 3–13: Speed Percentile Chart at Sensor 104.

The first attempt of using the pacers managed to obtain results closer to the field-gathered data between 2:00 pm and 7:00 pm. A side-to-side comparison can be found in Figure 3–14, each graph represents the right lane and the left lane respectively.



(a) right lane data: field vs. simulation



(b) left lane data: field vs. simulation

Figure 3–14: Set of VISSIM Model Run Results After First Calibration Attempt at 0.5 mi.

The work zone reduced speed areas based on the 15-min speed distribution data collected by sensor 104 have a noticeable impact on the speed averages within the model. The reductive effect kept the queued conditions for a substantially longer period when compared to the first trial of simulation. The general reduction of speed averages begins after 2:00 pm, and queued conditions dissipate 10 – 15 minutes indicated by after the field data. The right lane speed records corresponding to the period between 2:00 pm and 3:00 pm describe spikes in speeds reaching 67 mph followed by 15 mph averages within 10 minutes. Similarly, even more discrepant speed averages can be observed in the same time for the left lane field data (as supported by the field data). These spikes in speed prior to the harmonization due to queued conditions can be part of the overall merging behavior of freeway traffic. The right lane is expected to have slightly slower traffic than the left lane, as well as taking into consideration that PCMSs displaying the message 'Lane Closure Ahead' were already in place at milepost 72.6 and on the southbound on-ramp at Exit 68. Another explanation in regards to the speed spikes prior to queued conditions could be attributed to the fact that left lane vehicles are merging at higher speeds and within gaps into the right travel lane. These attributes obtained from field data can be attributed to randomness as well as driving behavior; this would be one of the major limitations of traffic simulation models in general. The next step in the calibration process focuses on the broader and more general speed trends based on simply when and by how much are speeds being reduced.

The next step towards achieving better queue dissipation timing would be to reduce the time lag that lies within the queue dissipation (15-min window). To force the queue to dissipate 15-min earlier and higher speeds were introduced earlier, the 17:00, 17:15, 17:30 and 17:45 were replaced with 18:00, 18:15, 18:30, and 18:45 respectively. This translates into the application of the four 15-min interval speed CDFs that correspond to 18:00, twice. Figure 3–15 describes the speed and percentiles used in the second version of the calibration process. This figure further illustrates the usage of the same speed distribution for 17:00 and 18:00, 17:15 and 18:15, 17:30 and 18:30, and 17:45 and 18:45.

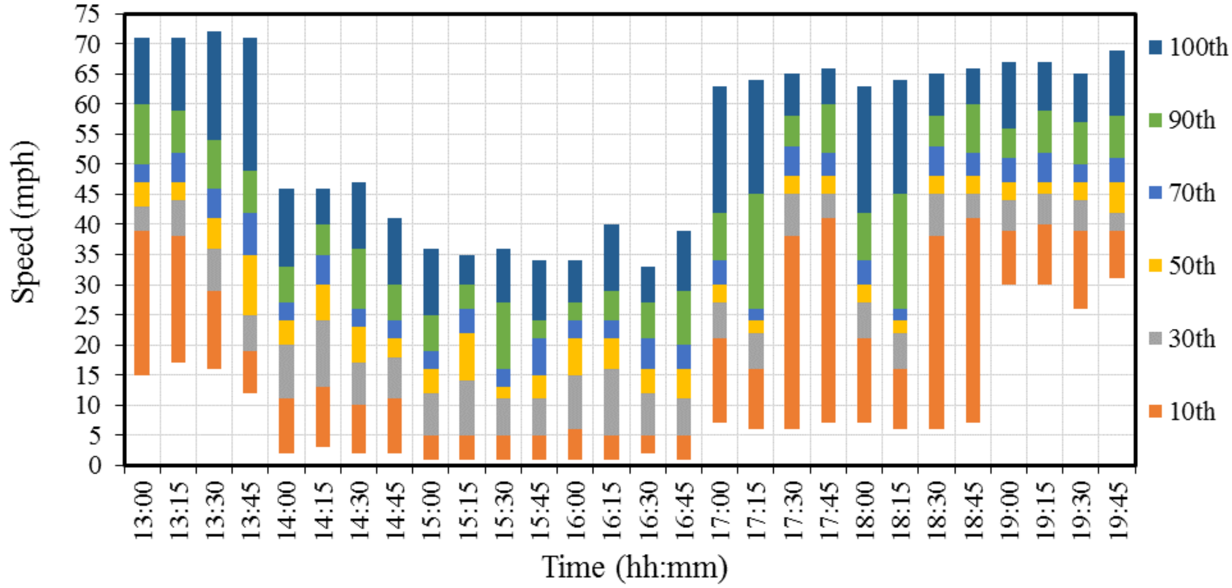


Figure 3–15: Calibrated Speed Cumulative Distribution Frequency Used in VISSIM

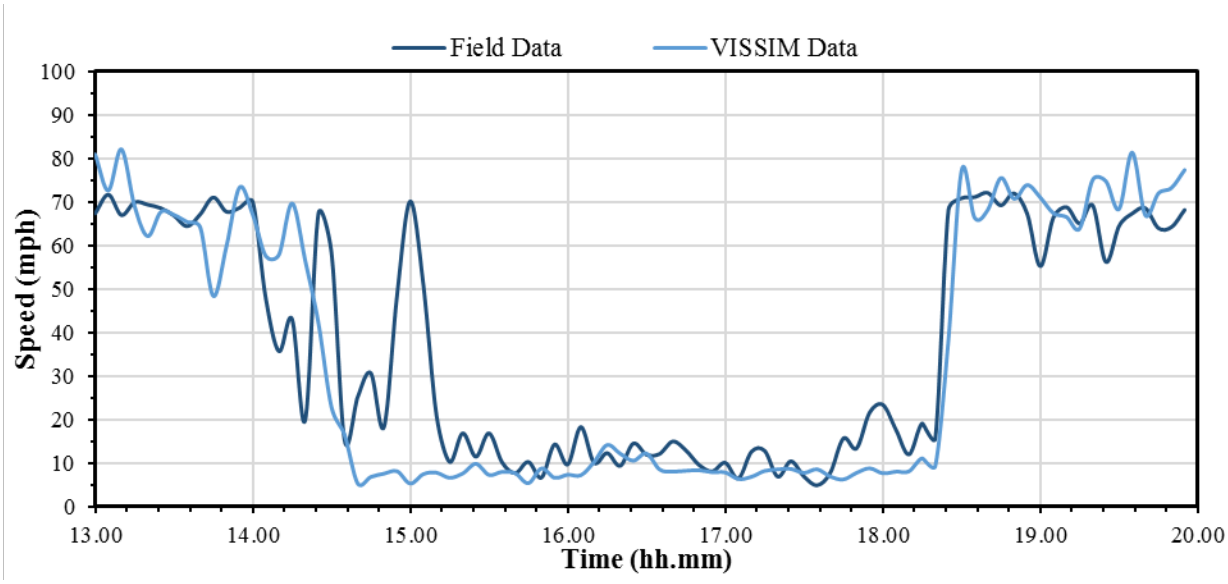
Figure 3–16 shows the DSD VISSIM interface for the right travel lane prior to the lane closure; the same applies to the left lane. The 28 15-min bins are described by the ‘TimeFrom’ and ‘TimeTo’ columns in units of seconds. The desired speed distribution column describes the cumulative speed frequency distribution of choice for the specific time.

Name	DesSpeedDistr(20)	Lane	Pos	TimeFrom	TimeTo
WZ_RIGHT_13.00	1300: 13:00 - 13:15	4: LANECLOSURE_CONNECT	3.000	0	900
WZ_RIGHT_13.15	1315: 13:15 - 13:30	4: LANECLOSURE_CONNECT	3.000	901	1800
WZ_RIGHT_13.30	1330: 13:30 - 13:45	4: LANECLOSURE_CONNECT	3.000	1801	2700
WZ_RIGHT_13.45	1345: 13:45 - 14:00	4: LANECLOSURE_CONNECT	3.000	2701	3600
WZ_RIGHT_14.00	1400: 14:00 - 14:15	4: LANECLOSURE_CONNEC	3.000	3601	4500
WZ_RIGHT_14.15	1415: 14:15 - 14:30	4: LANECLOSURE_CONNECT	3.000	4501	5400
WZ_RIGHT_14.30	1430: 14:30 - 14:45	4: LANECLOSURE_CONNECT	3.000	5401	6300
WZ_RIGHT_14.45	1445: 14:45 - 15:00	4: LANECLOSURE_CONNECT	3.000	6301	7200
WZ_RIGHT_15.00	1500: 15:00 - 15:15	4: LANECLOSURE_CONNECT	3.000	3601	4500
WZ_RIGHT_15.15	1515: 15:15 - 15:30	4: LANECLOSURE_CONNECT	3.000	4501	5400
WZ_RIGHT_15.30	1530: 15:30 - 15:45	4: LANECLOSURE_CONNECT	3.000	5401	6300
WZ_RIGHT_15.45	1545: 15:45 - 16:00	4: LANECLOSURE_CONNECT	3.000	6301	7200
WZ_RIGHT_16.00	1600: 16:00 - 16:15	4: LANECLOSURE_CONNECT	3.000	7201	8100
WZ_RIGHT_16.15	1615: 16:15 - 16:30	4: LANECLOSURE_CONNECT	3.000	8101	9000
WZ_RIGHT_16.30	1630: 16:30 - 16:45	4: LANECLOSURE_CONNECT	3.000	9001	9900
WZ_RIGHT_16.45	1645: 16:45 - 17:00	4: LANECLOSURE_CONNECT	3.000	9901	10800
WZ_RIGHT_17.00	1800: 18:00 - 18:15	4: LANECLOSURE_CONNECT	3.000	10801	11700
WZ_RIGHT_17.15	1815: 18:15 - 18:30	4: LANECLOSURE_CONNECT	3.000	11701	12600
WZ_RIGHT_17.30	1830: 18:30 - 18:45	4: LANECLOSURE_CONNECT	3.000	12601	13500
WZ_RIGHT_17.45	1845: 18:45 - 19:00	4: LANECLOSURE_CONNECT	3.000	13501	15300
WZ_RIGHT_18.00	1800: 18:00 - 18:15	4: LANECLOSURE_CONNECT	3.000	15301	16200
WZ_RIGHT_18.15	1815: 18:15 - 18:30	4: LANECLOSURE_CONNECT	3.000	16201	17100
WZ_RIGHT_18.30	1830: 18:30 - 18:45	4: LANECLOSURE_CONNECT	3.000	17101	18000
WZ_RIGHT_18.45	1845: 18:45 - 19:00	4: LANECLOSURE_CONNECT	3.000	18001	18900
WZ_RIGHT_19.00	1900: 19:00 - 19:15	4: LANECLOSURE_CONNECT	3.000	18901	19800
WZ_RIGHT_19.15	1915: 19:15 - 19:30	4: LANECLOSURE_CONNECT	3.000	19801	20700
WZ_RIGHT_19.30	1930: 19:30 - 19:45	4: LANECLOSURE_CONNECT	3.000	20701	21600
WZ_RIGHT_19.45	1945: 19:45 - 20:00	4: LANECLOSURE_CONNECT	3.000	21601	22500

