FINAL REPORT

PROJECT J

FEBRUARY 2022

Improving Work Zone Mobility through Planning, Design and Operations

Dr. Rod Turochy, Auburn University
Dr. Virginia P. Sisiopiku, University of Alabama at Birmingham
Dr. James Tsai, Georgia Institute of Technology
Dr. Billy Williams, North Carolina State University
Nicholas L. Jehn, Auburn University
Dr. Ossama Ramadan, University of Alabama at Birmingham
Troyee Saha, University of Alabama at Birmingham

STRIDE | Southeastern Transportation Research, Innovation, Development and Education Center

UF | Transportation Institute UNIVERSITY of FLORIDA

TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No. Project J	**2. Government Accession No.**	**3. Recipient's Catalog No.**
4. Title and Subtitle Improving Work Zone Mobility through Planning, Design, and Operations		**5. Report Date** February 28, 2022 **6. Performing Organization Code**
7. Author(s) Dr. Rod Turochy, Auburn University Dr. Virginia Sisiopiku, University of Alabama at Birmingham Dr. James Tsai, Georgia Institute of Technology Dr. Billy Williams, North Carolina State University Dr. Nicholas L. Jehn, Auburn University Dr. Ossama Ramadan, University of Alabama at Birmingham Dr. Troyee Saha, University of Alabama at Birmingham		**8. Performing Organization Report No.** STRIDE Project J
9. Performing Organization Name and Address Auburn University, Auburn Alabama 36849 University of Alabama at Birmingham, Birmingham, AL 35294 Georgia Institute of Technology, Atlanta, GA 30332 North Carolina State University, Raleigh, NC 27695		**10. Work Unit No.** **11. Contract or Grant No.** Funding Agreement Number 69A3551747104
12. Sponsoring Agency Name and Address **University of Florida Transportation Institute** **Southeastern Transportation Research,** **Innovation, Development and Education Center (STRIDE)** 365 Weil Hall, P.O. Box 116580 Gainesville, FL 32611 **U.S Department of Transportation/Office of Research, Development & Tech** 1200 New Jersey Avenue, SE Washington, DC 20590 United States		**13. Type of Report and Period Covered** 1/19/2017 to February 28, 2022 **14. Sponsoring Agency Code**
15. Supplementary Notes		

16. Abstract

This research project consists of a series of tasks intended to yield results that will better inform transportation agencies in their planning, design, and operations of freeway work zones. This report is comprised of four parts reflecting the work of the four institutions involved. First, a traffic simulation model of a freeway lane closure was developed and calibrated. This part of the research also addressed the validity of default parameters in Vissim for time headway and truck acceleration capabilities. Additionally, a stochastic approach to evaluating freeway work zone capacity was proposed and a test case developed. The second part of this report documents an effort to evaluate the effects of early merge and late merge scenarios across a range of freeway lane closure and traffic volume cases. The third part describes the use of video images to identify driver behavior patterns at freeway work zones across a range of traffic conditions and roadway geometric configurations. The fourth part describes mesoscopic traffic modeling of freeway work zones and how the results affected work zone operations to mitigate congestion. The report as a whole addresses many aspects of freeway work zones, including traffic simulation modeling and adjustment to several default model parameters, a simulation experiment comparing early and late merge scenarios, the use of video images processing to support modeling of driver behavior parameters, and a summary of other related work..

17. Key Words work zones, highway operations, freeways, driver behavior	**18. Distribution Statement** No restrictions		
19. Security Classif. (of this report) N/A	**20. Security Classif. (of this page)** N/A	**21. No. of Pages** 182 Pages	**22. Price** N/A

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the information presented herein. This document is disseminated in the interest of information exchange. The report is funded, partially or entirely, by a grant from the U.S. Department of Transportation's University Transportation Centers Program. However, the U.S. Government assumes no liability for the contents or use thereof.

ACKNOWLEDGEMENT OF SPONSORSHIP AND STAKEHOLDERS

This work was sponsored by a grant from the Southeastern Transportation Research, Innovation, Development, and Education Center (STRIDE), a Regional University Transportation Center sponsored by a grant from the U.S. Department of Transportation's University Transportation Centers Program. This work was also sponsored by the Alabama Department of Transportation (ALDOT).

Funding Agreement Number: 69A3551747104

LIST OF AUTHORS

Lead PI:

Dr. Rod Turochy, Auburn University
rodturochy@auburn.edu
ORCID 0000-0003-2294-7388

Co-PIs:

Dr. Virginia P. Sisiopiku, University of Alabama at Birmingham
vsisiopi@uab.edu
ORCID 0000-0003-4262-8990

Dr. James Tsai, Georgia Institute of Technology
james.tsai@ce.gatech.edu
ORCID 0000-0002-6650-2279

Dr. Billy Williams, North Carolina State University
billy_williams@ncsu.edu
ORCID 0000-0002-7599-1385

Additional Researchers:

Nicholas L. Jehn, Auburn University
Master's Student
nzj0017@auburn.edu

Dr. Ossama Ramadan, University of Alabama at Birmingham

Troyee Saha, University of Alabama at Birmingham
Master's Student
troyee91@gmail.com

Contents

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

This research project consists of a series of tasks intended to yield results that will better inform transportation agencies in their planning, design, and operations of freeway work zones. This report is comprised of four parts reflecting the work of the four institutions involved. First, a traffic simulation model of a freeway lane closure was developed and calibrated. This part of the research also addressed the validity of default parameters in Vissim for time headway and truck acceleration capabilities. Additionally, a stochastic approach to evaluating freeway work zone capacity was proposed and a test case developed. The second part of this report documents an effort to evaluate the effects of early merge and late merge scenarios across a range of freeway lane closure and traffic volume cases. The third part describes the use of video images to identify driver behavior patterns at freeway work zones across a range of traffic conditions and roadway geometric configurations. The fourth part describes mesoscopic traffic modeling of freeway work zones and how the results affected work zone operations to mitigate congestion. The report as a whole addresses many aspects of freeway work zones, including traffic simulation modeling and adjustment to several default model parameters, a simulation experiment comparing early and late merge scenarios, the use of video images processing to support modeling of driver behavior parameters, and a summary of other related work.

EXECUTIVE SUMMARY

The purpose of this research project was to produce information that transportation agencies could use to manage their work zones, particularly those involving lane closures on freeways. This research project examined several aspects of modeling traffic flow through freeway work zones so that agencies can ultimately develop more realistic traffic simulation models and be armed with information on the effectiveness of various temporary traffic control strategies.

Part 1 of this report documents research conducted at Auburn University involving the development and calibration of a traffic simulation model of rural freeway work zone with a 2-to-1 lane closure using the Vissim software. As the research progressed, the need to view capacity of the work zone as stochastic, rather than a static, deterministic value, became evident as the observed traffic flow rates immediately prior to breakdown conditions (and resulting queue formation) were not constant. During the model calibration process, adjustments were made to default values in Vissim for two key parameters, time headway and truck acceleration; recommendations for traffic modelers were made accordingly. Further exploring capacity stochastically, an experiment design that would result in estimation of the probability of a breakdown, as a function of key traffic parameters and work zone characteristics, was developed. A small portion of that design was then developed as a pilot case, setting the stage for future research that could result in a risk-based (probabilistic) approach to estimating freeway work zone capacity and queue formation probability that agencies could use to better inform scheduling of work zone activities.

In Part 2, research conducted at the University of Alabama at Birmingham, traffic simulation models of 3-to-1 and 3-to-2 lane closures were developed for several different merge control techniques in Vissim. These models were executed across a range of traffic flow rates with early and late merge strategies. The traffic impacts of these different combinations were then quantified. Recommendations regarding spatial and temporal placement, and selection of associated temporary traffic control strategies, were developed.

Part 3 describes work performed at Georgia Tech regarding the use of video log images to demonstrate the feasibility of using commonly available images of work zones to extract and study driver behaviors (merging timing/locations) in varying roadway geometries (straight and curved sections). On a curved road, cars and trucks tend to merge closer to a work zone taper than they do when traveling on straight roads; cars are more likely to merge closer to the work zone taper than trucks. This suggests that the traffic control devices should be placed at sufficient sight distances that drivers, especially on curved roadway segments, can have sufficient time to react to roadway conditions safely. Additional data and analysis are recommended to confirm the observations in this study.

Part 4 of this report describes an effort at North Carolina State University to describe and document key findings and lessons learned from a series of NCDOT and NCHRP work zone-related research projects. One project utilized a mesoscopic network modeling tool, DTALite,

to model the impact of various lane closure and work scheduling scenarios. The results of this project motivated a change by NCDOT and the construction contractor to the planned work zone design and schedule which significantly mitigated the negative work zone impacts. The second project coincided with the execution of the work zone plans that were informed by the first project. This project also used DTALite in an ongoing modeling effort that incorporated real-time traffic information and supplemented the network modeling with targeted FREEVAL modeling of key multi-segment facilities within the overall work zone. The final project provided a summary of freeway work zone capacity analysis methods.

PART 1 FREEWAY LANE CLOSURE: SIMULATION MODELING AND PROBABILISTIC FRAMEWORK TO ESTIMATE CAPACITY

1.0 INTRODUCTION

1.1 Background

As of 2015, it is estimated that there are 8.8 million lane-miles of public roadway in the United States. This constitutes only an 11% increase from the approximate 7.9 million lane-miles in 1980, whereas the number of vehicle miles traveled (VMT) has increased by 104% during the same 35-year period (Federal Highway Administration 2016). The Texas A&M Transportation Institute (TTI) recently partnered with INRIX to publish comprehensive nationwide congestion data showing the consequences of this trend. In 2014, 160 billion dollars and 3.1 billion gallons of fuel were lost to travel delay experienced by road users, who spent an average of 42 additional hours in traffic (Schrank et al. 2015). Increased travel and congestion also place an immense responsibility on state and local agencies, who have been forced to shift focus to the maintenance, rehabilitation, and expansion of the nation's crumbling infrastructure. During the 10-year period from 2002 to 2012 alone, the percentage of Federal-aid highways with an acceptable pavement ride quality decreased from 87.4% to 80.3% (Federal Highway Administration 2015). Thus, it is apparent that both traffic congestion and roadway deterioration are increasing faster than agencies can respond.

Accordingly, much of the National Highway System (NHS) is under construction each year. A survey of work zone activity during the summers of 2001 and 2002 found this to be the case for 20-27% of all public roadway mileage (Federal Highway Administration 2017). Given the trends discussed earlier, it would not be surprising if even more of the nation's highways were under construction during the peak season today. As such, it is concerning that work zones are responsible for approximately 24% of non-recurring congestion, a category including incidents, weather, and special events that accounts for 40% of all the delay discussed previously (Federal Highway Administration 2017). Although most non-recurring congestion is unplanned and uncontrollable, agencies can strive to design and operate work zones in a fashion that minimizes mobility implications. Of particular concern are freeway work zones, as such facilities carry 25.1% of all traffic while accounting for only 1.3% of all lane-miles on the NHS (Federal Highway Administration 2016). Fortunately, this disparity makes even small congestion mitigation efforts impactful.

1.2 Problem Statement

Despite compelling evidence supporting careful attention to freeway work zone design and operations, agency decision making practices are often not data driven. For example, a survey of state Departments of Transportation (DOTs) in 2016 found that traffic control strategies at freeway work zones were chosen based on experience alone by 40% of agencies (Sisiopiku and Ramadan 2016). Decision variables affecting the choice of daytime versus nighttime work have been relatively consistent, as several surveys have cited high daytime traffic, safety, traffic control, and road user costs as the most influential scheduling factors (Hancher and Taylor 2001; Park et al. 2002; Rebholz et al. 2004). However, data suggests that nighttime and off-peak operations will not always optimize safety and mobility. In 2014, 41% of all congestion occurred during off-peak hours, so shifting operations to these time periods does not always eliminate mobility issues (Schrank et al. 2015). Furthermore, drivers are more likely to expect free-flowing conditions during off-peak and overnight hours, so crash risk and severity are both increased despite decreased exposure. In 2015, only 15.3% of all crashes occurred between the hours of 9PM and 6AM, but this time period accounted for 32.8% of fatal work zone crashes (NHTSA 2015, 2016a).

Even when off-peak or nighttime operations improve safety and mobility for road users, they do so at the expense of decreased worker safety and productivity and increased construction costs. Several research efforts have performed sensitivity analyses and developed optimization models to strike a balance between these variables (Abdelmohsen and El-Rayes 2016; Jiang and Adeli 2003; Tang and Chien 2008). While the results of these studies show that work zone design and operations can be optimized to decrease both road user and construction costs, the results do not provide guidance applicable on a case-by-case basis. Such guidance requires a more precise measurement of freeway work zone capacity, queueing, and delay through field data or simulation. In any case, there is compelling evidence to support that work zone scheduling decisions should be better informed than they often are.

As will be discussed in the next section, the measurement and definition of freeway work zone capacity has been a topic of debate for several decades, leaving agencies with little formal guidance on predicting the behavior of traffic flow at given volumes for various work zone configurations. Recently, however, well-calibrated microsimulation models have shown promise as a work zone traffic analysis tool. Although these studies have provided guidance to practitioners on developing site-specific microsimulation models, many agencies may not wish to invest the time or have the resources required to carry out such analyses. Consequently, simple deterministic tools are still widely used in making freeway work zone design and operations decisions, even though traffic flow and breakdown are stochastic phenomena. This research aims to address these shortcomings by laying the groundwork for a freeway work zone traffic analysis tool

based on probabilistic capacity estimates to support agency decision making. Lastly, though urban freeway facilities carry approximately 70% of all interstate traffic in the United States, they compose less than half of the total lane mileage for this functional classification (Federal Highway Administration 2016). Despite the lack of traffic exposure, rural freeway facilities also exhibit a fatality rate 1.7 times higher than their urban counterparts, placing increased importance on their design and operation (NHTSA 2016b; NHTSA 2015). Therefore, the primary focus of this research task was on developing guidance for the modeling and analysis of rural freeway work zones. As the research progressed, specific focus areas for this research task were identified as characterization of freeway work zone capacity stochastically (rather than deterministically as has traditionally been done), examination of default traffic simulation model parameters pertaining to capabilities of heavy trucks, and a preliminary exploration of a freeway work zone lane closure analysis tool that characterizes the risk of traffic flow breakdown as a function of key traffic characteristics.

1.3 Research Objectives

This task, in support of the larger parent STRIDE project, involves the quantification of queueing and delay as a function of traffic demand and other explanatory variables so that agencies can determine the impact of scheduling lane closures by time of day and day of the week. While this research was not meant to be comprehensive, the following objectives were sought:

1. Develop, calibrate, and assess the validity of a microscopic simulation model for a 2-to-1 (two-lanes-to-one) rural freeway work zone lane closure. This effort includes potential modification of default values in the traffic simulation modeling software VISSIM pertaining to heavy truck characteristics and time headways.
2. Use microsimulation outputs to construct an initial base set of breakdown probability models for rural freeway work zones with varying demand volume, truck percentage, and lane closure side to determine the effect of these variables on the likelihood of queue formation. This is intended to capture the dynamic – or stochastic – nature of the capacity of freeway lane closures and provide a foundation for the development of models that characterize the risk – or probability – of traffic flow breakdown based on key traffic characteristics such as traffic volume and composition.

Each of the objectives above center around the fundamental idea that demand-based work zone planning, design, and operations decisions should be based on stochastic estimates of capacity, rather than deterministic pre-breakdown or queue discharge flow rates. In other words, rather than viewing the capacity of a freeway work zone with a

lane closure as a single value (below which traffic flow is uncongested and above which breakdown occurs and queues form), this study conceives of capacity as a range of volumes over which a breakdown condition is increasingly likely to occur as flow increases. As noted in Chapter Two, such an assertion agrees with the state of the art in capacity measurement and allows for agencies to make defensible, data-driven decisions, rather than experience-based assumptions. Furthermore, it focuses on the prevention of queueing, rather than just queue mitigation. While queueing and delay are not unavoidable at every site, they should be eliminated when possible and minimized when inevitable.

2.0 LITERATURE REVIEW

2.1 Introduction

This chapter synthesizes relevant literature on the topic of measuring and predicting freeway work zone capacity using field data and simulation. First, historical context will be given regarding past and current freeway work zone capacity methodology in the Highway Capacity Manual (HCM) and the research that has led to the state of the practice. Second, factors that have been found to influence freeway work zone capacity will be discussed to demonstrate where further research is necessary. The process by which breakdown probability models may be developed to estimate capacity from mathematical distributions will then be explored. Finally, case studies will be summarized in which microsimulation models have been created and calibrated to measure freeway work zone capacity under various conditions.

2.2 Historical Measurement and Definition of Freeway Capacity

The 6th edition of the HCM defines capacity as “the maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, environmental, traffic, and control conditions” (Transportation Research Board 2016). Although this definition has long remained unchanged, debate on the measurement of capacity at freeway facilities began in the 1960s, when several authors documented a discontinuity between capacity under stable flow and that under unstable flow (Drake et al. 1967; Edie 1961). Roess and Prassas synopsize nearly five decades of research on this topic, during which most researchers agreed the maximum throughput of a freeway facility drops after the transition from non-congested to congested conditions, but no consensus was drawn on how to define freeway capacity in the presence of bottlenecks such as work zones (Roess and Prassas 2016). This point of conflict has more commonly been referred to as “the two-capacity phenomenon” and has led to inconsistencies in practice, as documented in a more recent study (Yeom et al. 2015). Specifically, Yeom et al. argued that if capacity is to be defined as a

"sustainable" flow rate, the queue discharge rate after breakdown may be a more appropriate measure than pre-breakdown capacity.

In a similar way, there has been a growing body of research since the mid-1990s that suggests freeway capacity cannot be defined as a single value, but rather should be represented by a probability distribution (Brilon et al. 2005; Elefteriadou et al. 1995; Lorenz and Elefteriadou 2001; Minderhoud et al. 1997; Persaud et al. 1998). This concept was backed by research with field data providing evidence that breakdown is not a deterministic event but is stochastic in nature and can vary by several hundred vehicles per hour under identical prevailing conditions (Lorenz and Elefteriadou 2001). Thus, if capacity is to be identified by the onset of breakdown, both capacity and breakdown should be considered random variables and estimated using mathematical distribution functions.

In the early 2000s, TRB's Highway Capacity and Quality of Service Committee appointed a task force to provide clarity on these issues, and their findings were presented in 2006 in Yokohoma, Japan (Elefteriadou et al. 2006). Elefteriadou built on these findings and summarized the "state of the art in capacity measurement" in her book, An Introduction to Traffic Flow Theory (Elefteriadou 2014), where the following conclusions were drawn:

1. Breakdown is probabilistic and its occurrence does not necessarily coincide with the highest observed flow rate.
2. Capacity is a random variable and will vary by several hundred vehicles per hour per lane, even under identical prevailing conditions.
3. There are multiple time periods of interest during which flow measurements may be taken to define capacity: well in advance of breakdown, just prior to breakdown, and during congested conditions after breakdown.
4. Regardless of whether the pre-breakdown capacity (PBC) or queue discharge rate (QDR) is chosen to measure capacity, single values should be estimated from distributions obtained over many breakdown events.

Despite these discoveries, the stochastic nature of freeway work zone capacity has yet to be formalized in the core HCM methodology, though Chapter 26 of the 6th Edition demonstrates such a methodology for recurring bottlenecks (Transportation Research Board 2016). Most recently, a few studies have recommended that probabilistic methods be applied to describe work zone capacity but work in this area is limited to date (Heiden and Geistefeldt 2016; Weng and Yan 2016; Weng and Yan 2014). As such, measurement of capacity at freeway work zones remains ambiguous and practice varies among agencies. Studies conducted by the Indiana and Missouri Departments of Transportation suggest that PBC may be appropriate when practitioners wish to avoid congestion altogether, while QDR may be more meaningful when congestion is

expected but queue mitigation is desired (Bham et al. 2011; Jiang 1999). Nonetheless, recent studies have finally established a numerical connection between PBC and QDR by synthesizing freeway work zone capacity values in literature and from field data (Hu et al. 2012; Yeom et al. 2015), the results of which will be discussed next.

2.3 Work Zone Capacity in the Highway Capacity Manual

Given that freeway work zones often involve lane closures that function as bottlenecks, the issues discussed above are paramount in determining capacity at such locations. Both HCM 2000 and HCM 2010 presented short-term work zone capacity methodology based on studies conducted in Texas from 1987 to 1991 (Krammes and Lopez 1994). These studies were limited not only in the sense that they contained data from a single state, but that they only considered four variables: intensity of work activity, presence of ramps, presence of heavy vehicles, and number of open lanes through the work zone. Furthermore, the adjustments for work intensity and presence of on-ramps were to be done manually using engineering judgement, with little numerical guidance given. For long-term work zones, a table of average values under various lane closure configurations in several states was given and practitioners were advised to adjust these based on local experience (Chatterjee et al. 2009).

With the recent publishing of the 6th edition of the HCM, formal, detailed guidance was given on determining work zone capacity. Yeom et al. built on a past study conducted by several of the co-authors (Hu et al. 2012) to perform an extensive literature search, establish a relationship between QDR and PBC, and provide a regression model for estimating work zone capacity under various conditions (Yeom et al. 2015). The model currently included in the 6th edition of the HCM is given in Figure 1-1 and was created from 90 archival literature sources and 12 field-collected datasets (TRB 2016).

Most significant to note is that work zone capacity has now been formally defined in terms of the average queue discharge rate occurring after breakdown, rather than by the maximum pre-breakdown flow rate (TRB 2016). The language in the 6th edition of HCM and that of the authors of the works associated with NCHRP Project 03-107, the basis for the work zone capacity methodology update, still note the importance of PBC. In fact, wording within the HCM implies that freeway capacity should still be defined by the maximum flow prior to breakdown. However, as noted by Yeom et al., QDR is much easier to measure than PBC and provides a more practical means of obtaining freeway work zone capacity (Yeom et al. 2015). Thus, it was ultimately proposed that freeway work zone capacity be estimated in terms of QDR, then converted to PBC if desired by using a default conversion factor of +13.4% or one obtained from local data.

$$\text{average QDR} = 2{,}093 - 154 \times f_{\text{LCSI}} - 194 \times f_{\text{barrier}} - 179 \times f_{\text{area}} + 9 \times f_{\text{lateral\_12}} - 59 \times f_{\text{day\_night}}$$

where

$$f_{\text{LCSI}} = \frac{1}{\text{number of open lanes} \times \text{open ratio}}$$

average QDR = average queue discharge flow rate (pcphpl);
f_{barrier} = 0: concrete, 1: cone or drum;
f_{area} = 0: urban, 1: rural;
$f_{\text{lateral\_12}}$ = lateral distance—12 (minimum −11.9, maximum 0) (ft); and
$f_{\text{day\_night}}$ = 0: day, 1: night.

Figure 1-1: HCM 6th Edition work zone capacity model (Source: TRB 2016)

2.4 Factors Affecting Freeway Work Zone Capacity

Current understanding of freeway work zone capacity is primarily based on field-collected data in various states, where the effect of several traffic stream, environmental, roadway, and work zone characteristics on throughput have been studied. Weng and Meng performed an extensive literature search in a 2013 study, where the following factors were found to influence work zone capacity in past research (Weng and Meng 2013):

1. Traffic Stream Characteristics: heavy vehicle percentage, driver composition
2. Environmental Characteristics: time of day, weather, locale
3. Roadway Characteristics: roadway functional classification, grade, presence of on-ramps
4. Work Zone Characteristics: number of open and closed lanes, lane closure side, work zone length, work intensity, work zone duration, work zone speed limit

However, the significance and relative effect of many traffic- and work zone-related factors have been debated in the literature, as will be discussed next.

2.4.1 Traffic stream characteristics

It is well understood that heavy vehicles, which differ in size and performance from passenger cars, influence capacity. For this reason, HCM methodologies include passenger car equivalency factors (PCEs) to facilitate comparison of estimated capacity values when the percentage of trucks in the traffic stream

varies. However, the extent to which trucks affect freeway capacity in the presence of a work zone lane closure is complex and has been explored by several researchers. A study involving multiple reconstruction zones in Ontario, Canada in 2002 focused entirely on developing PCEs for congested freeways and noted that the effect of heavy vehicles is magnified during queue discharge flow (Al-Kaisy et al. 2002). Consequently, the researchers suggested that higher PCE values be applied during such conditions, where the specific factor is dependent on terrain. Later, Sarasua et al. used field-measured vehicle headways on freeways in South Carolina to calculate PCEs and found an average value of 1.93, which is significantly more than the value of 1.5 given in the 2010 HCM for level terrain (Sarasua et al. 2004). In phase two of the same study, however, the authors found that different PCEs should be used at different speeds and that these values do in fact increase during congested conditions (Sarasua et al. 2006).

Heavy vehicles have generally been found to decrease freeway work zone capacity, but whether this decrease is significant relative to decreases observed for basic freeway segments has been debated. A 2007 report for the Florida Department of Transportation developed microsimulation models to estimate freeway work zone capacity given 0%, 10%, and 20% trucks and found a strong, negative linear relationship with increasing heavy vehicle percentage (Elefteriadou et al. 2007). However, these findings were reported in vehicles per hour per lane (vphpl), rather than passenger cars per hour per lane (pcphpl), so the results are not surprising nor simply comparable. Additionally, only three truck percentages were modeled, whereas including several smaller increments may have provided a more accurate relationship. An earlier study conducted for freeway work zones in Indiana found that a decrease in capacity of approximately 4 vphpl occurs for each 1% increase in trucks, but that this trend was not significantly different than that found for non-work zone segments (Venugopal and Tarko 2001).

In addition to vehicle composition, the effect of driver population on capacity has been heavily studied. Al-Kaisy et al. researched this topic for various freeway work zones in Ontario, Canada and found that capacity is highest during peak hours and at long-term construction sites, when the traffic stream is composed mostly of commuters or those who are familiar with the ongoing work (Al-Kaisy and Hall 2001). The authors suggested a 7% capacity reduction during off-peak hours and a 16% reduction on weekends to account for these effects. An earlier study in North Carolina agreed with this notion and found that urban work zone sites had higher capacities, possibly due to increased driver familiarity (Dixon et al. 1996). Regardless of driver population, research has shown that driving

behavior in work zones is different from that outside of work zones due to frictional effects found in the changed driving environment (Yeom et al. 2015, 2016). These effects are particularly important when microsimulation is used to model work zone capacity, as will be discussed later.

2.4.2 Work zone characteristics

Although the temporal, behavioral, and traffic-related factors discussed previously have been shown to be significant, the geometric and environmental features of a specific work zone have the greatest impact on its capacity. Of these factors, the lane closure configuration has been given the most attention in literature and been shown to have the strongest influence. Several state-specific field data collection efforts from the mid-1990s through mid-2000s developed regression models to estimate work zone capacity and included the number of closed lanes as an input variable (Al-Kaisy and Hall 2003; Dixon et al. 1996; Kim and Lovell 2001; Sarasua et al. 2006, 2004). More recently, however, Yeom et al. have found that the lane closure severity index (LCSI) included in the 6th edition of HCM is a more distinguishing method of defining the work zone lane closure configuration (Yeom et al. 2015).

The LCSI is calculated using the inverse of the product of the number of open lanes and the ratio of open to closed lanes and allows for the effects of work on the shoulder or median that may not include a lane closure to be modeled. Moreover, this method differentiates lane closure configurations with the same ratio of open to closed lanes, such as 4-to-2 and 2-to-1 closures. Here, 4-to-2 and 2-to-1 lane closures refer to four-lane and two-lane freeway segments reduced to two and one open lane(s), respectively. Regardless, past studies agree that the per-lane work zone capacity decreases as the number of open and closed lanes decrease and increase, respectively. For instance, several authors have documented stark differences in per-lane capacity between 2-to-1, 3-to-1, and 3-to-2 lane closures, where a 3-to-2 closure has the highest capacity and 3-to-1 the lowest (Sarasua et al. 2006; Yeom et al. 2015).

While the effect of lane closure configuration on work zone capacity is well-documented and agreed upon, the influence of lane closure side and length is less clear. Al-Kaisy et al. found that right-side lane closures resulted in approximately 6% higher capacities than left-side lane closures but could not explain this phenomenon (Al-Kaisy and Hall 2003). On the contrary, Weng and Yan studied the relative effect of several factors on work zone capacity using archival literature sources and found that right-side lane closures will decrease capacity by approximately 2.7% relative to left-side lane closures (Weng and Yan 2016). Others found lane closure side to be insignificant but acknowledged that

this variable should be examined in future research (Heaslip et al. 2009; Kim and Lovell 2001).

Likewise, studies on the significance of lane closure length have been largely inconclusive. Data from South Carolina and Maryland freeway work zones found this variable to be insignificant but noted that insufficient sample size was an issue (Kim and Lovell 2001; Sarasua et al. 2004). Earlier research for North Carolina freeways, however, observed that the size of the activity area (i.e., length of the lane closure) is a driving factor in determining the magnitude of the drop in throughput under queue discharge flow as compared to pre-breakdown conditions (Dixon et al. 1996). This observation is thought to be explained by vehicles maintaining larger headways during congested conditions, especially in the presence of trucks, as discussed earlier (Al-Kaisy et al. 2002). Length and relative position of the advance warning area has been studied by others but without significant results (Elefteriadou et al. 2007; Heaslip et al. 2009).

Finally, work intensity has been shown to have a strong negative effect on work zone capacity, but there has been no consistency in how to measure or model this variable. To date, this variable has mostly been represented in deterministic equations for estimating capacity where the user must specify an adjustment using engineering judgement. Typically, this value has been suggested as +/- 10% of the base capacity of the work zone in question (Dixon et al. 1996; Sarasua et al. 2004). Thus, it is impractical to attempt to measure this variable's effect in most cases, as work intensity is typically described qualitatively as "light", "moderate", or "heavy". More recently, Heaslip et al. used the rubbernecking factor in CORSIM to attempt to capture the impact of work intensity on capacity more precisely (Heaslip et al. 2009). However, field data was not available to calibrate this factor, so the researchers relied on previous literature findings that indicated a 7% reduction in capacity when work activity was ongoing (Al-Kaisy and Hall 2003). The final model results indicated that rubbernecking factors of 0% and 5.6% should be used when work activity is not present and present, respectively.

2.5 Breakdown Probability Models

Past research dedicated to determining which factors influence freeway work zone capacity have played a vital role in shaping the methodology found in the HCM and improving the way that agencies design and operate their work zones. However, nearly every study to date has taken a deterministic approach to estimating freeway work zone capacity, despite recent findings that indicate probabilistic methods may be more appropriate. Therefore, defining freeway work zone capacity by the maximum

achievable flow rate prior to breakdown warrants further investigation. Given that instantaneous conditions within a freeway work zone are stochastic, such capacity may be best described by breakdown probability models (BPMs). To date, these models have mostly been applied to metered freeway ramp merge junctions, but the methodology presented is applicable to any freeway bottleneck.

2.5.1 Generating breakdown probability models

It is widely accepted that the development of BPMs requires the use of the product limit method (PLM) developed by Kaplan and Meier (Kaplan and Meier 1958). This methodology was first developed to describe the statistical properties of the lifetime of mechanical parts or human life but has a similar application in capacity estimation. This relationship is shown in Table 1-1, where breakdown can be described as the "failure" or "death" of a highway facility (Brilon et al. 2005). Earlier, it was mentioned that the maximum flow rate is not always synonymous with the breakdown flow rate. This phenomenon leads to incomplete observations referred to as right-censored data, because the upper end of the mathematical probability distribution is unattainable when the highest flow rates do not always result in breakdown. Consequently, the breakdown probability distribution appears truncated, but an incomplete empirical distribution can be obtained using Equation 1-1 because the PLM is non-parametric.

$$\widehat{\boldsymbol{S}}(\boldsymbol{t}) = \prod_{\boldsymbol{j}:\boldsymbol{t_j}<\boldsymbol{t}} \frac{\boldsymbol{n_j}-\boldsymbol{d_j}}{\boldsymbol{n_j}} \qquad (1\text{-}1)$$

Where:

$\hat{S}(t) =$ estimated survival function

$n_j =$ number of individuals with lifetime $T \geq t_j$

$d_j =$ number of deaths at time t_j

Table 1-1: Application of PLM to highway capacity analysis (Source: Brilon et al. 2005)

	Analysis of Lifetime Data	Highway Capacity Analysis
Lifetime Parameter	Time, t	Volume, q
Failure Event	Death at time t	Breakdown at volume q
Lifetime Variable	Lifetime, T	Capacity, C
Censoring	Lifetime, T, longer than duration of experiment	Capacity, C, greater than traffic demand
Survival Function	S(t) = 1 - F(t)	S(q) = 1 - F(q)
Probability Density Function	f(t)	f(q)
Cumulative Distribution Function	F(t)	F(q)

After using the PLM to obtain an initial probability distribution, it is recommended that the best-fit mathematical distribution be estimated to extrapolate the data (Kondyli et al. 2013). Studies of California and German freeway work zones found that the Weibull distribution was most appropriate after using maximum likelihood estimation to compare several candidate distributions (Brilon et al. 2005; Chow et al. 2009). Others have contended, however, that the lognormal or shifted lognormal distribution may also be suitable (Jia et al. 2010; Kondyli et al. 2013; Weng and Yan 2016). It should be noted that the most recent of these three studies was conducted for freeway work zones, but generated capacity distributions from archival literature rather than sequential field data. Nonetheless, the slight disagreement in the literature and site-specific variation suggests that several mathematical distributions should be considered. Elefteriadou et al. stressed this point in early research related to BPMs, where the authors also found that several hundred breakdown events may be necessary to validate such models. For example, to estimate the point corresponding to a 50% chance of breakdown with 95% confidence, it was determined that a minimum of 384 breakdown events should be observed (Elefteriadou et al. 1995). As such, developing BPMs for freeway work zones, even long-term, is likely only attainable using simulation.

2.5.2 Data collection and identification of breakdown

Correctly identifying the onset of breakdown and subsequent return to uncongested conditions is critical in developing meaningful BPMs. This process requires: (1) appropriate placement of data collection sensors, (2) proper choice of data observation and aggregation intervals, and (3) the combination of speed, occupancy, and volume algorithms to define breakdown and recovery periods.

BPMs have various practical applications, so the placement of data sensors has varied somewhat in literature. The bottlenecks in one study of United States and Canadian freeways were defined by several closely spaced ramp merging segments, so the authors placed multiple sensors just upstream and downstream of each bottleneck location in order to verify which ramp was the cause of each breakdown event (Kondyli et al. 2013). This placement strategy was crucial, as literature agrees that the onset of breakdown should be defined by observations made close to the bottleneck in question and not influenced by conditions downstream of the data collection point (Brilon et al. 2005; Elefteriadou et al. 1995; Jia et al. 2010; Kondyli et al. 2013). In the absence of on-ramps or other downstream influencing factors, most agree that the ideal sensor is one placed just downstream of the bottleneck in question (Jia et al. 2010; Lorenz and Elefteriadou 2001), although a study of German freeways contended that sensors should be placed just upstream of the bottleneck to avoid influence of conditions within the bottleneck (Brilon et al. 2005).

The choice of data aggregation periods is also important, as different probability distributions can be obtained from the same data when these intervals are changed. Lorenz et al. studied this concept for 1-, 5-, and 15-minute aggregation intervals and found that shorter intervals result in lower breakdown probability rates for a given flow rate, and vice versa. This phenomenon was explained by the fact that brief fluctuations in flow rates, even to above 2,000 vphpl, can be absorbed by the traffic stream over brief time periods. However, as the aggregation interval increases, an average flow rate of 2,000 vphpl would indicate sustained periods of high volume that are more likely to lead to congestion (Lorenz and Elefteriadou 2001). Later work by Kondyli et al. argued strongly for the use of 1-minute intervals to capture abrupt oscillations in traffic (Kondyli et al. 2013), while others concluded that 5-minute intervals provided the best compromise between accounting for brief spikes in volume and smoothing the data (Brilon et al. 2005; Persaud et al. 1998). Given that data availability and study objectives will vary, it seems that any choice is defendable so long as the researcher clearly defines which time interval was used.

While the breakdown mechanism can typically be identified from visual observation of the fundamental diagram of traffic flow or speed versus time plots, algorithms have generally been used to systematically pinpoint congested conditions. In the literature, these algorithms have involved combinations of speed, occupancy, and volume thresholds. Brilon et al. applied a constant speed threshold of 70 km/hr (43 mph) and classified all flow measurements that coincided with speeds equal to or less than this threshold as congested flow (Brilon et al. 2005). However, the study was conducted for German freeways, which the authors noted was a caveat of the research, as different speed values would likely apply in other countries. Others found that breakdown should be defined by both speed and density to avoid identifying congestion from anomalous free flow conditions (Chow et al. 2009; Jia et al. 2010). Jia et al. designated critical speed and density thresholds based on conditions where speeds were below 55 mph and densities above the level of service D threshold, or 26 passenger cars per mile per lane. 21

Modern studies have shown, however, that speed thresholds sustained over specified time periods may be most appropriate. The 6th edition of the HCM defines the onset of breakdown as a sudden speed drop at least 25% below the free flow speed (FFS) sustained for at least 15 minutes (Transportation Research Board 2016). Conversely, the recovery period is defined as a return to speeds within 10% of FFS for at least 15 minutes. Work by Elefteriadou et al. applied a 90 km/hr (56 mph) threshold to Canadian freeways but required that these speeds be maintained for a period of at least five minutes. Similarly, the authors stated that the return of stable traffic conditions should be signified by speeds above this value maintained for at least five minutes (Elefteriadou et al. 1995).