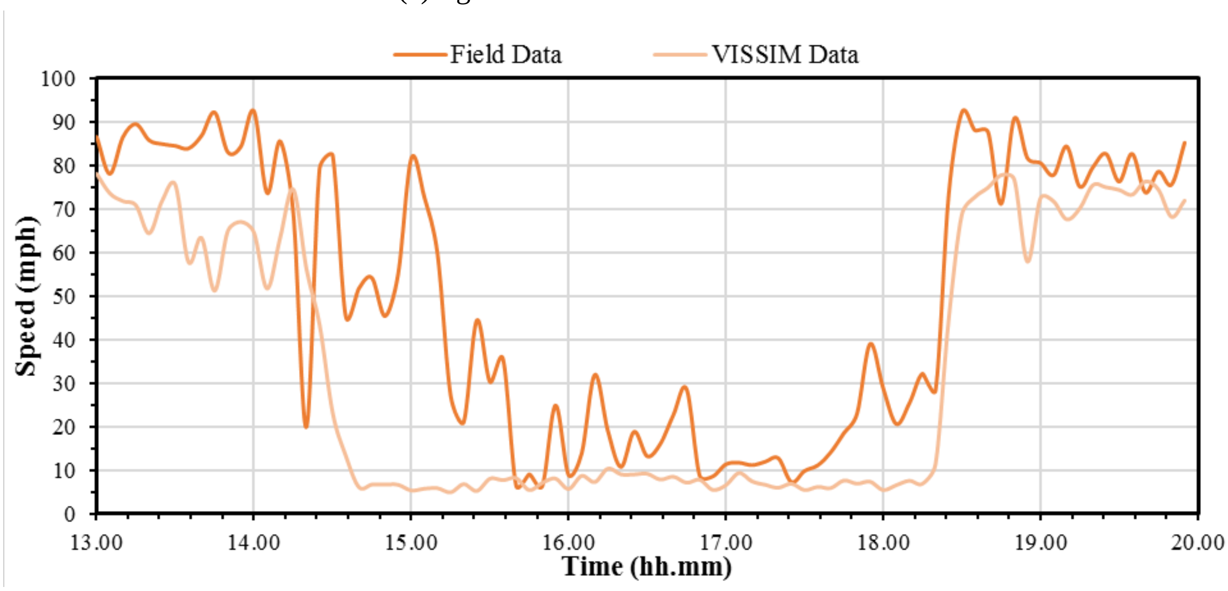
Figure 3–16: VISSIM Desired Speed Distribution Interface.

The purpose of this step of the calibration was to ‘fast-forward’ the distillation of queued conditions by an hour. By bridging the existing gap of queue dissipation during time periods between the VISSIM model and field-gathered data. The results of this step in the calibration process can be observed in Figure 3–17.

The initial speed reduction and preservation of slower speeds is maintained the identical to the previous calibration step, but the queue dispersion time gap is between field and VISSIM speeds is almost completely eliminated. This calibration effort allowed the model to better replicate field traffic behavior.



(a) right lane data: field vs. simulation



(b) left lane data: field vs. simulation

Figure 3–17: Set of VISSIM Model Run Results After Second Calibration Attempt

3.7 PROPOSED SPEED REDUCTION SCHEMES

This study proposes six schemes that are intended to reduce vehicle speeds 3.5 miles upstream of the work zone with the purpose of simulating the effects of a QWS. The six schemes differ in reduced speed ranges, speed reduction magnitude, and location of the reduction. The QWS relays reduced speed limits to drivers as vehicles approach the work zone environment when queues are present, similarly in this study, the speed limits stated in the simulation will be lowered as vehicles approach the work zone. The schemes, A-F, applied with three sets of non-compliant driver proportions, 0%, 10%, and 20%, this resulting in 18 proposed simulation cases. Each case is then repeated using a different random seed and then evaluated in regards of statistical significance.

3.7.1 SCHEMES AND SPEED REDUCTION LOCATION

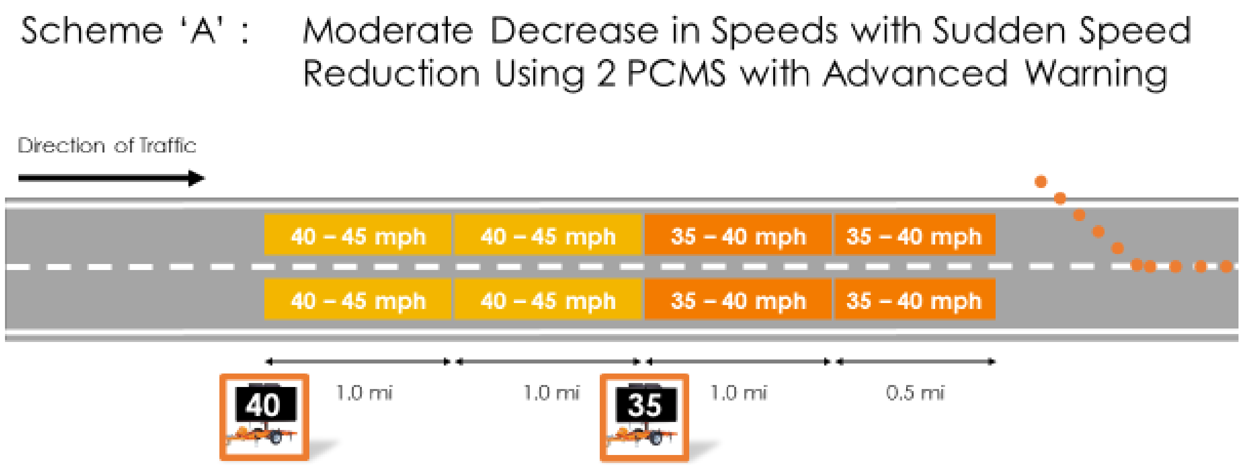
Speed reductions were established in the simulation model to cover a total link length of 3.5 miles. This length has been fragmented into four segments; one 0.5-mile long and three 1.0-mile long consecutive segments. Several combinations of reduced speed distributions are assigned to the different segments; always with speed distributions decreasing with the direction of traffic prior to entering the lane closure. Table 3-6 describes these six different combinations of speed distributions that represent different speed reduction approaches; speed reductions of 10 mph and 5 mph every mile are present.

Table 3-6: Speed Distribution Values and Location

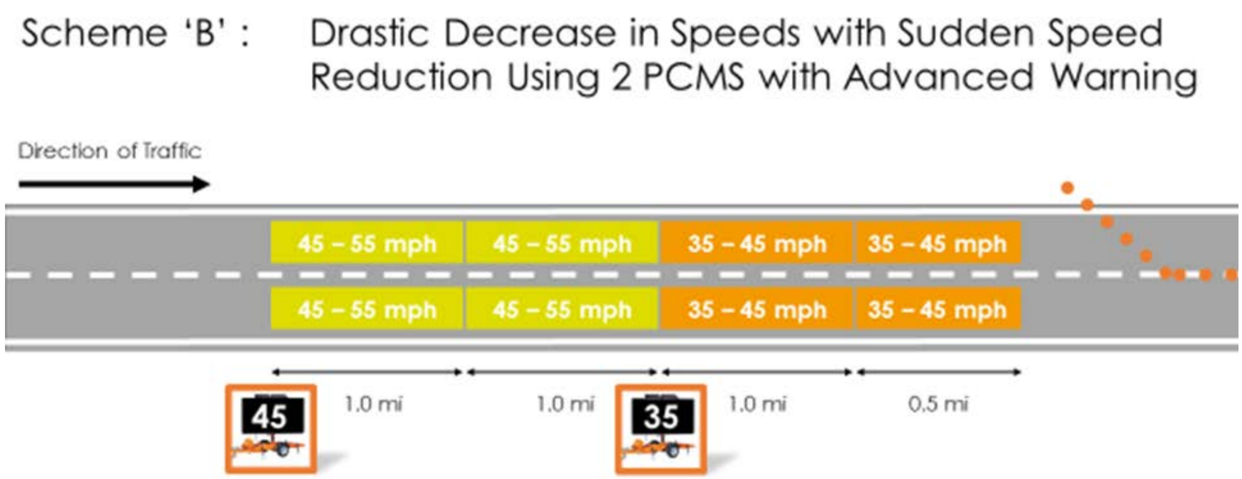
Scheme	Segment Located Miles Prior to Work Zone Taper (mi)			
	3.5	2.5	1.5	0.5
	Speed Range (mph)			
A	40 – 45	40 – 45	35 – 40	35 – 40
B	45 – 55	45 – 55	35 – 45	35 – 45
C	45 – 55	45 – 55	45 – 55	35 – 45
D	65 – 75	55 – 65	45 – 55	35 – 45
E	60 – 70	50 – 60	40 – 50	30 – 40
F	50 – 55	45 – 50	40 – 45	35 – 40

Figure 3–18 and Figure 3–19 illustrate the placement of the different combination of speed distributions used for the speed reductions in schemes A-F. The decrease in speeds varies gradually from 5 mph to 10 mph per segment. Conservative speed reductions are used with the intention of decreasing the abrupt change in speeds that is observed in the speed data prior to queued conditions. All schemes aim to test different driving behavior responses when introduced to a lane closure; the purpose is to test variety of speed reductions similar to those used in QWS deployments. The placement of the reduced speed areas is fixed in the model; nonetheless, the speed ranges described by this tool can be customized.

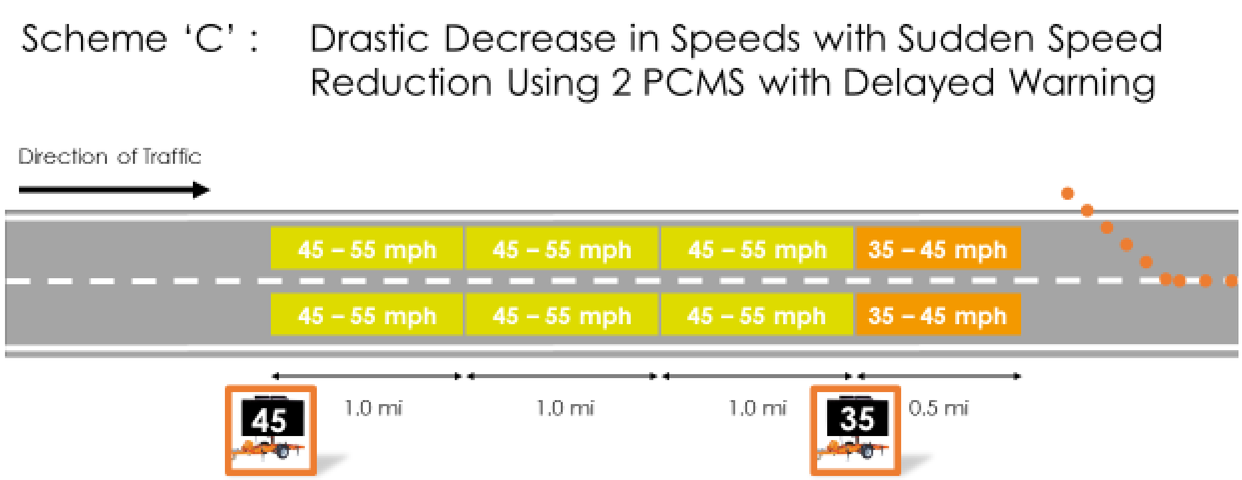
To achieve speed reduction schemes, the ‘Reduced Speed Area’ tool in VISSIM was used. ‘Reduced Speed Areas’ is the simulation instrument that forces entering vehicles to decelerate before entering the reduced speed area and enter it at a reduced speed; after leaving the area the vehicle accelerates until it reaches desired speed again. Reduced speed areas in VISSIM accomplish what QWSs aim to achieve on a real freeway. QWSs are based on the assumption that by reducing speeds enough in advance with messages displayed upstream of the work zone; a reduction in rear end crashes can be achieved and the overall safety of the work zone is increased.



(a) Scheme 'A'



(b) Scheme 'B'



(c) Scheme 'C'

Figure 3–18: Proposed Schemes Using 2 PCMSs with Sudden Speed Reductions

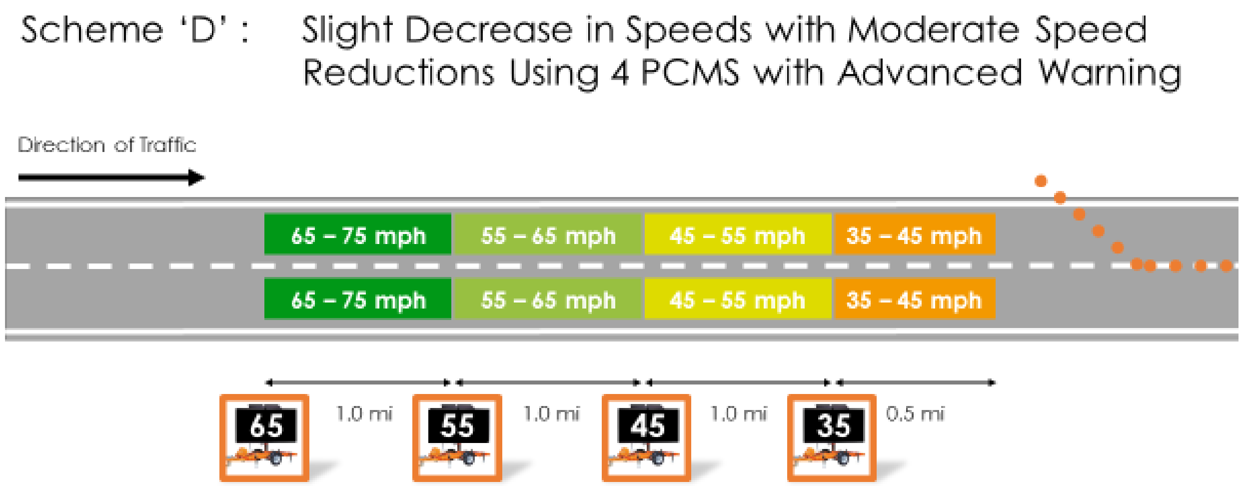
Figure 3–18 describes the first set of proposed speed reduction schemes that include two speed reductions for vehicles approaching the advanced warning area; from a practical perspective these three scenarios would translate into the use of two PCMSs. Incoming vehicles approach the reduced speed areas with an average speed of 70 mph in the simulation model.

Scheme ‘A’, as seen in Figure 3–18 (a), describes a speed range between 40 – 45 mph for the first 2 miles, followed by a reduced speed range of 35 – 40 mph for the subsequent 1.5 miles. Vehicles introduced to this scheme would initially reduce their traveling speed by an average of 30 mph when encountering the first PCMS located at 3.5 miles upstream of the work zone; followed by a second 5 mph speed reduction for the remaining 1.5 miles prior to entering the work zone where another PCMS is located. Vehicles enter the work zone at a range of speeds between 35 – 40 mph.

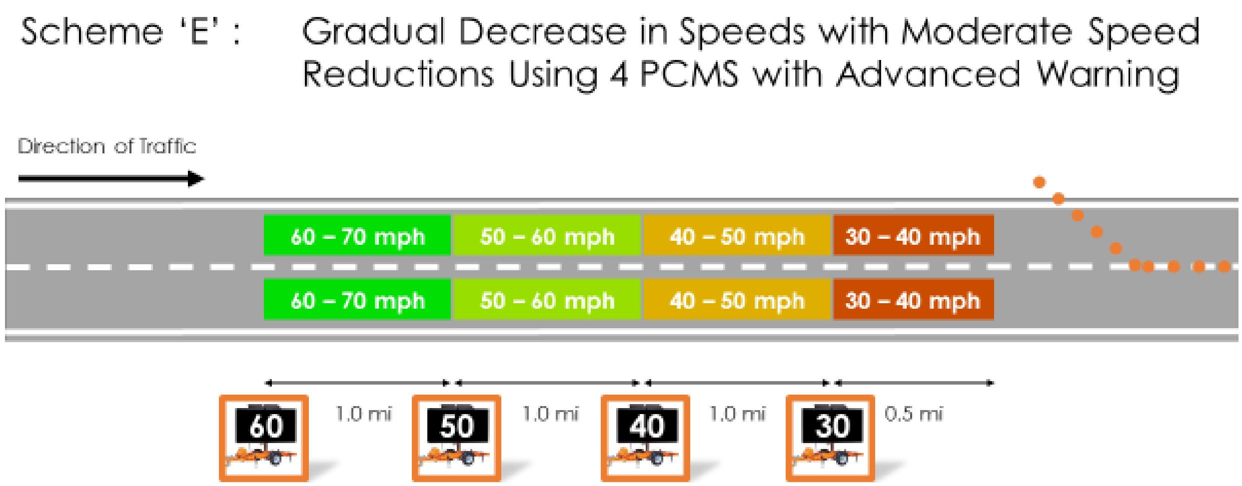
Scheme ‘B’, as seen in Figure 3–18 (b), describes a speed range between 45 – 55 mph for the first 2 miles, followed by a reduced speed range of 35 – 45 mph for the subsequent 1.5 miles. Vehicles introduced to this scheme would initially reduce their traveling speed by an average of 25 mph when encountering the first PCMS located at 3.5 miles upstream of the work zone; followed by a second 10 mph speed reduction for the remaining 1.5 miles prior to entering the work zone where another PCMS is located. Vehicles enter the work zone at a range of speeds between 35 – 45 mph.

Scheme ‘C’, as seen in Figure 3–18 (c), describes a speed range between 45 – 55 mph for the first 3 miles, followed by a reduced speed range of 35 – 45 mph for the subsequent 0.5 mile. Vehicles introduced to this scheme would initially reduce their traveling speed by an average of 25 mph when encountering the first PCMS located at 3.5 miles upstream of the work zone; followed by a second 10 mph speed reduction for the remaining 0.5 mile prior to entering the work zone where another PCMS is located. Vehicles enter the work zone at a range of speeds between 35 – 45 mph.

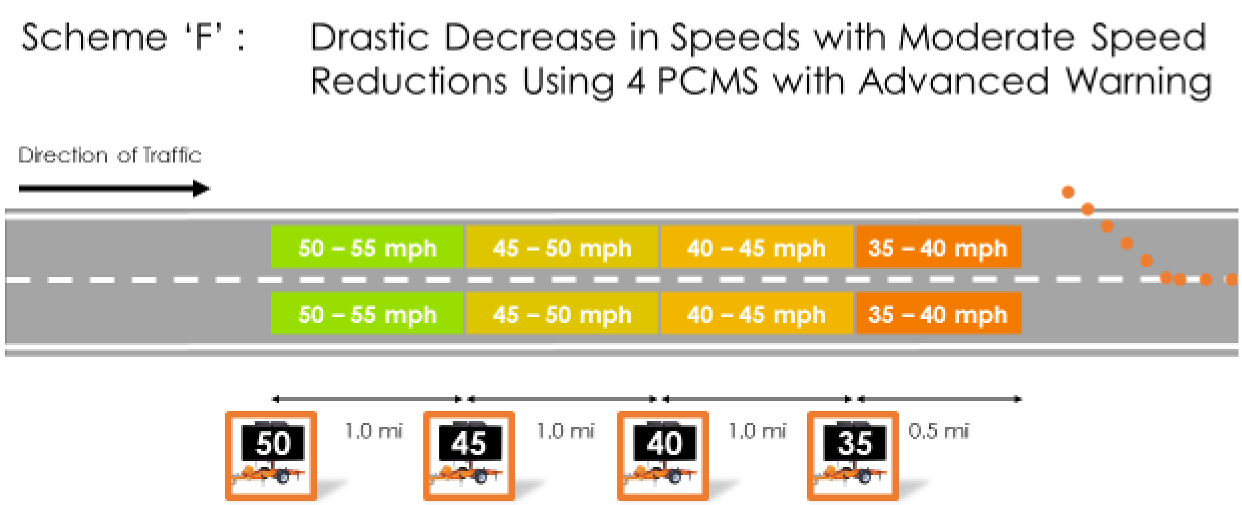
The first set of three speed reduction schemes previously described are considered as drastic reductions in speed. This attributed to the substantial speed difference between incoming vehicle speeds and the reduced speed areas. Delayed warning is attributed to the proximity of the second PCMS and the work zone. Advanced warning describes a scenario in which vehicles have at least 1.5 miles to travel at the suggested work zone speed. For brevity, these schemes will be referred to as schemes A, B, and C respectively.



(a) Scheme 'D'



(b) Scheme 'E'



(c) Scheme 'F'

Figure 3–19: Proposed Schemes Using 4 PCMSs with Moderate Speed Reductions

Figure 3–19 describes the second set of proposed speed reduction schemes that include four speed reductions for vehicles approaching the advanced warning area; from a practical perspective these three scenarios would translate in the use of four PCMSs. Incoming vehicles approach the reduced speed areas with an average traveling speed of 70 mph in the simulation model.

Scheme 'D', as seen in Figure 3–19(a), describes a speed range between 65 – 75 mph for the first mile, followed by a reduced speed range of 55 – 65 mph for the second mile, succeeded by a reduced speed range of 45 – 55 mph for the third mile, and a reduced speed range of 35 – 45 mph for the last 0.5 mile. Vehicles introduced to this scheme would minimally reduce their entering travelling speeds by an average of 5 mph when encountering the first PCMS located at 3.5 miles upstream of the work zone. Subsequent speed reductions of 10 mph are applied on the remaining speed reduction areas at 2.5, 1.5, and 0.5 miles respectively, PCMSs are located at the beginning of each reduced speed area. Vehicles enter the work zone at a range of speeds between 35 – 45 mph.

Scheme 'E', as seen in Figure 3–19(b), describes a speed range between 60 – 70 mph for the first mile, followed by a reduced speed range of 50 – 60 mph for the second mile, succeeded by a reduced speed range of 40 – 50 mph for the third mile, and a reduced speed range of 30 – 40 mph for the last 0.5 mile. Vehicles introduced to this scheme would minimally reduce their entering travelling speeds by an average of 10 mph when encountering the first PCMS located at 3.5 miles upstream of the work zone. Subsequent speed reductions of 10 mph are applied on the remaining speed reduction areas at 2.5, 1.5, and 0.5 miles respectively, PCMSs are located at the beginning of each reduced speed area. Vehicles enter the work zone at a range of speeds between 30 – 40 mph.

Scheme ‘F’, as seen in Figure 3–19(c), describes a speed range between 50 – 55 mph for the first mile, followed by a reduced speed range of 45 – 50 mph for the second mile, succeeded by a reduced speed range of 40 – 55 mph for the third mile, and a reduced speed range of 35 – 40 mph for the last 0.5 mile. Vehicles introduced to this scheme would minimally reduce their entering travelling speeds by an average of 20 mph when encountering the first PCMS located at 3.5 miles upstream of the work zone. Subsequent speed reductions of 5 mph are applied on the remaining speed reduction areas at 2.5, 1.5, and 0.5 miles respectively, PCMSs are located at the beginning of each reduced speed area. Vehicles enter the work zone at a range of speeds between 30 – 40 mph.

The first set of three speed reduction schemes previously described are considered as gradual reductions in speed. This attributed to the moderately low speed differences between incoming vehicle speeds and the reduced speed areas. Having a higher frequency of PCMSs displaying speed reduction warnings allows to have a more step-by-steps

A higher number of PCMSs allows more opportunities to warn incoming drivers about queued conditions. This translate into a higher number of speed reductions that can be implemented, by achieving a higher number of speed reductions the overall decrease of speeds can be accomplished in a gradual manner prior to the lane closure. For brevity, these schemes will be referred to as schemes D, E, and F respectively.