Later, a subsequent study suggested that breakdown identification algorithms should be based only on speed when sequential speed data is available because this method results in less variance among breakdown volumes. Their specific recommendations were to use a speed drop threshold of 16 km/hr (10 mph), where the reduced speed is sustained for at least five minutes, and a breakdown recovery time period of 10 minutes (Kondyli et al. 2013). The first of these two requirements prevent false identification of brief drops in speed and spikes in traffic flow that are ultimately absorbed, while the second ensures that multiple breakdown events are not identified from a single period of congestion. Accordingly, a similar set of speed-based breakdown identification algorithms will later be applied in this study.

2.6 Freeway Work Zone Simulation Models

The freeway work zone capacity methodology presented in the 6th edition of the HCM is substantially improved from that in previous editions but is still limited by the fact that it is a macroscopic model and cannot account for complex work zone configurations unique to specific sites. Capacities estimated from this model and those from past editions of the HCM are often used as input parameters in other deterministic work zone software such as QUEWZ and QuickZone to predict queueing, delay, and road user costs associated with various scheduling and traffic control strategies. QUEWZ, developed in 1998 by the Texas A&M Transportation Institute, and QuickZone, developed in 2001 by FHWA, have been heavily used of late, but have been shown by some studies to provide inconsistent and often inaccurate estimates of these parameters (Benekohal et al. 2003; Ramezani and Benekohal 2012). Furthermore, although field data collection and empirical capacity measurement are the most accurate means of depicting real traffic conditions, these efforts are often costly and difficult to obtain sufficiently large sample sizes from. Likewise, it has been shown previously that the development of BPMs for freeway work zones is likely not feasible using field data. Fortunately, the emergence of microsimulation software such as CORSIM and VISSIM has provided a means to more accurately and economically deal with such complexity as computing power has increased in recent years.

2.6.1 Simulation overview

Simulation is a valuable traffic analysis tool with wide-ranging applications, not just in work zones. The discussion to follow is based on principles outlined in Elefteriadou's An Introduction to Traffic Flow Theory and the FHWA Traffic Analysis Toolbox regarding the use of traffic analysis tools and simulation models (Dowling et al. 2004; Elefteriadou 2014).

At the most basic level, traffic analysis software can be divided into four categories. From most generalized to most complex, these are: sketch-planning, macroscopic, mesoscopic, and microscopic. Sketch-planning or analytical tools include software such as QUEWZ-98, QuickZone, FREVAL-WZ, or any spreadsheet created to calculate various performance measures. These deterministic tools primarily have high-level planning applications and should be used in situations where agencies wish to guide work zone design and operations decisions while spending the least time or money. However, these advantages are coupled with the disadvantage that randomness associated with individual driving behavior and other factors within work zones are unaccounted for. Thus, it is not surprising that studies have found these tools to be inaccurate in the past (Benekohal et al. 2003; Ramezani and Benekohal 2012). Although QUEWZ-98 and QuickZone apply now-outdated HCM methodology to determine various performance measures in work zones, FREEVAL-WZ applies the 6th edition

methodology and can model more complexity than the other two programs (Trask et al. 2015). Nonetheless, its effectiveness as an analysis tool has yet to be fully explored.

Macroscopic models replicate the movement of platoons of vehicles without analyzing individual vehicle movement. These simulation models are based solely on deterministic relationships between flow, speed, and density and include software such as the TRANSYT-7F package included with the Highway Capacity Software from McTrans. While these models may be useful in optimizing flow of the traffic stream and provide slightly more detail than the analytical tools mentioned previously, they are still generalized and ignore the stochasticity of work zone environments.

Mesoscopic simulation models are a hybrid of macroscopic and microscopic models. While they still only model platoons of vehicles, these models employ equations to indicate how different platoons interact. One such example of mesoscopic software is DYNASMART-P, another software package developed by McTrans in 2007 that can model the dynamic evolution of traffic flows as individual drivers make decisions about their best route. Such a tool may be useful for regional work zone management, but not as valuable for individual work zones.

Finally, microscopic simulation models imitate the movement of every vehicle in the network by accounting for how drivers respond to the surrounding roadway environment. Unlike the deterministic software discussed earlier, microsimulation tools are stochastic, meaning that each model run will produce a unique result. These characteristics make microsimulation the most valuable tool available for estimating freeway work zone capacity. Popular commercially available microsimulation software used by practitioners include CORSIM and VISSIM, developed by FHWA and the PTV Group, respectively. Each of these software packages follow three main algorithms to define the randomness of driving behavior: car-following, lane-changing, and gap acceptance. Recently, several studies have examined the effect of modifying these parameters on simulated work zone capacity and provided guidance for practitioners.

2.6.2 Driving behavior parameters

Although microsimulation is a powerful traffic analysis tool, the validity of any developed model is dependent on a strong calibration effort. Simulation models must be adjusted so that they can replicate field conditions before any hypothetical scenarios can be examined. The authors responsible for the work zone capacity methodology update mentioned earlier have already published work that supplements the analytical model in the 6th edition of HCM with

guidance on developing simulation models in VISSIM (Yeom et al. 2016), following suit with several other previous studies (Chatterjee et al. 2009; Chitturi and Benekohal 2008; Edara and Chatterjee 2010; Kan et al. 2014; Lownes and Machemehl 2006). While CORSIM has been applied in past freeway work zone capacity studies (Heaslip et al. 2009; Ramadan and Sisiopiku 2016), VISSIM was the chosen analysis tool for this study and will be the focus of the literature discussion to come.

The most heavily studied driving behavior parameters are those that pertain to car-following algorithms. These algorithms estimate the trajectory of a following vehicle given the behavior and position of a lead vehicle. Table 1-2 lists the 10 car-following parameters in VISSIM, which are based on the Wiedemann 99 car-following model (PTV Group 2017). The first three parameters are all related to determining the safety distance at which vehicles will follow each other and have been found to be the most influential in determining capacity. This relationship is described by Equation 1-2 (Edara and Chatterjee 2010).

$$\text{safety distance} = CC0 + CC1 * v + CC2 \qquad (1\text{-}2)$$

where v = velocity (ft/s)

Table 1-2: VISSIM car-following parameters (Adapted from Yeom et al. 2016)

Parameter	Description	Default Value
CC0	standstill distance between two vehicles	4.92 ft
CC1	desired headway time between lead and trailing vehicles	0.9 s
CC2	maximum additional distance over desired safety distance	13.12 ft
CC3	time in seconds to start of the deceleration process	-8.0 s
CC4	negative speed variations during the following process	-0.35 ft/s
CC5	positive speed variations during the following process	0.35 ft/s
CC6	influence of distance on speed oscillation	11.44
CC7	oscillation during acceleration	0.82 ft/s^2
CC8	desired acceleration from standstill	11.48 ft/s^2
CC9	desired acceleration at 50 mph	4.92 ft/s^2

In a non-work zone study of congested freeway segments in California, Gomes et al. also modified the emergency stop distance and waiting time before diffusion, two parameters that help to prevent unusual driving behavior during congested conditions. For example, the authors found that the initial model of one freeway merge junction consisted of stopped traffic in the rightmost lane with nearly free flowing conditions in the left lane. This phenomenon was explained by vehicles getting "stuck" while trying to merge and was corrected by decreasing the waiting time before diffusion. However, it was advised that these values be modified with caution so that queued vehicles which researchers do not desire to evaporate from the network are retained (Gomes et al. 2004). Other studies found that this issue was a result of unbalanced lane use upstream and verified that lane use balance thresholds were satisfied prior to validating a particular simulation run (Chatterjee et al. 2009; Yeom et al. 2016). Multiple case studies will be examined in the next section that address these topics in more detail.

2.6.3 Freeway work zone simulation case studies

The following section discusses case studies that have shaped the simulation methodology used in this study. The focus of recent work zone simulation research has been on the calibration effort, as the authors of those studies intended for practitioners to use the results to develop localized simulation models. Consequently, two of the studies presented were carried out to provide guidance on modifying driving behavior parameters and other elements of the simulation environment in VISSIM to replicate field-measured capacities. The third study was conducted using CORSIM but proceeded beyond calibration to investigate the effect of several factors on work zone capacity.

Calibration Case Study #1

In 2010, Edara and Chatterjee used data from Ohio work zones to evaluate default truck characteristics in VISSIM and develop regression models for determining driving behavior parameters based on the capacity, truck percentage, lane configuration, and upstream lane distribution of a given freeway work zone (Edara and Chatterjee 2010). The authors noted that the default length of a truck in VISSIM is 33.5 feet, but approximately 65% of truck-miles in the United States are driven by Class 9 tractor-trailers 73.5 feet in length (Harwood et al. 2003). As such, they recommend that any future studies consider a distribution of truck length based on local field data and power and weight specifications that are consistent with such trucks. Figure 1-2 shows the discrepancy in simulated capacity for a 2-to-1 lane closure when the default VISSIM truck characteristics were used versus adjusted values.

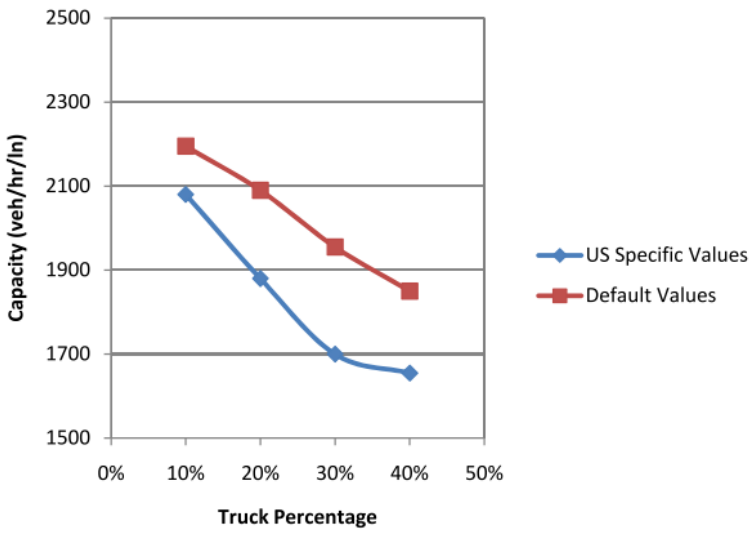


Figure 1-2: Sensitivity of simulated capacity to VISSIM truck characteristics (Source: Edara and Chatterjee 2010)

After using pilot simulation runs to guide modification of default truck characteristics, the authors performed a sensitivity analysis to determine the effect of select driving behavior parameters on simulated capacity. Based on past research (Chatterjee et al. 2009), the car-following, lane-changing, and gap acceptance variables ultimately examined were CC1, CC2, and SRF. Variations in lane-changing distance were not studied, but a value of 2500 feet was selected to mimic the expected location of a "___ LANE CLOSED ½ MILE" sign in the field. As shown in Figure 1-3, capacity was measured as the QDR just downstream of the bottleneck, while data collection points were placed at four locations to verify lane use balance upstream of the closure point. A total of 900 scenarios were simulated between 2-to-1, 3-to-2, and 3-to-1 lane closure configurations; however, only those which produced reasonable upstream lane balance consistent with field data were retained.

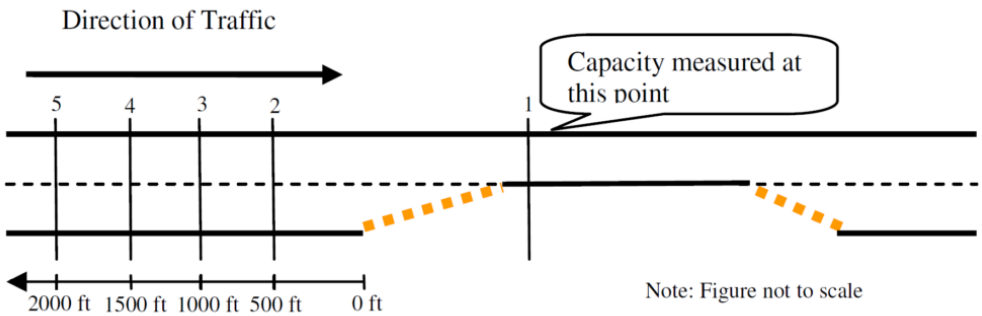


Figure 1-3: Lane closure network setup for 2-to-1 closure (Source: Edara and Chatterjee 2010)

The results of the study were three sets of regression equations, one for each lane closure configuration, that provide guidance for selecting the values of CC1, CC2, and SRF based on the field-measured capacity (Qc), truck percentage (PT), and lane distribution 1000 feet upstream of the lane closure (PCL). An example of these for a 2-to-1 closure is given in Equation 1-3 and is intended to provide practitioners with values of driving behavior parameters that are consistent with conditions in freeway work zones. It should be noted, however, that other research has found that these are not the only driving behavior parameters in VISSIM that affect capacity significantly (Chitturi and Benekohal 2008; Gomes et al. 2004; Kan et al. 2014; Lownes and Machemehl 2006; Woody 2006). Furthermore, while this guidance may serve as a good starting point, the total calibration effort will likely require more fine-tuning after adjusting the parameters studied here.

$$CC1 = 2.974 - 0.0009 * Q_C + 0.0267 * P_T + 0.0022 * P_{CL} - 0.000029 * Q_C * P_T$$

$$CC2 = 82.39 - 0.0266 * Q_C + 0.208 * P_T - 0.302 * P_{CL} - 0.00009 * Q_C * P_T$$

$$SRF = 0.656 - 0.0002 * Q_C + 0.0057 * P_T + 0.0078 * P_{CL} - 0.000009 * Q_C * P_T$$

(1-3)

Calibration Case Study #2

A subsequent study conducted in 2016 by Yeom et al. used nationwide field data from their NCHRP 03-107 research to perform a sensitivity analysis on a similar set of driving behavior parameters (Yeom et al. 2016). However, the authors

neglected trucks and instead focused solely on the modification of CC0, CC1, CC2, CC4, CC5, CC8, SRF, and lane-changing distance. A diagram of their network setup is given in Figure 1-5, where several key items should be noted. First, data collection points were placed 1000 feet upstream and 100 feet downstream of the lane closure point to allow for lane use balance verification and measurement of QDR, respectively. To ensure that the frictional effects within the work zone were accurately modeled, a reduced speed area and desired speed decision point were placed at the beginning of the lane closure. The former has a "look ahead" function that allows vehicles to begin slowing down prior to reaching the speed decision point. Speed distributions within and outside of the work zone were based on field data.

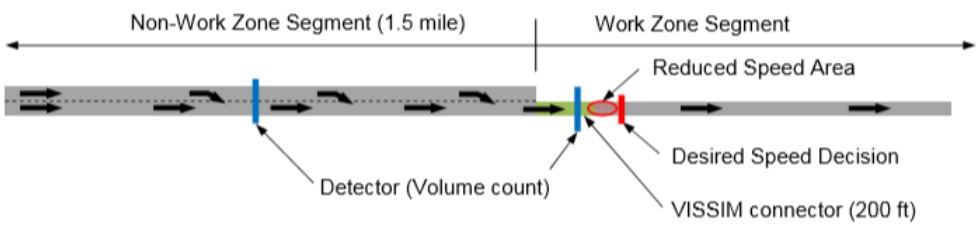


Figure 1-4: VISSIM network setup for 2-to-1 lane closure (Source: Yeom et al. 2016)

Given that capacity was to be measured as the average QDR, the network was coded with a demand of 2,000 vphpl to ensure that congested conditions would develop. Like the previous case study (Edara and Chatterjee 2010), lane use balance verification was critical in retaining realistic results; however, the researchers could not find sufficient guidance in the literature on appropriate lane use upstream of a work zone with a lane closure, so they used the thresholds given in Table 1-3 based on a late, or "zipper", merge strategy.

Table 1-3: Lane use balance thresholds (Source: Yeom et al. 2016)

Number of Lanes in Upstream Segment	Minimum	Expected Even Ratio	Maximum	Unconditional Minimum
2	0.35	0.5	0.65	N/A
3	0.23	0.33	0.43	0.15
4	0.18	0.25	0.33	0.05

A total of 8,478 experiments were planned by varying each of the parameters listed in Table 1-2 from 100 to 700 percent of their default values. The complete calibration effort included verification of lane use balance and cross-checking of simulated capacity against predicted capacity from the model in the 6th edition of the HCM. Only model runs that produced results within the thresholds in Table 1-3 and the minimum and maximum values predicted by the HCM model were retained. Ultimately, only CC1 and CC2 produced valid results under all the tested lane configurations, so it was recommended that the remaining parameters be held at their default value apart from lane-changing distance. Regression equations for determining CC1 while holding CC2 constant are given in Table 1-4, and generic guidance for driving behavior parameter settings is given in Table 1-5.

Table 1-4: Regression model for CC1 estimation (Source: Yeom et al. 2016)

Lane Configuration	LCSI	CC2 (ft)	CC1 (s) Estimation Regression Model	R^2 Value
4 to 3	0.44	39.36	-0.0015*avg. QDR + 3.9346	0.9950
3 to 2	0.75	26.24	-0.0020*avg. QDR + 5.0041	0.9807
4 to 2	1	26.24	-0.0019*avg. QDR +4.7155	0.9245
2 to 1	2	23.62	-0.0023*avg. QDR + 5.3146	0.9913
3 to 1	3	26.24	-0.0041*avg. QDR + 7.7741	0.9937
4 to 1	4	39.36	-0.0022*avg. QDR + 4.7177	0.9694

Table 1-5: VISSIM driving behavior parameter guidance (Source: Yeom et al. 2016)

Parameter	**VISSIM Default**	**Recommended WZ Setting**
Car Following Parameters		
CC0 (ft)	4.92	Default
CC1 (s)	0.90	Work Zone Configuration Specific
CC2 (ft)	13.12	Work Zone Configuration Specific
CC3 (s)	-8.00	Default
CC4 (ft/s)	-0.35	Default
CC5 (ft/s)	0.35	Default
CC6	11.44	Default
CC7 (ft/s^2)	0.82	Default
CC8 (ft/s^2)	11.48	Default
CC9 (ft/s^2)	4.92	Default
Lane-Changing Parameters		
Lane-Changing Distance (ft)	656.20	> 656.20
Necessary lane change, 1 ft/s^2 per distance (ft)	200.00	100.00
Maximum Deceleration for Cooperative Braking (ft/s^2)	-9.84	-20.00

CORSIM Case Study

Lastly, a study by Heaslip et al. in 2009 used CORSIM to develop analytical models for estimating freeway work zone capacity (Heaslip et al. 2009). The

authors felt that the methodology found in the 2000 edition of the HCM did not adequately address all factors that affect work zone capacity due to the limitations of field measurement and noted that simulation is a viable alternative. Their research objectives were: (1) develop analytical models for 2-to-1, 3-to-2, and 3-to-1 lane closures using the results of CORSIM simulation runs, (2) calibrate and refine these models using field data from a work zone in Jacksonville, Florida, and (3) compare the results to the HCM 2000 methodology.

In developing the simulation model, the following independent variables were initially considered: lane closure configuration, work zone length, lane closure side, work intensity, volume distribution among lanes, distance of first warning sign upstream of the closure, and percentage of trucks. Pilot simulation runs revealed that CORSIM results were not sensitive to work zone length or lane closure side, so these variables were ultimately eliminated. However, the authors did comment that this is a limitation of CORSIM and that past research has indicated that these factors may indeed be significant to work zone capacity. As such, they recommended that future research re-examine these variables. The dependent variables gathered from each simulation run were: speeds by lane, vehicle lane distributions, time headways, and maximum throughput under congested conditions.

A diagram of the simulated network is given in Figure 1-6. Links (3,4) and (6,7) were varied depending on the distance of the first upstream warning sign to the lane closure, where the former was adjusted to ensure that the overall length of the network remained the same for all simulation runs. Links (2,3) and (4,5) were used to verify headway values upstream of the closure and ensure that erratic driving behavior was not occurring. The lane closure itself was modeled as an incident because this feature in CORSIM allows for the specification of a rubbernecking factor, which the authors used to model work intensity.

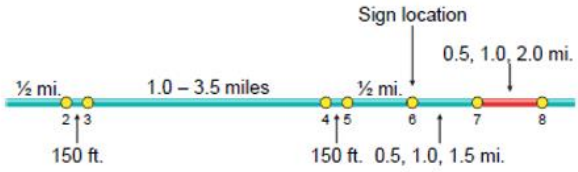


Figure 1-5: Diagram of simulated network (Source: Heaslip et al. 2009)

Lane Closure Type	CAPACITY MODEL FOR OPERATIONAL PROCEDURE
2 TO 1	 $R^2 = 0.915$ $C^{2\text{-}to\text{-}1}_{unadj} = 1855 - 693 * SignDist + 191 * f_{HV\text{-}F} - 12.3 * Rubber\% - 467 * DistrLan1(6,7) + 829 * DistrLan1(6,7) * SignDist + 7.43 * SpeedLan1(6,7)_{adj} * SignDist$
3 TO 2	 $R^2 = 0.932$ $C^{3\text{-}to\text{-}2}_{unadj} = 917 + 461 * SignDist + 854 * f_{HV\text{-}F} - 20.4 * Rubber\% - 611 * DistrLan1(6,7) * SignDist - 4.03 * SpeedLan1(5,6)_{adj} * SignDist$
3 TO 1	 $R^2 = 0.895$ $C^{3\text{-}to\text{-}1}_{unadj} = 1177 + 550 * f_{HV\text{-}F} - 14.5 * Rubber\% + 157 * DistrLan3(6,7)$
ALL	$C_{adj} = f_l * f_d * f_r * (C_{unadj} - v_R)$

Figure 1-6: Analytical models by lane configuration (Source: Heaslip et al. 2009)

The results of the study yielded three analytical models, one for each of the studied lane closure configurations, which the authors found to predict capacity to within 1% of field-measurements. Using field data from the Jacksonville, Florida site and literature, further adjustments were recommended to account for variation in lighting conditions, weather, and driver population, all of which could not be modeled in CORSIM. The analytical models are presented in Figure 1-7, but the reader is directed to Heaslip et al. for the full procedure.

Despite the seemingly promising results, several caveats of the research were presented. For example, it was noted that CORSIM constrained the simulation effort due to its lack of versatility in accounting for the effects of variables such as work zone length, lane closure side, and lane width. Furthermore, this study only used a single 3-2 lane closure in Florida to calibrate the simulation model, which brings the results for other lane configurations into question. Thus, the authors recommended that future research apply a larger field dataset and that CORSIM algorithms be modified to aid in work zone simulation modeling.

2.7 Summary

In summary, an expansive literature review was undertaken to assess the state of the practice in defining, measuring, and modeling freeway work zone capacity and motivate the objectives of this study. On a broad level, it is evident that researchers still cannot agree on the most appropriate measure of freeway work zone capacity. While the most recent HCM methodology points to QDR as the most reliable and conservative measure

of capacity at freeway work zones, research and agency practice strongly suggests that breakdown and queue formation be considered to drive work zone design and operations decisions. Since variability in instantaneous driving behavior and conditions in the work zone environment led to variations in breakdown flow and QDR, there may be advantages to describing each with probability distributions rather than a single value.

The most accurate model of real traffic conditions comes from field data collection, which was the focus of most early freeway work zone capacity studies. However, these efforts require substantial time and money, and the extent of observations necessary to provide adequate data for constructing breakdown probability models is immense. Fortunately, recent research has suggested that simulation models may be a viable alternative to obtaining a large sample of capacity data under various work zone configurations. The focus of most of this work has been to provide practitioners with guidance in developing and calibrating such simulation models so that they may be applied to individual, localized work zones. That said, many agencies may not have the time or resources to generate these models, and might prefer to lean on deterministic methods, charts, or look-up tables when making work zone scheduling decisions. Therefore, past literature has revealed the need for full-scale work zone simulation models that provide data for practice-ready application.

3.0 METHODS AND ANALYSES

3.1 Introduction

The methods by which findings from the literature review were used to drive field data collection efforts, VISSIM model development, and final experiment design are discussed in this chapter. The first section provides an overview of the study work zone near Tuscaloosa, Alabama from which sequential speed, length, volume, and headway data were collected, screened, and processed. Next, VISSIM network coding and calibration are described in significant detail, as the validity of the results depended most strongly on the model development process. In this step, Vissim default values for time headway and heavy truck capabilities are evaluated and alternative values proposed. Lastly, the design of the factorial experiment used to generate breakdown probability models under various freeway work zone conditions is presented, and a small pilot or test case is developed.

3.2 Site Overview and Data Collection Plan

One of the primary objectives of this study was to produce realistic, generalizable results that could be transformed into a tool to be used by agencies and practitioners for rural freeways. As such, it was critical that simulation model outputs were validated using field-collected data. The research team coordinated with the Alabama

Department of Transportation (ALDOT) to identify potential study work zones and ultimately selected a 2-to-1 lane closure on Interstate 59/Interstate 20 (I-59/I-20) just south of Tuscaloosa, Alabama. A map of the site is provided in Figure 1-7 along with the location of key pieces of temporary traffic control (TTC), as these TTC devices were influential in coding the final VISSIM model. Specifically, simulated lane-changing behavior and vehicle speed distributions were determined in large part by the location of warning signage and reduced speed limits throughout the work zone. Note that in the figure, AADT and TDHV refer to annual average daily traffic and proportion of trucks in the traffic stream during the design hour, respectively. Likewise, K and D represent the proportion of daily traffic occurring during the peak hour and in the peak direction, respectively. It should be noted that this data was obtained from the nearest ALDOT permanent counting station and may or may not reflect exact conditions at the study work zone.

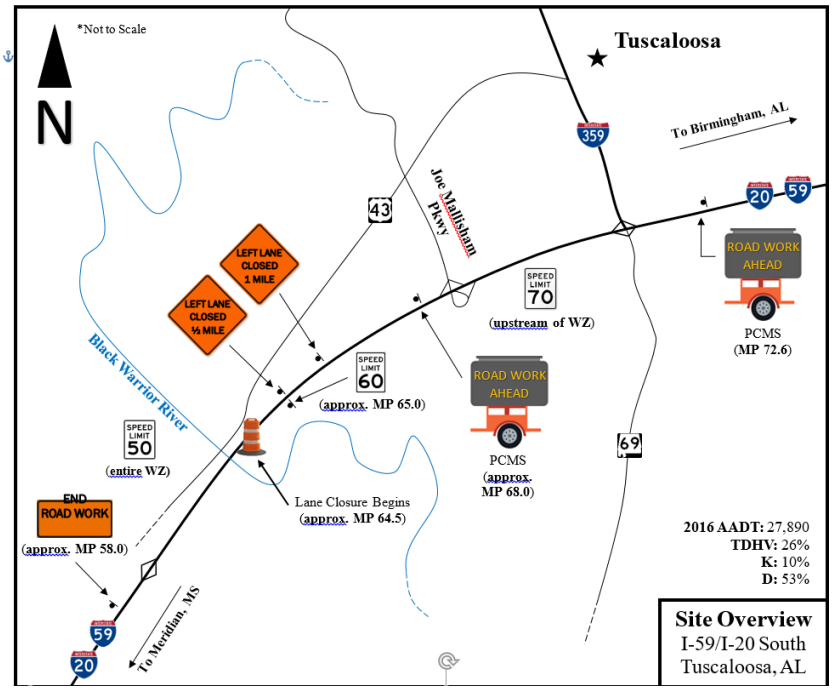


Figure 1-7: Map of study work zone

Data was collected during the 14-day period from October 3, 2016, to October 16, 2016 using a total of nine traffic sensors, deployed as shown in Figure 3-2. The NC350 BlueStar Portable Traffic Analyzer, manufactured by M.H. Corbin, was the chosen sensor for the study. Each sensor had the capability to collect speed, volume, length, and headway data at one-second intervals for up to 300,000 vehicles or 21 days, whichever occurred first. The product information specifies that each sensor is also accurate to

within 4 mph for vehicle speeds, 4 feet for vehicle lengths, and 1% for vehicle counts (MH Corbin 2016).

The sensor deployment scheme shown in Figure 1-8 was developed based on findings from the literature review suggesting that there are three primary locations at which data should be collected: well upstream of the lane closure, between a quarter and half mile upstream of the lane closure, and just downstream of the lane closure. It should be noted that the labels given to each sensor in Figure 1-8 correspond to the last two or three digits of their respective serial numbers, which will be used throughout the remainder of this report for brevity. Sensors 96, 97, 98, 100, 101, and 102 were placed with the intent to collect vehicle speeds in locations not influenced by downstream congestion due to the bottleneck; such speeds were ultimately used to develop free flow speed distributions in VISSIM for the non-work zone segment of the network. Sensors 99 and 103 were placed one half mile upstream of the bottleneck to observe queue propagation and dissipation due to breakdown events and to study lane distributions within the advance warning area. Lastly, sensor 104 was installed approximately 100 feet downstream of the beginning of the full lane closure to build desired speed distributions in VISSIM for the work zone segment of the network, identify breakdown events, and measure queue discharge flow rates.

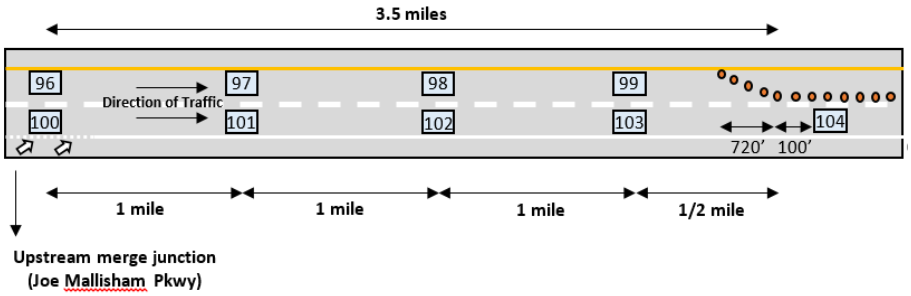


Figure 1-8: Traffic sensor deployment scheme

Despite the relatively high accuracy of the NC350 traffic sensors, there were limitations to the data collection effort. Sensor 103 stopped collecting data before the end of the study period, the cause of which was determined to be a firmware issue after discussions with M.H. Corbin staff. In addition, the research team moved sensor 104 from the right lane to the left lane on October 6th, 2016, around 4:30PM because resurfacing operations were scheduled to shift to the right lane. Since the traffic switch did not actually occur until 7:00AM on October 8th, 2016, the number of available days to observe breakdown events was reduced. Finally, a comparison of the 14-day traffic volumes observed at each data collection location revealed significantly lower vehicle counts at sensors 96 and 100, most likely due to their proximity to the on-ramp at Joe

Mallisham Parkway. Consequently, these records were used exclusively for free flow speed data and VISSIM volume inputs were obtained from other upstream sensors.

3.3 Data Screening and Processing

Even the most accurate sensors will occasionally produce erroneous measurements, so data screening was an especially vital component of this research. A combination of threshold- and traffic flow theory-based screening methods as proposed by Turochy and Smith were initially explored to identify obvious errors in the sensor data (Turochy and Smith 2000). Quick inspection of the dataset revealed that there were numerous vehicle records for which the speed or length was measured as zero or as an exceptionally high value. Further discussions with M.H. Corbin led to the discovery that the NC350 traffic sensors tend to measure unrealistic vehicle speeds and lengths during congestion, when prevailing speeds may be less than the equipment's stated minimum of 5 mph. As a result, the first threshold set was a minimum speed of 5 mph, although other speed thresholds would later be considered.

Regarding traffic flow theory-based methods, the observed values of headway (in seconds, from front bumper to front bumper) were often found to be inconsistent with the speed differential between successive vehicles. For instance, a headway of 1 second or less was frequently measured for two vehicles whose speeds differed by as much as 50 mph, suggesting that at least one of these records were erroneous. However, headway-based screening methods were ultimately abandoned for two reasons: (1) it was discovered that headway values were all rounded down to the nearest second, meaning that a value of 1 second could screening methods would invalidate the headway value for a given record. Therefore, several threshold- and statistical-based screening methods were applied to the dataset as will be discussed in the following sections.

3.3.1 Vehicle lengths

The first detailed screening effort that took place pertained to the distribution of vehicle lengths at the study work zone. For a VISSIM model to accurately replicate field conditions, it is crucial that simulated traffic is composed of the correct percentage of each vehicle class and that such vehicles are represented by their true lengths. While it was expected that there would be inconsequential variation among sensors with regards to vehicle lengths and that a prevailing distribution would be easily identified, this was not the case. Figure 1-9 provides an example of this discrepancy for sensors 101 and 102. The figure shows that sensor 101 consistently measured longer passenger cars and trucks than sensor 102 and supports the decision to pursue a statistical means of developing accurate length distributions. The disagreement between sensors 101 and 102

was common among other sensors—the mean length for passenger cars was as small as 15 feet and as large as 20 feet, while the mean length for trucks varied between 63 feet and 82 feet. To address this issue, a multi-step procedure was utilized to address sensor error and estimate the true distribution of vehicle lengths:

1. Common vehicle lengths were identified from literature (AASHTO 2011) to gain an idea of how lengths should be distributed for each vehicle class, and extreme values were discarded.
2. The mean frequency of each observed vehicle length was found to generate an average distribution among the nine sensors.
3. The bounds for a normal distribution were approximated for each vehicle class and the mean and standard deviation calculated.
4. Upper and lower bounds for each vehicle class were set at two standard deviations away from the mean for each distribution.

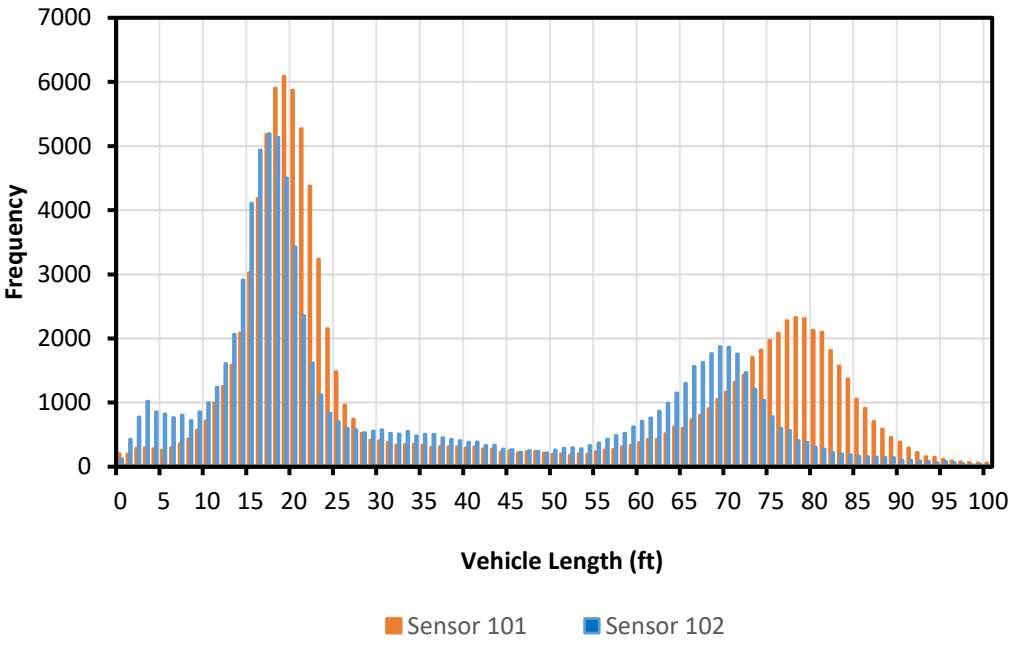


Figure 1-9: Vehicle length distributions for sensors 101 and 102

The frequency distribution representing the combination of all nine sensors is given in Figure 1-10, where the average frequency of each length was calculated by dividing its total number of occurrences by nine. The disparity in vehicle length measurements for heavy trucks is underscored by the bimodal distribution found between 50 and 100 feet. However, the dotted line drawn to

approximate a unimodal distribution has a peak between 73 and 75 feet, which is close to the 73.5-foot length of a WB-67 interstate semitrailer, a vehicle class composing 65% of all truck-miles in the United States (Harwood et al. 2003).