3.7.2 NON-COMPLIANT DRIVER PROPORTIONS

Driver compliance in VISSIM with a set speed distribution is of 100% unless stated otherwise. For the purpose of this research, three different driver proportions are introduced into the simulation model; these contain a different percentage of non-compliant drivers. Non-compliant drives are those drivers that do not follow the reduced speeds stated by the reduced speed areas, which simulate the effects of a QWS, as well as the work zone pacers. The non-compliant driver proportion (NCDP) is intended to follow the original PV and HV speed distribution throughout the entire 7-hour analysis period. Three different NCDP are introduced to all six schemes in the simulation model; 0%, 10%, and 20%. Guidance in regards to non-compliant driver proportion values was obtained by a study that attempted to simulate rubbernecking effects on 20% of its drivers (Ramadan and Sisiopiku 2016).

A 100% compliant driver behavior case will be used as the link between control and treatment schemes. The control scenario's purpose is to represent field gathered data in the simulation model; meaning that this case had a 100% of compliance rate in regards to speed and following the work zone speed distribution. This is comparable to all variations of cases that have 0% NCDP. Nevertheless, to better represent random driver behavior especially under work zone conditions non-compliant drivers are introduced into the model for PV and HV respectively.

The volume inputs used in the control case were of 3,208 vehicles (during the 7-hour simulation period) for PV and 1,111 for HV. In order to introduce the non-compliant drivers into the model a separate set of vehicle type was created in the model; same general default characteristics as the PV and HV used in the control case. The only difference lies in the speed distribution that this set of vehicle types are intended to follow, non-compliant PV (NC-PV) and non-compliant (NC-HV).

For the 10% NCDP case, 10% of the volumes for PV and HV were replaced with NC-PV and NC-HV volume inputs, as shown in Table 3-7. The same process was followed for the 20% non-compliance case.

Table 3-7: Relative Flow Values for Three Different NCDP

Non-Compliant Drivers	Volume Inputs			
	PV	HV	NC-PV	NC-HV
0%	3208	1111	0	0
10%	2887	1000	321	111
20%	2566	889	642	222

Table 3-7 presents the amount of vehicles that were proportionally split into all four of the vehicle type categories. The proposed treatment can be summarized into a six by three matrix comprised of six schemes (A-F) crossed with three NCDP; 18 different treatment cases will be introduced into the simulation model, and differences among the scenarios will be tested for significance in this chapter.

3.7.3 Designated Time Window

One of the main characteristics of a QWS is that it is activated only when slower speeds are detected; an actuated system can detect reduced speeds associated with queues and display appropriate warning messages to upcoming drivers. VISSIM is unable to directly simulate PCMSs with warning signs that would warn drivers about queued conditions and eventually affect driver behavior. Nonetheless, the modeler is able to identify a window of time for which the slower speeds, using reduced speed areas, can be implemented. For the purposes of this research, a time window for which queues are present is identified based on field data, with the goal of modeling the effects of a QWS. The time window for which the QWS is “ON” starts at 2:05 pm (4500 simulation seconds) and ends at 6:05 pm (21,900 simulation seconds), as shown in Figure 3–20.

The QWS simulation and its effect will be conducted and evaluated throughout six different schemes scenarios with different non-compliant driver proportions. While all 18 cases will differ regarding these two aspects, the time window for which the reduced speed areas are programmed to start and end will be the same for all cases.

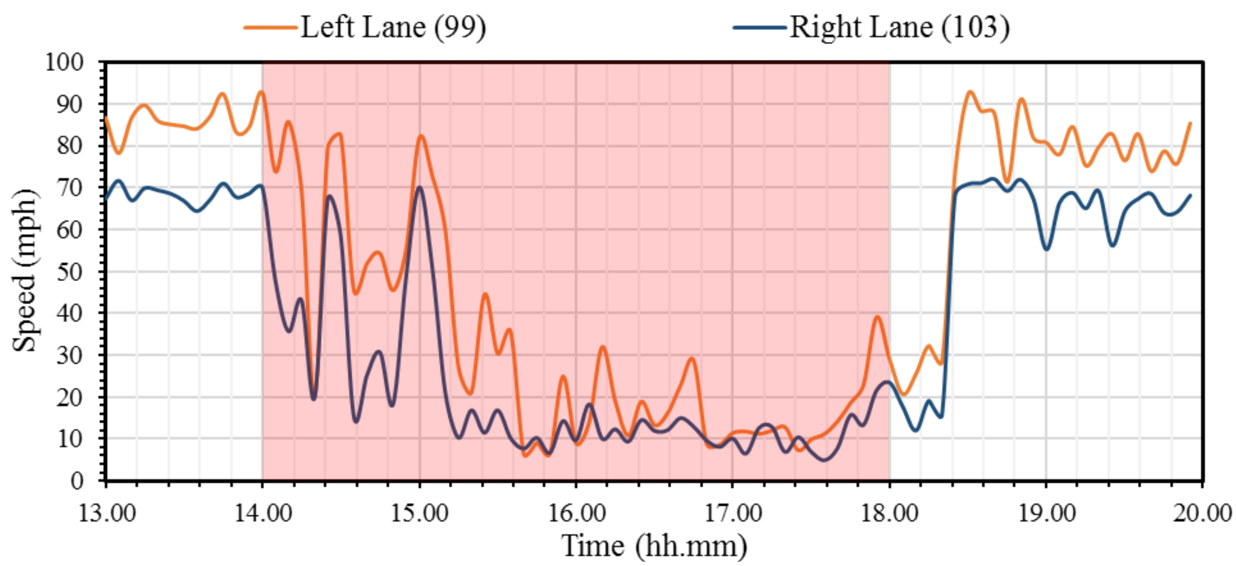


Figure 3–20: Window of Time for Which 'Reduced Speed Areas' are Activated.

3.8 MEASURES OF EFFECTIVENESS

This section illustrates the measures of effectiveness that were used in this study to validate or abandon the hypothesis that the reduced speed areas have a significant effect in the result of the simulation models as well as a meaningful indication of safety improvements that may be attainable in the field.

3.8.1 SPEED AND DELTA SPEED

In early stages of this study, only measurements of speed and time were mentioned and considered throughout the development and calibration of the simulation model. Although speed can be an indication that the model is achieving the desired speed range area boundaries and maintaining the speed under predetermined thresholds, it is not exactly what would indicate an increase in safety. A rear-end crash is more likely to happen when there is a large speed differential in a small amount of time. The selected approach to identify vulnerable conditions in which rear end crashes are likely to happen would be to identify when the difference between consecutive 5-min interval speed averages is considerably larger. A negative and large speed differential (ΔS) indicates high-speed vehicles encountering slowed and queued conditions while approaching the work zone.

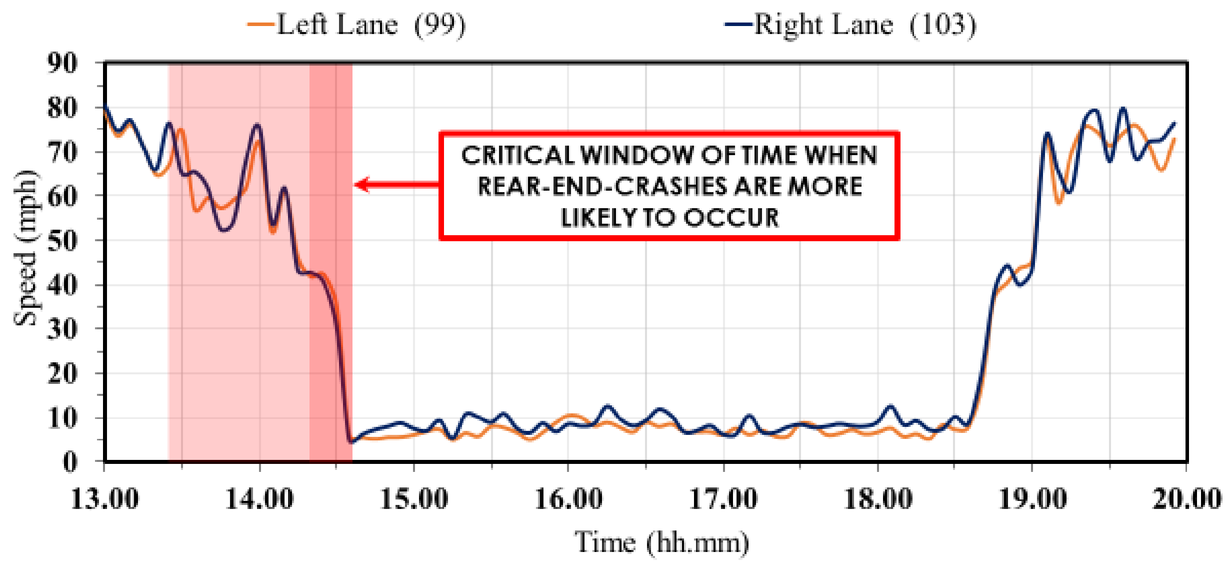
$$\Delta S = Speed_{i+1} - Speed_i \qquad \text{(Eq. 3-1)}$$

Where,

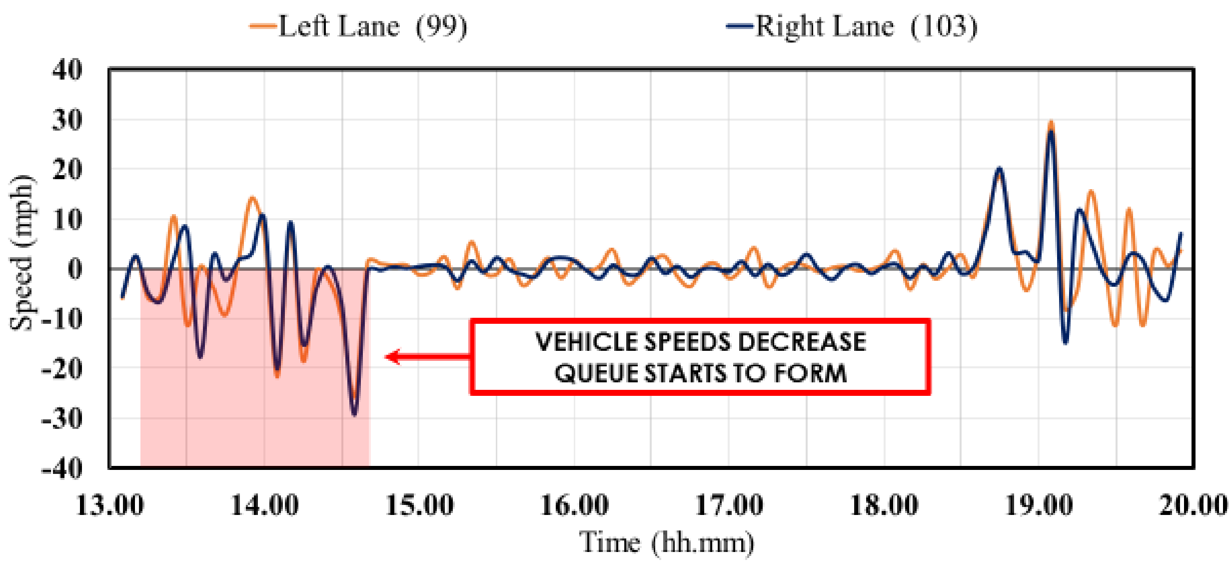
Speed: refers to the speed average of each of the 5-min intervals (mph)

i: refers to the 5-min time interval

This equation is used to calculate ΔS for each interval of each for the control case as well as each of the different combinations of control cases; graphical representation of ΔS vs. time can be observed in Figure 3–21(b). The speed vales used to calculate ΔS are shown in Figure 3–21(a).



(a) sample simulation result Δ speed vs. time (sensors 99 & 103)



(b) sample simulation result Δ speed vs. time (sensors 99 & 103)

Figure 3–21: Measures of Effectiveness - Speed and Delta Speed Sample Model Run Results.

An increase in speed would be identified as a large and positive number, and these are expected to be found within the time when the dissipation of the queue takes place. Field data reveal that large and negative numbers of ΔS are likely to be found within the vicinity of the 2:00 pm time of day. This particular time window is where queues begin to occur both in the field data and in the simulation model.

3.9 DETERMINATION OF NUMBER OF REPETITIONS

Simulation by definition, incorporates variability through the use of input value distributions and random number seeds. Multiple repetitions of the same case are required since each simulation run incorporates built in variations with the use of different random number seed. The random number seed describes a selection of random number sequences that influence numerous decisions in regards to following distance, merging time gap and distance throughout the simulation process. The overall result of each simulation run will likely be close to the average of all runs, nonetheless each run will be different from one another. The required number of repetitions is obtained through an iterative process and three main components need to be known for this computation: sample standard deviation, desired length of confidence interval (CI) and desired level of confidence.

To estimate the sample standard deviation, a minimum number of repetitions need to be conducted. For the purpose of determining the number of runs, the control case model will be used with different number random seeds in order to first attempt to calculate a sample standard deviation. It is recommended that at least four runs be performed for the initial estimation of the standard deviation (Dowling et al. 2004).

The standard deviation of a critical time interval in the control case was calculated. For the purpose of this research, the critical point in time is defined as to where the difference in speed averages between two consecutive 5-min intervals is substantial as well as negative. The most critical period of time can be found when the queue is starting to propagate. This critical period does not necessarily need to be the same time stamp for each simulation run, but are likely to be within the vicinity of the same period. This difference in speeds would eventually lead to identifying time periods where rear-end crashes are more likely to occur due to a sudden and dramatic decline in speeds. Standard deviations resulting from using four different random number seeds within the control case scenario were calculated.

The selection of the desired confidence level is the probability that the true mean can be found within the target confidence interval; the usual approach is 95% level of confidence. The desired CI is described as the range of values within which the true mean value may lie; for the purpose of this research, the margin of error (ME) used to determine the number of runs is ± 5 mph. The computation of minimum number of repetitions is obtained by making use of the following equation:

$$CI_{1-\alpha\%} = 2 \times t_{(1-\alpha/2),\ N-1} \times \frac{s}{\sqrt{N}} \qquad \text{(Eq. 3-2)}$$

Where,

$CI_{1-\alpha\%}$ = Confidence interval for the true mean, where alpha equals the probability of true mean not lying within the confidence interval

$t_{(1-\alpha/2),\ N-1}$ = t- statistics for the probability of a two sided error summing to alpha with N-1 degrees of freedom

N = number of repetitions

s = standard deviation of the model results

By solving for N in equation 3-2, the number of runs required in order to obtain a ME of ± 5 mph within a 95% confidence interval is 15 runs per case; resulting in 270 simulations for proposed scenarios and 15 simulations for the control case.

3.10 SUMMARY

The data collection plan replicates previous QWS deployments in Texas. Quality control efforts described in this section have the purpose of reducing the possibility of incorrectly simulating the behavior observed in the field when coded into VISSIM. Calibration methods are described in depth, intended to illustrate the use of several tools within the software to customize the control case to replicate more accurately the observed behavior. The proposed speed reduction schemes (A-F) in this study are intended to reduce vehicle speeds approaching the work zone; this aims to replicate the effects that a QWS would have in the field. The non-compliant driver proportions are intended to represent drivers that in the field would not follow the speed reduction messages prompted by a PCMS in case of queued conditions.

CHAPTER FOUR:
ANALYSIS

4.1 INTRODUCTION

This chapter presents the methodology for and implementation of testing the statistical significance of differences observed in speed differentials by the effect of proposed speed reduction schemes and non-compliant driver population. The measure of effectiveness that is used to evaluate the QWS simulation proposed schemes is the change in speed from one 5-min interval to the next (ΔS). The largest, and negative, 5-min speed average differences between two consecutive intervals were drawn from the simulation results. Eighteen treatments with two independent factors were evaluated for statistically significant difference. The first factor is described by six speed reduction schemes (A, B, C, D, E, and F), and the second factor is described by three non-compliant driver proportion (0%, 10%, and 20%).

First, this study investigated the statistical significance of differences of the effects of the speed reduction scheme factor between its six different values, the effects of non-compliant driver proportion factor between its three different values, and the effects of the interaction within the two factors on the selected measure of effectiveness (ΔS).

Second, this study examined the extent of statistically significant differences between the speed differential results obtained from the 21 possible pairwise combinations between the six different speed reduction schemes and the control case; this analysis addresses difference between baseline control scenario and the proposed speed reduction schemes.

Third, this study examined the extent of statistically significant differences between the speed differential results obtained from the three possible combinations of driver compliance factors.

4.2 COMPILATION OF MODEL RESULTS

This section illustrates the preliminary findings and summary statistics obtained from the simulation model based on ΔS results. Table 4-1 and Table 4-2 provide the average and the variance of the negative and largest speed differential (ΔS) obtained from the 15 simulation runs per treatment case. A summary for the three non-compliant driver proportions is listed at the bottom of the table and the summary of the six different schemes is on the right most column. Table 4-1 describes the results obtained from the data collector placed on the left lane and Table 4-2 describes the results obtained from the right lane; both located 0.5 miles prior to the beginning of the taper in the simulation model. These tables contain summary statistics, the columns represent the different proportions used for non-compliant drivers and the rows describe all six schemes proposed.

Table 4-1: Summary Statistics Results Based on Largest and Negative ΔS (Left Lane)

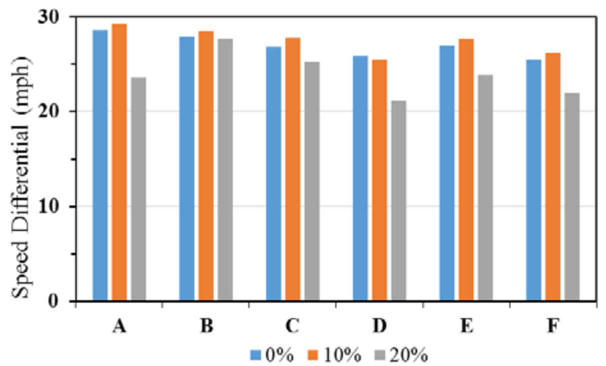
Schemes	Summary	Non-Compliant Driver Proportion 0%	10%	20%	Total
A	Mean	**28.6**	**29.2**	**23.6**	**27.1**
	Variance	*23.8*	*18.8*	*20.1*	*26.3*
B	Mean	**27.9**	**28.5**	**27.7**	**28.0**
	Variance	*63.1*	*27.7*	*23.5*	*36.5*
C	Mean	**26.8**	**27.8**	**25.3**	**26.6**
	Variance	*64.1*	*23.3*	*25.0*	*36.9*
D	Mean	**25.9**	**25.4**	**21.1**	**24.2**
	Variance	*48.5*	*17.1*	*16.6*	*30.9*
E	Mean	**26.9**	**27.7**	**23.9**	**26.2**
	Variance	*54.5*	*24.2*	*23.4*	*35.2*
F	Mean	**25.4**	**26.2**	**21.9**	**24.5**
	Variance	*24.3*	*21.2*	*17.3*	*23.4*
Total	Mean	**26.9**	**27.5**	**23.9**	
	Variance	*45.0*	*22.5*	*24.5*	

Table 4-2: Summary Statistics Results Based on Largest and Negative ΔS (Right Lane)

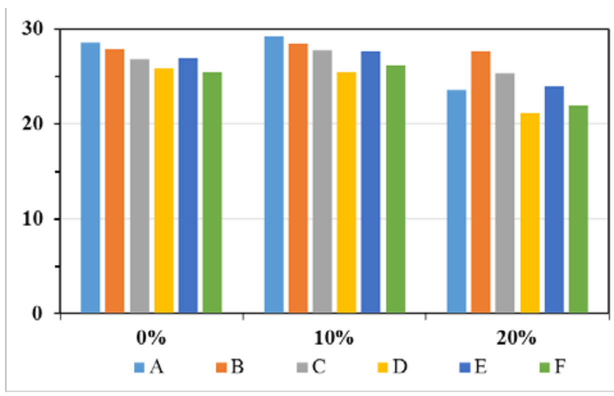
Schemes	Summary	Non-Compliant Driver Proportion 0%	10%	20%	Total
A	Mean	**29.7**	**29.1**	**23.7**	**27.5**
	Variance	*19.8*	*26.9*	*21.9*	*30.0*
B	Mean	**29.3**	**28.6**	**27.7**	**28.5**
	Variance	*43.5*	*27.8*	*31.7*	*33.2*
C	Mean	**28.5**	**26.9**	**26.2**	**27.2**
	Variance	*38.9*	*26.3*	*39.9*	*34.4*
D	Mean	**26.4**	**25.2**	**20.8**	**24.1**
	Variance	*30.8*	*13.2*	*23.2*	*27.4*
E	Mean	**28.3**	**28.8**	**24.5**	**27.2**
	Variance	*33.8*	*14.71*	*20.8*	*25.8*
F	Mean	**25.2**	**25.4**	**22.6**	**24.4**
	Variance	*11.8*	*20.4*	*22.5*	*19.2*
Total	Mean	**27.9**	**27.3**	**24.2**	
	Variance	*30.7*	*22.9*	*30.4*	

4.2.1 Speed Differential Results: Interaction between Schemes

As described previously, the proposed speed reduction schemes differ in length of the reduced speed areas as well as the speeds that are assigned to each of the reduced speed area segment. The magnitude of the speed reduction is also different from one reduced speed area to the next. Given that the proposed schemes contain both similarities and differences, it is interesting to note the effect of the same scheme applied to three different proportions of non-compliant drivers. Examining the first section of Table 4-1 it can be observed that in the case of the left lane when all drivers are compliant with the reduced speed area the averaged ΔS obtained is 25.4 mph. ΔS increases to 26.2 mph when the non-compliant driver proportion is increased to 10%; this translates into a larger speed differential from one 5-min interval to the next. When the non-compliant driver proportion is increased to 20% the ΔS declines to 21.9 mph, meaning that the ΔS is smaller when 20% of the driving proportion is non-compliant with the reduced speed areas. Scheme F, when introduced to the different driver proportions, averages ΔS of 24.5 mph, with a variance of 23.4 mph^2, resulting in a standard deviation of 4.84 mph.