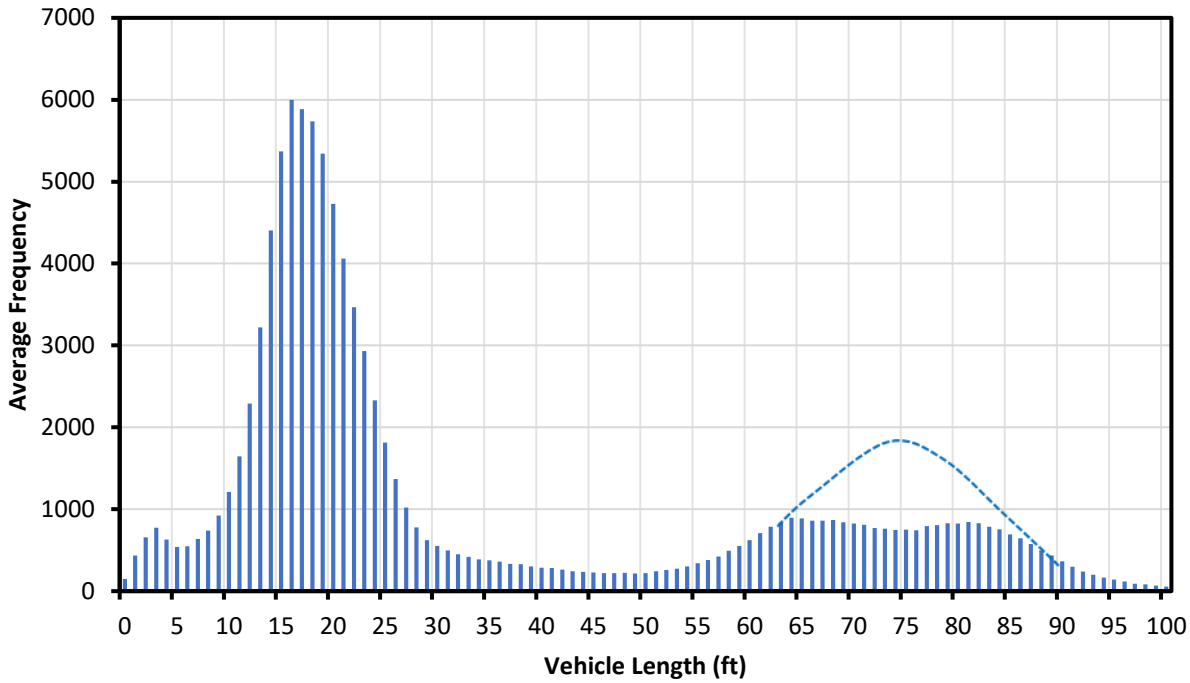


Figure 1-10: Volume-weighted vehicle length frequency distribution

Of more interest during the screening process, however, was defining upper and lower bounds beyond which vehicles would be declassified and not considered in determining overall proportions of each vehicle class in the traffic stream. The results of the process outlined above are presented in Table 1-6, where the absolute upper and lower lengths for passenger cars and heavy trucks are defined. Based on the data shown in Figures 1-9 and 1-10, and shown in Table 1-6, the decision was made to use 32 ft as the breakpoint between passenger cars and trucks. Single unit trucks will later be accounted for in coding the VISSIM model but were not significant to the data screening process. To account for sensor error and eliminate the possibility of falsely rejecting accurate values, the upper and lower bound lengths were conservatively defined as 99 feet and 5 feet, respectively. All records with values of length outside of these bounds were discarded for subsequent analyses not involving raw vehicle volumes. For example, when calculating free flow speed distributions, even vehicles with

reasonable values of speed were not considered if their length was unreasonable.

Table 1-6: Vehicle classification bounds

	Mean (ft)	Standard Deviation (ft)	Lower Bound (ft)	Upper Bound (ft)
Passenger Cars	18.92	6.57	5.79	32.06
Heavy Trucks	72.57	11.14	50.29	94.85

3.3.2 Vehicle speeds

The second data screening process that took place was the validation and filtering of vehicle speeds to be used in constructing free flow speed distributions. Like the screening of vehicle lengths, a multi-step procedure was performed to ensure that only valid speeds were included in the non-work zone and work zone desired speed distributions to be coded in VISSIM. Specifically, three types of tests were used to eliminate erroneous records or those occurring under non-free flow conditions:

1. *Threshold Screening:* Given the sensor manufacturer's specifications and site conditions, the upper and lower bound speeds were defined as 99 mph and 5 mph, respectively.
2. *Statistical Screening:* Speeds outside of two standard deviations from the mean speed for each 5-minute interval were discarded.
3. *Density Screening:* Only speed records occurring during intervals with a density of less than or equal to 11 passenger cars per mile per lane (pc/mi/ln; equivalent to LOS "A") were considered. While not erroneous, these records were not applicable to modeling free flow speeds.

The number of records eliminated during each screening procedure are summarized in Table 1-7. The threshold and statistical tests left most of the data intact, eliminating only 14% of the records, and would be applied for other portions of VISSIM model development such as the construction of time headway distributions. Density screening eliminated an additional 27% of all speed records, primarily from sensors 99, 103, and 104, where queueing was frequently observed, and traffic flow was constrained even during marginal flow

rates due to the presence of the lane closure. The speed-flow diagram for sensor 104 revealed a that there were still a few unusually low speeds remaining during periods of low flow and density, but these were found to minimally affect cumulative distribution curves and were retained.

Table 1-7: Vehicle speed screening results

Sensor Group	Raw Data	After Threshold Screening		After Statistical Screening		After Density Screening	
	Records	Records Retained	Cumulative% Removed	Records Retained	Cumulative% Removed	Records Retained	Cumulative% Removed
96, 100 (3.5 Miles)	179,194	163,594	8.7%	155,099	13.4%	149,994	16.3%
97, 101 (2.5 Miles)	187,845	179,339	4.5%	170,747	9.1%	150,632	20%
98, 102 (1.5 Miles)	188,440	170,639	9.4%	162,914	13.5%	120,975	36%
99, 103 (0.5 Miles)	161,573	146,346	9.4%	117,132	27.5%	87,808	46%
104 (Lane Closure)	163,598	158,767	3.0%	151,773	7.2%	22,118	86%
Total	*880,650*	*818,685*	*7.0%*	*757,665*	*14.2%*	*531,527*	*41%*

Once only valid free flow speeds remained, desired speed distributions could be built and later applied in the VISSIM model. Given differences in driving behavior and vehicle performance, separate distributions were necessary for heavy trucks and passenger cars. However, single unit trucks were assumed to account for a negligible proportion of the traffic stream and follow the same speed distribution as passenger cars. Since each sensor was found to measure different truck length distributions, the lower bound length defining the cutoff between passenger cars/single unit trucks and heavy trucks was calculated independently for each, as summarized in Table 1-8.

Table 1-8: Vehicle length cutoff values

Sensor	Mean Truck Length (ft)	Std. Dev. (ft)	Lower Bound Length (ft)
96	68	7	55
97	65	9	48
98	81	9	64
99	79	9	61
100	82	9	64
101	76	8	59
102	69	8	54
103	76	7	62
104	63	7	50
Weighted Average	73	11	50

Using these thresholds, a total of 18 speed distributions (nine sensors x two vehicle types) were constructed with the intent to use the weighted average of the eight upstream sensors for the non-work zone segment of the model and utilize the downstream sensor for the work zone segment. That said, observation of the lane-specific speeds upstream of the closure taper revealed that the mean speed of vehicles traveling in the left lane was much lower than that of those traveling in the right lane—a counterintuitive relationship. These findings are highlighted in Table 1-9, which shows this to be the case for the sensors 2.5 and 3.5 miles upstream of the lane closure. It was hypothesized that the portable changeable message signs shown in Figure 1-7 may have been responsible for higher-speed traffic merging into the right lane well ahead of the closure, but this theory could not be confirmed. As such, free flow speeds at sensors 98, 102, 99, and 103 were assumed to be most representative of speed behavior upstream of the lane closure and were adopted in VISSIM. This distribution is shown in Figure 1-1. For brevity, the free flow speed distribution for sensor 104 will not be shown here but will be described in the next section.

Table 1-9: Summary of lane-specific mean free flow speeds

Location	Vehicle Class	Sensor	Eligible Volume	Mean Speed (mph)
3.5 Miles Upstream	**Passenger Cars**	96	53924	71
		100	54738	81
	Trucks	96	6523	67
		100	34808	75
2.5 Miles Upstream	**Passenger Cars**	97	53851	64
		101	54102	72
	Trucks	97	8705	60
		101	33973	69
1.5 Miles Upstream	**Passenger Cars**	98	52797	79
		102	36452	63
	Trucks	98	12715	73
		102	19011	59
1/2 Mile Upstream	**Passenger Cars**	99	44730	78
		103	19278	72
	Trucks	99	13605	71
		103	10195	67
Downstream of Taper	**Passenger Cars**	104	13505	51
	Trucks	104	8613	48

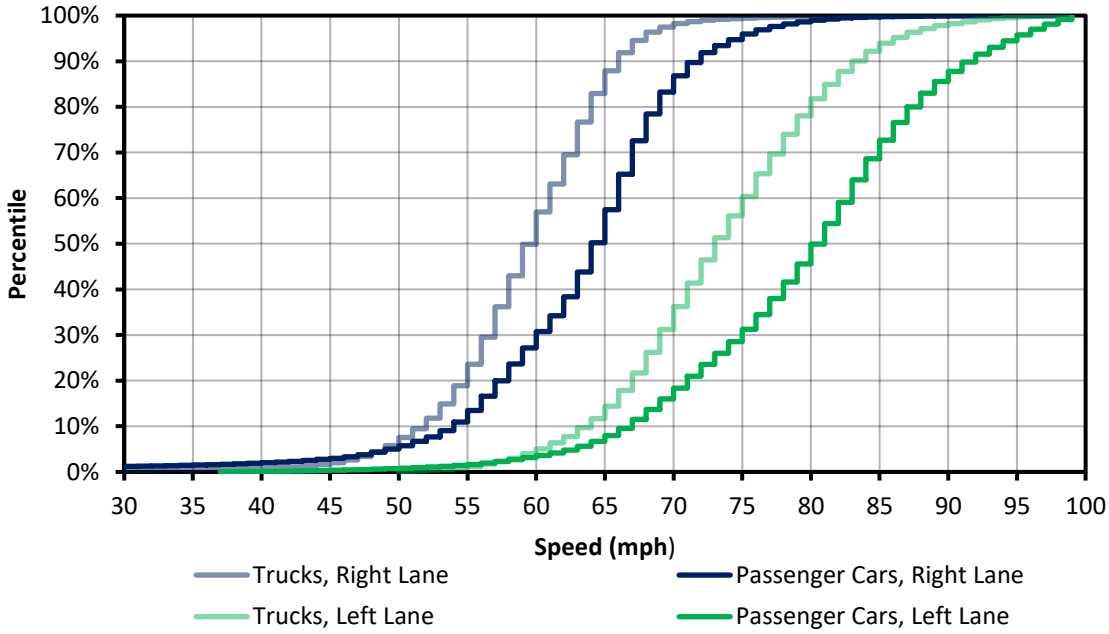


Figure 1-11: Upstream free flow speed distributions

Figure 1-11 highlights a significant difference in speed by lane and vehicle type, reinforcing the need to include separate desired speed distributions for passenger cars and trucks and for the right and left lanes. Though a single, volume-weighted distribution could have been applied to both lanes in VISSIM, it was determined that the use of two distinct distributions would more accurately distribute fast- and slow-moving vehicles in the model. This conclusion was drawn based on the assumption that upstream lane distributions in the field reflected the desire of aggressive, fast-moving vehicles to travel in the left lane, while most heavy vehicles and slow-moving passenger cars traveled in the right lane.

3.3.3 Exploration of field data

Prior to developing and evaluating the VISSIM model, field data was examined so that typical traffic conditions could be characterized and understood. Since this research focused on studying the breakdown phenomenon at rural freeway work zones, time intervals just prior to and during congestion were of the most interest. Over the course of the data collection period, 12 major breakdown events were observed using a breakdown identification algorithm that required average speeds to be below 35 mph for at least 15 minutes. This algorithm was selected based on suggestions from the literature and the definition of

breakdown in the HCM, which requires that speeds be maintained at least 25% below the free flow speed for at least 15 minutes.

The declaration of breakdown and recovery from breakdown both occasionally required the use of engineering judgement, particularly when prevailing speeds prior to sustained periods of congestion hovered near the threshold for several time intervals. A complete summary of every breakdown event at 5- and 15-minute aggregation intervals is provided in Appendix A, but an abbreviated synopsis is given in Table 1-10 to show the range of variation in PBC, QDR, and truck percentage for each congested period. From this point forward, a PCE of 2.0 (the default for level terrain in the 6th edition of the HCM) was used in all volume conversions unless stated otherwise.

Table 1-10: Summary of breakdown events at study work zone

	Maximum Pre-Breakdown Flow Rate (pcphpl)		**Breakdown Flow Rate (pcphpl)**		**Average QDR (pcphpl)**		**% Trucks**
	15-Minute Intervals	*5-Minute Intervals*	*15-Minute Intervals*	*5-Minute Intervals*	*15-Minute Intervals*	*5-Minute Intervals*	
All	1161	1324	1115	1169	1049	1052	25%
Left Side Closure	1125	1309	1065	1140	1035	1045	27%
Right Side Closure	1206	1342	1178	1205	1066	1060	22%
Minimum	1071	1256	989	802	936	990	16%
Maximum	1270	1392	1270	1338	1256	1256	32%

Table 1-10 was organized so that differences in traffic conditions could be highlighted for varying lane closure side, truck percentage, and aggregation interval. Several interesting findings were made that would ultimately drive subsequent decisions during model development and data analysis. For example, despite the study site having a TDHV of 26%, the proportion of heavy trucks in the traffic stream varied between 16% and 32% during the two hours prior to

each breakdown event and throughout the duration of queue discharge. This can partially be explained by the fact that work zones are a source of non-recurring congestion and may cause queueing at any point throughout the day, during which truck percentage varies greatly. That said, one should still expect the truck percentage for the same hour to vary by a relatively large amount from day to day, so this finding was not surprising. Nonetheless, care was taken to ensure that an appropriate range of truck percentages were observable in VISSIM, even with static inputs for vehicle compositions, so that field conditions could be accurately replicated.

Regarding lane closure side, a stark difference in all three measures of throughput was observed between the right- and left-side closure. The maximum pre-breakdown flow rate, breakdown flow rate, and QDR were 7.2%, 10.6%, and 2.9% higher, respectively, when the right lane was closed if measured using 15-minute aggregation intervals. Figures 1-12 and 1-13 give another visualization of this trend by showing the approximate flow vs. density curve for each lane closure configuration, where the flow rate and density at capacity are both higher for the right-side lane closure. The fitted curves shown in the figure were developed based on Greenshields's model, which assumes the relationship between flow and density is parabolic. Even though small sample size made it difficult to discern how much of this difference was due to chance or exogenous factors, this finding helped reaffirm the decision to include lane closure side as a variable of interest in the final experiment.

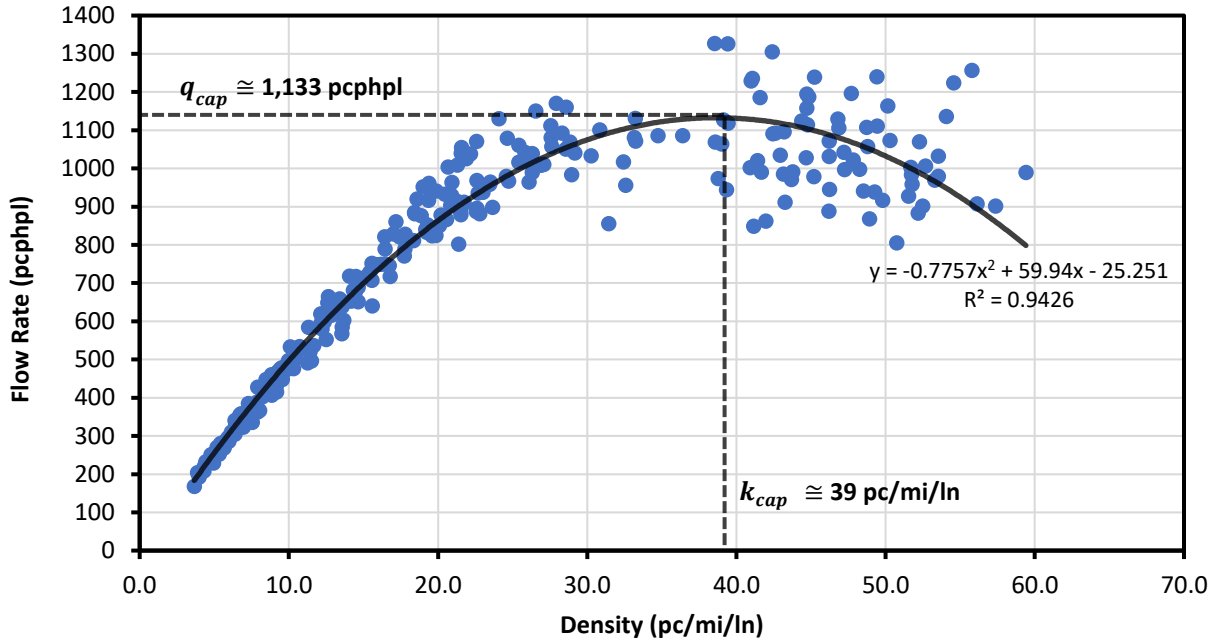


Figure 1-12: Flow vs. density curve (left-side closure)

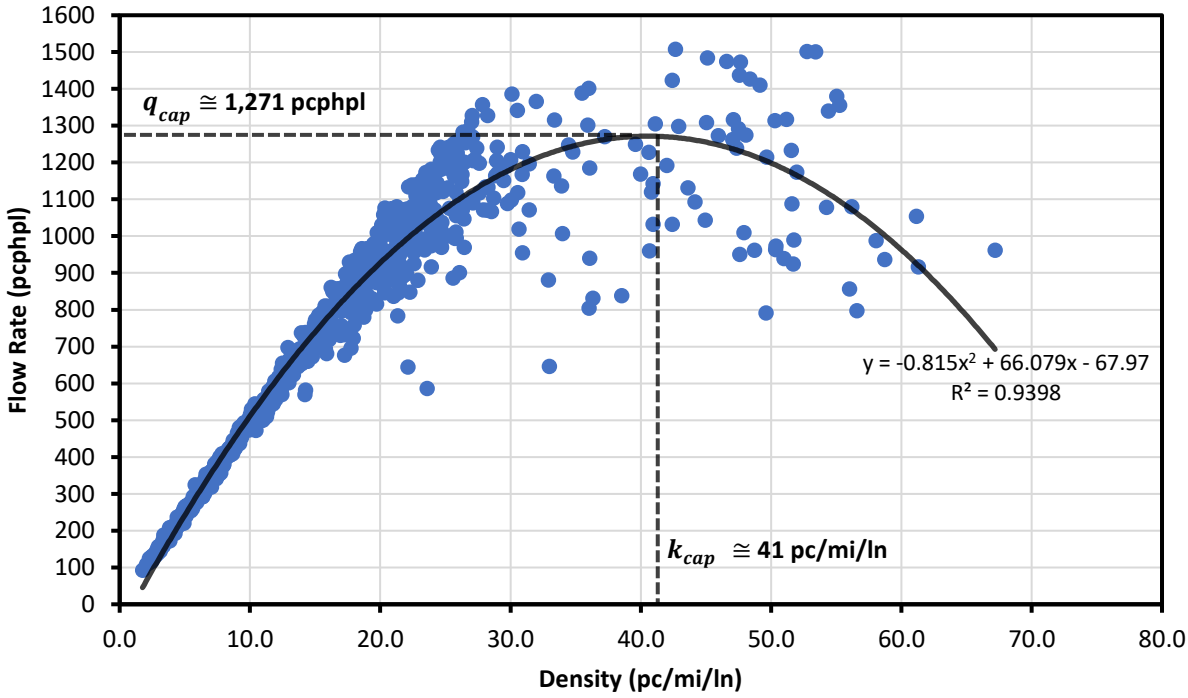


Figure 1-13: Flow vs. density curve (right-side closure)

Lastly, it was found that the choice of aggregation interval had a significant effect on flow rates measured prior to breakdown, while queue discharge flow rates were relatively consistent between the two. This coincides well with findings from the literature review, where several authors noted that aggregation intervals smaller than 15 minutes will capture abrupt oscillations in traffic volume, which often consist of high flow rates sustained over short periods of time. Since traffic flow generally stabilizes in the presence of a queue, it is intuitive that there is less variation among 5-minute flow rates in these situations and that such volumes are approximately equivalent to their corresponding 15-minute flow rates. When formulating the factorial experiment in VISSIM, an aggregation interval needed to be selected, so a dilemma arose: from a research perspective, the use of 5-minute aggregation intervals would allow for more precision when declaring breakdown events and thus most accurately account for the effect of the studied variables on work zone capacity. However, from a practical standpoint, agencies are most likely to have 15-minute volumes on hand, and such flow rates would more conservatively estimate breakdown probability distributions. Given the benefits of both options, it was decided that 5- and 15-minute aggregation intervals would both be explored and compared.

3.4 VISSIM Model Development

After screening, processing, and aggregating the field data, the traffic inputs for a base VISSIM model could be developed. In addition to the sources cited within the literature review previously, microsimulation guidelines from several state DOTs including Florida, Oregon, Washington, and Virginia were utilized to help inform coding and calibration decisions (Dowling et al. 2004; Florida Department of Transportation 2014; Park and Won 2006; Washington State Department of Transportation 2014). Though the guidance provided by past work zone simulation case studies and sensitivity analyses provided a starting point from which to work, characteristics of the work zone used to calibrate the model in this study required many of the software's default parameters to be adjusted manually. This section will discuss setup of basic VISSIM network geometry, selection of volume inputs, fine-tuning of desired speed distributions, modification of key truck characteristics, and construction of time headway distributions.

3.4.1 Basic network coding

The first step in generating the work zone simulation network was coding basic geometric elements such as links, connectors, and their respective lengths and widths. Using the Bing Maps interface within VISSIM, it was possible to draw the

entire network to scale with a high degree of accuracy. Once drawn, the network was inspected to make sure that link lengths and lane widths matched those measured in the field and that other key components such as static routing decisions, desired speed decisions, and data collection points were at their ideal locations. It should be noted that despite having data from field sensors at 3.5 miles upstream of the lane closure, the VISSIM network began 2.5 miles upstream, primarily due to the volume measurement error for sensors 96 and 100 mentioned in Section 3.2. A simple drawing of the network is provided in Figure 1-14, which was sketched outside of the VISSIM software and does not include representation of the horizontal curvature found in the field.

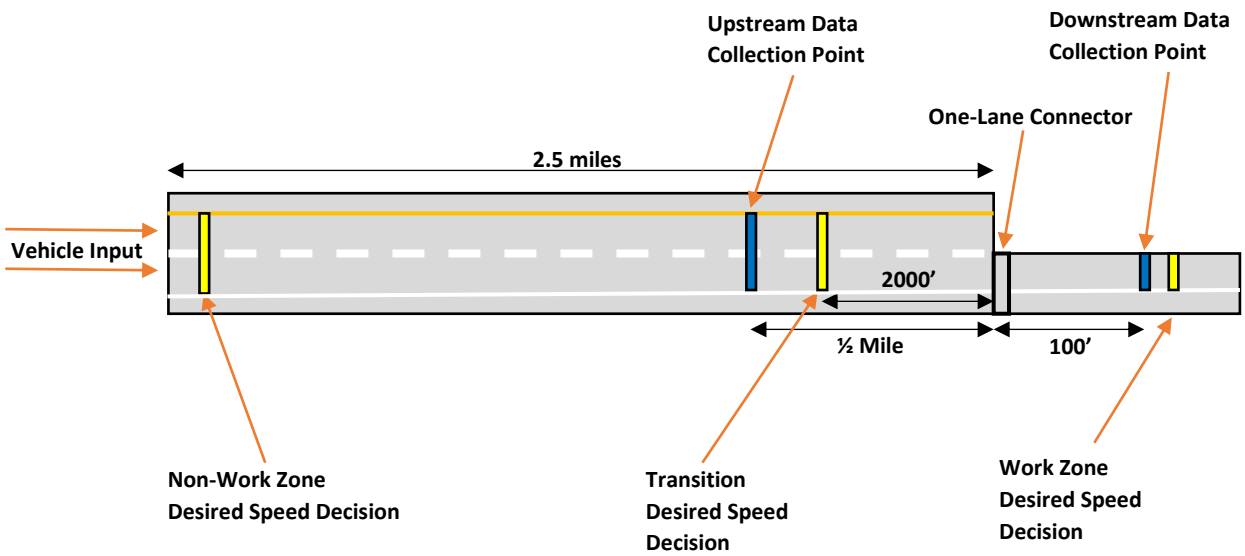


Figure 1-14: Diagram of VISSIM network (drawing not to scale)

As noted previously, the location of TTC devices in the field played a key role in the network setup process. Referring to Figure 1-7, the "transition" desired speed decision point pictured above was included so that the model could replicate the effects of the drop from a speed limit of 70 mph to a speed limit of 60 mph 2000 feet upstream of the lane closure. The "___ Lane Closed ½ Mile" sign in the field also motivated the decision to set the lane-changing distance upstream of the one-lane connector to 3000 feet, where it was anticipated that drivers could first visualize this sign. The last notable component of the diagram is the placement of data collection points at ½ mile upstream and 100 feet downstream of the lane closure. The former would be used in the calibration process to verify upstream speeds and queue propagation, while the latter would be used to detect breakdown events, measure queue discharge flow

rates, and gather average vehicle speeds within the work zone during each simulation run. Each of these components will be discussed in greater detail in the sections that follow, as extensive thought was involved in most model development decisions.

3.4.2 Volume inputs and traffic stream composition

The breakdown events summarized in Table 1-10 revealed a high amount of variability in the field data, so it was desired to develop a calibrated VISSIM model from a day representing typical conditions at the study work zone. After examining the full dataset, it was determined that traffic characteristics on October 3rd were the most representative, so volume inputs and relative vehicle class proportions were extracted from that day's data. Figure 1-15 provides a plot of 15-minute average speed and flow versus time on October 3rd and shows that a significant breakdown event was observed from approximately 1:45PM to 6:30PM. Available microsimulation guidance suggests that periods of congestion should be modeled with uncongested time intervals at the beginning and end of the study period to produce realistic results, so the model was coded to run from 11:45AM to 8:00PM. An additional 15 minutes from 11:45AM to 12:00PM was included despite not having field data for these intervals as a warm-up period to allow the model to reach equilibrium. Since literature has shown that freeway segments change state from stable flow to breakdown in brief time increments (Elefteriadou et al. 1995), 5-minute input volumes were coded in VISSIM to capture the same oscillations in traffic demand that were observed in the field.

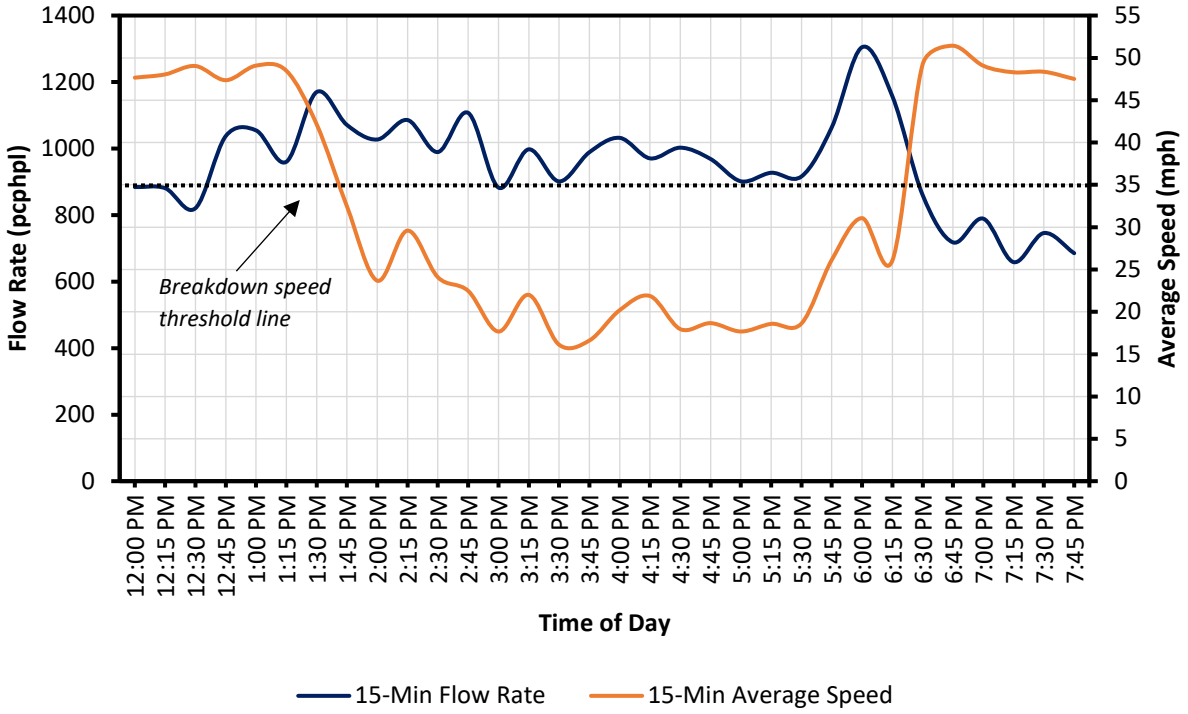


Figure 1-15: Speed and flow vs. time on October 3rd, 2016

As discussed earlier, the proportion of each vehicle class in the traffic stream was crucial for replicating field conditions, since heavy vehicles perform differently than passenger cars and queue length is sensitive to vehicle lengths. To account for sensor error and capture the actual percentage of heavy vehicles measured in the field during each time interval, the screening thresholds set in Section 3.3.1 were adopted to reduce the full set of raw vehicle records to a set of "classified" records from which proportions of vehicle classes would be determined. The number of trucks in each time interval were counted using the lower bound lengths of each sensor as defined in Table 1-8, then divided by the "classified" volume to calculate a best estimate of the truck percentage during each 5-minute interval. Finally, this percentage was multiplied by the raw volume of vehicles observed to obtain an adjusted truck volume for use in calculating passenger car equivalent flow rates. The same process was applied for 15-minute interval data, as these flow rates would eventually be used as a measure of calibration. The results of this process are exemplified in Table 1-11.

Table 1-11: Example of adjusted truck volumes

Time	Raw Volume	Raw Truck Volume	Classified Volume	% Trucks	Adjusted Truck Volume	Adjusted Flow Rate (pcphpl)
1:30:00 PM	75	19	69	28%	21	1148
1:35:00 PM	65	16	61	26%	17	985
1:40:00 PM	94	20	90	22%	21	1379
1:45:00 PM	71	6	68	9%	6	927
1:50:00 PM	73	14	70	20%	15	1051
1:55:00 PM	80	22	77	29%	23	1234
2:00:00 PM	67	13	60	22%	15	978
2:05:00 PM	61	17	58	29%	18	947
2:10:00 PM	83	13	80	16%	13	1158
2:15:00 PM	63	14	63	22%	14	924
2:20:00 PM	81	23	80	29%	23	1251
2:25:00 PM	78	12	77	16%	12	1082
2:30:00 PM	63	8	61	13%	8	855

The percentage of single unit trucks for each time interval was calculated using a similar procedure, where the length bounds for such vehicles were determined from the upper bound passenger car lengths and lower bound truck lengths for each individual sensor. The reader may refer to Table 1-8 and to Appendix A to see how these values were determined, but in general, single unit trucks were classified as vehicles 25-50 feet in length. The relative proportion of each vehicle class is given in Table 1-12, along with the static, rounded length assigned to each. It was assumed that defining multiple classes of vehicles within passenger cars, trucks, and single unit trucks to account for a finer distribution of lengths was not necessary, and those included in the table were believed to produce nearly identical results in the model.

Table 1-12: Base VISSIM model vehicle composition input

Vehicle Class	Relative Proportion of Traffic Stream			
	Right Lane	*Left Lane*	*Total*	*Length (ft)*
Passenger Cars	54%	71%	62%	16
Heavy Trucks	37%	12%	26%	74
Single Unit Trucks	9%	17%	12%	32

3.4.3 Desired speed distributions

The desired speed decision points pictured in Figure 1-15 were initially coded to match field-calculated distributions, such as those shown in Figures 1-11 and 1-12. In fact, the "transition" speed decision was omitted at first, and two desired speed distributions were applied—non-work zone and work zone. However, pilot simulation runs conducted at low input volumes revealed that speeds measured at the downstream data collector were approximately 10-20 mph higher than indicated by the field data under free flow conditions. As mentioned earlier, inspection of project traffic control reports revealed that there was a 60-mph reduced speed limit in place upstream of the closure for the duration of the work, so an additional desired speed decision point was added at this location. Since none of the sensors measured speeds between the speed limit change and the beginning of the lane closure, a representative speed distribution was built with a mean of 62 mph and standard deviation equal to that of upstream conditions. Trial and error in VISSIM revealed that these specifications produced speeds closest to those observed in the field at the bottleneck. Each of the three distributions were coded in VISSIM using the 10th, 25th, 35th, 50th, 75th, 85th, and 95th percentile speeds and are presented in Table 1-13.

Table 1-13: VISSIM desired speed distributions

Speed Distribution	Vehicle Class	Mean Speed (mph)	Percentile						
			10th	25th	35th	50th	75th	85th	95th
Non-Work Zone (Right Lane)	Passenger Cars	65	59	65	68	71	76	78	83
	Trucks	60	57	61	63	65	69	70	74
Non-Work Zone (Left Lane)	Passenger Cars	79	65	73	76	80	88	91	97
	Trucks	72	62	66	69	72	77	80	86
Transition (All Lanes)	All	62	53	57	60	62	67	68	73
Work Zone (All Lanes)	Passenger Cars	51	42	46	48	51	55	59	64
	Trucks	48	42	44	46	47	51	53	55

The complete free flow speed distributions for every sensor are provided in Appendix A, where one will note that the percentile values for the non-work zone distributions do not match those for any one sensor. Instead, the desired speed distribution upstream of the work zone was calculated from a weighted average of sensors 98, 99, 101, and 102. Additionally, the right-lane non-work zone speed distribution percentiles each had to be increased by 5 mph to replicate field-measured speeds in VISSIM. Even with such adjustments, it will be seen in a later section that the model could not be calibrated to match speeds perfectly, which literature has cited as a limitation of VISSIM (Kan et al. 2014). Nonetheless, the distributions above were deemed adequate since the results of this study were intended to be generalizable and prevailing speeds will vary between work zone sites.

3.4.4 Modification of key truck characteristics

The components detailed in the previous three sections were combined with default driving behavior parameters in VISSIM and the initial performance of the model was evaluated. After further research, it was found that the default power, weight, and acceleration distributions for heavy trucks in VISSIM are not representative of the U.S. truck fleet, but rather of the lighter and faster trucks found in Europe (Edara and Chatterjee 2010; Harwood et al. 2003; PTV Group 2017). By modifying truck acceleration capabilities to reflect those of the U.S.

fleet, it was found that congestion could be modeled at much more reasonable values of other calibration parameters, so assigning proper truck characteristics warranted further investigation.

First, potential sources of accurate truck weight distributions were explored. Fortunately, a past study by Turochy, Timm, and Mai applying weigh-in motion (WIM) data from ALDOT infrastructure revealed that there is a WIM station (ID 918) on I-59/I-20 at MP 100.0, just 30 miles north of the study work zone (Turochy et al. 2015). Data from this station was imported into Microsoft Access, where a weight distribution was calculated and exported to VISSIM. The power distribution for U.S. trucks was generated by assuming an average value of 328 horsepower, which matches that provided in NCHRP Report 505 for interstates in the eastern United States (Harwood et al. 2003). Table 1-14 compares the default weight and power distributions for heavy trucks in VISSIM to those applicable to the southeastern U.S. truck fleet and shows only a slight difference between the two. However, since truck power-to-weight ratios define maximum truck acceleration in VISSIM, they are critical to accurately modeling scenarios with a high percentage of trucks in the traffic stream.

Table 1-14: Comparison of truck power and weight in VISSIM

Percentile	Weight (lb), Default	Weight (lb), Adjusted	Power (kW), Default	Power (kW), Adjusted
0th	6,174	13,402	150	168
5th	10,275	29,831	163	171
25th	26,681	41,890	213	208
35th	34,883	47,483	238	221
50th	47,187	57,620	275	256
75th	67,694	70,203	338	282
85th	75,896	72,825	363	303
95th	84,099	76,146	388	387
100th	88,200	115,819	400	406

Conversely, there is a dramatic difference between the default truck acceleration curve in VISSIM and that of typical U.S. interstate semitrailers. Researchers at TTI found this to be the case in a 2006 study, but chose not to modify default values in VISSIM due to a lack of recent truck performance data (Middleton et al. 2006). A 2016 study of truck standstill acceleration at ramp meters in Florida, however, contended that the acceleration capabilities of heavy trucks in the U.S. have not improved much over time and provided detailed, updated curves for trucks with various weight-to-power ratios (Yang et al. 2016). To further justify the modification of these values in VISSIM, the truck acceleration curves in CORSIM were checked and found to align with those in the literature, as shown in Figure 1-16. Specifically, the acceleration curves for medium-loaded and fully loaded tractor-trailers is represented by performance indices 5 and 6 (PI 5 and PI 6), where acceleration values from a standstill are just above 2 ft/s^2.