(a) Scheme



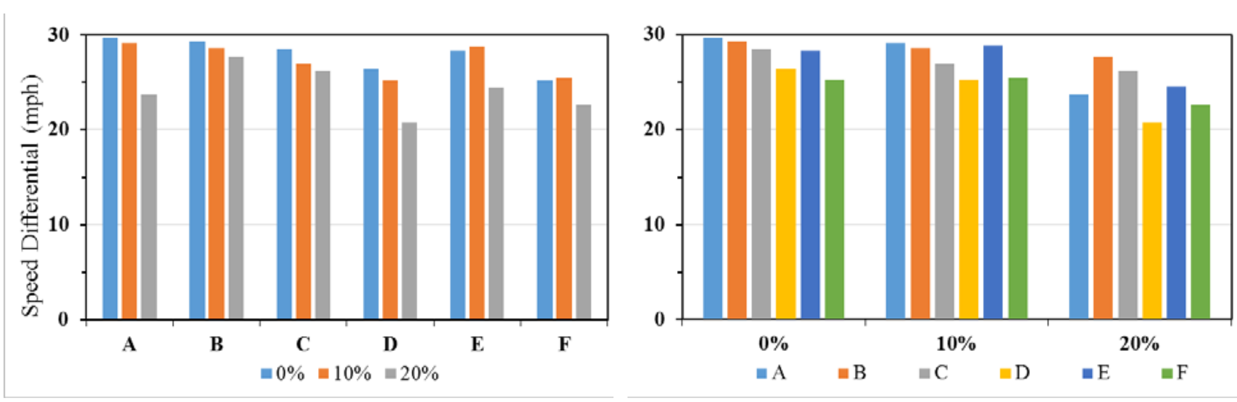
(b) Non-Compliant Driver Proportion

Figure 4–1: Speed Differential Results (Left Lane)

As observed in Figure 4–1(a) most schemes seem to have a slight increase in ΔS when a 10% of non-compliant driver proportion is introduced and then a larger decline in ΔS when the non-compliant driver proportion is increased to 20%, except for scheme D. Scheme D with a 0% non-compliant driver response averages at 25.9 mph, slightly decreases to 25.4 mph when introduced to 10% of non-compliant proportion, and continues to decrease even more drastically to 21.1 mph when non-compliant driver proportion is increased to 20%.

The right lane ΔS follows a similar trend to the simulation modeling of the left lane, as shown in Figure 4–2. Results of the six schemes seemed to have similar reductive effects on both 0% and 10% NCDPs. An increase in ΔS can be observed for the first increase of non-compliance in Table 4-1, except for scheme D that had a 0.5 mph decrease. When the second increase in NCDP is introduced, a significant reduction in ΔS is observed, the magnitude of this reduction varied from less than 0.8 mph to 5.7 mph.

Speed differential measures for the right lane are of slightly larger magnitude than those on the left lane by approximately 1 mph, and thus, the QWS is believed to have no significant effect on merging behavior.



(a) Scheme (b) Non-Compliant Driver Proportion

Figure 4–2: Speed Differential Results (Right Lane)

The main purposes of a QWS is to warn drivers prior to queued conditions so that drivers can reduce their speeds in a safe manner. It is reasonable to assume that the less abrupt the speed reduction is, the smaller the required reaction time would be. Speed differential results for both lanes indicate that the most preferred scheme under these conditions is scheme D based on a ΔS across all extents of non-compliant drivers, followed by speed reduction schemes F, E, C, A, and B. QWS applications are believed to benefit more from those schemes with lower speed differentials, as the likelihood of and end-of-queue crash occurring should decrease when speed differentials decrease.

4.2.2 SPEED DIFFERENTIAL RESULTS: NON-COMPLIANT PROPORTIONS

Three groups of non-compliant driver proportion are set to follow the speed reduction schemes, 0%, 10% and 20%. A 0% non-compliant compliant driver behavior within the VISSIM model intends to represent baseline conditions; all vehicles entering the 3.5-mile segment prior to the lane closure are set to follow the reduced speed areas. The non-compliant driver proportions of 0% and 10% present result in similar trends across all proposed schemes for both travel lanes.

When non-compliant proportion is increased to 20%, results observed for ΔS decrease in magnitude. A decrease in ΔS for 20% non-compliant drivers does not translate into a milder deceleration; it would represent freeway conditions when $1/5^{th}$ of the traveling public is not following the messages displayed by a QWS and therefore not reducing their speeds accordingly. Since VISSIM is unable to simulate crashes, this study is unable to directly prove whether the increase in non-compliance proportion would lead to an increase in rear-end crashes as well as a decrease in motorist and worker safety. Nonetheless, it is assumed that when non-compliant behavior is increased, the general risk of rear-end collisions at the end of the queue also increases.

4.3 TESTING THE RELATIVE IMPACTS OF DIFFERENT TREATMENTS

This section intends to determine whether the factors and combination of factors have an influence in the speed differential results obtained from the simulation model. The ANOVA test is a statistical technique that evaluates the potential variation between and within the interaction of independent factors. An ANOVA tests whether the statistical mean of several groups is equal. For the purpose of this test is able to determine if there is a statistically significant difference between the ΔS results based on the interaction between the two independent factors (schemes and NCDPs), as well as the effects due to the interaction within the factors.

4.3.1 ANOVA SUMMARY

The ANOVA test provides an answer to whether the interaction between the independent factors has a statistically significant influence on speed differential results. The ANOVA also determines if the interaction within the treatments has a statistically significant effect on ΔS results. In order to determine statistical significance this test is based on the assumptions stated by the following hypotheses statements:

$$H_1 : \mu_A = \mu_B = \mu_C = \mu_D = \mu_E = \mu_F$$

$$H_2 : \mu_{0\%} = \mu_{10\%} = \mu_{20\%}$$

$$H_3 : \mu_{A\&0\%} = \mu_{A\&10\%} = \ldots = \mu_{F\&20\%}$$

The first null hypothesis, H_1, states that there is no significant statistical difference among schemes and that they are all equal at the 95% confidence level, using α=0.05 as the threshold for statistical significance. There is sufficient evidence to reject the null hypothesis when one of the population means becomes not equal at α=0.05. H_2 proposes a similar statement, in regards to the three NCDPs. H_3 states that there is no statistically significant difference between each of the 18 cases.

This test also provides the following set of summary statistics: sum of squares (SS), degrees of freedom (df), mean squares between groups (MS), variation between sample means (F), as well as the p-value. The p-value determines that there is less than a 0.1% of observing our sample assuming that the null hypotheses are true. Assumptions made for the use of the two way ANOVA with replication are:

- Speed differential results are normally distributed
- Both groups of factors, speed reduction schemes and driver compliance proportions, have the same sample size
- The variances of the two groups of factors are equal
- Speed differential results obtained are independent from each other

4.3.2 DATA SET

The data set introduced to this analysis is comprised of speed differential results obtained from the 270 traffic simulation model runs. The ANOVA method first compared the variances observed in the speed differential results based on the effects of speed reduction schemes, resulting in 15 comparisons made per lane. Secondly, compared the variances observed in the speed differential results based on the effects of the non-compliant driver populations, resulting in three comparisons made per lane. Lastly, this statistical test compared the variances observed within the interaction of the two previously mentioned independent factors, thus evaluating the statistically significant difference between 153 possible combinations per lane.

4.3.3 COMPARISON

As shown in Table 4-3, in the case of the left lane the comparison between the different schemes resulted in a p-value of 0.005; meaning that the different proposed schemes prove to be significantly different from each other based on the speed differential results. Similarly, the comparison between different proportions of non-compliant drivers showed to be significantly different with a p-value of less than 0.001.

Table 4-3: ANOVA Two-Factor with Replication Results (Left Lane)

	SS	*df*	*MS*	*F*	*p-value*
Scheme	515	5	103	3.46	**0.005**
% Driver	653	2	326	11.0	**< 0.001**
Interaction	160	10	16.0	0.538	0.862

Table 4-4 shows that in the right lane, interaction among speed reduction schemes A-F was statistically significant with a p-value of less than 0.001, and differences in NCDPs resulted in a p-value of less than 0.001. Thus, indicating that same set of independent factors have a statistically significant effect on ΔS for both lanes of the modeled freeway.

Table 4-4: ANOVA Two-Factor with Replication Results (Right Lane)

	SS	*df*	*MS*	*F*	*p-value*
Scheme	721	5	144	5.55	**< 0.001**
% Driver	723	2	362	13.9	**< 0.001**
Interaction	206	10	20.6	0.791	0.637

The following statements can be claimed based on the analysis of variances:

- There is greater than 99.99% confidence that there is a statistically significant difference in ΔS between the six proposed speed reduction schemes for both lanes; $\text{p-value}_{left} = 0.004838$ and $\text{p-value}_{right} < 0.001$.
- There is 99.99% confidence that a statistically significant difference ΔS exists between the three non-compliant driver proportions for both lanes; $\text{p-value}_{left} < 0.001$ and $\text{p-value}_{right} < 0.001$.

- The interaction between proposed the six schemes and the three non-compliant driver proportions has no statistically significant effect on the ΔS results; p-value $_{left}$ = 0.862 and p-value $_{right}$ = 0.637.

4.3.4 RESULTS

Given the results obtained from the comparison section, it can be concluded that the when different sets of speed reduction schemes are compared based on its variance, an influence on the speed differential results does exist for both lanes. A parallel conclusion can be drawn when comparing the NCDPs; there is a statistically significant difference observed in speed differential results based on the comparison between the three groups of drivers for both lanes. However, when the speed differential results obtained from each of the 18 treatments are compared between each other, no statistically significant difference was observed. It can be concluded that there is no particular combination between the two independent factors that is deemed statistically significant different; no combination of the two independent factors is influential enough to cause speed differential to be statistically significantly different from the rest of the possible combinations.

For the purposes of this study, a secondary set of statistical measures was taken to determine which of the comparisons among speed reduction schemes and non-compliant driver proportions were statistically significantly different from each other, as previously indicated by the ANOVA method.

4.4 TESTING THE RELATIVE IMPACT OF SPEED REDUCTION SCHEMES

This section intends to take a step further to determine the identification of which combination of speed reduction schemes have a significant effect on the speed differential results obtained from the simulation model. In addition, this section also provides a comparison between the control case and the proposed scheme speed reduction ΔS results.

4.4.1 T-TEST SUMMARY

A two-sample t-test can determine which of the interactions of the speed reduction schemes and non-compliant driver proportion were most significantly distinctive from each other. In order to determine statistical significance this test is based on the assumptions stated by the following hypotheses statements:

$$H_0 : \mu_{CONTROL} = \mu_A = \mu_B = \mu_C = \mu_D = \mu_E = \mu_F$$

$$H_A : \mu_{CONTROL} \neq \mu_A \neq \mu_B \neq \mu_C \neq \mu_D \neq \mu_E \neq \mu_F$$

For this test, the null hypothesis, H_0, proposes that there are no differences between the population means drawn from the sample.

The alternative hypothesis, H_A, proposes that proportion means are not equal. As a result, the one-tailed hypothesis of H_0 at a significance level of α = 0.05 when a computed t – statistics exceeds $t_{\alpha/2,n-1}$ or is less than - $t_{\alpha/2,n-1}$. The method used to obtain the z score for the comparison between the different cases is described below; this equation is used to evaluate the statistical significant difference between the sample means.

$$Z = \frac{(\bar{X}_1 - \bar{X}_2) - (\mu_1 - \mu_2)}{\sqrt{\sigma_1^2/n_1 + \sigma_2^2/n_2}} \qquad \text{(Eq. 4-1)}$$

Where:

X: sample mean

μ: population mean

σ: standard deviation

n: sample size

Z-scores are then translated into cumulative probability areas under the probability density function curve for the theoretical normal distribution with the use of a one-tailed standard normal table where a p-value is then used in order to determine the significance of the interactions among both schemes and non-compliant driver proportion. Assumptions made for the use of this test are:

- Speed differential results are representative of the freeway segment in the study
- Random number seeds were randomly selected
- Speed differential results are independent from each other
- Speed reduction schemes and the control case are approximately normally distributed
- The variances of the control case and the speed reduction schemes are equal

4.4.2 DATA SET

The data set introduced to this test is comprised of speed differential results obtained from the 285 traffic simulation model runs, 270 for proposed schemes and 15 for control case. The control case conditions assumes that there is a wide variety of non-compliant driver proportion given that this case has not been introduced to speed reduction schemes compliance settings in the simulation model. The t-test method compares the variances observed in the speed differential results based on the effects of proposed speed reduction schemes and the control case, resulting in 21 comparisons made per lane.

4.4.3 COMPARISON

The six different speed reduction schemes results were compared against each other and as well as against the results from the control case in the simulation model. 21 different comparisons were analyzed and the results can be observed in Table 4-5 for the left lane and in Table 4-6 for the right lane. The probability of observing the sample assuming the null hypothesis is true is indicated by a p-value of 0.05 or less, interaction between factors that are statistically significantly different from each other are highlighted in bold in Table 4-5.

Table 4-5: Resulting p-Values among Schemes (Left Lane)

Scheme	Control	A	B	C	D	E	F
Control		0.052	0.147	**0.033**	**< 0.001**	**0.017**	**< 0.001**
A			0.219	0.341	**0.004**	0.208	**0.006**
B				0.136	**0.001**	0.070	**0.001**
C					**0.022**	0.357	**0.032**
D						**0.048**	0.379
E							0.071
F							

In the left lane, a statistically significant difference was found for seven interactions between schemes, and four schemes were found to have statistically significant difference when compared with the control case. The following statements can be derived from the information provided in Table 4-5:

- There is a statistically significant difference between the following combinations of treatment schemes based on speed differential results: A and D, B and D, C and D, D and E, F and A, F and B, and F and C.
- There is a statistically significant difference between the following list of schemes and the control case based speed differential results: C, D, E, and F for the left lane.

Table 4-6: Resulting p-Values among Schemes (Right Lane)

Scheme	Control	A	B	C	D	E	F
Control		0.352	0.374	0.305	**0.005**	0.284	**0.007**
A			0.182	0.424	**0.002**	0.402	**0.002**
B				0.143	**< 0.001**	0.118	**< 0.001**
C					**0.004**	0.483	**0.005**
D						**0.002**	0.382
E							**0.003**
F							

In the case of the right lane, seven statistically significant different interactions between schemes were observed, and only two speed reduction schemes showed to be statistically significant different from the control case. The following conclusive statements can be drawn from the information provided in Table 4-6:

- There is a statistically significant difference between the following combinations of treatment schemes based on speed differential results: A and D, A and F, B and D, B and F, C and D, C and F, D and E, and E and F for the right lane.
- There is a statistically significant difference between the following list of schemes and the control case based speed differential results: D and F for the right lane.

4.4.4 RESULTS

Several sets of combinations of speed reduction schemes have resulted to have a significant impact on speed differential results in one of the two lanes. The left lane resulted in 11 significantly different interactions between schemes, while the right lane indicated to have nine significantly different interactions between schemes this could be attributed to the occupational disparity between lanes.

The disparity between total amount of significant interactions for both lanes could be attributed due to the disparity in volumes between them. Given that, speeds at 0.5 mile prior to the beginning of the taper are affected by the incoming merging maneuvers. Speed differential could be affected by the right lane merging behavior due to an increase in volume and a decrease in traveling speeds. In the other hand, the left lane speed reductions are affected by an increase in traveling speeds due to a reduction in volume.

4.5 TESTING THE RELATIVE IMPACT OF NCDPs

This section intends to take a step further in order to determine the identification of which combination of non-compliant driver proportions have a significant effect on the speed differential results obtained from the simulation model.

4.5.1 t-TEST SUMMARY

A two-sample t-test can determine which of the interactions of the non-compliant driver proportions were most significantly distinctive from each other. To determine statistical significance this test is based on the assumptions stated by the following hypotheses statements:

$$H_0 : \mu_{0\%} = \mu_{10\%} = \mu_{20\%}$$

$$H_A : \mu_{0\%} \neq \mu_{10\%} \neq \mu_{20\%}$$

For this test, the null hypothesis, H_0, proposes that there are no differences between the population means drawn from the sample. This section essentially follows the same methodology included in the previous section.

4.5.2 DATA SET

The data set used in the determination of the relative impact of NCDPs on speed differential values, is obtained from the simulation model runs. Three sets of driver behaviors are introduced to the six speed reduction schemes, thus obtaining speed differential 90 results for each NCDP per lane. The t-test method compares the variances observed in the speed differential results based on the effects of driver compliance, resulting in 3 comparisons made per lane.

4.5.3 COMPARISON

The impact of the three NCDP on speed differential were compared with each other using the same methodology described in the previous section. The probability of observing the sample assuming the null hypothesis is true is indicated by a p-value of 0.05 or less, interaction between factors that are statistical significantly different from each other are highlighted in bold in Table 4-7.

Table 4-7: Resulting p-Values of Two-Way t-Statistics among Driver Proportions

Left Lane				Right Lane			
Proportion	0%	10%	20%	Proportion	0%	10%	20%
0%		0.238	**<0.001**	0%		0.266	**<0.001**
10%			**<0.001**	10%			**<0.001**
20%				20%			

The 20% non-compliant driver proportion was found to be significantly different from the other proportions for both lanes separately. No significantly different set of results were obtained from the 0% and 10% non-compliant proportion of drivers; this is also true for both lanes.

4.5.4 RESULTS

Findings obtained from t-statistical evaluation indicated that the 0% and 10% non-compliant driver proportion are influenced in a very similar manner by the proposed speed reduction schemes and thus, proving to not be statistical significantly different from each other. Evidently, the 20% non-compliant driver proportion obtained significantly different speed differential results when compared to the 0% and 20% NCDPs.

4.6 SUMMARY OF ANALYSES

The findings presented in this section indicate the significance of the contrast in speed differential results obtained from the simulation models of the different scenarios. First, it was concluded by the ANOVA that there is a statistically significant difference between the six speed reduction schemes and between the three non-compliant driver proportions. It was also found that there was no suggestion of statistical significance indicating that any interaction within the two independent factors would have an influence on speed differential results.

Second, the application of the t-test determined the specific interactions between speed reduction schemes that proved to be statistically significantly different from the control case (C, D, E, and F for the left lane and D, and F for the right lane). In addition, the interactions between the following list of speed reduction schemes showed to be statistical significantly different from each other: A and D, A and F, B and D, B and F, C and D, C and F, and D and E for both lanes.

Third, the use of t-tests found which interactions of NCDPs presented statistically significantly different speed reduction results. The 20% NCDP obtained significantly different speed differential results when compared to the 0% and 10% NCDPs for both lanes.

CHAPTER FIVE:
CONCLUSIONS AND RECOMMENDATIONS

5.1 INTRODUCTION

The goal of this study was to evaluate the effects of a queue warning system in a freeway work zone environment using traffic simulation software. This purpose was served by collecting field data, creating and calibration a simulation model, comparing the effect of different sets of speed distributions and driver compliance proportions prior to the lane closure. In addition, the set of results was then analyzed for statistical significant differences among all cases.

5.2 CONCLUSIONS

Work zone simulated conditions developed in the traffic simulation software VISSIM were able to replicate field-gathered traffic behavior, as well as, proposed schemes that simulated the effects of QWS on freeway traffic. Six schemes with different speed reduction locations and magnitudes were introduced to three proportions of non-compliant drivers. The QWS effects were measured according to the largest and negative speed average difference between the average traffic speeds in two consecutive 5-min intervals (ΔS); this was identified to be the moment where vehicles would be more likely to be involved in an end-of-queue crash. The statistical analysis was comprised of a two-way ANOVA with replication and two t-statistic evaluations. The ANOVA method was able to determine the statistically significant differences between proposed schemes, non-compliant driver proportions, and within treatment cases. The t-statistic test was then able to determine which proposed scheme and which non-compliant driver proportion was significantly different among each group. The major findings of the analysis are stated below:

- There is greater than 99.99% confidence that there is a statistically significant difference in ΔS between the six proposed speed reduction schemes for right lane ($p\text{-value}_{right} < 0.001$).
- There is greater than 99.95% confidence that there is a statistically significant difference in ΔS between the six proposed speed reduction schemes for left lane ($p\text{-value}_{left} < 0.005$).
- There is 99.99% confidence that a statistically significant difference in ΔS exists between the three non-compliant driver proportions for both lanes; $p\text{-value}_{left} < 0.001$ and $p\text{-value}_{right} < 0.001$.
- The interaction between proposed the six schemes and the three non-compliant driver proportions has no statistically significant effect on the ΔS results; $p\text{-value}_{left} = 0.862$ and $p\text{-value}_{right} = 0.637$.
- There is a statistically significant difference between schemes E and F for the right lane; $p\text{-value}_{right} = 0.0029$.
- There is a statistically significant difference for both lanes between the following combinations of schemes: A and D, A and F, B and D, B and F, C and D, C and F, and D and E for both lanes.
- There is a statistically significant difference between non-compliant driver proportions of 0% and 20% for both lanes; $p\text{-value}_{left} < 0.001$ and $p\text{-value}_{right} < 0.001$.
- There is a statistically significant difference between non-compliant driver proportions of 10% and 20% for both lanes; $p\text{-value}_{left} < 0.001$ and $p\text{-value}_{right} < 0.001$.

5.3 RECOMMENDATIONS FOR FUTURE RESEARCH

This research is a component of a larger ALDOT QWS statewide implementation effort. In continuation of this study, a deeper exploration of the traffic simulation tools and effectiveness evaluation can stem from the results of this research. Field applications of the speed reduction schemes proposed in this study could potentially be tested and evaluated in future QWS deployments. As well as more specific recommendations listed below:

- Explore a wider variety of non-compliant driver proportions and its effects on speed difference at the onset of queue formation.
- Explore a wider variety of speed reduction locations, lengths, and magnitudes that are used as inputs for 'Reduced Speed Areas'.
- Explore the differences among different work zone layouts, roadway classification, sites and its effects on the magnitude in speed differentials.
- For the purposes of this study, speed differential was chosen to be the most appropriate measure of effectiveness in order to explore the effects of a QWS simulation. For further research, queue-derived measurements can be included in the analysis as an indication of mobility and/or capacity.

Future work can originate from this project. The model developed through this effort can be applicable to other state highway agencies. Data collected at sites in other states or regions could increase the generalizability of the results. The following set of variables could be adjusted to customize the simulation:

- Roadway classification
- Percent of heavy vehicles
- Roadway geometry
- Work zone layout

- Traffic volume
- Speed limit
- Duration of the roadwork

Regarding application to current practice, in response to ΔS variable, the following list ranks in order, from most to least preferable, the recommended schemes D, F, E, C, A, and B (note this is true for both lanes). Modeling of sites in other states or regions could be used to verify this recommendation.

From a practical standpoint, the use of scheme 'D' proposes the use of 4 PCMSs in real deployments, which could translate into higher equipment costs for the agency, similarly with F and E. Although this is the preferred deployment configuration based on a moderate and gradual descent of travelling speeds, scheme 'C' has an average of speed differential 2.4 mph higher than the preferred configuration for the left lane and 3.1 mph higher for the left lane. The main practical difference between these interactions is the increase in equipment cost and deployment by a factor of 2, and only observing a decrease in speed differentials of less than 5 mph. These interactions were determined to be statistically significantly different from each other in based on the results obtained from the simulation model, however when this might not be considered to be of practical significant by some agencies.