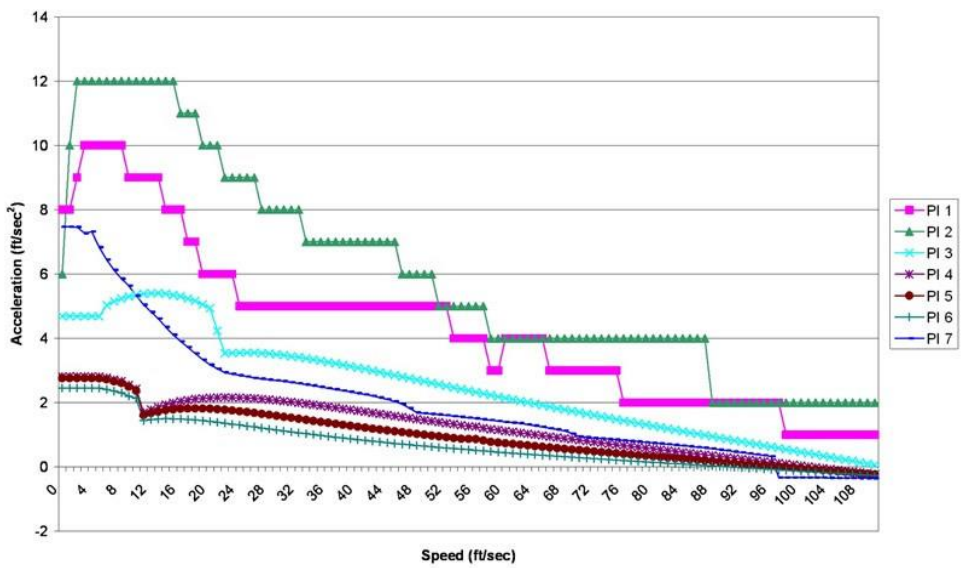


Figure 1-16: CORSIM vehicle acceleration curves (Source: FHWA)

In VISSIM, the acceleration curve for trucks starts at 8.2 ft/s^2, which is nearly equivalent to that used for buses in CORSIM. Therefore, starting acceleration values between 2 and 3 ft/s^2, followed by a decreasing function similar to that in Figure 1-16, were evaluated in VISSIM through trial and error until the most appropriate distribution was settled on. Figure 1-17 compares the default desired acceleration function in VISSIM with that developed from literature review and calibration to illustrate the disparity between the two.

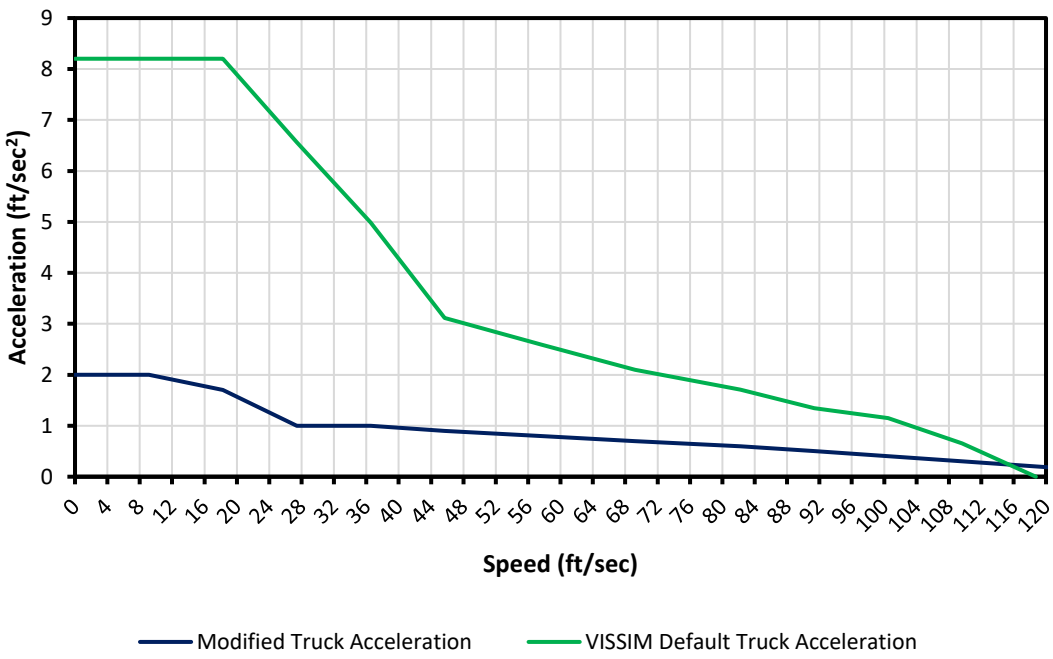


Figure 1-17: VISSIM default vs. calibrated truck acceleration

When comparing Figure 1-16 and Figure 1-17, it may be noted that the modified truck acceleration curve used in VISSIM is more conservative than that used in CORSIM. Review of the VISSIM User's Manual led to the finding that minimum and maximum acceleration values can be set at each speed, where the power-to-weight ratio of the heavy vehicle determines the exact value modeled (PTV Group 2017). Therefore, while the graph reflects only the mean acceleration at each speed value, maximum values were set that match more closely to the values in Figure 1-16. Furthermore, acceleration curves closer to the default in VISSIM were initially tested but determined not to replicate field speeds and flow rates without modifying other driving behavior parameters beyond reason. Lastly, the aforementioned work by Yang et al. in 2016 yielded a mean and 85th percentile standstill acceleration of 1.93 ft/s^2 and 2.24 ft/s^2, respectively, for heavy trucks. Since this was found to be the most recent vehicle performance study conducted to date, the values applied in VISSIM were thought to be reasonable and accurate.

3.4.5 Time headway distributions

Finally, the parameter found most critical to model development was desired time headway. Given that headway is the inverse of traffic flow, it is not surprising that sensitivity analyses have typically concluded that the

corresponding value of CC1 in VISSIM is the most influential in determining modeled throughput. Despite this, literature review of calibration methodology showed that many analysts select candidate values of time headway and other driving behavior parameters at random, then simply choose the values that best replicate field conditions. While this may produce valid data for a specific site and time period, it does not adequately capture the randomness of traffic flow and inherent variability that would be observed at a single site over time. Dong et al. noted this issue in a 2015 report and proposed that time headway parameters in microsimulation models be set based on field distributions if available (Dong et al. 2015). Such methodology would eliminate combinations of model parameters that may reproduce field conditions only with infeasible values of time headway, while also potentially reducing the time required for calibration.

The same study measured vehicle class-separated time headway and standstill distance on Iowa highways and found that both values depend on vehicle following pairs. Specifically, passenger cars were found to maintain shorter headways in general than heavy vehicles, but also maintained longer headways when following tractor-trailers. Several other studies have been conducted in the past and drawn similar conclusions, suggesting that traffic flow can be modeled more accurately if separate headway distributions are constructed for passenger cars and heavy vehicles (Houchin 2015). Despite the validity of these claims, field-calculated headway distributions could not be modeled in VISSIM prior to version 9, as the value of CC1 was static. Consequently, this study is believed to be one of the first work zone simulation studies to apply stochastic, vehicle class-specific headway distributions measured from field data.

Like the calculation of speed distributions, constructing time headway distributions required significant filtering of the data. Unlike for speed data, however, headway calculations were only valid if the order of vehicle records was maintained. To ensure that this condition was not violated, the data was not sorted or deleted during the entire process. Rather, several indicator columns were populated to designate whether a specific following pair was to be considered in developing headway distributions. Using traffic flow theory principles and the data screening procedure detailed in Section 3.3, three screening tests were created to determine if a given headway value was valid:

1. *Speed Screening:* Only following pairs where the speed of both vehicles was greater than 5 mph, less than 100 mph, and within two standard deviations of the average speed for a given 5-minute time interval were considered.

2. *Length Screening:* Both vehicles in a following pair were required to have lengths greater than 5 ft and less than 100 ft.
3. *Flow Screening:* To ensure that a following vehicle's choice of speed was constrained by a leading vehicle, only time intervals with flow rates greater than or equal to 1,000 pcphpl were examined.

This procedure was executed only for data from sensor 104, as it was expected that driving behavior within the work zone would be different from that upstream due to the changes in roadway environment. After applying the three tests listed above, approximately 65,000 vehicle following pairs remained and were used to calculate time headway distributions for each vehicle class. Initially, however, it was discovered that large headways greater than 6 seconds were unusually common at low following speeds. Further investigation led to the hypothesis that longer headways were measured for trucks when following passenger cars due to differences in vehicle acceleration capabilities. Thus, vehicles with speeds less than 35 mph (the speed threshold later used to determine the onset of breakdown) were ultimately excluded from headway analysis to eliminate this potential scenario.

Histograms of time headway for passenger cars and trucks at the study work zone are provided in Figures 1-18 and 1-19, where a clear difference may be observed between the two distributions. Interpretation of these graphs and the final VISSIM input require understanding of two main ideas: (1) the sequential data collected by sensors in this study measured the arrival time of vehicles to the nearest whole second, so analysis at finer increments was not possible; (2) the CC1 parameter in VISSIM is a portion of the desired safety distance between a lead and following vehicle, which is measured from front bumper to rear bumper, rather than front bumper to front bumper. As shown in Figure 1-20, this value is also shortened by the value of the standstill distance, CC0. That said, it was assumed that the distributions shown in the figures would need to be reduced by several tenths of a second to accurately reproduce field conditions.

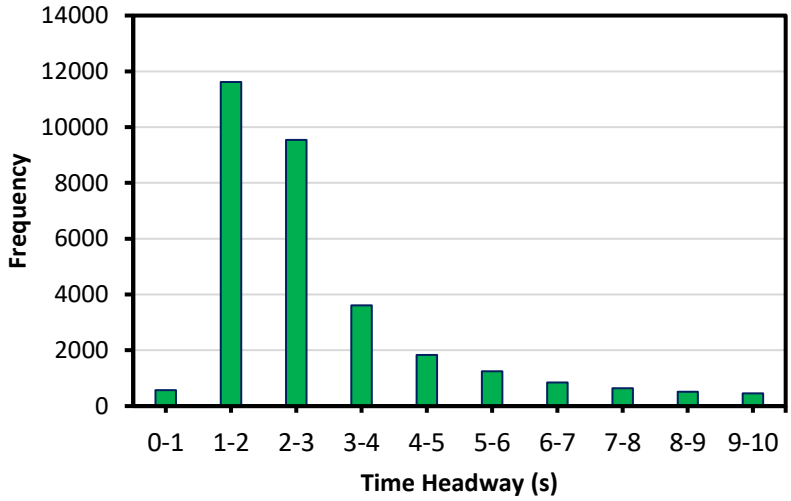


Figure 1-18: Field headway distribution (passenger cars)

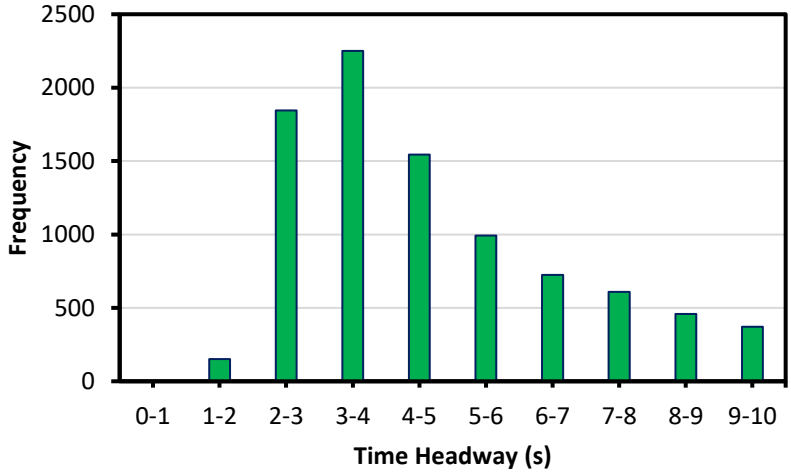


Figure 1-19: Field headway distribution (trucks)

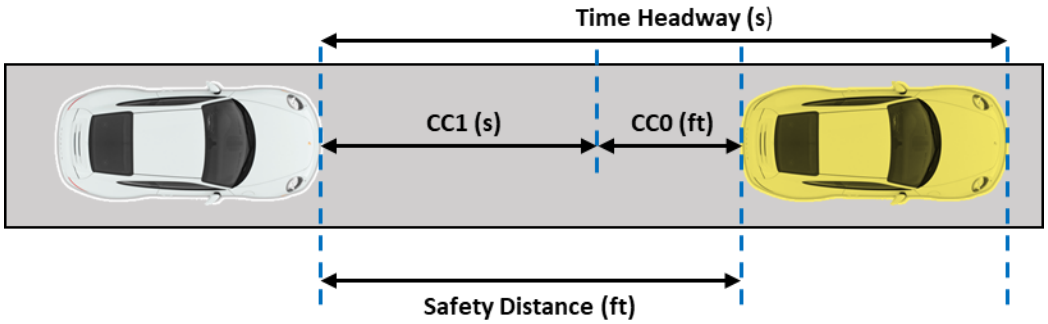


Figure 1-20: Desired safety distance in VISSIM

The last step in developing appropriate headway input for VISSIM was reducing the distributions shown in the figures to a set of reasonable values. For example, even at high flow rates, a headway of 10 seconds between two vehicles is likely due to circumstances beyond the selection of such a distance by drivers. Although literature has suggested typical maximum desired headways of 4-6 seconds, the field data was further scrutinized to set this threshold. For both passenger cars and trucks, the difference between successive headway intervals was plotted to observe the point at which no meaningful change in frequency occurred. These graphs are given in Figures 1-21 and 1-22.

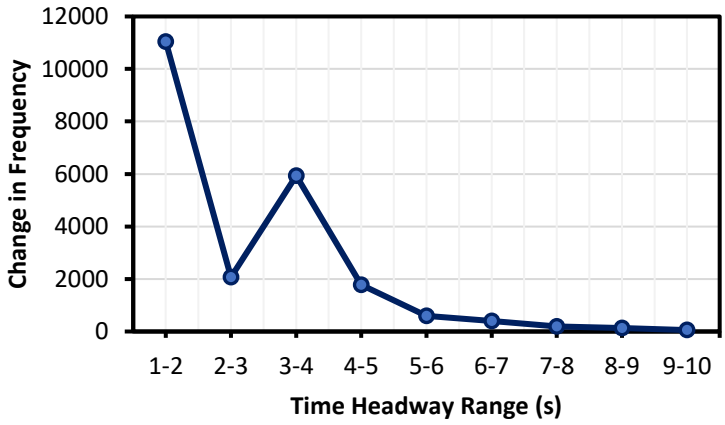


Figure 1-21: Change in headway frequency (passenger cars)

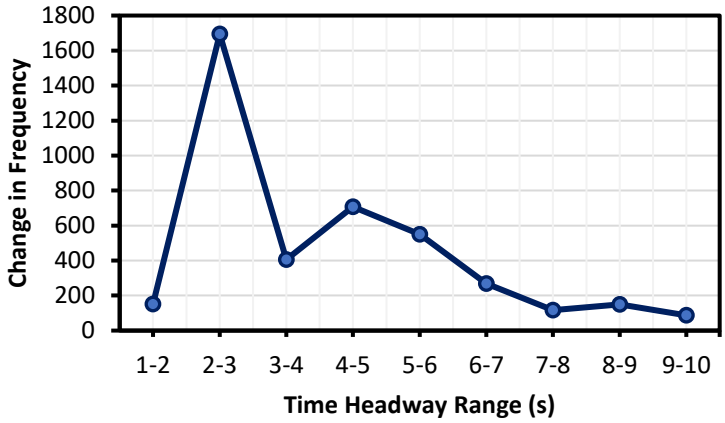


Figure 1-22: Change in headway frequency (trucks)

Based on the figures, the maximum desired headway ranges used in VISSIM were 5-6 seconds and 6-7 seconds for passenger cars and trucks, respectively. In model calibration, each point on these distributions was reduced uniformly to consider the portion of desired safety distance accounted for by vehicle length and standstill distance. It was determined that doing so would change measures of central tendency without changing the variances of the distributions. Table 1-15 summarizes the initial headway distributions applied in VISSIM for the CC1 parameter, while calibrated values will be discussed in the next section.

Table 1-15: VISSIM input desired headway distributions

Headway (s)	Passenger Cars		Trucks	
	Frequency Below	% Below	Frequency Below	% Below
1	577	1%	0	0%
2	11618	43%	151	2%
3	9545	76%	1845	27%
4	3607	89%	2251	57%
5	1835	96%	1543	77%
6	1246	100%	993	90%
7	--	--	724	100%

3.5 Calibration and Validation

Even when field geometry, volume, and speed data are carefully entered, simulation models may not acceptably replicate field conditions if default vehicle characteristics and driving behavior parameters are used. Accordingly, these parameters must be iteratively modified until the model is deemed to be reasonably calibrated. After achieving calibration, it is common practice to validate a simulation model using independent data from the same site to test its predictive abilities. While this process is often the most time-consuming part of developing microsimulation models, it is paramount to producing realistic outputs and drawing meaningful conclusions. This section describes the procedure used to evaluate the base VISSIM model described in Section 3.4, define and modify significant input parameters, and validate the calibrated parameter set against field-collected measures of effectiveness (MOEs).

3.5.1 Calibration methodology

Since the end users of a potential tool developed from this study are agencies and practitioners, guidance was taken from typical reference material prepared by and for these groups. As mentioned in the previous section, these sources included the FHWA Traffic Analysis Toolbox, reference manuals from state DOTs, and literature specific to freeway work zones (Dowling et al. 2004; Florida Department of Transportation 2014; Park and Won 2006; Washington State Department of Transportation 2014). Such methodology involves generating a reasonable number of calibration parameter combinations and choosing the set of values that best matches to field collected MOEs. However, the time required for calibration can quickly balloon if parameters are not selected carefully and objectives are not well-defined.

To maximize efficiency, the literature generally suggests that the following steps be followed when calibrating microsimulation models:

1. <u>Define calibration objectives:</u> Select at least two MOEs (e.g. throughput, travel time, speed) that may be used to compare simulation outputs to field data. Define an acceptable amount of deviation from field data using statistical measures such as root mean square normalized error (RMNSE) or mean absolute percentage error (MAPE).
2. <u>Perform multiple simulation runs with the default parameter set:</u> Verify that the model is not adequately calibrated with default parameters.

3. Select Calibration Parameters: Determine which parameters are most significant to the calibration objectives. The number of remaining parameters should be minimized.
4. Determine Feasible Range of Values: Use pilot simulation runs to determine the range of values that should be explored for each parameter.
5. Search for Optimal Parameter Set: Iterate with each parameter set, collect MOEs, and choose the combination that best reproduces field data.
6. Fine-Tune: Visually inspect the simulation animation and make minor changes as necessary to ensure realistic driving behavior and best match outputs to field conditions.

The six steps listed above were adhered to throughout calibration to minimize the effort necessary to achieve a satisfactory model. Based on the variability observed in the field data, sensor error, and past practice in literature, modest calibration objectives were set. Since most analyses performed by researchers and practitioners are conducted for facilities with recurring sources of congestion, it was expected that calibrating to a non-recurring source of congestion such as a work zone would be challenging. Therefore, the objectives summarized in Table 1-16 were found acceptable for this research.

Table 1-16: VISSIM input desired headway distributions

Measure of Effectiveness	Measurement Location	Calibration Metric(s)
15-Minute Average Speed (mph)	Lane Closure (Sensor 104)	RMSNE < 0.20 MAPE < 20%
Mean Queue Discharge Rate (pcphpl)	Lane Closure (Sensor 104)	Within 10% of Field Value
Queue Propagation and Dissipation	1/2 Mile Upstream (Sensors 99, 103)	Qualitative

The findings from the literature and early experimentation in VISSIM indicated that the most significant parameters to replicating the congested conditions observed in the field were CC0, CC1, CC2, SRF, lane-changing distance, and desired acceleration for heavy vehicles (the reader may refer to Chapter Two for

definitions of these parameters). As such, the calibration effort focused on modifying these six variables until model performance was acceptable. However, even six changeable parameters were thought to be excessive, so strategies were developed to reduce the number of candidate variables and the ranges of their values. First, though many have found the CC2 parameter to be influential to modeled throughput, guidance from literature and pilot simulation runs suggested that more realistic driving behavior could be observed by holding this value at its default of 13.12 feet (Washington State Department of Transportation 2014). Second, the SRF is highly dependent on the value of the lane-changing distance, which was held constant at 3000 feet, so this value was also held static at its default value of 0.60.

This logic reduced the final set of calibration parameters to CC0, CC1, and desired truck acceleration. Though the latter two parameters were initially modified as outlined in Sections 3.4.4 and 3.4.5, it was expected that additional changes would be necessary before reaching calibration. For example, it was noted earlier that the time headways calculated from field data would likely need to be reduced since a portion of time headway in VISSIM is accounted for by the standstill distance and following variation parameters. For desired truck acceleration, six candidate distributions were developed based on literature review and intended to bracket typical values with mean standstill acceleration between 2 and 3 ft/s^2. Finally, the range for CC0 was based on anecdotal experience and field-measured values from a study in Iowa (Dong et al. 2015; Houchin 2015). Table 1-17 shows the ranges and search increments explored for each parameter, where the empirical distribution referenced for CC1 corresponds to the values found in Table 1-15.

Table 1-17: Calibration parameter ranges

Parameter	Default	Feasible Range (Literature)	Explored Range	Increment
CC0 (ft)	4.92	> 4.92	8 - 16	2
CC1 (s)	0.9[b]	0.9 - 4.0[b]	Empirical distribution reduced by 0.1 – 0.7	0.1
Desired Truck Acceleration (ft/s^2)[a]	8.2	< 8.2	2.0 - 3.0	0.2

[a] Mean acceleration from a standing start

[b] Static values

3.5.2 Calibration results

If every possible combination of the three parameters in Table 1-17 was checked, the result would be 210 unique cases (five values of CC0 x seven-time headway distributions x six desired truck acceleration distributions), which would require extensive effort. Fortunately, since each parameter is potentially correlated with the others, several extreme scenarios could be eliminated. For example, high values of CC0 and low values of desired truck acceleration were not necessary for the model to produce congestion like that observed in the field when paired with longer time headways. Conversely, if smaller values of CC0 or larger values of desired truck acceleration were used, longer time headways could be applied. Initially, five simulation runs were conducted for each candidate parameter set to identify a shorter list of combinations that should be examined further. Once the list had been narrowed, it was necessary to calculate the number of simulation runs required to be statistically confident in model outputs using equation 1-4 (Washington State Department of Transportation 2014).

$$N = \left(\frac{2 * t_{1-\frac{\propto}{2}, N-1} * S}{CI_{1-\propto\%}}\right)^2 \qquad (1\text{-}4)$$

Where:

N = number of required repetitions

$t_{1-\alpha/2,\ N-1}$ = t-statistic at a confidence level of 1-α and N-1 degrees of freedom

s = standard deviation of model outputs

$CI_{1-\alpha\%}$ = confidence interval for the true mean of a given parameter

For the purposes of calibration, the mean QDR was chosen as the determining metric for calculating the required number of repetitions. Multiple simulation runs yielded a sample standard deviation of 70 pcphpl, so at a 95% confidence level, a minimum of 10 runs was required to estimate the mean QDR to within 10% of the field data. Other metrics were not examined as carefully because it was hypothesized that traffic flow is too variable to expect a high degree of precision and consistency in modeled speeds and queue lengths. Nonetheless, to best capture fluctuations in the onset of breakdown between simulation runs, the number of repetitions was increased to 20.

The final parameter set is presented in Table 1-18, and plots of speed vs. time for the field data, default VISSIM model, and calibrated VISSIM model are compared in Figure 1-23. The slight difference between the calibrated model outputs and field measurements supports the theory proposed previously, as it was not possible to match queue duration and speeds more so than what is shown in the figure. Particularly, the sharp drop in speed signaling the onset of breakdown was consistently observed too soon in the model, suggesting that simulated traffic may not be able to absorb brief spikes in volume as real traffic does in the field. That said, observation of speed differentials showed that the greatest discrepancy actually existed later in the simulation period. Since traffic flow characteristics well after the occurrence of breakdown were not relevant to research objectives, only average speed and QDR data from 12:00PM to 3:30PM were required to meet calibration objectives. A similar truncated time window would be examined for data from October 6th during the validation phase for the purposes of consistency.

Table 1-18: Calibrated driving behavior parameters

Parameter	Description	Default Value	Calibrated Value
Car-Following Parameters			
CC0	desired standstill distance	4.92 ft	10 ft
CC1	desired time headway	0.9 s	Empirical Distribution with 0.35 s subtracted[a]
CC2	additional distance over desired safety distance	13.12 ft	Default
CC3 - CC9	--	--	Default
Lane-Changing Parameters			
Lane-Changing Distance	distance upstream of a required lane change that drivers will begin looking for gaps to merge	656.2 ft	3000 ft
SRF	safety distance reduction factor	0.6	Default
Cooperative Braking	check box (yes or no)	No	Yes
Maximum Deceleration for Cooperative Braking	maximum accepted deceleration when braking cooperatively	-9.84 ft/s^2	-20 ft/s^2
Waiting Time Before Diffusion	maximum waiting time before vehicle removed from network	60 s	200 s
All others	--	--	Default

[a]See Table 1-15 for original empirical distribution

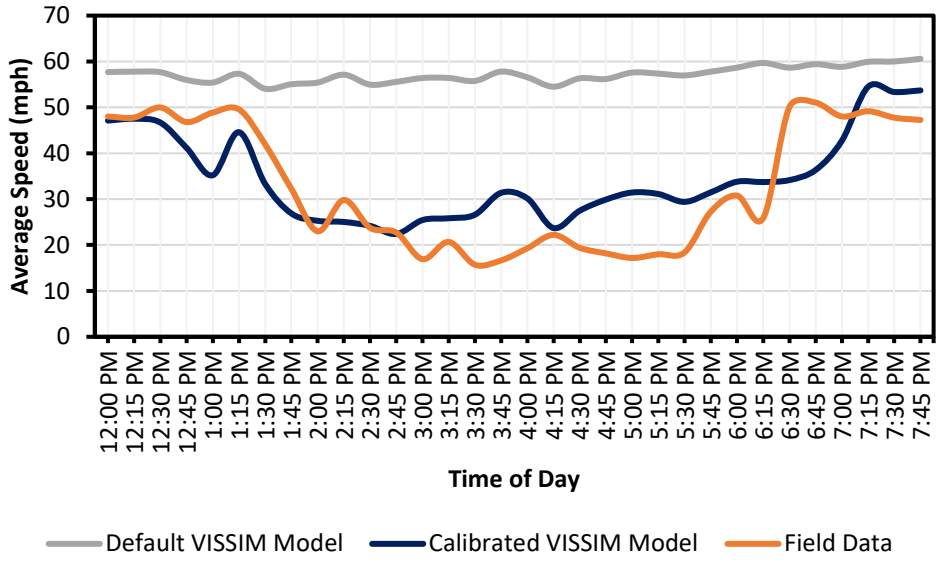


Figure 1-23: Field, default VISSIM, and calibrated VISSIM speed profile comparisons

In Table 1-18, the adjusted empirical distribution for time headway involved subtracting 0.35 seconds from each entry in Table 1-15, resulting in values of 0, 0.65, 1.65, 2.65, 3.65, etc. in the final input distribution. Though increments of 0.1 seconds were originally explored, smaller increments of 0.05 seconds were necessary during the fine-tuning stage for the final model to meet calibration objectives. These objectives are summarized in Table 1-19 and Figure 1-24 for 15-minute average speeds and mean QDR.

Despite slight discrepancies in speed profiles, both the RMSNE and MAPE were well within calibration objectives for the period from 12:00PM to 3:30PM on October 3rd and considered adequate. The mean QDR in VISSIM was only 2% higher than that measured in the field, and an overlay of the flow rate histograms shows that the variance of the two distributions are somewhat similar. The most notable difference between the field data and simulation outputs shown in Figure 1-24 is that the highest and lowest queue discharge flow rates were unobservable in the model. This finding substantiates the claim that the variability of real-world driving behavior cannot be fully replicated using microsimulation.

Table 1-19: Calibration summary (October 3rd, 2016)

Time	VISSIM Speed (mph)	Field Speed (mph)	RMSNE	MAPE
12:00:00 PM	47.1	48.0	0.000	2%
12:15:00 PM	47.5	47.8	0.000	1%
12:30:00 PM	46.7	50.0	0.004	7%
12:45:00 PM	41.2	46.8	0.015	12%
1:00:00 PM	35.2	48.9	0.078	28%
1:15:00 PM	44.6	49.6	0.010	10%
1:30:00 PM	33.3	41.9	0.042	21%
1:45:00 PM	26.9	32.2	0.028	17%
2:00:00 PM	25.3	23.0	0.010	10%
2:15:00 PM	25.0	29.8	0.026	16%
2:30:00 PM	24.2	23.8	0.000	2%
2:45:00 PM	22.4	22.7	0.000	1%
3:00:00 PM	25.4	16.9	0.253	50%
3:15:00 PM	25.8	20.7	0.062	25%
Total			**0.194**	**14%**
Mean QDR (pcphpl)				
Calibrated VISSIM Model	**Field**		**Error**	
1032	1016		2%	

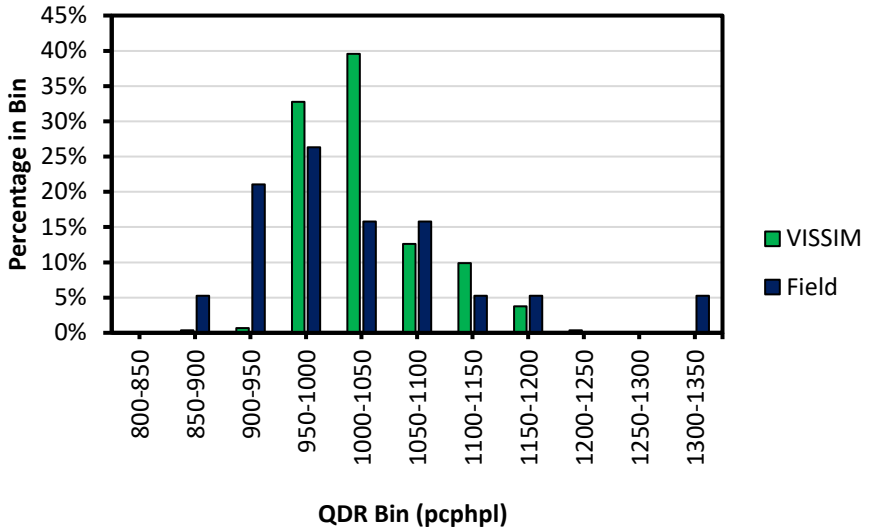


Figure 1-24: QDR distribution comparison (October 3rd, 2016)

3.5.3 Model validation

While the results from the calibrated model were promising, they only applied to data from October 3rd and needed to be validated using input from a different day. Data from October 6th, 2016, was ultimately selected because congestion was observed during off-peak hours, providing a unique set of traffic volume characteristics to be modeled. Field data showed a continuous period of congestion beginning at approximately 9:45AM, so VISSIM was coded to run from 7:45AM to 11:30AM, which included a 15-minute warm-up period and 3.5 hours of independent volumes and vehicle compositions. A comparison of the speed profiles generated in VISSIM and observed in the field is provided in Figure 1-25.

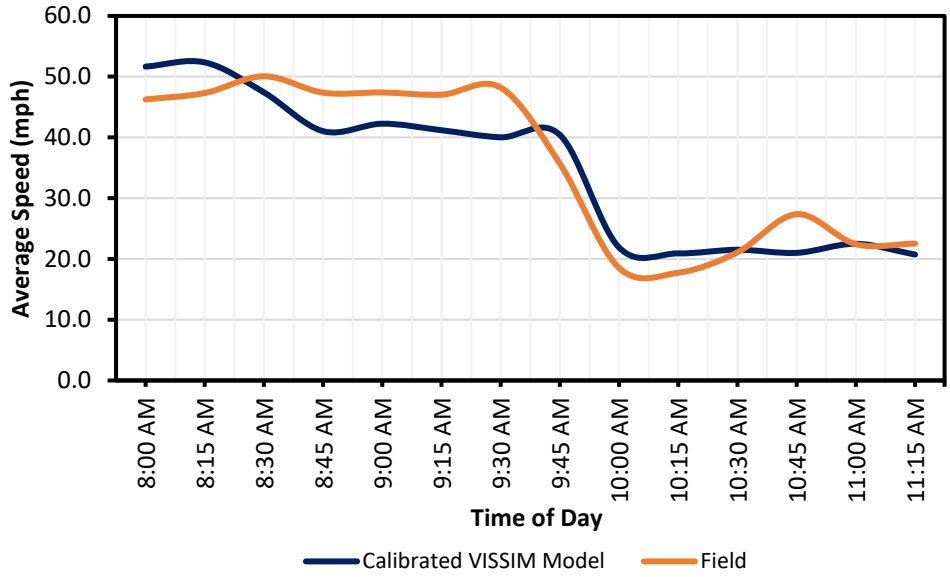


Figure 1-25: Field and validated VISSIM speed profile comparisons

Figure 1-25 shows that speeds in VISSIM dropped slightly below those observed in the field approximately 45 minutes into the simulation, but generally matched field speeds otherwise. In fact, the validated model adhered to objectives even more closely than those for October 3rd, further increasing confidence in the validity of the model. The same number of simulation runs (20) were conducted to achieve statistical confidence in the validation results presented in Table 1-20, as the standard deviation of the QDR was nearly identical to that observed during the calibration process. Following the table, Figure 1-26 provides another overlay of QDR histograms to show that the simulated distribution contains most of the field distribution. Like for October 3rd, VISSIM was unable to capture flow rates near the lower and upper bounds of the distribution, emphasizing the assertion that simulation models cannot mimic the variability of real-world traffic. Nonetheless, the model was deemed successfully validated.

Table 1-20: Validation summary (October 6th, 2016)

Time	VISSIM Speed (mph)	Field Speed (mph)	RMSNE	MAPE
8:00:00 AM	51.7	46.3	0.014	12%
8:15:00 AM	52.3	47.3	0.011	11%
8:30:00 AM	47.4	50.1	0.003	5%
8:45:00 AM	41.0	47.4	0.018	13%
9:00:00 AM	42.3	47.4	0.012	11%
9:15:00 AM	41.2	47.0	0.015	12%
9:30:00 AM	40.0	48.2	0.029	17%
9:45:00 AM	40.4	35.7	0.018	13%
10:00:00 AM	21.9	18.5	0.033	18%
10:15:00 AM	20.9	17.7	0.032	18%
10:30:00 AM	21.5	21.1	0.000	2%
10:45:00 AM	21.0	27.4	0.054	23%
11:00:00 AM	22.5	22.4	0.000	0%
11:15:00 AM	20.7	22.5	0.006	8%
Total			**0.132**	**12%**

Mean QDR (pcphpl)		
Validated VISSIM Model	**Field**	**Error**
1024	1053	3%

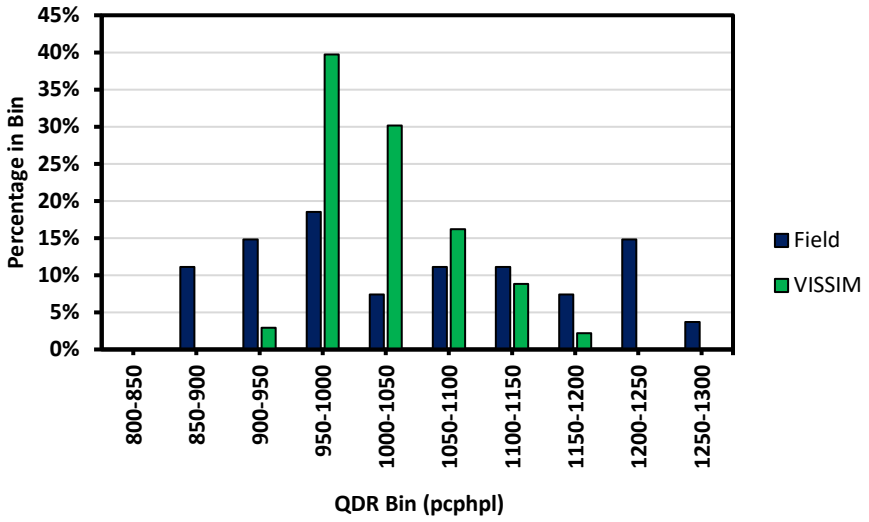


Figure 1-26: QDR distribution comparison (October 6th, 2016)

3.6 Exploration of Probabilistic Model for Freeway Work Zone Capacity

After development, calibration, and validation of the traffic simulation model of the study site, exploration of a breakdown probability model at the site could begin. This section lays the groundwork for development of the model that can then form the basis of a freeway work zone lane closure analysis tool. Based on the literature review, the following factors (that can be modeled) may have a substantial effect on the probability of breakdown and subsequent queueing at freeway work zones involving lane closures: traffic volume, upstream lane distributions, free flow speed, speed variance, truck percentage, and lane closure configuration. Since field data were only available for a single site with a 2-to-1 lane closure, scenarios involving other configurations and traffic characteristics were not considered in the planning for a traffic simulation experiment but may be topics for further research, some of which will be explored part of a subsequent study, STRIDE Project P2. Traffic volume, truck percentage, and lane closure side would be included as explanatory variables and expected to be sufficient for capturing the variability in breakdown flow rates that could reasonably be expected to occur at similar sites in the United States. This section describes the characterization of typical traffic conditions on rural freeways and application of those conditions to input parameters in the traffic simulation package VISSIM.