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APPENDICES

APPENDIX A

SPEED DIFFERENTIAL SIMULATION RESULTS

Table A-1: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'A'		Non-Compliant Driver Proportion 0%	10%	20%
Random Number Seed	42	31.9	29.3	13.6
	43	26.6	24.4	22.7
	44	26.8	19.3	18.6
	45	15.4	26.1	25.9
	46	26.4	27.3	23.3
	47	26.2	24.9	25.7
	48	29.7	32.1	25.9
	49	26.3	31.5	24
	50	21.5	22.7	19
	51	26.1	29.1	19.9
	52	30.7	32	19.5
	53	26.5	26.6	20
	54	15.3	26.5	23.8
	55	22.2	25.1	29.8
	56	29.4	15.5	17.3

Table A-2: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'B'		Non-Compliant Driver Proportion 0%	10%	20%
Random Number Seed	42	23.9	20.2	16.2
	43	31.9	24.4	23.3
	44	29.1	23.1	19.8
	45	17.5	27.9	28.3
	46	18.5	28.7	25.7
	47	23.6	31	28.7
	48	35.9	35.5	30.6
	49	35.2	33.8	29.1
	50	26	32.5	23.1
	51	33.4	23.5	20.2
	52	37	31.6	26
	53	29.5	28.7	26
	54	21.3	27.9	16.5
	55	28.5	28.1	27.9
	56	12.1	18.6	17.6

Table A-3: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'C'		Non-Compliant Driver Proportion		
		0%	10%	20%
Random Number Seed	42	42.1	23.6	15.7
	43	33	25.4	19.7
	44	32.1	26.2	19.1
	45	25.5	30.8	27.4
	46	25.5	30.7	25.7
	47	32.6	30.1	26.8
	48	22.7	35	30.3
	49	28.6	35.2	27.6
	50	27.6	34.2	18.5
	51	26.6	24.7	21.5
	52	27.8	30.3	26.1
	53	30	32.2	24.9
	54	26	28.3	21.1
	55	25.5	30.2	29.6
	56	23.6	20.5	20.1

Table A-4: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'D'		Non-Compliant Driver Proportion		
		0%	10%	20%
Random Number Seed	42	26.2	22.7	21.2
	43	29.2	22.7	19.7
	44	26.4	25.9	14.5
	45	19.5	24.8	23.5
	46	23.5	27.7	17.1
	47	12.5	29.3	22.6
	48	34.5	32.3	21.8
	49	34.3	20.1	23.5
	50	25.3	31.4	16.8
	51	30.9	21.6	16.1
	52	35.8	27.5	21.9
	53	27.7	27.7	21.4
	54	22.3	22.8	31.1
	55	27.1	27.2	24.7
	56	13.4	17.8	21.2

Table A-5: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'E'		Non-Compliant Driver Proportion		
		0%	10%	20%
Random Number Seed	42	18.5	25.3	17.6
	43	34.3	24.3	22.4
	44	26.5	24.4	21.8
	45	19	25.9	29
	46	19.6	30.8	26.4
	47	18.2	29.4	27.9
	48	37	34.4	32.4
	49	35.9	33.3	29.3
	50	27.9	33.6	22.8
	51	33.9	22.9	22.5
	52	37.8	34.9	30.7
	53	30.5	28.3	29.7
	54	22.2	26.1	16.5
	55	27.8	25.8	29.6
	56	13	18.2	20.5

Table A-6: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'F'		Non-Compliant Driver Proportion		
		0%	10%	20%
Random Number Seed	42	24	21.4	19.8
	43	35.3	26.9	27.5
	44	29.1	24.7	22.9
	45	19.4	22.5	33.8
	46	19.9	31.1	29.1
	47	15.5	31.4	31.9
	48	37.7	36.4	34.8
	49	36.4	35.5	31.5
	50	27.5	31.1	24.2
	51	34.2	24.3	25.4
	52	39.2	33	28.5
	53	31.5	32.2	28.3
	54	23.1	29.8	23.1
	55	30.3	28.6	33.7
	56	15.9	18.6	20.9

Table A-7: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'A'		Non-Compliant Driver Proportion		
		0%	10%	20%
Random Number Seed	42	28.7	25.6	20.5
	43	28.8	31.6	25.6
	44	24.1	24.6	12
	45	17.7	24.5	20.6
	46	27.9	21.1	25
	47	24	27	23.5
	48	30.6	27.9	22.8
	49	22.7	30.2	26.8
	50	24.9	18.2	16.7
	51	24.6	27	20.5
	52	26.4	25.3	30.7
	53	26.2	26.1	17.5
	54	24.2	28.4	23.6
	55	19.9	29.5	25.9
	56	27.8	14.7	26.9

Table A-8: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'B'		Non-Compliant Driver Proportion		
		0%	10%	20%
Random Number Seed	42	18.6	25.8	18.6
	43	35.9	31.6	26.5
	44	25.7	28.3	13.6
	45	18.7	31.6	24.3
	46	24.1	25.6	29.7
	47	26.4	28.9	26.5
	48	38.4	33.6	26.6
	49	29.6	35	27.7
	50	31.4	28	26.4
	51	28.8	30.1	23.3
	52	33.5	27.2	28.5
	53	33	28.7	24.1
	54	30.7	27	19.1
	55	25.8	31.4	22.1
	56	23.2	19.1	30.1

Table A-9: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'C'		Non-Compliant Driver Proportion		
		0%	10%	20%
Random Number Seed	42	41.9	22.4	21.6
	43	28.1	31.1	25.2
	44	23.6	27.8	12
	45	23.8	31.6	22.2
	46	30.1	27.9	27.7
	47	33.1	32.3	24.2
	48	28.7	34.5	24.6
	49	30.7	36	27.8
	50	32.4	23.9	18.7
	51	25.8	32	20.6
	52	28	29.6	29.2
	53	29.7	29.4	23.8
	54	32.4	29.8	22.6
	55	27.1	33.1	20.8
	56	30.8	15.7	31

Table A-10: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'D'		Non-Compliant Driver Proportion		
		0%	10%	20%
Random Number Seed	42	22.2	21.7	20.3
	43	33.6	31.6	23.8
	44	25	25.1	12
	45	16.8	23.7	16.8
	46	25.1	22.2	24.6
	47	24.8	26.2	21
	48	36.4	29	21.4
	49	27.8	29.7	22.8
	50	29.5	20.6	13.2
	51	18.5	25.5	16
	52	31.8	24.3	24.4
	53	31	24.7	22.6
	54	28.1	24.2	27.1
	55	25	30	17.3
	56	20.5	19	28.2

Table A-11: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'E'		Non-Compliant Driver Proportion		
		0%	10%	20%
Random Number Seed	42	21.5	25.5	22
	43	36.3	30.3	30.5
	44	25.5	27.7	14
	45	15.3	26.4	23.5
	46	24.6	22.8	33.9
	47	27.7	29.1	26.7
	48	38.7	32.2	28.7
	49	29.8	32.8	29.6
	50	31.7	22.3	19.5
	51	29.5	28.5	25
	52	34.7	25.6	37
	53	33.1	28.2	30.3
	54	31.2	27.9	16.8
	55	27.1	32.4	24.7
	56	21.4	12.4	30.6

Table A-12: Magnitudes of Speed Differential Results (ΔS) for the Left Lane (mph)

Scheme 'F'		Non-Compliant Driver Proportion		
		0%	10%	20%
Random Number Seed	42	16.8	24.8	20.6
	43	38.6	29	32.5
	44	27.4	29.6	24.8
	45	19.1	31.6	27.8
	46	24.5	26.2	35
	47	28.1	30.9	29
	48	39.5	35.1	31.5
	49	30.6	35.4	30.4
	50	31.2	23.6	19.6
	51	29.6	30.4	24.1
	52	35.6	28.9	37.6
	53	34.1	28	30.4
	54	32	26.8	19.5
	55	29.9	34.7	22.4
	56	21.9	14.6	30.1

APPENDIX B

FIELD-GATHERED VOLUMES AND VISSIM INPUTS

Table B-1: Field Gathered Volumes and VISSIM Inputs from 1:00 PM - 8:00 PM

Time Interval	Original Volume (96)	Volume vph (96)	Original Volume (100)	Volume vph (100)
13:00 - 13:05	36	432	42	504
13:05 - 13:10	23	276	42	504
13:10 - 13:15	24	288	37	444
13:15 - 13:20	26	312	30	360
13:20 - 13:25	17	204	37	444
13:25 - 13:30	30	360	41	492
13:30 - 13:35	28	336	36	432
13:35 - 13:40	30	360	47	564
13:40 - 13:45	38	456	39	468
13:45 - 13:50	26	312	47	564
13:50 - 13:55	21	252	40	480
13:55 - 14:00	32	384	40	480
14:00 - 14:05	33	396	46	552
14:05 - 14:10	24	288	37	444
14:10 - 14:15	30	360	39	468
14:15 - 14:20	28	336	41	492
14:20 - 14:25	30	360	44	528
14:25 - 14:30	25	300	38	456
14:30 - 14:35	24	288	43	516
14:35 - 14:40	27	324	41	492
14:40 - 14:45	27	324	39	468
14:45 - 14:50	38	456	38	456
14:50 - 14:55	28	336	31	372
14:55 - 15:00	25	300	35	420
15:00 - 15:05	25	300	37	444
15:05 - 15:10	24	288	39	468
15:10 - 15:15	35	420	44	528
15:15 - 15:20	27	324	40	480
15:20 - 15:25	21	252	39	468
15:25 - 15:30	28	336	35	420
15:30 - 15:35	29	348	40	480
15:35 - 15:40	29	348	39	468
15:40 - 15:45	31	372	41	492
15:45 - 15:50	20	240	32	384
15:50 - 15:55	24	288	42	504
15:55 - 16:00	23	276	37	444
16:00 - 16:05	23	276	37	444
16:05 - 16:10	29	348	41	492
16:10 - 16:15	24	288	36	432
16:15 - 16:20	42	504	44	528
16:20 - 16:25	25	300	41	492
16:25 - 16:30	43	516	26	312
16:30 - 16:35	28	336	29	348
16:35 - 16:40	33	396	34	408

16:40 - 16:45	29	348	39	468
16:45 - 16:50	28	336	29	348
16:50 - 16:55	26	312	44	528
16:55 - 17:00	29	348	39	468
17:00 - 17:05	24	288	35	420
17:05 - 17:10	24	288	36	432
17:10 - 17:15	30	360	26	312
17:15 - 17:20	32	384	38	456
17:20 - 17:25	25	300	33	396
17:25 - 17:30	32	384	36	432
17:30 - 17:35	23	276	36	432
17:35 - 17:40	26	312	46	552
17:40 - 17:45	21	252	25	300
17:45 - 17:50	25	300	42	504
17:50 - 17:55	25	300	35	420
17:55 - 18:00	24	288	38	456
18:00 - 18:05	18	216	28	336
18:05 - 18:10	24	288	35	420
18:10 - 18:15	14	168	25	300
18:15 - 18:20	24	288	36	432
18:20 - 18:25	20	240	34	408
18:25 - 18:30	14	168	31	372
18:30 - 18:35	26	312	28	336
18:35 - 18:40	20	240	36	432
18:40 - 18:45	16	192	33	396
18:45 - 18:50	17	204	26	312
18:50 - 18:55	18	216	31	372
18:55 - 19:00	18	216	29	348
19:00 - 19:05	19	228	33	396
19:05 - 19:10	17	204	36	432
19:10 - 19:15	17	204	21	252
19:15 - 19:20	14	168	30	360
19:20 - 19:25	13	156	23	276
19:25 - 19:30	20	240	31	372
19:30 - 19:35	14	168	26	312
19:35 - 19:40	19	228	26	312
19:40 - 19:45	19	228	35	420
19:45 - 19:50	15	180	23	276
19:50 - 19:55	18	216	25	300
19:55 - 20:00	15	180	24	288

APPENDIX C

DATA COLLECTION FIELD PICTURES

Figure C-1: Installation of NC-350 on I-59/I-20 Nick Jehn and Dr. Rod Turochy

Figure C-2: Rolling Road Block Operation Assisted by AL State Tropper and ALDOT

Figure C-3: Installation of NC-350 on I-59/I-20 Veronica Ramirez and Dr. Rod Turochy

Figure C-6: Installation of NC-350 on I-59/I-20 Veronica Ramirez and Dr. Rod Turochy