Section 3.3.3 underscored the importance of fully exploring field data from the study site before setting calibration and validation objectives for the VISSIM model. That discussion centered on flow rates immediately prior to breakdown and during congestion, but it was equally vital to the experiment design phase to understand the natural rise and fall of volumes and truck percentages from hour-to-hour and day-to-day. All key pieces of literature related to studying breakdown at freeway facilities

emphasized the need to collect field data for up to one year if possible (Kondyli et al. 2013; Lorenz and Elefteriadou 2001), during which a myriad of traffic conditions may be observed. Since such an extensive data collection period is not feasible for most work zones, simulation was utilized to accomplish this task. As diverse of a set of traffic volumes and truck percentages as possible, given that data were available from one site for a two-week period, were used. Table 1-21 contains daily and peak hour volumes for the two weeks of field data collection conducted as part of this research and shows that conditions vary greatly even over a short period.

Table 1-21: Study work zone volume summary

Day	Total Daily Volume (vehicles)	Peak Hour Volume (vehicles)	K Factor (%)	Trucks During Peak Hour (%)
Monday, October 3, 2016	7,614[b]	891	--	26
Tuesday, October 4, 2016	12,615	828	6.6	36
Wednesday, October 5, 2016	13,340	915	6.9	39
Thursday, October 6, 2016	14,153	922	6.5	30
Friday, October 7, 2016	16,693	1,244	7.5	19
Saturday, October 8, 2016	13,762	984	7.2	23
Sunday, October 9, 2016	13,330	1,105	8.3	16
Monday, October 10, 2016	12,714	1,000	7.9	13
Tuesday, October 11, 2016	12,500	968	7.7	37
Wednesday, October 12, 2016	13,041	866	6.6	36
Thursday, October 13, 2016	13,703	948	6.9	22
Friday, October 14, 2016	16,792	1,224	7.3	21
Saturday, October 15, 2016	13,871	1,030	7.4	21
Sunday, October 16, 2016	13,717	1,105	8.1	18
Averages	**13,864**	**1,002**	**7.3**	**25**

[b]Traffic data was collected for only 12 hours on this day

The table was created using demand volumes from sensors 97 and 101, 2.5 miles upstream of the lane closure at the study site and revealed several interesting trends. First, both total and peak hour volumes were substantially higher on the weekends, especially Fridays, while truck percentages were greater on weekdays. Second, the values in the table do not coincide with those calculated using the AADT, K factor, and D factor given in Figure 1-7. The nearest permanent counting station maintained by ALDOT had a 2016 AADT of 27,890 vehicles, K factor of 10%, and D factor of 53%, suggesting that the peak directional hourly volume at the study work zone should be more than 1,300 vehicles, even if I-59/I-20 southbound is not the peak direction. Sensor data contradicted these calculations and showed an average peak hour volume of 1,002 vehicles, which would require a K and D factors of approximately 7.3% and 50%, respectively, using an AADT of 27,890 vehicles. Given that there was only one minor interchange between the study work zone and counting station in question, it is unclear why such large discrepancies were observed. Nonetheless, this finding stresses the need for practitioners to verify traffic conditions at work zones of interest.

Finally, the data show that a site averaging 25% trucks during the peak hour may produce as few as 5% or as many as 40% trucks during an individual peak period. Accordingly, for the development of a probabilistic model for the capacity of a freeway lane closure, this window is recommended as the minimum range to be explored to ensure that VISSIM output would capture the full range of site conditions that would be observed from several weeks or months of data collection. Trial simulation runs were found to produce comparable variance in truck percentage by time interval even when a single value was used as input, ultimately motivating the use of several static percentages to represent sites with different average truck volumes. Likewise, a range of target peak hour volumes would be applied so that fluctuations of up to several hundred vehicles per hour between hours of the day and days of the week could be accounted for.

To best mimic real-world traffic conditions, it would also be necessary to gradually increase modeled demand volumes in the same pattern observed in the field. Since this study asserts that capacity is not a static value, lower traffic volumes would need to be simulated to allow for opportunities for breakdown at flow rates less than the expected 'average' (static) capacity. Conversely, the construction of breakdown probability models also requires that ample uncongested, or censored, flow rates be observed at these lower volumes. Lastly, just as microsimulation models are typically coded with initialization periods, a similar extended period of steady volume increase would allow the model to reach equilibrium and ensure realistic outputs. To inform volume inputs, demand profiles for each day of data at the study work zone were examined and assumed to be representative of other four-lane rural freeways across the southeastern

United States. Figure 1-27 provides a representative example of traffic volumes throughout the day at the study site.

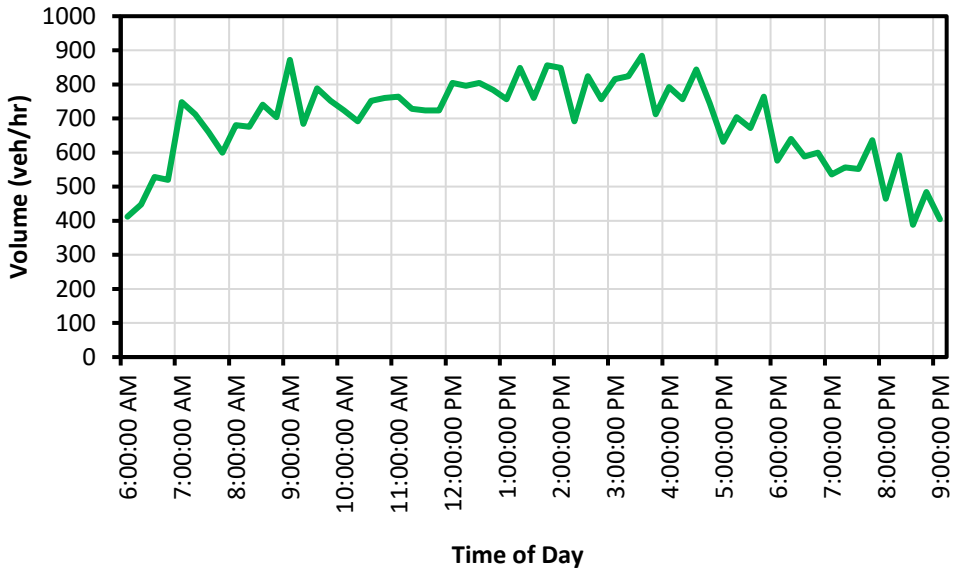


Figure 1-27: Traffic demand profile (October 4th, 2016)

Given the traffic data from the study site, it is recommended that traffic volumes ranging from 700 to 1300 vehicles per hour should be simulated in a larger scale study proposed as further research. Such an experiment involves simulating cases across this range of traffic volumes and a range of truck percentages between 5% and 40%, as well as left and right side 2-to-1 lane closures. While execution of the full experiment is beyond the scope of the current study, a pilot or test case, specifically for traffic comprised of 10% heavy trucks, for a left lane closure, has been developed.

3.7 Development of Test Case for Probabilistic Model

A plan for thorough study of rural freeway work zone capacity from a stochastic perspective, and development of associated breakdown probability models, was outlined in the previous section. Execution of this process across a range of traffic volumes, but specifically for a single value of heavy truck percentage (10%) and a left-side lane closure, in described in this section.

3.7.1 Data preparation

As stated previously, the transition from stable flow to congestion has been found to occur suddenly, so 5-minute data aggregation intervals were anticipated to shed more light on the underlying relationship between traffic flow, truck percentage, and the probability of queue formation. Conversely, 15-

minute intervals were expected to be less precise in capturing the breakdown phenomenon but necessary to ensure that agencies and practitioners could apply readily available traffic data to a lane closure analysis tool once developed.

Once the simulated data were appropriately aggregated, individual flow records were subdivided into one of three categories: breakdown flow rates, uncongested flow rates, and queue discharge flow rates. This step was necessary because congested flow rates provided no information about the likelihood of breakdown at a given level of demand and could be disregarded. On the contrary, stable flow rates up to those immediately preceding breakdown were significant because they combined to indicate the sustainability of a given traffic volume. The preceding literature review or external sources provide further discussion on the issue of capacity measurement at freeway facilities (Elefteriadou 2014; Lorenz and Elefteriadou 2001; Roess and Prassas 2016).

Classification of records into the bins mentioned above was accomplished using three distinct breakdown identification algorithms, one for each aggregation interval. In all cases, a 35-mph speed threshold was applied based on the definition of breakdown in the 6th edition of the HCM, which specifies “a sudden drop in speed at least 25% below the free flow speed for a sustained period of at least 15 minutes” (Transportation Research Board 2016). Since the free flow speed at the lane closure bottleneck was approximately 50 mph, a reduction to at least 37.5 mph would be required to meet this definition. For simplicity, this value was rounded down to 35 mph. While the 15-minute period of sustained congestion cited by the HCM was adhered to for 5-minute data, it was found that too many false breakdown events were identified at 15-minute aggregation intervals when such criteria were used. To prevent this occurrence, it was required that speeds be maintained below 35 mph for 2 consecutive intervals, or 30 minutes, in the latter case. Similarly, recovery from breakdown was signaled by an increase in speeds above 35 mph for the same number of consecutive time intervals. Table 1-22 provides an example of the Excel output generated after executing the breakdown identification algorithm for a set of 5-minute simulated data.

Table 1-22: Example breakdown identification

Simulation Run	5-Minute Average Speed (mph)	5-Minute Flow Rate (vphpl)	Breakdown Flow Rate (1 = Yes)	Queue Discharge Flow Rate (1 = Yes)
1	51.2	1032	0	0
1	52.1	936	0	0
1	46.0	1056	0	0
1	50.4	912	0	0
1	44.9	1104	0	0
1	49.3	996	0	0
1	47.2	1092	1	0
1	22.6	1032	0	1
1	22.0	1116	0	1
1	20.5	1104	0	1
1	23.7	1200	0	1
1	24.2	1140	0	1
1	21.0	1116	0	1
1	20.4	1032	0	1
1	19.7	1056	0	1

The sample data in the table confirms that the breakdown identification algorithm successfully identified the flow rate immediately prior to the sudden drop in speed from 47 mph to 23 mph, then classified all subsequent flow records as congested. If the table were to continue vertically, a “1” would be recorded in the last column until average speeds recovered above 35 mph for at least three consecutive 5-minute intervals (thereby indicating queue clearance and a return to uncongested flow) or data from the second simulation run began, whichever occurred first. The example shown in Table 1-22 indicates that the breakdown flow rate did not coincide with the highest flow rate observed prior to congestion, confirming one of the fundamental concepts of traffic flow

theory from the literature. Namely, capacity is stochastic and may be represented by a wide range of flow rates even under identical prevailing conditions.

3.7.2 Survival analysis methodology

Aggregated, classified simulation data was evaluated using Kaplan-Meier survival analysis, also known as the product-limit method. This statistical methodology uses data on the lifetime of individuals to determine the approximate probability of reaching a terminal state at a given point in time. Here, "individuals" refer to traffic flow records, and the "terminal state" is the onset of breakdown. Theoretical aspects of this approach were presented in the literature review, but the focus of this section will be on its practical application to simulated data. Construction of breakdown probability models using the product-limit method was accomplished by following the steps below, each referencing Table 1-23 (as an example for cases with 10% trucks).

1. Simulated data was aggregated by time interval and classified by one of three flow regimes: breakdown, uncongested, or congested.
2. After discarding congested flow rates, breakdown and uncongested flow rates were summed at each observed volume (**Columns A, B, and C**).
3. Flow rates at all volumes were summed to obtain the total risk set, or number of flow rates which had the potential to be followed by a breakdown event (**Column D, row 1**).
4. In each subsequent row of the table, the remaining risk set was determined using equation 1-5 (**Column D, rows 2 through end)**:

$$Risk\ Set_i = Risk\ Set_{i-1} - Breakdowns_{i-1} - Uncongested_{i-1} \qquad \textbf{(1-5)}$$

5. Cumulative survival and breakdown probabilities were calculated using equations 1-6, 1-7, and 1-8 (**Columns E, F, and G**):

$$Factor = \frac{Breakdowns_i}{Risk\ Set_i} \qquad \textbf{(1-6)}$$

$$Probability\ of\ Survival = (1 - Factor_i) * Probability\ of\ Survival_{i-1} \qquad \textbf{(1-7)}$$

$$Probability\ of\ Breakdown_i = 1 - \ Probability\ of\ Survival_i \qquad \textbf{(1-8)}$$

Table 1-23: Survival analysis table (10% trucks, 5-minute aggregation interval, left-side lane closure)

A	B	C	D	E	F	G
Volume (vphpl)	*# Breakdown Flow Rates at Volume*	*# Uncongested Flow Rates at Volume*	*Risk Set*	*Factor*	*Probability of Survival*	*Probability of Breakdown*
708	1	47	6584	0.000152	100.0%	0.0%
780	1	103	6070	0.000165	100.0%	0.0%
816	1	139	5744	0.000174	100.0%	0.0%
900	5	160	4718	0.00106	99.8%	0.2%
912	5	156	4553	0.001098	99.6%	0.4%
924	6	182	4392	0.001366	99.5%	0.5%
936	8	212	4204	0.001903	99.3%	0.7%
--	--	--	--	--	--	--
1140	10	92	981	0.010194	92.3%	7.7%
1152	7	66	879	0.007964	91.6%	8.4%
1164	5	61	806	0.006203	91.0%	9.0%
1176	5	39	740	0.006757	90.4%	9.6%

-- Table abbreviated to allow higher breakdown probabilities to be visible

An empirical breakdown probability distribution can be built, for a specific truck percentage and lane closure side, by combining the data gathered from each corresponding input volume. For example, Table 1-23 was developed from the simulated data for a left-side lane closure with 10% trucks, aggregated into 5-minute time intervals. Since this combination was simulated at input volumes of 1000, 1100, 1200, and 1300 vph, four sets of flow data were used to complete the table. The distribution associated with this table is given in Figure 1-28, and reflects trends observed for data aggregated into 15-minute time intervals.

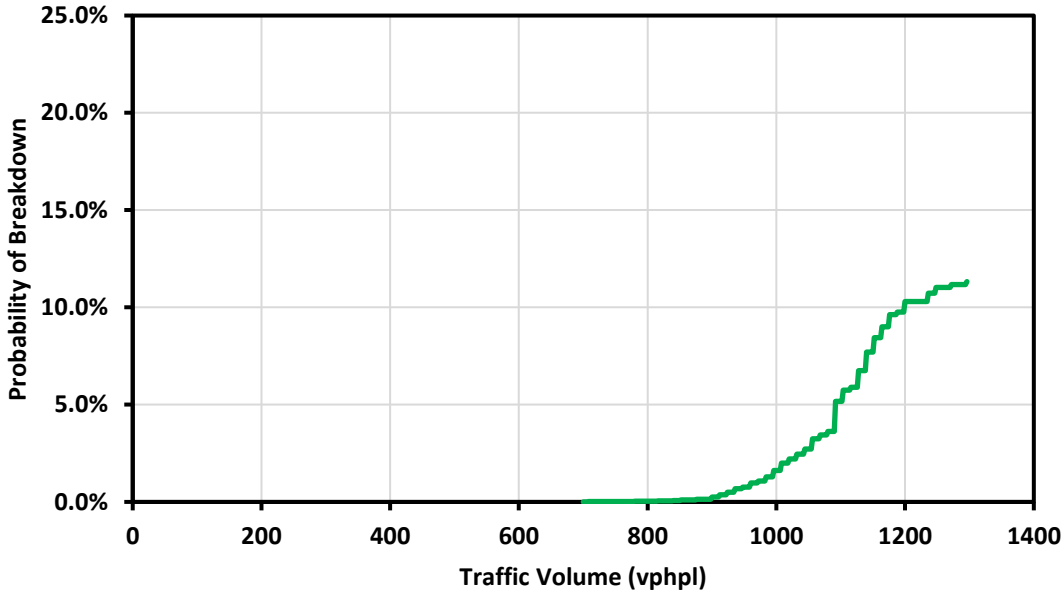


Figure 1-28: Empirical breakdown probability distribution (10% trucks, 5-minute aggregation interval, left-side lane closure)

The most evident feature of the figure is the fact that the probability distribution terminates at just above 10% on the y-axis. This is common even for distributions built from substantially larger datasets. For instance, the methodology provided in the supplemental volume of the HCM is demonstrated using over 23,000 flow rate observations, yet the associated empirical distribution still truncates around a 40% probability of breakdown (Transportation Research Board 2016). A plot of this dataset was recreated to emphasize this point and is given in Figure 1-29. In the figure, β and γ are the scale and shape parameters of the best-fit Weibull distribution, respectively.

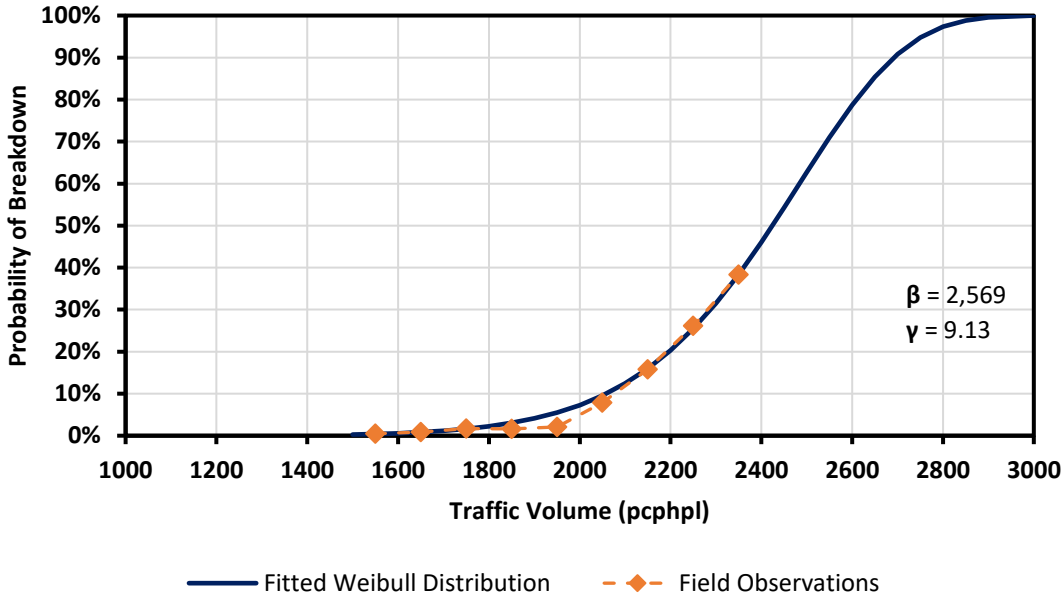


Figure 1-29: Example fitted weibull distribution (Source: TRB 2016)

The figure demonstrates the enormous data requirements for developing breakdown probability models, but also the quality of fit achieved by assuming a Weibull distribution. Based on this observation and findings from past research, it was assumed that the empirical distributions in this study could be sufficiently extrapolated using best-fit Weibull distributions. There are two caveats associated with Figure 1-29 and the methodology found in the HCM, however. First, field-measured volumes were binned into 100 pcphpl increments, reducing the number of data points available for curve fitting. Second, breakdown probabilities were calculated for each bin by dividing the number of breakdown flow rates by the total number of observed flow rates. Since this calculation occurs independently for each bin, the effect of previously observed flow rates is unaccounted for. As documented in literature, this leads to an overly conservative (low) estimate of capacity (Asgharzadeh and Kondyli 2018). For these reasons, the product-limit method was deemed the more appropriate technique for use in this research.

3.7.3 Curve fitting

Once all survival analysis tables and empirical probability distributions were completed, simulated data points were fitted to Weibull distributions to develop complete cumulative distribution functions. The Weibull cumulative distribution function is given in equation 1-9, where λ is the probability of breakdown, q is the flow rate in vphpl, β is the scale parameter, and γ is the shape parameter.

Solving this expression for q yields equation 1-10, which was ultimately used to calculate goodness of fit statistics.

$$\lambda = 1 - e^{-(\frac{q}{\beta})^{\gamma}} \quad \textbf{(1-9)}$$

$$q = \beta * \sqrt[\gamma]{-\ln(1-\lambda)} \quad \textbf{(1-10)}$$

To simplify the analysis, basic curve fitting with Excel's Solver function was applied in lieu of maximum likelihood estimation, the methodology typically used in literature. In this case, instead of maximizing the log-likelihood value, the MAPE between simulated data points and those on the best-fit Weibull curve was minimized. This statistic was calculated using equation 1-11.

$$MAPE = \frac{1}{n} * \sum_{i=1}^{n} \left(\frac{\left| Flow\ Rate_{Weibull,i} - Flow\ Rate_{Simulated,i} \right|}{Flow\ Rate_{Simulated,i}} \right) \quad \textbf{(1-11)}$$

Here, the empirical flow rate corresponding to a known probability of breakdown was compared to the flow rate calculated for the same probability of breakdown on a cumulative Weibull distribution with given shape and scale parameters. This process was iterated by Excel's Solver add-in until the smallest MAPE was achieved, typically at a value of less than 2%. The results of these calculations for a simulated work zone with 10% trucks, a left-side lane closure, and 5-minute aggregation intervals are shown as an example in Figure 1-30.

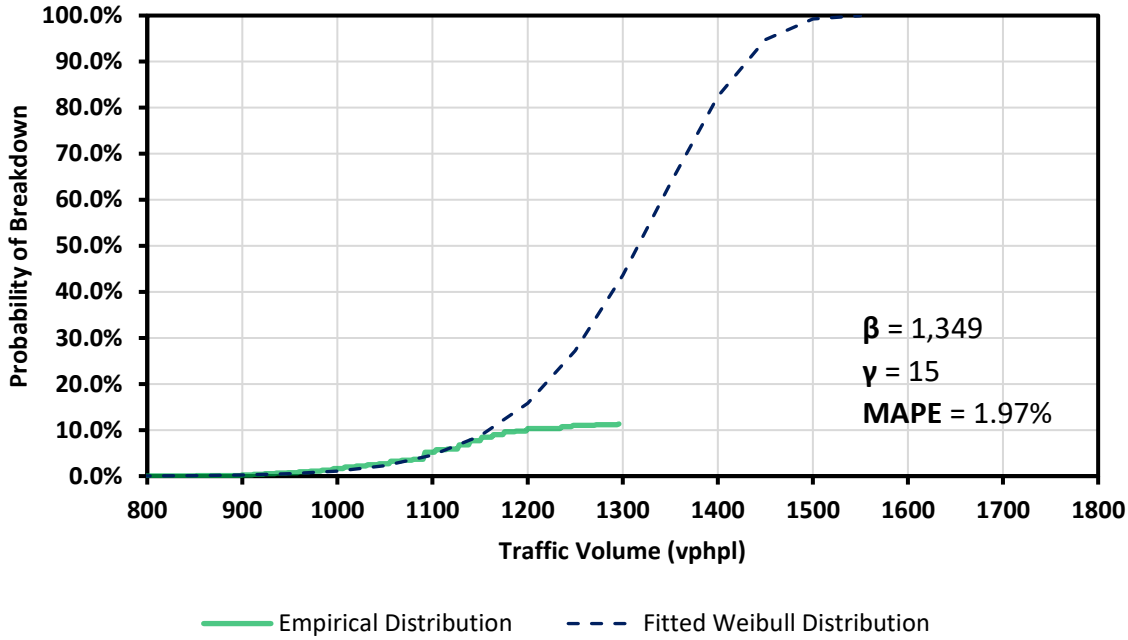


Figure 1-30: Curve fitting example (10% trucks, left-side lane closure, 5-minute aggregation intervals)

In the figure, the dashed Weibull distribution diverges from the truncated survival curve approximately midway between volumes of 1100 and 1200 vphpl, illustrating a key component of the curve fitting process. That is, because the simulated sample size was too small to create true cumulative distributions, survival curves typically flattened out near their maximum value. To avoid misleading results, data points corresponding to flow rates at or above which breakdown was seldom observed were excluded from MAPE calculations. Such points were identified by binning flow rates into 50 vphpl increments, where bins with fewer than 50 observations were generally excepted from curve fitting. It should be noted that the quality of fit for each Weibull distribution was inflated by the number of censored data points present at lower flow rates. For example, 6,390 of the 6,584 data points (97%) used to generate Figure 1-30 were censored, with 56% of such records occurring at flow rates of less than 1,000 vphpl. Similar trends were also observed for data aggregated into 15-minute intervals.

3.8 Summary

To summarize, Chapter Three began by outlining the field data collection effort that was ultimately used to inform model development. Data collected from a 2-to-1 freeway

lane closure near Tuscaloosa, Alabama in October 2016 was screened and analyzed to produce volume inputs, vehicle compositions, desired speed distributions, and time headway distributions in VISSIM. Then, the model was calibrated and validated by manually adjusting key driving behavior and vehicle performance parameters until simulated speed profiles and queue discharge rates replicated field observed values to the specified degree of accuracy. Ultimately, modeled speeds and queue discharge rates produced error within acceptable ranges using data from two different days, so the model was deemed suitable for analysis.

Finally, a plan for a partial factorial experiment was developed to study the probability of queue formation at rural freeway work zones as a function of traffic volume, truck percentage, and lane closure side. Recommendations were made pertaining to the range of values for each independent variable. A pilot or test case from the proposed experiment was then executed to illustrate the concept. For a specific truck percentage and lane closure side, a breakdown probability model was developed, and a best-fit Weibull distribution was applied to the empirical distribution generated from traffic simulation model runs.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 Introduction

The current state of the National Highway System often necessitates that agencies interrupt normal traffic operations for maintenance and capacity improvements. With nearly 9 million lane-miles of public roadway and an economy driven by the automobile, these interruptions are inevitable, but the significant safety and mobility impacts associated with queueing at freeway work zones are mitigable. The current methodology in the 6th edition of the HCM is a vast improvement over historical work zone capacity guidance but approaches the issue differently than research suggests agencies and practitioners should. Namely, a capacity defined by the mean queue discharge is deterministic and fails to account for the stochastic nature of traffic flow and breakdown. Rather, the frequency of rear-end crashes and high-speed differentials at freeway work zones warrants that the risk of queue formation always be minimized.

Rural freeway facilities are particularly important, as they compose more than half of all interstate lane-miles in the United States and account for 30% of all interstate vehicle miles of travel (Federal Highway Administration 2016). Despite lessened exposure compared to urban facilities, rural freeway segments pose an increased safety risk because drivers are less expectant of changes to the roadway environment and given increased opportunities to travel at high speeds. In 2015, NHTSA findings substantiated

this claim by finding that 43% of all fatal interstate crashes occurred in rural areas (NHTSA 2016a).

4.2 Conclusions

This research sought to accomplish three main objectives:

1. Assess the validity of microsimulation inputs and outputs as a means to obtain probabilistic estimates of capacity at rural freeway work zone lane closures
2. Develop a plan for breakdown probability models, and execute a test case, for a 2-TO-1 lane closures, with varying truck percentage and lane closure side to determine the effect of these variables on the likelihood of queue formation
3. Provide a framework for the continuation of future research, which will develop models for other lane closure configurations commonly experienced on rural and urban freeway facilities

First, field data was collected at a single-lane work zone on I-59/I-20 southbound near Tuscaloosa, Alabama for 14 days in October 2016. Site characteristics were determined to be typical of rural freeways in the southeastern United States and used to calibrate and validate a model in VISSIM. Critical components of model development included the use of time headway distributions and modified truck characteristics obtained from field observations and literature. Comparison of simulation outputs to field-collected data matched speed profiles reasonably well and yielded differences in mean queue discharge rates of less than 2%. Based on these findings, it was determined that microsimulation was an appropriate tool for collecting large samples of data for hypothetical freeway work zones.

Second, a plan for exploration of a series of breakdown probability models, which could then serve as the basis for a freeway work zone lane closure analysis tool, was developed. This exploration could include several traffic variables, such as volume, percentage of traffic comprised of heavy trucks, and work zone configuration, as inputs. A pilot or test case, for a specific truck percentage (10%) and lane closure side (left lane) was then developed and a breakdown probability model generated. Execution of the full experiment is proposed as part of a subsequent research project.

4.3 Recommendations for Traffic Modelers

The traffic data collected for this study provided an opportunity to examine the appropriateness of the use of default values for several parameters in VISSIM. Specifically, truck performance characteristics and time headway distributions were developed from the field data and then compared with the VISSIM default values.

VISSIM was developed in Germany, and the default values for truck acceleration and headway are based on the vehicle fleet in western Europe. The fleet vehicle size and weight distribution in the United States is heavier than that of western Europe, and driver behavior in terms of car following and gap acceptance may also differ. Specific recommendation include:

1. As shown in Figure 1-17, the recommended truck acceleration profile for use in traffic simulation models on freeways in the southeastern U.S. is substantially different than the default profile in VISSIM. It is recommended that traffic modelers in the southeastern U.S. consider using the acceleration profile shown in Figure 1-17 instead of the VISSIM default.
2. It is recommended that traffic modelers in the southeastern U.S., in the absence of field data from the vicinity of the freeway segment being modeled, use the time headway distributions for cars and trucks as shown in Figures 1-18 and 1-19 respectively and then summarized in Table 1-15.
3. It is recommended that traffic modelers in the southeastern U.S., in the absence of field data from the vicinity of the freeway segment being modeled, use a value of 10 ft for VISSIM parameter CC0 (standstill distance) instead of the default value of 4.9 ft.
4. It is recommended that traffic modelers in the southeastern U.S., in the absence of field data from the vicinity of the freeway segment being modeled, use a value of 3000 ft for lane-changing distance, instead of the default value of 6.6 ft.

4.4 Recommendations for Future Research

This study approached the issue of work zone capacity measurement in a unique manner by producing throughput estimates based on the probability of queue formation, rather than through traditional deterministic methods. While the results of this work make a significant contribution to the existing body of literature, they only provide a framework for the completion of a larger project funded by the Southeastern Transportation Research, Innovation, Development, and Education Center (STRIDE). Other phases of this project and future research may build upon the findings of this research by:

1. Conducting the full experiment outlined in Section 3.6
    a. Execute simulation models for the other combinations of truck percentage and lane closure side not included in the pilot or test case described in Section 3.7.
2. Extending field data collection efforts

   a. Data should be collected at rural freeway work zone sites in different states and with various lane closure configurations (e.g., 2-to-1, 3-to-2, 3-to-1), traffic characteristics (e.g. upstream lane distributions by vehicle type), and work types (e.g. resurfacing, bridge repair, major widening projects).
   b. Video cameras should be used to verify driving behavior near the bottleneck location and identify atypical occurrences such as traffic incidents or the movement of construction equipment in and out of the work area.
3. Increasing the number of modeled input variables
   a. Data collection at other sites will allow for additional variables such as lane closure configuration, work type and intensity, free flow speed, merge control strategy, and terrain to be modeled and included as user inputs in an expanded lane closure analysis tool.
   b. Breakdown probability distributions should be combined with estimated queue discharge rates to provide approximations of queue length and delay for agencies that will tolerate a queue but wish to minimize its impacts.
4. Accounting for future changes to vehicle characteristics and travel behavior
   a. The effect of varying levels of market penetration of automated and connected passenger cars and trucks should be incorporated in a supplemental model given that these technology advancements may soon be realized.

The methodologies, results, conclusions, and recommendations presented provide promise for future study and application of rural freeway work zone safety and mobility best practices. The decline of the structural and functional adequacy of the National Highway System suggests that work zones will become more prevalent and that careful attention to their design and operation is critical. Therefore, agencies and practitioners should make data-driven decisions based on the results of this study and similar research.

5.0 PART 1 REFERENCE LIST

1. AASHTO. (2011). “A Policy on Geometric Design of Highways and Streets”. Washington, D.C.

2. Abdelmohsen, A. Z., and El-Rayes, K. (2016). “Optimal Trade-Offs Between Construction Cost and Traffic Delay for Highway Work Zones.” ASCE Journal of Construction Engineering and Management, 142(7).

3. Al-Kaisy, A., and Hall, F. (2001). "Examination of Effect of Driver Population at Freeway Reconstruction Zones." Traffic Flow Theory and Highway Capacity 2001: Highway Operations, Capacity, and Traffic Control, (1776), 35–42.

4. Al-Kaisy, A., and Hall, F. (2003). "Guidelines for Estimating Capacity at Freeway Reconstruction Zones." ASCE Journal of Transportation Engineering, 129(5), 572–577.

5. Al-Kaisy, A., Hall, F. L., and Reisman, E. (2002). "Developing Passenger Car Equivalents for Heavy Vehicles on Freeways During Queue Discharge Flow." Transportation Research Part A: Policy and Practice, 36A(8), 725–742.

6. Asgharzadeh, M. A., and Kondyli, A. (2018). "Comparison of Highway Capacity Estimation Methods." Presented at the Transportation Research Board 97th Annual Meeting, Washington, D.C.

7. Benekohal, R. F., Kaja-Mohideen, A.-Z., and Chitturi, M. V. (2003). "Evaluation of Construction Work Zone Operational Issues: Capacity, Queue, and Delay." Report No. ITRC FR 00/01-4.

8. Bham, G. and Khazraee, S. (2011). "Missouri Work Zone Capacity: Results of Field Data Analysis." Report No. MATC-MST: 118.

9. Brilon, W., Geistefeldt, J., and Regler, M. (2005). "Reliability of Freeway Traffic Flow: A Stochastic Concept of Capacity." Proceedings of the 16th International Symposium on Transportation and Traffic Theory, (July), 125–144.

10. Chatterjee, I., Edara, P., Menneni, S., and Sun, C. (2009). "Replication of Work Zone Capacity Values in a Simulation Model." Transportation Research Record: Journal of the Transportation Research Board, 2130, 138–148.

11. Chitturi, M. V, and Benekohal, R. F. (2008). "Calibration of VISSIM for Freeways." Presented at the Transportation Research Board 87th Annual Meeting, Washington, D.C.

12. Chow, A. H. F., Lu, X. Y., and Qiu, T. Z. (2009). "An Empirical Analysis of Freeway Traffic Breakdown Probability Based on Bivariate Weibull Distribution." Presented at the 12th IFAC Symposium on Transportation Systems, IFAC, Redondo Beach, CA.

13. "Comprehensive Truck Size and Weight Study." (2000). Report No. FHWA-PL-00-029, FHWA, Washington, D.C.

14. Dixon, K., Hummer, J., and Lorscheider, A. (1996). "Capacity for North Carolina Freeway Work Zones." Transportation Research Record: Journal of the Transportation Research Board, 1529, 27–34.

15. Dong, J., Houchin, A., Shafieirad, N., Lu, C., Hawkins, N., and Knicherbocker, S. (2015). "VISSIM Calibration for Urban Freeways." Report No. INTRANS 14-487. Ames, IA.

16. Dowling, R., Skabardonis, A., and Alexiadis, V. (2004). "Traffic Analysis Toolbox Volume III : Guidelines for Applying Traffic Microsimulation Modeling Software." Report No. FHWA-HRT-04-040. Washington, D.C.

17. Drake, J. S., Schofer, J., and May, A. D. J. (1967). "A Statistical Analysis of Speed-Density Hypotheses." Highway Research Record, (154), 112–117.

18. Edara, P., and Chatterjee, I. (2010). "Multivariate Regression for Estimating Driving Behavior Parameters in Work Zone Simulation to Replicate Field Capacities." Transportation Letters, 2(3), 175–186.

19. Edie, L. (1961). "Car-Following and Steady-State Theory for Non-Congested Travel." Operations Research, 9(1), 66–76.

20. Elefteriadou, L. (2014). An Introduction to Traffic Flow Theory. Springer, New York.

21. Elefteriadou, L., Arguea, D., Heaslip, K., and Kondyli, A. (2007). "Impact of Trucks on Arterial LOS and Freeway Work Zone Capacity (Part B: Freeway Work Zone Capacity)." Report No. FDOT BD-545-51. Gainesville, FL.

22. Elefteriadou, L., Hall, F., Brilon, W., Roess, R. P., and Romana. (2006). "Revisiting the Definition and Measurement of Capacity." Presented at the 5th International Symposium on Highway Capacity and Quality of Service, Yokohoma, Japan.

23. Elefteriadou, L., Roess, R. P., and McShane, W. R. (1995). "Probabilistic Nature of Breakdown at Freeway Merge Junctions." Transportation Research Record: Journal of the Transportation Research Board, 1484, 80–89.

24. Federal Highway Administration. (2015). Status of the Nation's Highways, Bridges, and Transit: Conditions and Performance.

25. Federal Highway Administration. (2016). Highway Statistics 2015. Washington, D.C.

26. Federal Highway Administration. (2017). "Work Zone Facts and Statistics." Federal Highway Administration, <https://ops.fhwa.dot.gov/wz/resources/facts_stats/mobility.htm> (Aug. 17, 2017).

27. Florida Department of Transportation. (2014). Traffic Analysis Handbook: A Reference for Planning and Operations. Tallahassee, FL.

28. Geistefeldt, J. (2009). "Estimation of Passenger Car Equivalents Based on Capacity Variability." Transportation Research Record: Journal of the Transportation Research Board, 2130, 1–6.

29. Gomes, G., May, A., and Horowitz, R. (2004). "Congested Freeway Microsimulation Model Using VISSIM." Transportation Research Record: Journal of the Transportation Research Board, 1876, 71–81.

30. Hancher, D., and Taylor, T. (2001). "Nighttime Construction Issues." Transportation Research Record: Journal of the Transportation Research Board, 1761, 107–115.

31. Harwood, D. W., Torbic, D. J., Richard, K. R., Glauz, W. D., and Elefteriandou, L. (2003). NCHRP Report 505: Review of Truck Characteristics as Factors in Roadway Design. Washington, D.C.

32. Heaslip, K., Kondyli, A., Arguea, D., Elefteriadou, L., and Sullivan, F. (2009). "Estimation of Freeway Work Zone Capacity Through Simulation and Field Data." Transportation Research Record: Journal of the Transportation Research Board, 2130, 16–24.

33. Heiden, N. Von Der, and Geistefeldt, J. (2016). "Capacity of Freeway Work Zones in Germany." Transportation Research Procedia, Elsevier B.V., 15, 233–244.

34. Houchin, A. J. (2015). "An Investigation of Freeway Standstill Distance , Headway , and Time Gap Data in Heterogeneous Traffic in Iowa." Iowa State University Theses and Dissertations.

35. Hu, J., Schroeder, B. J., and Rouphail, N. M. (2012). "Rationale for Incorporating Queue Discharge Flow into Highway Capacity Manual Procedure for Analysis of Freeway Facilities." Transportation Research Record: Journal of the Transportation Research Board, 2286, 76–83.

36. Jia, A., Williams, B. M., and Rouphail, N. M. (2010). "Identification and Calibration of Site-Specific Stochastic Freeway Breakdown and Queue Discharge." Transportation Research Record: Journal of the Transportation Research Board, 2188, 148–155.

37. Jiang, X., and Adeli, H. (2003). "Freeway Work Zone Traffic Delay and Cost Optimization Model." ASCE Journal of Transportation Engineering. 129(3), 230–241.

38. Jiang, Y. (1999). "Traffic Characteristics and Estimation of Traffic Delays and User Costs at Indiana Freeway Work Zones." Report FHWA/INDOT No. SPR-2121.

39. Kan, X. D., Ramezani, H., and Benekohal, R. F. (2014). "Calibration of VISSIM for Freeway Work Zones with Time-Varying Capacity." Presented at the Transportation Research Board 93rd Annual Meeting. Washington, D.C.

40. Kaplan, E. L., and Meier, P. (1958). "Nonparametric Estimation from Incomplete Observations." Journal of the American Statistical Association, 53(282), 457–481.

41. Kim, T., and Lovell, D. J. (2001). "A New Methodology to Estimate Capacity for Freeway Work Zones." Presented at the Transportation Research Board 80th Annual Meeting.

42. Kondyli, A., Elefteriadou, L., Brilon, W., Hall, F. L., Persaud, B., and Washburn, S. (2013). "Development and Evaluation of Methods for Constructing Breakdown Probability Models." ASCE Journal of Transportation Engineering, 139(9), 931–940.

43. Krammes, R. A., and Lopez, G. O. (1994). "Updated Capacity Values for Short-Term Freeway Work Zone Lane Closures." Transportation Research Record: Journal of the Transportation Research Board, 1442, 49–56.

44. Lorenz, M., and Elefteriadou, L. (2001). "Defining Freeway Capacity as Function of Breakdown Probability." Transportation Research Record: Journal of the Transportation Research Board, 1776, 43–51.

45. Lownes, N. E., and Machemehl, R. B. (2006). "Sensitivity of Simulated Capacity to Modification of VISSIM Driver Behavior Parameters." Transportation Research Record: Journal of the Transportation Research Board, 1988, 102–110.

46. Middleton, D., Venglar, S., Quiroga, C., and Lord, D. (2006). "Strategies for Separating Trucks from Passenger Vehicles." Report No. FHWA/TX-07/0-4663-2. College Station, TX.

47. Minderhoud, M., Botma, H., and Bovy, P. (1997). "Assessment of Roadway Capacity Estimation Methods." Transportation Research Record: Journal of the Transportation Research Board, 1572, 59–67.

48. NHTSA. (2015). "Fatality Analysis Reporting System (FARS)." <https://www.nhtsa.gov/research-data/fatality-analysis-reporting-system-fars> (Aug. 21, 2017).

49. NHTSA. (2016a). Traffic Safety Facts 2015. Washington, D.C.

50. NHTSA. (2016b). Rural/Urban Comparison of Traffic Fatalities. Washington, D.C.

51. Park, B. B., and Won, J. (2006). Microscopic Simulation Model Calibration and Validation Handbook. Virginia Transportation Research Council. Charlottesville, VA.

52. Park, S.-B., Douglas, K. D., Griffith, A. S., and Haas, K. J. (2002). "Factors of Importance for Determining Daytime Versus Nighttime Operations in Oregon." Transportation Research Record: Journal of the Transportation Research Board, 1813, 305–313.

53. Persaud, B., Yagar, S., and Brownlee, R. (1998). "Exploration of the Breakdown Phenomenon in Freeway Traffic." Transportation Research Record: Journal of the Transportation Research Board, 1567, 64–69.

54. PTV Group. "PTV VISSIM 10 User Manual." (2017). Karlsruhe, Germany.

55. Ramadan, O., and Sisiopiku, V. (2016). "Impact of Bottleneck Merge Control Strategies on Freeway Level of Service." Transportation Research Procedia, Elsevier B.V., 15, 583–593.

56. Ramezani, H., and Benekohal, R. F. (2012). "Work Zones as a Series of Bottlenecks." Transportation Research Record: Journal of the Transportation Research Board, 2272, 67–77.

57. Rebholz, F. E., Al-Kaisy, A., Nassar, K., Liu, L. Y., El-Rayes, K., and Soibelman, L. (2004). "Nighttime Construction: Evaluation of Construction Operations." Report No. ITRC FR 00/01-5.

58. Roess, R. P., and Prassas, E. S. (2016). The Highway Capacity Manual: A Conceptual and Research History. Springer, New York.

59. Sarasua, W. A., Davis, W. J., Clarke, D. B., Kottapally, J., and Mulukutla, P. (2004). "Evaluation of Interstate Highway Capacity for Short-Term Work Zone Lane Closures." Transportation Research Record: Journal of the Transportation Research Board, 1877, 85–94.

60. Sarasua, W., Davis, W., Chowdhury, M., and Ogle, J. (2006). "Estimating Interstate Highway Capacity for Short-Term Work Zone Lane Closures: Development of Methodology." Transportation Research Record: Journal of the Transportation Research Board, 1948, 45–57.

61. Schrank., D., Eisele., B., Lomax., T., and Bak., J. (2015). 2015 Urban Mobility Scorecard. Texas A&M Transportation Institute.

62. Shojaat, S., Geistefeldt, J., Parr, S. A., Wilmot, C. G., and Wolshon, B. (2016). "Sustained Flow Index." Transportation Research Record: Journal of the Transportation Research Board, 2554, 158–165.

63. Sisiopiku, V. P., and Ramadan, O. E. M. (2016). "Evaluation of Traffic Control Options in Work Zones." Report No. 2016-001. Southeastern Transportation Research, Innovation, Development, and Education Center.

64. Tang, Y., and Chien, S. (2008). "Scheduling Work Zones for Highway Maintenance Projects: Considering a Discrete Time-Cost Relation." Transportation Research Record: Journal of the Transportation Research Board, 2055, 21–30.

65. Transportation Research Board. "Highway Capacity Manual 6th Edition: A Guide for Multimodal Mobility Analysis." (2016), Washington, D.C.

66. Trask, L., Aghdashi, B., Schroeder, B., and Rouphail, N. (2015). "FREEVAL-WZ Users Guide." North Carolina State University, Raleigh, NC.

67. Turochy, R. E., and Smith, B. L. (2000). "New Procedure for Detector Data Screening in Traffic Management Systems." Transportation Research Record: Journal of the Transportation Research Board, 1727, 127–131.

68. Turochy, R. E., Timm, D. H., and Mai, D. (2015). Development of Alabama Traffic Factors for Use in Mechanistic-Empirical Pavement Design. Auburn University Highway Research Center, Report No. 930-685. Auburn, AL.

69. Venugopal, S., and Tarko, A. (2001). "Investigation of Factors Affecting Capacity at Rural Freeway Work Zones." Presented at the Transportation Research Board 80th Annual Meeting, Washington, D.C.

70. Washington State Department of Transportation. (2014). Protocol for VISSIM Simulation.

71. Weng, J., and Meng, Q. (2013). "Estimating Capacity and Traffic Delay in Work Zones: An Overview." Transportation Research Part C: Emerging Technologies, 35, 34–45.

72. Weng, J., and Yan, X. (2016). "Probability Distribution-Based Model for Work Zone Capacity Prediction." Journal of Advanced Transportation, 50(2), 165–179.

73. Weng, J., and Yang, X. (2014). "New Methodology to Determine Work zone Capacity Distribution." Transportation Research Record: Journal of the Transportation Research Board, 2461, 25–31.

74. Woody, T. (2006). "Calibrating Freeway Simulation Models in VISSIM." University of Washington Theses and Dissertations.

75. Yang, G., Xu, H., Wang, Z., and Tian, Z. (2016). "Truck Acceleration Behavior Study and Acceleration Lane Length Recommendations for Metered On-Ramps." International Journal of Transportation Science and Technology, 5(2), 93–102.

76. Yeom, C., Hajbabaie, A., Schroeder J, B., Vaughan, C., Xuan, X., and Rouphail M, N. (2015). "Innovative Work Zone Capacity Models from Nationwide Field and Archival Sources." Transportation Research Record: Journal of the Transportation Research Board, 2485, 51–60.

77. Yeom, C., Rouphail, N. M., Rasdorf, W., and Schroeder, B. J. (2016). "Simulation Guidance for Calibration of Freeway Lane Closure Capacity." Transportation Research Record: Journal of the Transportation Research Board, 2553, 82–89.

PART 2 TEMPORARY TRAFFIC CONTROL OPTIONS FOR WORK ZONE TRAFFIC MANAGEMENT

1.0 INTRODUCTION

Maintenance and rehabilitation work are very important for proper functioning of roadways. Several states opt for construction work on the existing roadways to improve roadway condition and serve the traveling public. Often, lane closures are required whenever there is an active work zone. In fact nearly 20% of the U.S. National Highway System roads have construction work during the peak construction season [1] and there is a possibility that a driver would confront one active work zone out of every hundred miles driven on the highway [2]. Work zones reduces the capacity of roadways and interrupts traffic flow at merge point [3]. During the uncongested situation, vehicles can drive at regular speed, but speeds may decline by 31.6% to 56.1% of the regular speed at work zones [4].

Researchers estimate that work zone causes approximately 24% of non-recurring delays which ranks work zone as the second largest factor of non-recurring delay on arterials [5]. The lane closure not only affects mobility of regular travelers on the specific roadway, but also affects local business and community, and causes noise and environmental issues. Researchers indicated that the cost of congestion combining travel delay and extra fuel cost was estimated to be $115 billion in 2009 [6]. They calculated total travel delay cost of users with passenger cars for a hypothetical 2-mile work zone to be $196,342.10 /day, assuming six lane interstate and 88% passenger cars in the regular traffic flow. Therefore, transportation agencies are trying to ensure efficient traffic flow through work zones and research is going on to enhance mobility diminishing interruptions in the traffic flow.

One strategy used to manage traffic through work zone lane closures is conventional Temporary Traffic Control (TTC). TTC methods can be static or dynamic. The choice depends on time period of work zone, traffic volume, driving behavior, etc. Selection of TTC should be based on evaluation of operational impacts of the specific strategy at specific work zone and safety considerations. Few strategies perform well with lower traffic volumes but may not be efficient when demand volume exceeds capacity. The situation becomes even worse when the queue goes upstream beyond warning signs and possibility of collision increases [7].

Given the time and money lost by travelers due to work zone induced traffic congestion, it is critically important to efficiently plan temporary traffic control (TTC) at work zones. In fact, the Federal Highway Administration (FHWA) has developed a rule according to which all state and local highway organizations should have guidelines for evaluating mobility impacts and managing safety at work zone locations for each project [6, 8]. Many state Departments of Transportation (DOT) fix a certain user delay as a measure of mobility [8]. To reduce impact on

traffic operation and safety, many agencies select off-peak hours for lane closures. As a result, project duration increases and setup related time and cost also increase [9].

Lane closures at work zones may take place for pavement repair, resurfacing, installation of pavement markers, asphalt removal, etc. Hence, planning of work zone traffic control should be done cautiously so that lane closures have minimum impacts on mobility [10]. However, earlier research confirms that the majority of State Departments of Transportation do not have formal guidelines for selecting TTC strategy for work zones; instead, they rely on their earlier experience without consideration of operational nor safety impacts [11].

Some earlier studies looked at TTC strategies for short-term work zones under a 3-to-2 lane drop scenario. However, very few studies focused on assessing the impact of TTC strategies for 3-to-1 lane configuration and no study focused on long-term work zone for 3-to-1 lane closure. Therefore, there is an evident need to study different TTC strategies for long-term and short-term work zones for 3-to-1 lane closure configuration for various work zone length so that transportation agencies can ensure maximum flow and minimum delay of road users at work zone under the 3-to-1 lane closure scenario.

The objective of this task is to investigate the operational impacts of two TTC strategies for work zones, namely static late- and early merge control, under varying traffic demand with 3-to-1 lane-drop configurations. The tasks performed to meet this objective are as follows:

1. Model a study freeway corridor under typical traffic demand conditions to establish a baseline for comparisons using the VISSIM microscopic traffic simulation platform.
2. For the same study corridor and under similar geometric and traffic conditions, develop VISSIM simulation models that represent work zone operations with 3-to-1 and 3-2 lane drops with late- and early merge control for varying work zone lengths.
3. Evaluate late- and early merge control for 3-to-1 and 3-to-2 lane closures at work zones throughout 24-hr time period and compare operational impacts on the basis of travel time, flow, speed and density as a function of traffic demand as well as length of the work zone.
4. Evaluate late- and early merge control for a 3-to-1 lane closure at work zones with lane closures imposed during peak- and non-peak time periods and compare operational impacts
5. Develop recommendations for spatial and temporal placement of freeway work zones with 3-to-1 lane closures and selection of TTC strategies.

More specifically, the study first reviewed available TTC strategies at lane closures and the current practice in different states and obtained the needed data to develop and calibration a traffic simulation model of a study corridor under current conditions. Then simulation models of the corridor representing work zone conditions with 3-to-1 and 3-to-2 lane closures were developed for both static late- and early merge control. Comparisons of performance measures

were performed between these two strategies and used to develop recommendations about conditions and TTC strategies that produce best results.

The research bridges the gap between construction work scheduling and transportation planning research. Earlier simulation studies concentrating on static early and late merge mostly considered short-term work zones during off-peak. Additionally, research and analysis of the 3-to-1 lane closure configuration is very limited. Therefore, this study contributes to better understanding of long-term work zone operation under constrained roadway capacity (3-to-1 lane closure configuration). Overall, the findings of this study yield information that transportation agencies can use to better plan future work zones, particularly those involving partial closures on freeways. Examining mobility impacts of various combinations of lane closures, TTC strategies, and workspace lengths, is expected to identify those combinations that will have minimal impacts on mobility. The study findings are expected to provide valuable guidance for agencies responsible for planning, design, and operations of work zones.

2.0 LITERATURE REVIEW

2.1 Available Merge Control Strategies

Various State Departments of Transportation (DOTs) implement merge control strategies at work zones according to the procedures described in the Manual of Uniform Traffic Control Devices (MUTCD) [12]. The literature review reveals that temporary traffic control (TTC) methods used by DOTs include static or dynamic early merge, static or dynamic late (zipper) merge, reduced speed when flashing, closure of entrance ramps during construction [13-16]. The main features of commonly used TTC methods are discussed next.

2.1.1 Static or dynamic early merge

Static early merge technique is employed by placing lane closing signs a few miles ahead of the actual closed lane at one-mile intervals. This set up gives drivers that are approaching a work zone advanced information about which lane is closed and enough time to merge in the open lane. The reason to adapt this strategy is to facilitate an orderly merge to the open lanes in advance rather than encouraging drivers to merge close to the lane drop over a short distance. Since advanced warning is placed well upstream of closed lanes, early merge can also decrease the rate of rear-end collision [14]. Static early merge control increases the amount of free merges and when this is incorporated with effective warning distance of 1-mile, the percentage of free merge becomes greater than or equal to 90 [17]. However, early studies report that congestion may occur in the open lane(s) since many drivers merge very early abandoning the closed lane for a distance more than required, also leading to higher delays and travel time [18]. This strategy may also result in lane-change crashes since drivers in the open lane might use the portion of closed lane to overtake other

vehicles stuck in congestion [14]. It has also been reported that drivers face confusion about the instructions to merge early in low-volume situations [19]. Another disadvantage is that maintenance of signage and other control measures becomes difficult [20].

Dynamic early merge technique incorporates real time traffic information in warning signs, thus the distance of warning from the closed lane varies according to the traffic demand level. Sonic detectors are attached on warning signs at ¼ to ½ mile intervals along the closed lane and the queue in open lane is monitored. Whenever any detector finds out the existence of queue, it transfers the signal to the next upstream sign. Then the flashing light turns on at the next signpost to warn drivers at that location to merge into the open lane. The length of the no-passing zone varies depending on the queue in open lane, hence it is called dynamic early merge. Field tests in Indiana confirmed smoother lane merging with dynamic early merge control than with the conventional method. The capacity of the work zone is increased during uncongested conditions and the number of free merges improved because of enough space before closure. But this control strategy becomes ineffective if the queue goes beyond the detectors when traffic demand is very high [14, 15]. Several studies revealed that capacity reduces by using this control, thus travel times increase and longer queue develops during high demand volume [21].

2.1.2 Static or dynamic late (zipper) merge

Static late merge is a strategy where drivers remain in their respective lane up to a certain merge point closer to the lane closure and then enter into the open lane in a zipper fashion, hence it is called “zipper merge” as well. Opposed to early merge, drivers do not merge into open lane much in advance of the lane closure and do not need to worry about which lane is closed ahead of time, since all lanes can be used before the merge point. Thus, drivers of open lane feel more comfortable since they would not be passing vehicles beside them using closed lanes. The Minnesota Department of Transportation (MnDOT) has evaluated this strategy and concluded that queue length can be reduced by 40% by implementing late merge control [16]. Other potential benefits of this method are improvements related to travel time, delay and rear-end collisions. More space can be utilized by vehicles in late merge, which reduces queue length and potential conflicts. Some studies revealed that late merge becomes more efficient when traffic volume is much higher, while early merge is less effective at congested locations [14, 22]. Studies report that congestion lasts longer under early merge and hence travel time is lessened using late merge, especially under high traffic demand conditions. Although these are some observed benefits of late merge, more study is recommended in the literature to focus on drivers’

behavior under lower traffic volume with higher speed scenarios and also on safety impacts related to the use of late merge strategies. With respect to safety, the possibility of conflict about right-of-way at merge point is also mentioned in one research [23].

Dynamic late merge strategy attempts to address the problems associated with static late merge during high-speed and low traffic volumes conditions. So, dynamic late merge considers real-time information of traffic and accordingly changes merge points and controls the traffic in a fashion similar to early merge control during off-peak. When the queue is detected on open lanes, an advanced warning sign informs drivers to drive in their lane instead of merging early. But when there is no queue, this sign would be changed to warn drivers to merge to open lanes. To ensure effective use of dynamic late merge (DLM), volume and speed threshold points need to be identified for interchanging between early and late merge control [14].

2.1.3 Joint merge

Joint merge is a new type of traffic control strategy where there are tapers on two sides of the road instead of one side as in early or late merge. In this technique, two lanes are reduced to form one lane, thus vehicles on the both lanes of the road get equal priority [24]. One field study was conducted in Louisiana and comparison was made with a conventional merge control on the exactly the same location [24]. It was reported that when the traffic volume was between 600 vph to 1200 vph, both strategies performed similar to each other. The researchers concluded that volumes were equally distributed in two lanes because both were getting reduced and recommended that drivers in both lanes should drive carefully while merging.

2.1.4 Reduced speed when flashing

This control strategy uses a flashing sign to advise drivers to reduce their speed when they approach a work zone. For the rest of the time, they can maintain normal speed. The reduced speed is set at a minimum of 10 mph less than the posted limit. This strategy is used so that drivers are not forced to always drive at lower speed at work zones, rather they drive at lower speed only when some activity is going on and workers are on the roadways. If workers are not there, drivers can drive at greater speed, thus mobility is enhanced and compliance with the speed limit restriction increases.

2.1.5 Closure of entrance ramps during construction

During lane closures, traffic entering through ramps increases merging maneuvers and contributes to increased congestion. That is why on-ramps are kept closed at some locations to heighten mobility through work zones. Safety

also increases along the corridor. Moreover, the likelihood of accidents is reduced by closing entrance ramps and congestion delay along the mainline is minimized.

2.1.6 Mainline merge metering

In mainline merge metering systems, a meter is mounted adjacent to the closed lane of freeway to instruct drivers to change lane at merging points. This is similar to late merge, except for the fact that merging is controlled by metering. A study [25] considered this method in order to increase overall throughput when volume became greater than capacity. The ALINEA ramp metering algorithm was utilized and microscopic simulation was used to validate the control system. Another study [26] identified greater throughput using the same logic as the previously mentioned study, under some fixed configuration of traffic flow. But the limitation is that trucks were set to use only one lane and calibration was not reported in that study. The dynamic merge metering concept was developed by combining dynamic late merge, merge meter, and wireless technologies to be used at merge points. The method was introduced in a VISSIM study by Wei et al., but they recommended further studies to optimize operations [3].

2.1.7 Temporary ramp metering strategies

Since capacity of a roadway is decreased due to lane closure, vehicles entering from on-ramps can create turbulence by their turning movement. Temporary ramp metering is a way to control the vehicles entering on freeways, thus increasing the mainline flow on the freeway. One researcher [27] studied the performance of ramp meter on the interstate with work zones using simulation and found that shorter queues on the arterials and the rightmost lane closer to mainline merge point of the interstate. A study based on seven actual lane closure locations in Missouri where temporary ramp meters were installed was used to collect data. The data were used in a simulation model that analyzed performance under the temporary ramp metering strategy for off-peak hour [28]. However, the outcome of this strategy is unknown for peak hours. More studies need to be conducted to evaluate its performance for a lane closure segment on freeway with on-ramps [29].

Table 2-1 presents a summary of available strategies focusing on their potential benefits and limitations as reported in the literature.

Table 2-1: Summary of different traffic control options

Type of the traffic control strategy	Potential benefits	Limitations
Static Early Merge	Reduce merge related collision, rear-end collision; Increase free merges at higher traffic volume	Congestion, Lane-change crashes, difficult maintenance of signage
Dynamic Early Merge	Varying no-passing zone based on detectors, smooth merging	Ineffective for queues beyond the detectors, higher travel time
Static Late Merge	Reduce queue length and rear-end crashes	Safety issues in low-volume and high-speed situations
Dynamic Late Merge	Changeable merge point based on real-time information	Threshold volume and speed needs to be evaluated accurately
Joint Merge	All lanes get equal priority	Insignificant change compared to conventional merge
Reduced Speed When Flashing	Reduce speed only during active work zone	Driver's perception study is needed
Closure of Entrance Ramps During Construction	Enhanced mobility	Adverse impact on arterials
Mainline Merge Metering Temporary Ramp Metering Strategies	Increase throughput Increase mainline flow	Further study is required Peak-hour study is needed

2.2 Current State-of-Practice in the United States

A detailed study about the current practice to control traffic at work zones around the states had been conducted by questionnaire survey among DOTs [29]. The study gathered information from 27 states over a period at more than one month. The participating states were Alabama, California, Colorado, Illinois, Iowa, Michigan, Minnesota, Mississippi, Missouri, North Carolina, South Carolina, South Dakota, Utah, Vermont, West Virginia, Wisconsin, and Wyoming, and another 10 States that remained anonymous. The response from these states about the merge control strategy during work zone lane closures identified that most of them schedule maintenance work during off-peak period and more than half of the DOTs adapt static early merge as merge control strategy, followed by static late merge control adapted by almost 20% of the DOTs that responded in the survey. The rest of the responders used conventional merge

control and a few attempted dynamic lane merge controls. The reason that they follow any of these strategies is merely their previous experience. Their decision also depended on safety, mobility, policy, and cost, but primarily they relied on their past knowledge. Another study indicated that the majority of states carry on the work on a lane during night or off-peak based on the type of roadway [30].

Another study was conducted to identify the current practice of the Texas DOT using a questionnaire survey [14] which revealed that more than half of the responders used arrows to encourage vehicles to merge on the open lane and 25% of them close the left lane. Almost 30% of the respondents have used late merge in lane closure situation.

Earlier studies had identified some DOTs that developed manuals for work zone traffic control and few are enriching their guidelines [29]. For example, the Illinois Department of Transportation (IDOT) has maintained a manual for location, and design of roadway projects that maximize the safety of workers and travelers. They have set their target to decrease the fatality rates in work zones by 10% each year. The Texas Department of Transportation (TxDOT) has also established a manual for recognizing the project requirements so that it can be finished within the proper timeline. The Florida Department of Transportation (FDOT) has a training plan to ensure safety at work zones for workers and motorists. Their rule enforces prohibition of lane closure on interstate with two lanes to decrease delay for the travelers [31]. The California Department of Transportation (Caltrans) also has a manual with criteria to categorize the significance level of work zone based on the traffic delay [32].

2.3 Earlier Studies Considering Different Temporary Traffic Control (TTC)

Several studies have been conducted on the Temporary Traffic Control (TTC) strategies available around the states. Some conducted field tests and some used simulation studies to evaluate the performance of different TTCs. Earlier studies and their findings are important for enabling efficient work zone planning.

2.3.1 Studies to compare static late merge and early merge

Researchers had conducted several studies to compare static early merge and late merge strategies at work zones. One team [33] from Nebraska performed a detailed study of both traffic control options at the time when the state utilized conventional merge control for its work zone situations. Computer simulations were conducted for typical work zone scenarios such as 2-to-1, 3-to-1, and 3-to-2 lane closure configurations using both traditional merge and late merge strategies to identify the difference between these two strategies. Variations in free flow speed, volume, and heavy vehicle percentage were considered to identify their effect on operational performance. Field tests were also conducted for the 2-to-1 lane closure configuration. The simulation study indicated that late merge strategy holds promise for 3-to-1 lane closure configuration for any demand volume or any percentage of heavy vehicles. The other configurations (3-to-2 and 2-to-1) also performed better under late merge control when

compared to the conventional merge strategy with more than 20% of heavy vehicle. In that study, the volume in the closed lane increased by 30% with static late merge. Mean speed decreased by 7 mph and 16.1 mph under static late and static early merge respectively, when compared with standard MUTCD under uncongested traffic situation. This means vehicles can travel at higher mean speed with static late merge than with static early merge, thus highlighting the benefits of late merge TTC. It was also reported that under late merge the forced merge was reduced by 75% and the queue decreased by half. More research is recommended before actually implementing late merge in the field, especially where various amount of traffic volume occurs.

As a part of a project sponsored by the Texas Department of Transportation, researchers utilized the simulation platform VISSIM to compare impacts at work zones operating under early, late and signalized merge control strategies [34]. Network performance, safety issues, driver behavior, and driver operations were investigated to differentiate among these controls, both based on field observations and microsimulation studies. They utilized field data to increase the accuracy of the simulation model. From their study, they observed that early merge control is better for lower demand conditions. This is because the vehicles get sufficient space to merge to the open lane if less traffic are present on the roadway. This strategy also proves to increase safety and reduces delay because of the smooth merging with lower traffic. On the other hand, under increased traffic demand conditions, late merge is reported to perform better. The reason is that late merge utilizes the available lane capacity more effectively, right until the reach the actual lane closure. The researchers concluded that use of any strategy depends on the volume (V) and capacity (C) of the roadway. When V/C becomes close to 1 or greater than 1, the roadway may perform better with late merge control according to their research. The authors studied late merge with a signal system to direct drivers through work zones and found out that signalized late merge works better if volume is more than 1,800 vph/lane.

2.3.2 Studies to compare dynamic early and dynamic late merge

Some state DOTs try to improve mobility and safety near or at work zones locations through the use of Intelligent Transportation System applications. Researchers in Florida compared conventional practice followed by the Florida DOT, termed as Motorist Awareness System (MAS), dynamic early merge, and dynamic late merge. For the simulation, they used a 2-to-1 lane closure configuration and field data were collected for all three types of merge control and entered in the VISSIM simulator using vehicle-actuated programming (VAP) to represent the algorithm [35]. The length of the work zone was 13 miles and workers moved the work zone and worked on almost 3 mile each day. The experimental design considered various drivers' compliance rates, different truck

percentages, and different traffic volumes in the VISSIM environment. Comparison of the three controls revealed that simplified dynamic early merging system (early SDLMS) had higher throughput and lower travel time compared to late SDLMS and MAS. They recommended to study the control strategies with varying speed and other parameters in follow up studies.

Another study performed similar research focusing on 3-to-2 lane closure configuration [36] and focused on speed and measured speed variations in the closed lane. This variation was maximum with MAS strategy under all demand volumes and minimum using dynamic early merge control. Dynamic early merge worked better than conventional MAS when volumes ranged from 500 veh/hr to 2000 veh/hr. Moreover, dynamic late merge worked better than conventional MAS when volumes ranged between 1500 veh/hr and 2000 veh/hr but did not perform well with low volumes. Furthermore, the comparison between the two forms of dynamic merge control showed that dynamic late merge has a superior performance than dynamic early merge with higher volumes between 1500 veh/hr and 2000 veh/hr [36].

2.3.3 Studies focusing on late merge

The North Carolina DOT investigated different types of strategies on the roadways that impact merging behavior. The focus of the study was to reduce travel time. Among the various techniques considered in the scope of the project, late merge was one strategy that was included [37]. Two types of lane closure were considered, namely 2-to-1, 3-to-2 on rural arterial, rural freeway, and suburban freeway. The sites that had zipper merge control showed an increased speed by 11 mph, meaning reduced travel time. The study reported that after implementing late merge, vehicles continued to drive in closed lanes further than without late merge strategy, thus more roadway capacity was utilized. An improvement on the safety was also observed. The study categorized types of merge and found that the most dangerous type of merge was reduced when zipper merge was used.

One study in Kentucky compared early merge with zipper merge with some documented case studies [38]. They investigated two case studies in Kentucky, one at an interstate, and another at a bridge. When they compared the data for the interstate, they found out that late merge had better traffic flow compared to early merge and had better safety [38]. Overall, they found that late merge performs better and results in greater throughput in the area affected by construction. They recommended application of late merge in other locations, and collection and analysis of more field data that will reveal the appropriate location for implementing late merge in the future.

2.3.4 Studies focusing on dynamic late merge

The Maryland State Highway Administration evaluated the performance of DLM on a freeway lane closure in 2003 using occupancy as control thresholds. Portable Changeable Message Signs (PCMS) were active when occupancy was over 15% occupancy. The researchers estimated work zone throughput, volume distribution, and queue length. The results showed that DLM performs better than conventional merge control with respect to throughputs [39].

Another study investigated the usefulness of a synchronized system to warn drivers about work zones. The warning-light system proved advantageous at urban freeway for new work zones, but not at rural roadways where lane has been closed for long duration. So, it was concluded that this strategy might have better performance for short or intermediate work zones [40].

The Michigan Department of Transportation (MDOT) applied the Dynamic Late Lane Merge System (DLLMS) on three freeways in 2006 and made a comparison with conventional merge control at a site. All work zones considered had the 2-to-1 lane closure configuration. Traffic volumes, speeds, travel time, crashes, and queues were measured using video cameras and radar guns and floating cars were used for several days to gather average travel time along the work zones. Crash reports were also collected to assess any crashes at the study site. The researchers concluded that traffic performance was enhanced and percentage of vehicle merging closer to taper increased using DLLMS instead of the conventional method. The average travel time decreased by almost 40% using DLLMS compared to conventional method and delay and speed were both increased by 60% compared to the conventional merge control. Overall, each measure of effectiveness considered showed that DLLMS outperforms early merge [41].

2.3.5 Studies focusing on joint merge

The Louisiana DOT has utilized joint merge for an interstate work zone with 2-to-1 lane closure configuration and compared the operational impacts of this control to those under early merge at work zones. Field data such as traffic volumes, speeds, and type of vehicles were collected and analyzed. Overall, merging speeds were found to be relatively similar at volumes ranging from 600 to 1,200 vehicles per hour and did not affect the discharge rate at the merge outflow point. However, the experimental results did suggest that drivers were more cautious in their merging maneuvers [24].

Table 2-2 shows the summary of earlier studies discussed above that are conducted by researchers focusing different temporary traffic controls and their findings are included as well.

Table 2-2: Summary of earlier studies on various temporary traffic control strategies

Researchers	Strategy	Period	Lane closure configuration	Approach of analysis	Findings
(Beacher et al., 2005) [33]	Static Late merge vs early merge	Short-term	2-to-1, 3-to-1, and 3-to-2	Microscopic simulation	Late merge is better
(Kurker et al., 2014) [34]	Static Late merge, early merge, signalized merge	Short-term	2-to-1, 3-to-1, and 3-to-2	Microscopic simulation	Late merge is better for high volumes, Early merge is better for low volume
(Harb et al., 2012) [35], (Harb, 2009) [36]	Conventional practice, Dynamic early and Dynamic late	Short-term	2-to-1, 3-to-2	Microscopic simulation	Dynamic early merge is better
(Vaughan et al., 2018) [37]	Zipper merge	Congested time period at field site	2-to-1, 3-to-2	Field test on Freeways, arterials	Increased safety, reduced travel time
(Kang et al., 2006) [39]	Dynamic Late merge	Field site	2-to-1	Microscopic simulation	Dynamic late merge is better than conventional
(Grillo et al., 2008) [41]	Dynamic Late merge	Field site	2-to-1	Field test on Freeways	Reduced travel time; better than early merge
(Idewu & Wolshon, 2010) [24]	Joint merge and early merge	Field site	2-to-1	Field test on Highways	Joint merge had higher volume in closing lane

2.4 Tools to Measure Mobility and Safety at Lane Closures

A wide variety of tools are available to measure safety and mobility at work zones. For example, the Highway Capacity Manual (HCM) 2000 has developed a system to measure capacity of work zones. It takes base capacity, work zone adjustment factors, heavy

vehicle factors, and existence of nearby ramp as its input. However, this manual does not have any method for evaluating queue length [36].

Queue and user cost evaluation of work zone (QUEWZ) is another system that uses HCM 2000 to estimate work zone capacity and HCM 1994 to estimate queue length. QuickZone is one tool that was developed for the Federal Highway Administration (FHWA) to calculate traffic impacts of work zones. This tool can be tailored to represent a particular work zone under any state DOT. DELAY Enhanced 1.2 is another tool developed for FHWA as well to identify traffic impact of work zone. Queue length can be plotted for short-term work zones.

Several other microscopic simulation software can be used to analyze traffic impacts on work zones namely, VISSIM, CORSIM, SimTraffic, AIMSUN, ARENA, VISTA, SSAM etc. Among them, VISSIM has been used to code work zone in many studies available in the literature and several researchers provided recommendations on how to calibrate driving behavior parameters in VISSIM for closely matching to the real lane closure condition [30]. Table 2-3 shows a summary of earlier studies that coded various lane closure merge control strategy using these available simulation platforms.

Table 2-3: Some earlier studies using various simulation platform

Researchers	Strategy	Simulation platform
(Beacher et al., 2005) [33]	Static Late merge vs early merge	VISSIM
(Kurker et al., 2014) [34]	Static Late merge, early merge, signalized merge	VISSIM
(Pesti et al., 2008) [14]	Dynamic merge	VISSIM
(Wei et al., 2010) [3]	Dynamic merge with merge meter	VISSIM
(Lentzakis et al., 2008) [25]	Mainline metering	AIMSUN
(Tympakianaki et al., 2014) [26]	Mainline metering	AIMSUN
(Harb et al., 2012) [35], (Harb, 2009) [36]	Conventional practice, Dynamic early and Dynamic late	VISSIM, VAP
(Sun et al., 2013) [28]	Temporary Ramp Meter	ARENA
(Kang et al., 2006) [39]	Dynamic Late merge	CORSIM
(Oner, 2009) [27]	Temporary Ramp Meter	ARENA

Based on the features and capabilities of the various software options, the literature suggests that VISSIM or CORSIM could meet the modeling needs of the current study the best. Thus, those two options are reviewed in greater detail next.

VISSIM

VISSIM is a microscopic, stochastic, multi-modal simulation model that was developed in Karlsruhe, Germany by Planning Transport Verkehr (PTV) [30, 36]. PTV VISSIM distributes the software in the United States. This software takes traffic volume, composition, lane distribution, speed, type of roadway and other parameters as inputs and analyzes roadway traffic operations based on the coded network [42]. The model is based on Wiedemann's work [11]. The advantage of using this software is that it takes "psycho-physical" driver behavior into account in the simulation. The accuracy of the model relies on modeling of vehicle and driver behavior. Freeway condition is coded on the basis of the Wiedemann 99 car following model (W-99) where there are 10 user defined driving behavior parameters. Drivers take the decision to increase or decrease the speed based on threshold value of speed and distance in W-99. The model is developed in such way that drivers perceive speed, safe distance, and desired speed between two vehicles. Gap acceptance criteria are also included in the model which ensures that a driver would change lanes only when the gap is more than a set critical gap [30].

Corridor-Microscopic Simulation Program (CORSIM)

CORSIM is a part of Traffic Software Integrated System (TSIS) which is a combination of NETSIM (surface street simulation) and FRESIM (freeway simulation). It is a microscopic time-step simulation model used to evaluate operation of traffic on roadways and is based on car-following and lane-changing logic [11]. It is a stochastic model and takes drivers' behavior, traffic system, and vehicles into account while analyzing.

CORSIM is developed on behalf of FHWA by combining other simulation platforms into one platform. Researchers report that there might be some problems with managing high on-ramp volume with metering or managing off-ramp high volume with backups [43] but the Minnesota DOT provided a solution to such problem by addressing the integration of nodes between freeways and surface streets [44].

Researchers have compared the most commonly used simulation software, VISSIM and CORSIM based on technical aspects and features [45] and determined these two are actually similar types of platforms. It was reported that VISSIM has an advantage over CORSIM as it has the ability to simulate dynamic merge control by using the Vehicle

Actuated Programming (VAP) feature. Another study was conducted to compare these two software [43] and their findings are summarized below:

- VISSIM has path-based routing option that ensures correct use of lane for closer intersections. CORSIM has link-based routing that may not perform well with lane utilization for closer intersections.
- VISSIM has built-in a detailed three-dimensional animation features to enrich visual understanding. CORSIM offers two-dimensional animation.
- Both software tools report total delay by link and are unable to evaluate control delay for turn maneuvers. Yet, CORSIM can estimate control delay for each approach.
- CORSIM can cause incorrect output at high on-ramp volume with metering or managing off-ramp high volume with backups because of the barrier between freeways and surface streets.
- VISSIM requires more time for set-up compared to CORSIM.

Earlier studies conclude that there is no perfect software that is applicable for all various types of work zone [46] but developing a network is easier with VISSIM as it allows the user to build a network on the aerial photo of actual location by drawing links and connectors [45]. Another benefit is that VISSIM runs based on psycho-physical driver behavior developed by Wiedemann, instead of setting a desired headway like CORSIM. Finally, VISSIM gives the user more flexibility to collect output data by specifying location of data collection points [45]. Based on the literature, VISSIM was selected as the simulation platform in this study for its superiority and ability to meet the study goals and needs.

3.0 METHODOLOGY

3.1 Introduction

The purpose of the study is to investigate operational performance of two temporary traffic controls namely early and late merge for 3-to-1 and 3-to-2 lane closure configurations for various work zone lengths. This chapter presents how the study was conducted, and discusses the study segment, experimental design, network coding in VISSIM, calibration and validation efforts, and measures of effectiveness considered.

3.2 Study Corridor

In this study, a decision was made to employ simulation modeling for the study of operational performance of early and late merge for 3-to-1 and 3-to-2 lane closure

configurations for various work zone lengths. Simulation modeling provided an opportunity for controlled experimentation and analysis as an actual work zone set up on the field was not a feasible or desirable option. Changing the traffic control strategy or length of an existing work zone frequently can be very confusing for drivers and might have undesirable effects on the safety and convenience road users. A microsimulation study, on the other hand, allowed to make changes in the work zone set up, TTC or other parameters and to observe their impacts on traffic operation through the collection and evaluation of performance measures.

In this study, a 12-mile corridor of Interstate 65 (I-65), passing through the city of Birmingham in the state of Alabama, was selected as the study site (

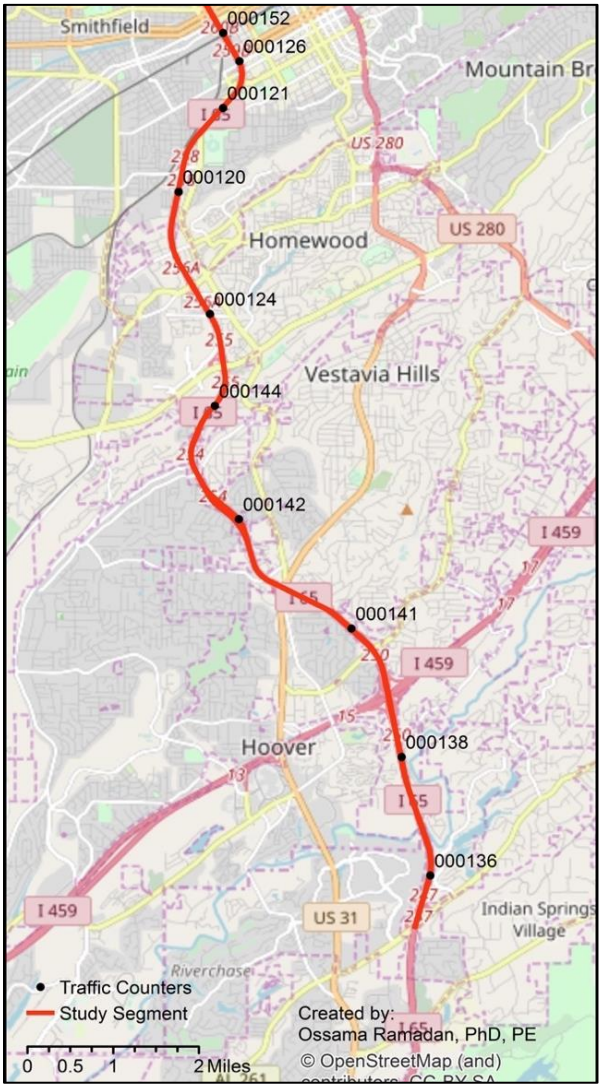


Figure 2-1: Study corridor

).

The I-65 corridor considered in this study is located within Jefferson County, Alabama, and extends from downstream of Exit 247 near Valleydale Road to just before Exit 261A on the northbound direction where the interstate passes over 1st Ave North. The interstate has 3 traffic lanes per direction, with occasional acceleration/ deceleration lanes added near ramps to facilitate the vehicle movements. The typical lane width is 12-ft and the posted speed limit is 60 mph on the interstate and 45 mph on the ramps. Table 2-4 gives detailed information about the study corridor.

Table 2-4: Geometric information of the study corridor

Description of the segment	Number of lanes	Total width (ft)	Total length (ft)
From exit 247 to exit 250	4	48	12777.6
From exit 250 to exit 252	4	48	9715.2
From exit 252 to exit 254	3	36	9187.2
From exit 254 to exit 255	3	36	7339.2
From exit 255 to exit 256	3	36	5491.2
From exit 256 to exit 258	3	36	7656
From exit 258 to exit 259	3	36	6652.8
From exit 259 to exit 260	4	48	3696

3.3 Traffic Characteristics

Recent traffic volumes for the study corridor mainline were obtained through the Alabama Department of Transportation (ALDOT) for Thursday April 19, 2018, a typical weekday. The volumes were obtained for 24hrs on an hour-by-hour basis starting from 12 midnight. The Traffic Monitoring section of the Maintenance Bureau at ALDOT routinely collects and maintains traffic volume for the major roadways in the state of Alabama. For traffic coding

purposes, in addition to mainline counts, on-ramp and off-ramp volumes were also needed. Those are collected periodically by ALDOT and were obtained by contacting with the personnel of ALDOT. Many of those ramp counts were not collected on April 19, 2018, but all were collected in 2018.

When the base data sheet was obtained from ALDOT for the month of April, it was observed that the data source contained both the mainline and ramp volumes for many stations in the state. Each station provided a description about its location that includes details such as which county it is located in, its position with respect to highway, etc. To identify the stations that fell within the study segment, a description of each station was checked and those falling in Jefferson County were extracted. These were again scrutinized to identify which are located on I-65 and a description about nearby streets was used to get an accurate idea about the stations within the study site. Finally, to be completely assured, the ArcGIS Shapefile was collected from the Regional Planning Commission of Greater Birmingham which has length and location of each traffic management center (TMC) along I-65. These were matched and checked with the findings from the raw data sheet so that no traffic counter remained unidentified. Once these were confirmed, mainline volume and ramp volumes were obtained only for these stations located within the study site and arranged together in the sequence along the I-65 study corridor (northbound and southbound direction).

After organizing the traffic volumes according to the segments along the study corridor, it was noticed that some traffic volumes did not balance. This was expected, as the ramp volumes and mainline volumes were not collected simultaneously and there is variation in traffic volumes from day to day. Therefore, while the actual mainline volumes were kept exactly as reported by the ALDOT April 19, 2018, data, a few ramp volumes were adjusted to ensure that no vehicles were disappearing from the network. The balancing exercise ensures that when off-ramp volume is subtracted from a mainline segment upstream of an exit and it is added to the on-ramp volume of that exit, it should be equal to the volume of mainline segment downstream of that exit. Finally, the traffic composition was assumed to be 90% passenger vehicles and 10% trucks throughout the simulation experiments.

3.4 Experiment Design

Once the geometric properties of the study corridor and traffic data were established, the base model was developed in VISSIM to simulate existing conditions during all 24 hours of the day. Moreover, simulation models were developed for the work zone situation for a 3-to-1 and 3-to-2 lane closure configuration. Three different work zone lengths were considered namely 500 ft, 1,000 ft, and 1,500 ft. The shortest work zone length (500ft) was considered for long-term work zone operation (24hrs), as well as for short-term work zone operation during morning peak hour and non-peak hour analysis. Based on the findings from the 500 ft work zone length, other work zone lengths were tested for peak and non-peak short-term work zones. The longest work zone was tested for non-peak hour work zone only.

Error! Reference source not found. shows the experiment design for this study. Each of the scenarios had 5 runs in VISSIM resulting in 60 runs total for work zone scenarios, and the predetermined measures of effectiveness were collected and averaged across the five runs. The base model was run separately for a 24hrs period as well as for the 3-hr AM peak period (6:00-9:00 AM) and a 3-hr PM non-peak period (9:00-12:00 PM), resulting in a total of 85 runs for the entire analysis.

Table 2-5:Experimental design for the study

Scenario Number	Work zone length (ft)	Merge control strategy	Lane drop configuration	Duration
1.	500	Early merge	3-to-1	24-hr
2.	500	Late merge	3-to-1	24-hr
3.	500	Early merge	3-to-1	3-hr peak
4.	500	Late merge	3-to-1	3-hr peak
5.	500	Early merge	3-to-1	3-hr non-peak
6.	500	Late merge	3-to-1	3-hr non-peak
7.	1000	Early merge	3-to-1	3-hr peak
8.	1000	Late merge	3-to-1	3-hr peak
9.	1000	Early merge	3-to-1	3-hr non-peak
10.	1000	Late merge	3-to-1	3-hr non-peak
11.	1500	Early merge	3-to-1	3-hr non-peak
12.	1500	Late merge	3-to-1	3-hr non-peak
13.	1000	Early merge	3-to-2	24-hr
14.	1000	Late merge	3-to-2	24-hr

3.5 Base Model Development

Base model development procedure included network coding with proper geometry, entering of traffic volumes, and setting driver behavior parameters. Details of all steps are described below.

3.5.1 Network Coding

The roadway network geometry was obtained from available aerial map of VISSIM. Moreover, the number of lanes, location of auxiliary lanes, and curves

were confirmed by field visits. All segments of the freeway of I-65 are drawn using links and the connector feature of VISSIM. One link can have same number of lanes throughout it. Therefore, separate links are drawn from one exit to the next one since the total number of lanes remains same for such segments except for locations where auxiliary lanes are needed. The width of each lane was set to be 12 ft. On-ramp and off-ramp segments are drawn using the link feature. To represent extra lanes for deceleration, a connector was drawn starting from the end of freeway link end extending until the starting point of off-ramp link. Acceleration lanes are also coded in a similar manner. Due to the unavailability of grade information of the interstate, no grades were added for each of the links.

In VISSIM, the type of the link selected controls the type of driver behavior. There are five types of links based on behavior namely, urban (motorized), right-side rule (motorized), freeway, footpath, and cycle-Path. Since the study corridor is on interstate, “freeway” was selected as the type of the links and connectors. Lane change behavior of vehicles was coded changing the default lane change parameters and emergency stop for connectors. Lane changing distance was varied for each connector from the default value. This distance means how much ahead of a turn the vehicle would try to change its lane to reach its destination. Various combinations were used to replicate the actual scenario. Finally, when travel time for each segment became closer to the field value, that distance was set for the next runs.

3.5.2 Traffic Coding

Traffic coding refers to the process of inputting correct traffic volume with correct vehicle composition for each link. Traffic demand data set was prepared based on field observed data retrieved from ALDOT data sources. In VISSIM, traffic can enter at the starting point of a link. To represent actual conditions, vehicles’ entrance was coded for the very first link of interstate and for the entrance at each on-ramps. The data sheet was prepared for on-ramp volumes so that the mainline volume for all links match with actual data. Since the study focused on 24-hour duration, 24 “time intervals” were created with each interval having 3600 seconds, i.e., 1-hour slots. Then traffic volume was loaded carefully for each time interval at those above-mentioned source nodes.

As mentioned earlier, the segment typically contains 10% heavy vehicle, and the posted speed limit is 60 mph. So, default vehicle types - car and HGV (truck) were used to create a composition with desired speed of 60 mph. VISSIM allows a distribution of speeds instead of a fixed one for all time, because vehicles’ speed varies from time to time. Some links were coded with different ranges of

desired speeds at certain time periods of the day to closely match field observations. The change of speed was based on real data collected from the National Performance Management Research Data Set (NPMRDS) and obtained through the Regional Planning Commission of Greater Birmingham (RPCGB). Thus, vehicle composition was created and assigned to the vehicle input for the links.

Vehicles exit behavior also needed careful coding. VISSIM has various route choice decisions. Vehicle routes have a sequence of links and connectors that direct vehicles in the desired direction. For this study, the static route decision was coded before each exit. One route directs vehicle towards the off-ramp, and another route directs vehicle along the interstate. Based on the ALDOT off-ramp data, exit volumes were coded using route choice decision and the rest were entered in the straight direction of the route to make sure that no vehicles were disappearing or added automatically in the network.

3.6 Model Calibration and Validation

VISSIM is a broadly accepted microsimulation too that has been utilized for analyzing many freeways in North America. Still, in every new study, calibration and validation is needed for ensuring that VISSIM is coded properly so that it accurately replicating real field conditions.

Calibration parameters fall into two categories- one is system calibration parameters, and another is operation calibration parameters. System calibration refers to checking model input. The most commonly used calibration parameters are related to driver behavior which falls under operational calibration. Driver behavior affects how the model works and the output changes based on different driver behavior parameters. In this study, Wiedemann 99 car following model is used to control drivers' characteristics in the freeway segments. There are ten calibration parameters in VISSIM labeled as CC_0, CC_1, CC_2, CC_3, CC_4, CC_5, CC_6, CC_7, CC_8, CC_9 for freeway behavior. The operational calibration parameters that were changed in the study include car following behavior, lane changing behavior, and lane changes distances. Default parameters for lane change distance were used initially to run the model, but those values did not represent the study area close to reality. Therefore, parameters related to driver behavior were changed along with different lane changing distances and a total 25 versions of the model were run for the simulation, each having 5 runs, with various parameter combinations until the model was validated. Following are the values of the above-mentioned parameters used in the model:

- CC_0: This is the standstill distance that defines desired distance between two consecutive vehicles when vehicles have a speed, v=0 mph. The value for this one is 4.92 ft in the model.

- CC_1: This refers to the desired headway time in seconds between two consecutive vehicles. 0.9 seconds is used for this parameter in the model. Higher value of this means drivers drive in a safer manner.
- CC_2: This represents the following distance that a vehicle would maintain for safety. The model uses 13.12 ft as the following variation.
- CC_3: This refers to the time in seconds when any driver starts to decelerate because of slower moving lead vehicle to reach safety distance. The value of it is -8 seconds.
- CC_4 and CC_5: These parameters specify the speed variation between leading and following vehicle. If the value is small, this indicates more sensitive reaction from following drivers due to acceleration or deceleration of lead vehicle. The model uses -0.35 ft/s for CC_4 and 0.35 ft/s for CC_5.
- CC_6: Represents how speed oscillation depends on the distance of following condition. Higher value of the parameter means that speed oscillation increases because of increased distance from the leading vehicle. Once this threshold following value is surpassed, then the following vehicle's speed does not depend on the leading vehicle.
- CC_7: Defines the acceleration rate during oscillation. The model uses 0.82 ft/s^2.
- CC_8: Defines standstill acceleration that is desired from standstill condition. The value for this parameter is 11.48 ft/s^2.
- CC_9: Defines rate of acceleration that is desired at a speed of 50 mph. Value of CC_9 is 4.92 ft/s^2.

The total travel time along the study corridor was selected as the validation parameter. RPCGB has a Shapefile with location and length of traffic management center (TMC) along the study segment. Travel time data for each TMC for 24-hour time periods were collected for the month of April and averaged for typical weekdays (Monday, Tuesday, Wednesday, and Thursday) to make sure that the travel time is representative for a typical day. Then "Vehicle travel time measurements" segments were configured in VISSIM retrieving data from ArcGIS Shapefiles and locations of each TMCs starting point and ending point were placed carefully in the model. Table 2-6 shows the length of TMCs which were created in identical manner in VISSIM. These segments can measure travel time for the vehicles passing at different time slots. All versions of the model had the same travel measurement segments and was run for 5 times with each combination of parameters. Finally, one model that generated the closest value of validation parameter, i.e., travel time, was selected as the base model and work zone with different merge control strategies and zone lengths were coded by making the necessary adjustments to the main model.

Table 2-6: Length of each TMC according to RPCGB record

Number	TMC	Length (ft)	Length (mile)
1.	101+04371	5524	1.05
2.	101+04372	5541.13	1.05
3.	101+04373	7584.48	1.44
4.	101+04374	5081.41	0.96
5.	101+04375	3256.79	0.62
6.	101+04376	5104.85	0.97
7.	101+04377	4137.99	0.78
8.	101+04378	130.18	0.02
9.	101+04379	373.69	0.07
10.	101+04380	1431.45	0.27
11.	101+05095	4611.84	0.87
12.	101P04371	5628.26	1.07
13.	101P04372	1130.58	0.21
14.	101P04373	2956.17	0.56
15.	101P04374	2101.63	0.40
16.	101P04375	2976.87	0.56
17.	101P04376	2262.86	0.43
18.	101P04377	1468.54	0.28
19.	101P04378	468.18	0.09
20.	101P04379	755.74	0.14

The target of the calibration and validation efforts was to fine-tune the model so that travel time from VISSIM for each hour falls within ± 15% range of actual travel time values obtained from the National Performance Management Research Data Set (NPMRDS) through RPCGB. This range demonstrates the tolerance of acceptability and is reasonable for traffic studies as there is great variation in traffic and. The calibration effort was carried on until all 24 travel times fall within acceptable range.

The plot depicted in **Error! Reference source not found.** shows the comparison between the actual travel time obtained from the NPMRDS dataset through RPCGB for April 19, 2018, and the travel times obtained from the VISSIM base model. One line is generated with 15% increase of RPC data which is termed as upper control limit (UCL) and another line is drawn with 15% decrease of RPC travel time that is defined as lower control limit (LCL).

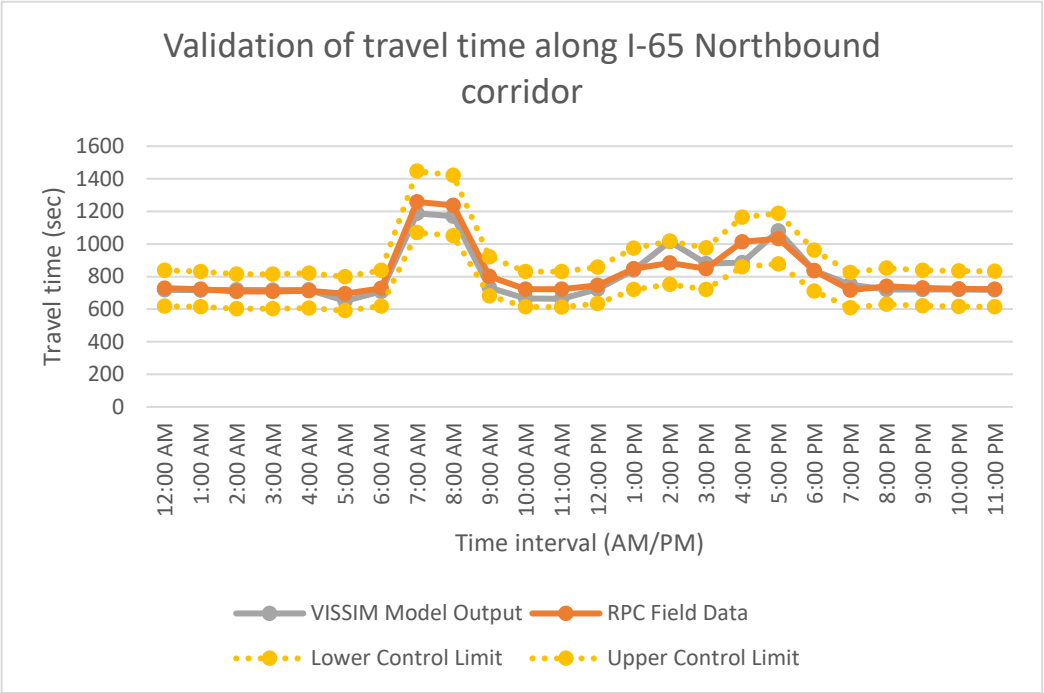


Figure 2-2: Validation of travel time along northbound corridor

As shown in**Error! Reference source not found.**-2, the line that represents the VISSIM model's travel time along the study corridor nicely matches the field data and falls within the boundary of upper limit and lower limit. This confirms that the model output is within the preset acceptable range and thus the model can be utilized for further analysis. This validation effort ensured that the model represents the actual traffic behavior observed on the selected study segment.

3.7 Work Zone Setup

Once the base model is calibrated and validated, the work zone is placed in the middle portion of the study corridor in the northbound direction. The 3-to-1 lane closure configuration closed the two right most lanes, leaving the left lane in operation through the work zone. Since early merge control typically puts advanced warning signs 1-mile ahead of the work zone and encourages driver to merge at this point, the location of the work zone is carefully selected so that it affects the minimum number of ramps. This is because the primary objective of the study is to compare the operational impact of merge control options. The researchers wanted to reduce impacts that are attributable to closure of a ramp.

The segment of I-65 between US-31 and Lakeshore Pkwy was the location selected for placing the work zone. For this purpose, exit 254 had to be closed. This was the best way to minimize the number of ramp closures in the middle segment of the total corridor and observe the extent of destruction caused by the work zone placement on the links upstream. If the hypothetical work zone was set up at other location, more than one ramps would have to be closed. Therefore, the work zone was set up at this location. When this ramp was closed, distribution of the off-ramp and on-ramp traffic demand was taken care of by assuming that 40% of the volumes would use the previous off and on-ramp, another 40% volume would shift to the next exit after the closure and 20% of the ramp volumes would use alternative routes. To represent the distribution pattern, the volume for the previous exit (252 exit) of ramp closure and volume of the following exit (255 exit) had to be adjusted accordingly. The other ramp volume distributions and volumes along the mainline were similar as in the base model.

3.7.1 Setting up late merge control

For setting up the work zone under late merge for the 3-to-1 lane drop of the two right most lanes of the northbound direction considered in this study, the segment between exits 252 and 255 was divided into three segments. The starting segment had three lanes and after certain distance, one lane was dropped using a taper. From this point onwards, the segment maintained two lanes. After vehicles merge into two lanes, they would find another taper ahead instructing them to merge again to one lane. This is the practice for 3-to-1 closure in Alabama. The reason for having a portion with two lanes is to give the drivers a transition length to merge to one open lane from all three lanes in a smooth and orderly manner.

Error! Reference source not found. shows the typical position of the sign “merge here” for late merge control and the overall configuration of arranging for dropping 2 out of 3 lanes resulting in a 3-to-1 configuration. This is how late merge control is coded in VISSIM using link structures and connectors, along with route decisions.

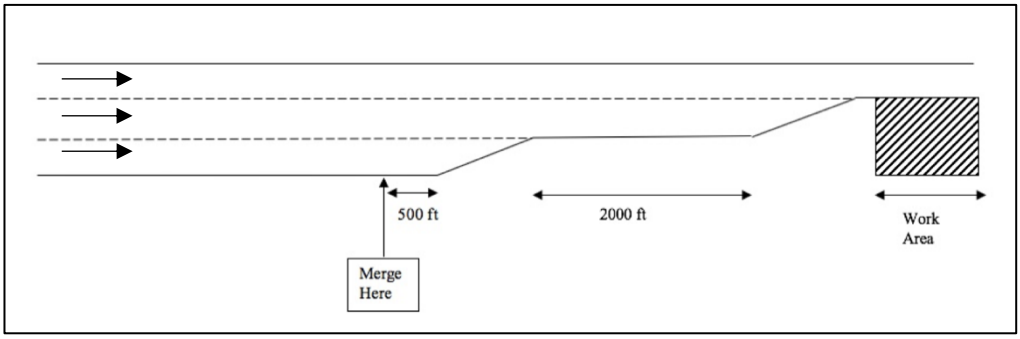


Figure 2-3: Work zone with late merge control for 3-to-1 closure scenario

The set-up of 3-to-2 closure is done in the similar manner in VISSIM except for there is no transition zone in 3-to-2. The configuration just drops one lane before the work zone and continues as a two-lane segment until the end of work zone.

3.7.2 Setting up early merge control

The position of the work zone for early merge was exactly the same as it was with late merge control (between exit 252 and 255). The only difference here was that vehicles are directed to merge very early to the open lanes leaving the closed lane unused for almost one mile. Advanced warning signs are placed 1.0 mile ahead of the work zone. The advance warning sign encourages merging early and helps direct drivers along with the signs “merge here” or “2 right lanes closed”. The typical practice for 3-to-1 closure is shown in **Error! Reference source not found.**. The intermediate segments are drawn in the same fashion for both late and early merge, but the lane change decision point is varied for these two, since early merge control requires coding that influences drivers not to use the closed lane.

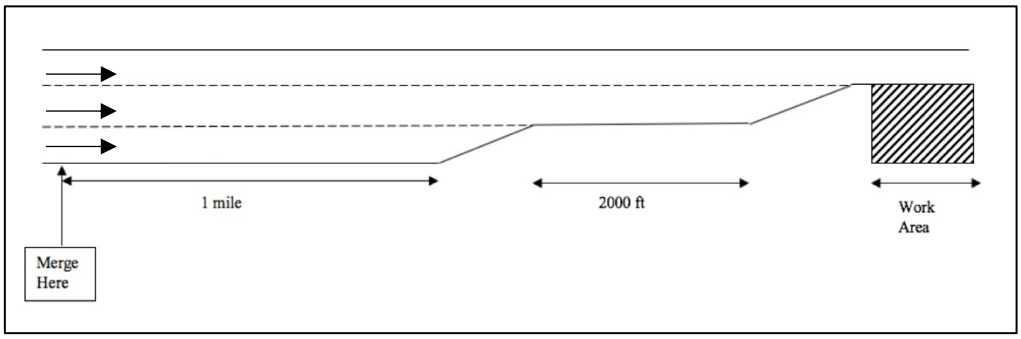


Figure 2-4: Work zone with early merge control for 3-to-1 closure scenario

The set-up of 3-to-2 early merge closure has the "merge here" sign 1 mile ahead of the lane drop and after it drops one lane, that's continued through the work zone.

3.8 Measures of Effectiveness

After placing work zone in the study segment and coding the model for early merge and late merge control strategy, measures of effectiveness (MOE) were estimated and used to compare the performance of the two strategies. The MOEs considered herein are travel time, speed, density, and flow. Density refers the number of vehicles per unit length of a roadway segment. This is a very important parameter to understand the level of service of the roadway. Delay is defined as the difference between expected travel time at free flow speed and the actual travel time along a roadway. Travel time is estimated for all vehicles that enter a system during a specific period. It is a measure of travelers' perception about the performance of the routes. Traffic flow specifies the number of total vehicles passing a certain point. This measure refers to the throughput and is very important parameter for better understanding of the performance of roadway. Travel time and speed are MOEs that relate to the user's satisfaction while density and traffic flow are good measures of the facility's performance.

3.9 Simulation Parameters and Evaluation Configuration

When the base model and work zone models for temporary traffic control were coded with proper geometry and traffic characteristics, and measures of effectiveness were also decided, the researchers focused on getting the expected data from VISSIM runs correctly. In an effort to do so, a few simulation parameters and evaluations criteria were fixed for each model so that the outcome could be representative of exact similar conditions. At first, a warm-up period was needed to be set up as commonly done with traffic simulation models. The warm-up period indicates the time after which the model will start collecting data so that the model does not start from an empty network state. This technique helps to ensure that the simulation model better mimics traffic conditions in real life. Therefore, warm-up period ensures that the simulation doesn't generate output for an empty network. Warm-up period is added to the actual simulation period and the starting time of day of the simulation is set earlier by the

warm-up period time. This model has an actual simulation period of 24-hour (86400 seconds), and 1 hour is taken as warm-up period. Thus, the simulation period was set as 90,000 seconds. For peak and non-peak segments, 900 seconds were taken as the warm-up period.

Simulation resolution is another parameter which means how many times the model would calculate any vehicle's position within one simulated second. The usual range can be 5–10-time steps per simulation second. If the value is higher, it ensures a smoother simulation. The model for this study used 10-time steps per simulation second.

Random seeds were also needed in the model to ensure that differences in results obtained by different models where due to the differences in the configurations studied, rather than variations in the streams of vehicles generated by the software. The random seed is a number linked to the arrival time of vehicles in the network, stochastic variability of driver behavior, etc. and ensures that the exact same sequence of vehicles is generated in each scenario that is using the same seed. In other words, if the same random seed is used for separate runs with identical inputs, the model would generate the same results. Since the purpose of the study was to compare operational impact of two merge control strategy, the same 5 random seeds were maintained for all versions of models (one for each iteration) to ensure that no change in the results were attributable to the vehicle arrival patterns. The number 42 was the random seed number for the first run (iteration) of the models. As mentioned earlier, to get more accurate results, each scenario of the models was run 5 times and results were averaged across those 5 runs in the final analysis. VISSIM gives the flexibility to use separate random seeds for various runs. The amount by which the first seed number increases for the next runs can be defined as random seed increment. The base model and models with work zone with early and late merge had used 20 as the increment. This means the first run started from random seed of 42, and the second run started for 62 and so on. Simulation speed was set as maximum.

Evaluation configuration specifies parameters that the model needs to evaluate from the simulation. For the study, density, vehicle travel time, speed, and flow were selected for evaluation. The time interval for which the simulation should generate results can also be defined here. The simulation model had traffic volumes for 24-hour entered by each of 24 1-hr long intervals and analysis could be by hour over the 24-hr time span. Therefore, the evaluation time interval was set as 3,600 seconds which means that the simulation model was asked to keep record for each measure of effectiveness on an hour-by-hour basis.

4.0 RESULTS

4.1 Introduction

This chapter summarizes the study findings based on measures of effectiveness (MOE), namely density, flow, speed, and travel time. Comparisons are performed for MOEs obtained under early merge, late merge, and base model scenarios for same time period of the day and same work zone length. For any given time period, the MOEs are extracted for the study segments

located upstream of the starting point of the work zone, the segment including the work area, and the segment 1 mile after the work zone ends. Flow and density are estimated per lane and then compared against base model output.

4.2 Comparison of 3-to-1 Merge Control for 500 ft Work Zone Length

According to the experimental design of the study, the shortest length for the work zone to be considered was 500 ft. Initially, both early merge and late merge scenarios were coded for long-term work zones, meaning the work zone was active on the roadway segment for a 24-hr period. Then based on the findings from long-term work zone study, separate models of peak and non-peak hour were also considered.

4.2.1 Long-term work zone analysis for 500 ft work zone

The simulation models of early and late merge scenarios with a 500ft work zone closure were run for a 24-hr period.

shows the flow along the segments from the start of the study corridor (i.e., upstream of the work zone) and up to one mile downstream of the work zone (a total of 7.61 mile from exit 247 to exit 255). The results show that when the volume was much less than the capacity at midnight, the placement of the work zone did not have much impact on the flow. However, when the volume started to increase from 6:00 AM in the morning, the scenario gets worse for both TTC options considered leading quickly to oversaturated conditions. One noteworthy observation is that the system is overwhelmed from the 2-lane closure and cannot recover throughout the day. This is because the one open lane cannot manage the demand and cannot clear the traffic accumulated from previous time intervals. Therefore, there is not much improvement in the flow even in the non-peak hour.

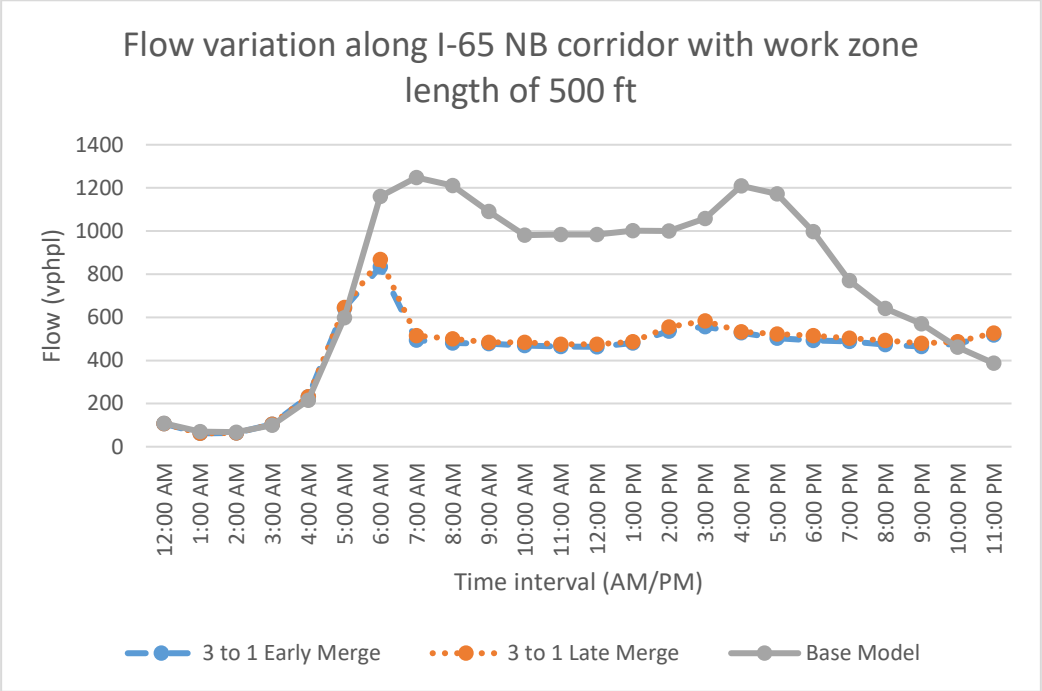


Figure 2-5: Flow variation along I-65 NB corridor (WZ length: 500 ft; 3-to-1 closure)

For better understanding of the extent of flow reduction with respect to the base flow condition, the following formula is used:

Percent reduction in flow = $\frac{Base\ flow - Flow\ with\ merge\ control}{Base\ flow} * 100$

Error! Reference source not found. shows the amount by which the flow throughput is reduced under the two TTC strategies considered. Up to a 60% reduction in flow was recorded in the 3-to-1 closure scenario.

Table 2-7: Percentage reduction in flow for long-term 3-to-1 closure

Number	Time period starting at	%reduction in flow with Early merge control	% reduction in flow with Late merge control
1.	7 AM	60 %	58 %
2.	8 AM	60 %	58 %
3.	4 PM	56 %	56 %
4.	5 PM	57 %	55 %

Density was also analyzed as it gives a clearer idea about the level of service of freeways. **Error! Reference source not found.** shows the density profile of the base model. This is the average density of the northbound direction from the start of the study section and up to 1.0 mile downstream of the work zone. The density profiles for the same section under late and early merge for 3-to-1 closure are superimposed to allow for comparisons

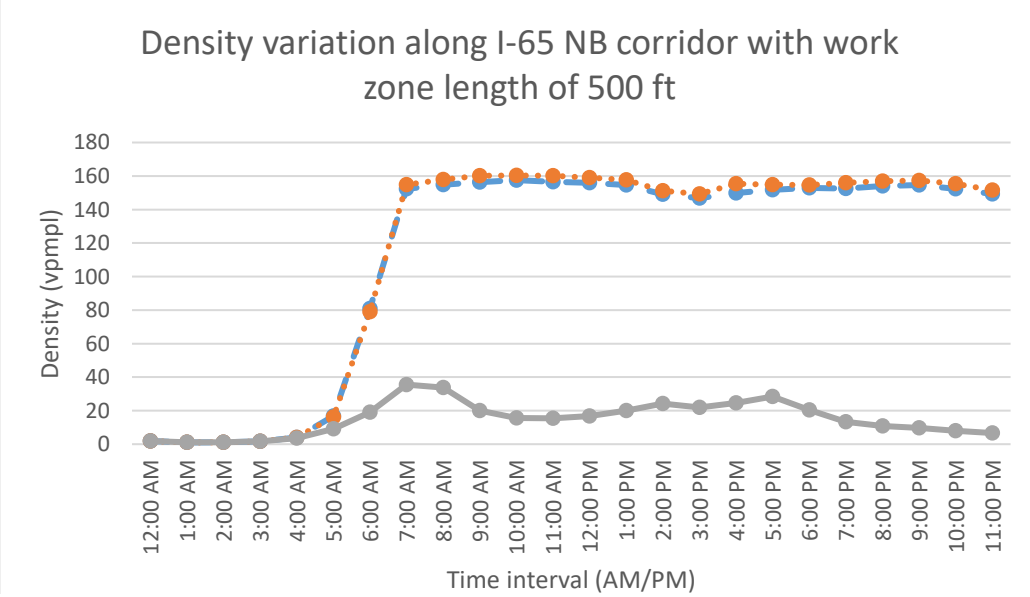


Figure 2-6: Density variation along I-65 NB corridor (WZ length: 500 ft; 3-to-1 closure)

Error! Reference source not found. shows that in the presence of the 3-to-1 work zone the density starts getting affected after 5:00 AM in the morning. The density reaches the jam density during the morning peak and remains the same during the day as the roadway cannot recover and remains congested until the end of the study period (12:00 midnight).

4.2.2 Short-term work zone analysis (Peak period) for 500 ft work zone

Besides investigation of the impact of a long-term work zone with two lanes closing, the researchers also looked at the impact of a 3-to-1 closures for short-term work zone. Short-term work zones were analyzed for the morning peak period (6:00 AM to 9:00 AM) which is the most severe peak period along the northbound direction. For the peak period analysis, flow, speed, density, and travel time were compared for early and late merge and the results are depicted in Figures 2-7 through 2-12.

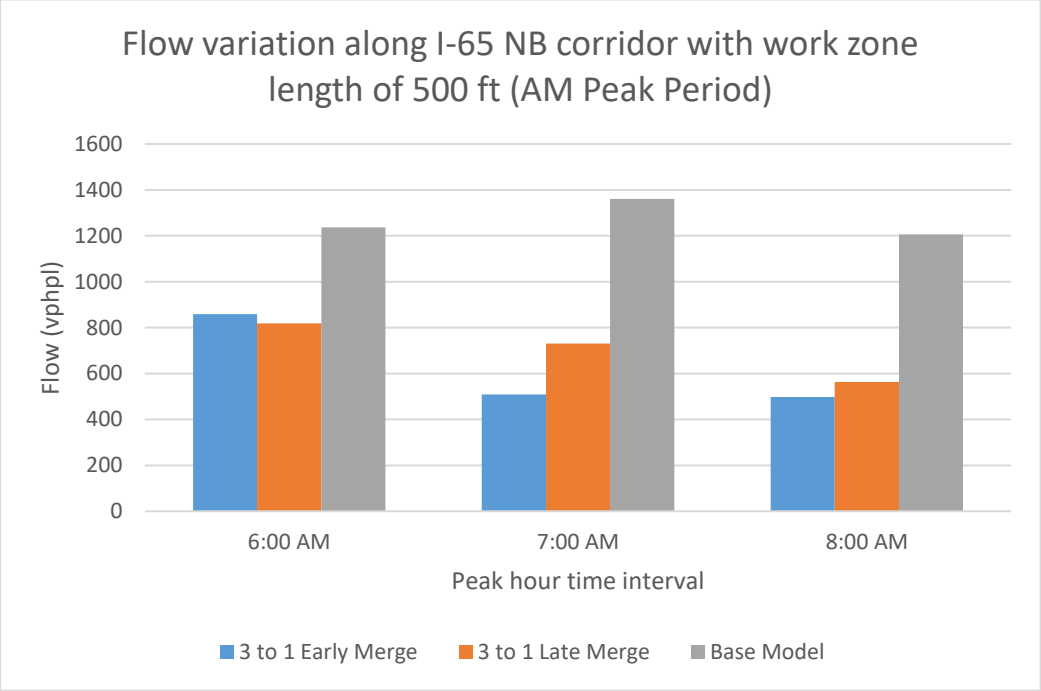


Figure 2-7: Flow variation along I-65 NB corridor (WZ length: 500 ft; Peak period; 3-to-1 closure)

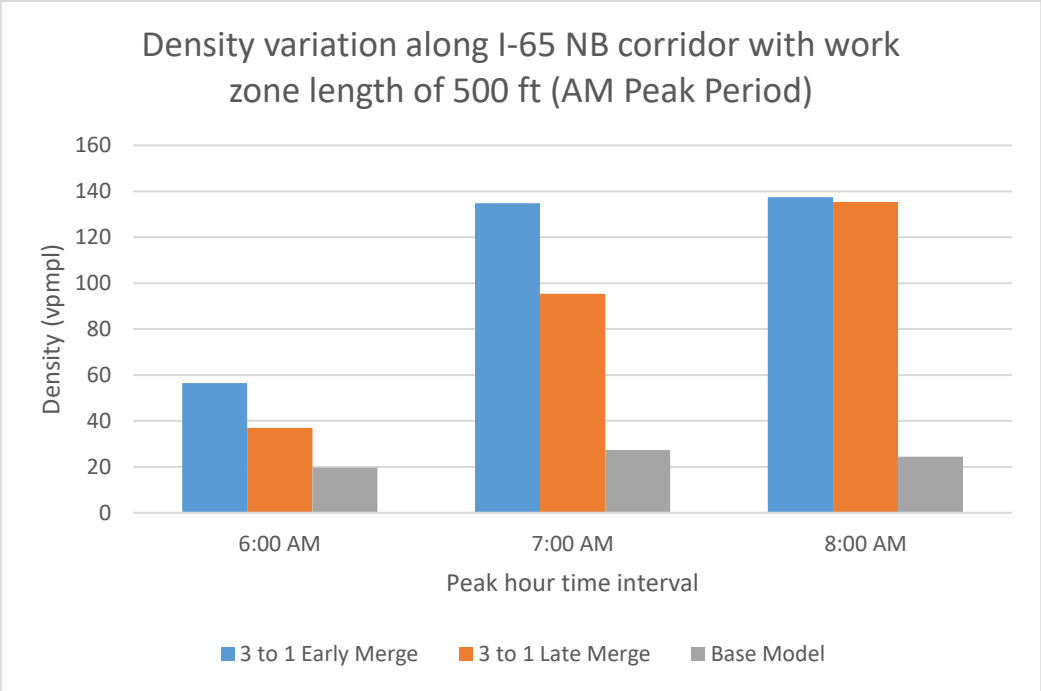


Figure 2-8: Density variation along I-65 NB corridor (WZ length: 500 ft; Peak period; 3-to-1

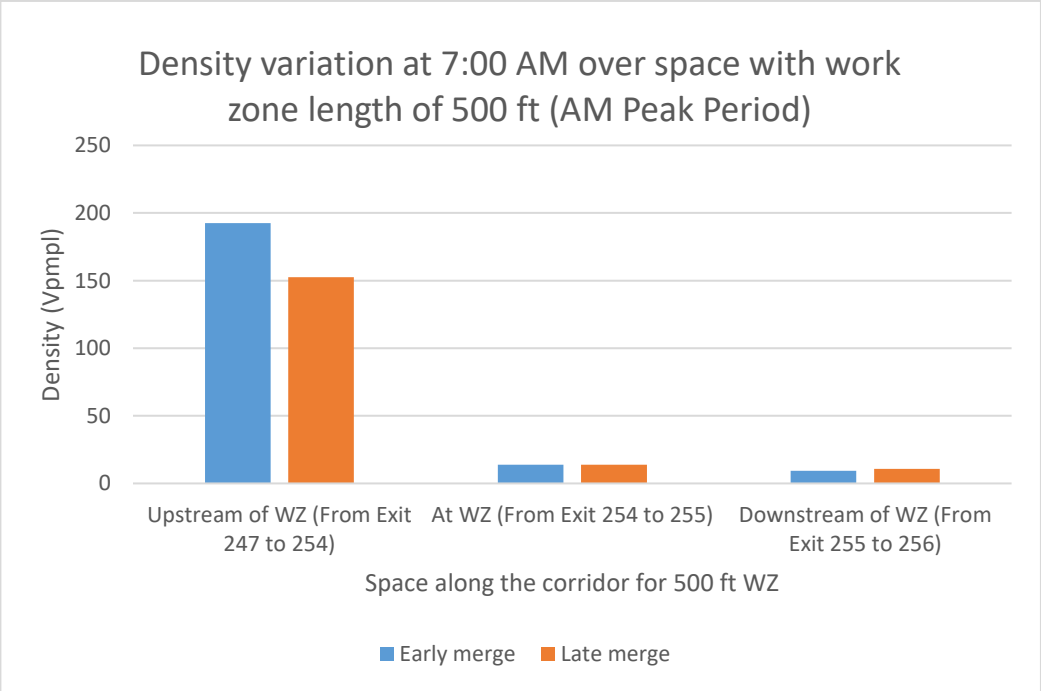


Figure 2-9: Density variation at 7:00 AM over space (WZ length: 500 ft; Peak period; 3-to-1 closure)

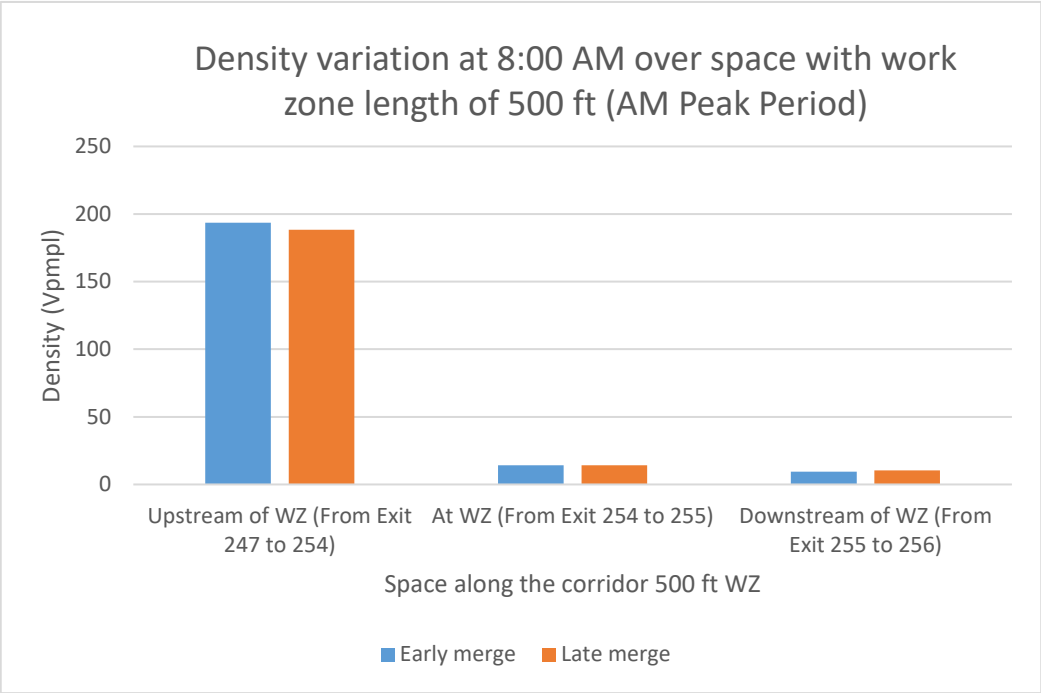


Figure 2-10: Density variation at 8:00 AM over space (WZ length: 500 ft; Peak period; 3-to-1 closure

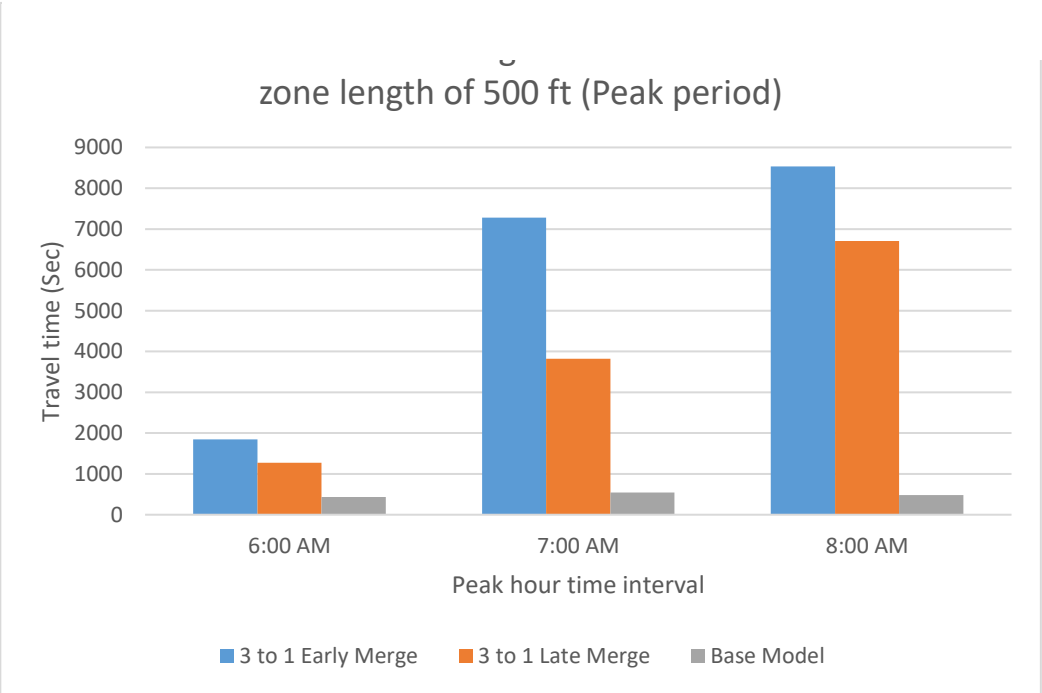


Figure 2-11: Travel time variation along I-65 NB corridor (WZ length: 500 ft; Peak period; 3-to-1 closure)

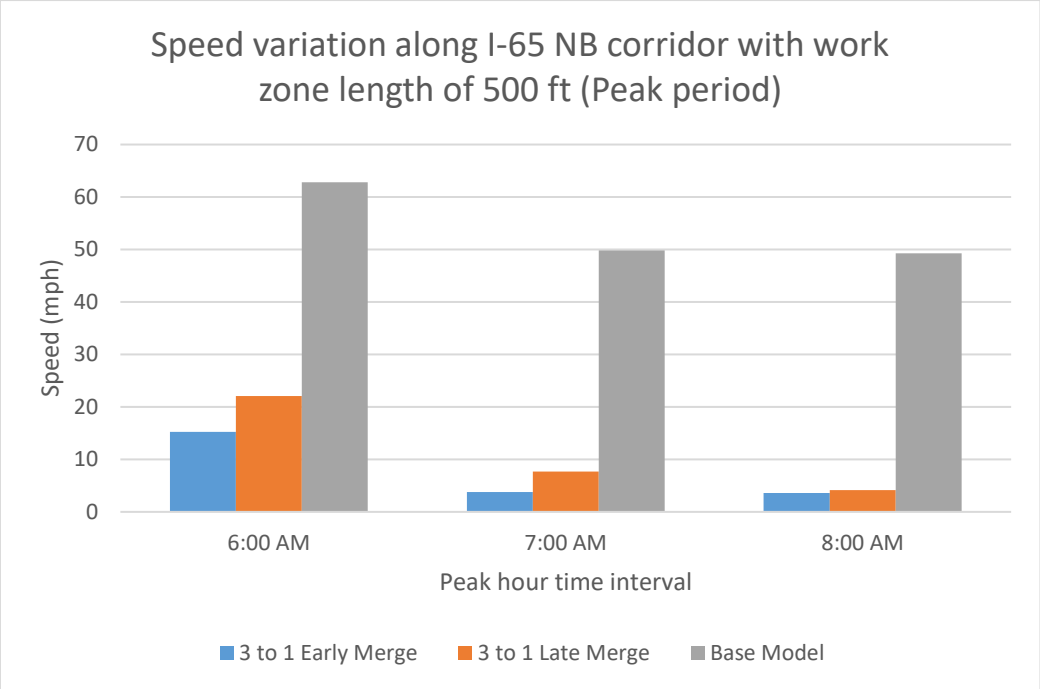


Figure 2-12: Speed variation along I-65 NB corridor (WZ length: 500 ft; Peak period; 3-to-1 closure)

The findings from the short-term 3-to-1 work zone lane closure of 500 ft length during the AM peak showed that the base model – as expected - had clearly the highest flow and speed, and lowest density and travel time. When early merge and late merge performance were compared, it was observed that late merge resulted in slightly better performance than early merge for all MOEs considered as long as the volume-to-capacity ratio was under 1.0. However, under oversaturated conditions both types of merge controls failed to accommodate the demand.

4.2.3 Short-term work zone analysis (Non-peak period) for 500 ft work zone

Since late merge strategy performs slightly better than early merge at onset of the peak hour, it is important to confirm if the behavior remains similar for non-peak hours or not. Therefore, MOEs were collected for a non-peak time interval (9:00 PM-12:00 PM) and compared, as shown in Figures 2-13 through 2-16.

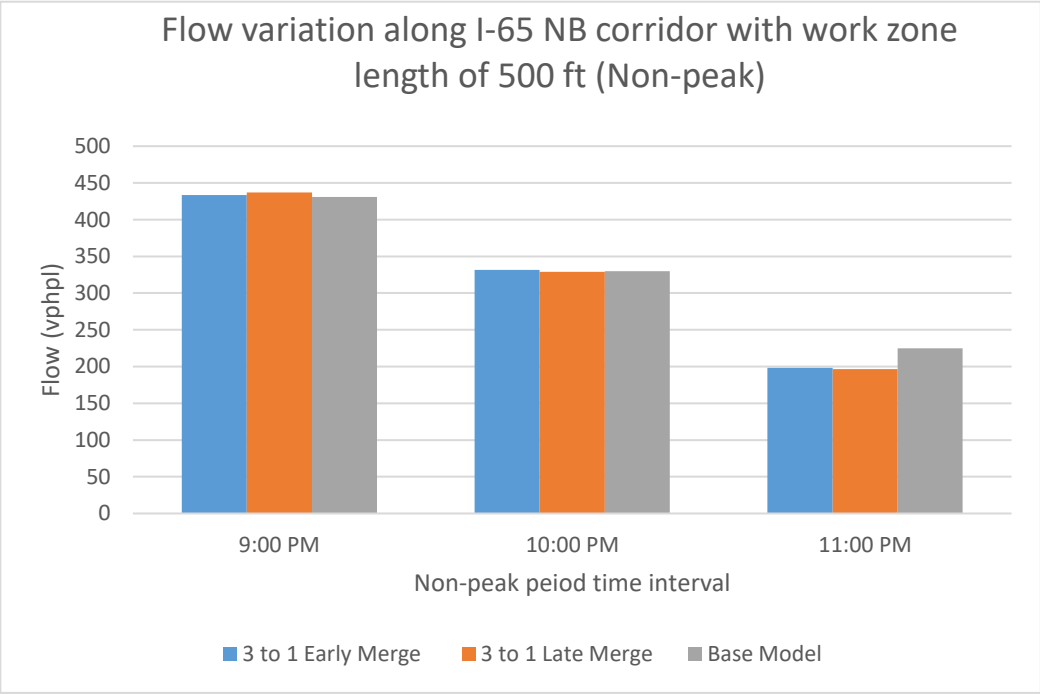


Figure 2-13: Flow variation along I-65 NB corridor (WZ length: 500 ft; Non-peak period; 3-to-1 closure)

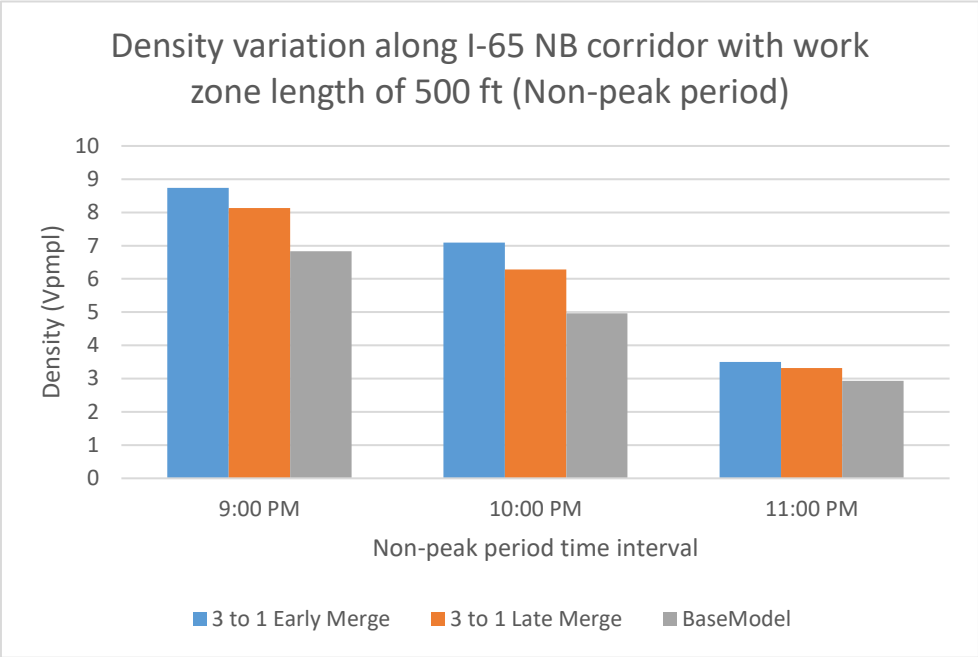


Figure 2-14: Density variation along I-65 NB corridor (WZ length: 500 ft; Non-peak period; 3-to-1 closure)

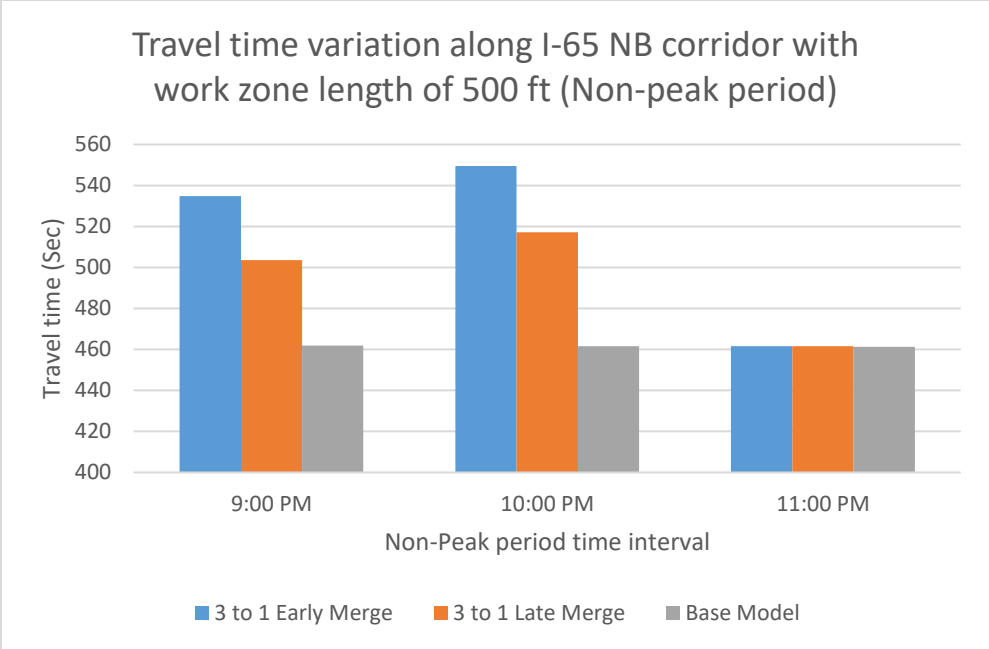


Figure 2-15: Speed variation along I-65 NB corridor (WZ length: 500 ft Non-peak period; 3-to-1 closure)

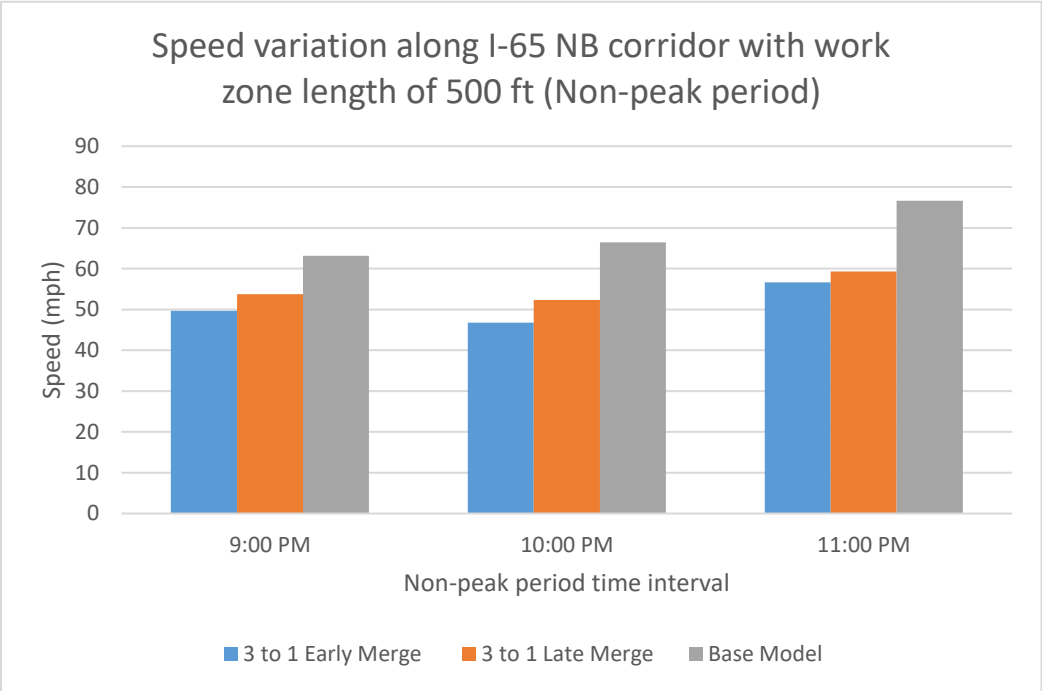


Figure 2-16: Speed variation along I-65 NB corridor (WZ length: 500 ft Non-peak period; 3-to-1 closure)

As far as the thought put is concerned, all options perform similarly during the night non-peak time period. Given the low demand during this time period (i.e., low volume-to-capacity ratio) the performance of the network is not affected by the 3-to-1 lane closure and the remaining open lane can handle the traffic demand at that time. This observation is consistent in the results presented in Figures 2-14 through 2-16 and all MOEs provide similar results with the late merge control showing a small advantage over early merge control, but not enough to justify its selection over its counterpart.

4.3 Comparison of 3-to-1 Merge Control for 1000 ft Work Zone Length

After observing the impact of merge control strategy on 3-to-1 lane closure with 500 ft work zone length for long-term and short-term, it was clearly seen that long-term work zone with 3-to-1 lane closure collapses the whole system, even with the shortest work zone length of 500ft. Therefore, efforts to evaluate the performance of work zones of longer length concentrated only on short-term work zones.

4.3.1 Short-term work zone analysis (Peak period) for 1000 ft work zone

A short-term work zone with 1000 ft length and a 3-to-1 lane drop was modeled under the AM peak period and for both late and early merge strategies. The purpose is to observe the impact of length of work zone on the performance. Figures 2-17 through 2-20 show the density, flow, speed, and travel time for 1000 ft work zone with late and early merge controls in place.

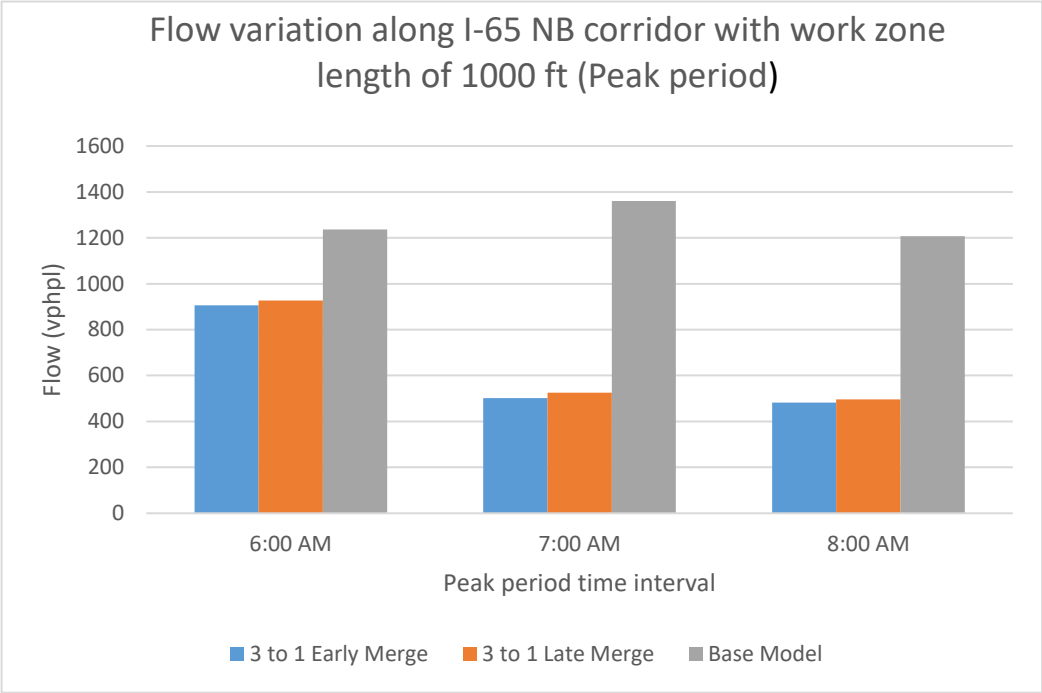


Figure 2-17: Flow variation along I-65 NB corridor (WZ length: 1000 ft; Peak period; 3-to-1 closure)

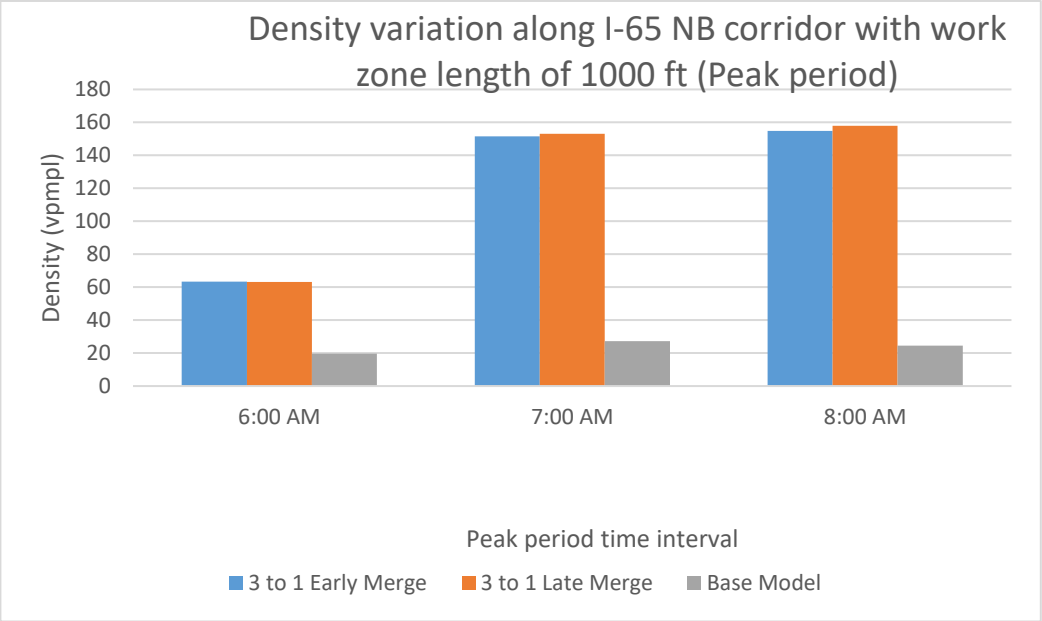


Figure 2-18: Density variation along I-65 NB corridor (WZ length: 1000 ft; Peak period; 3-to-1 closure

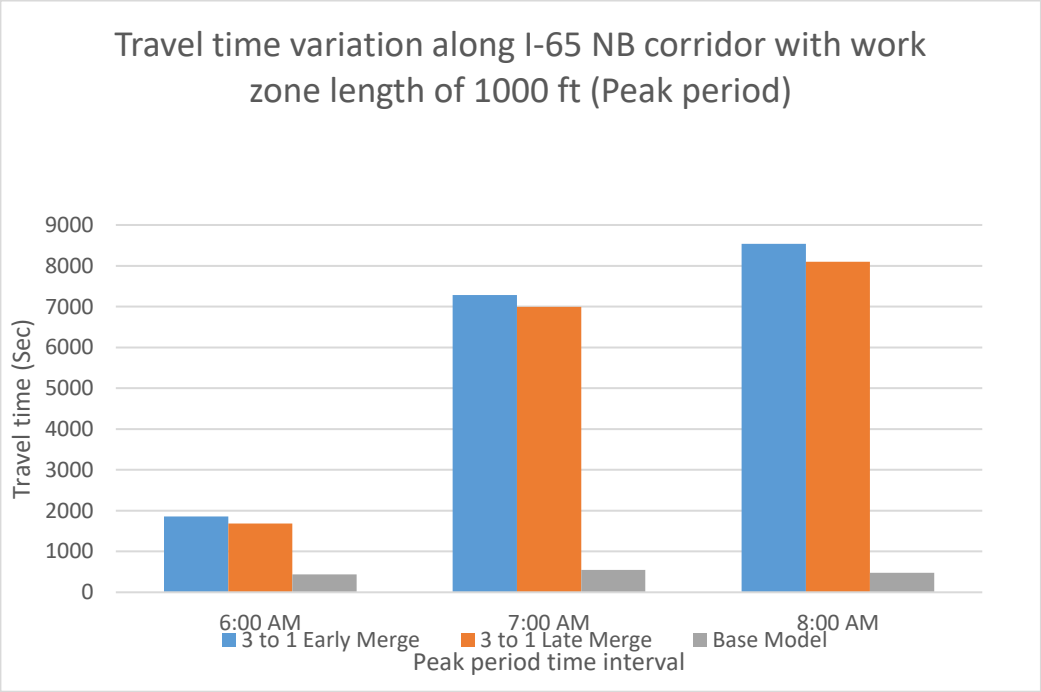


Figure 2-19: Travel time variation along I-65 NB corridor (WZ length: 1000 ft; Peak period; 3-to-1 closure)

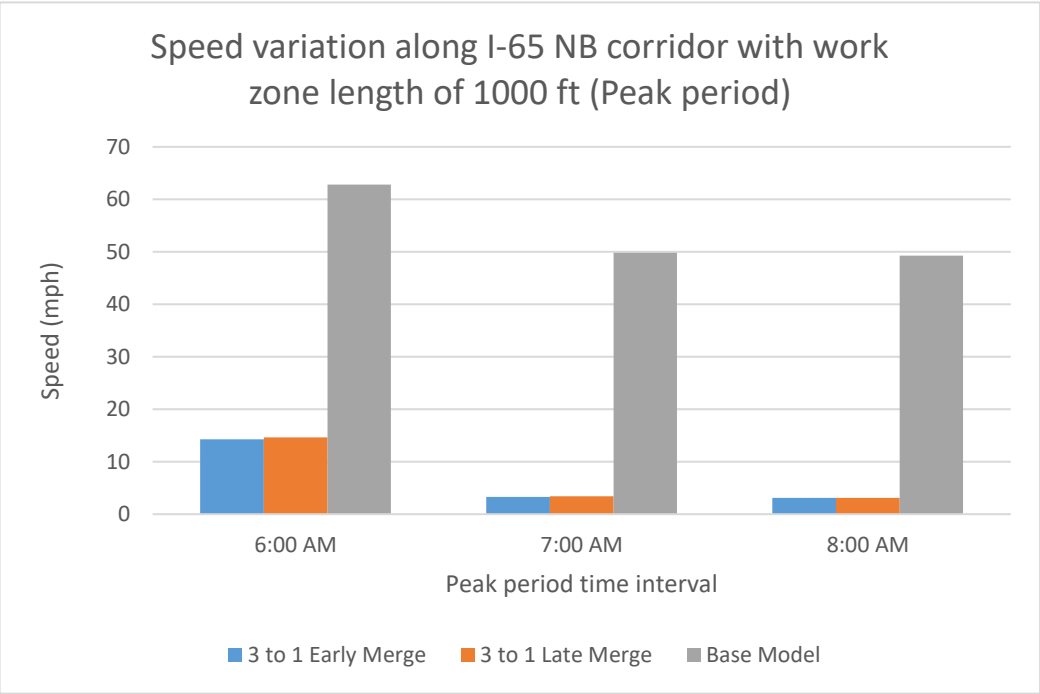


Figure 2-20: Speed variation along I-65 NB corridor (WZ length: 1000 ft; Peak period; 3-to-1 closure)

It is found from the analysis that short-term work zones with 1000 ft or longer lane closure during peak could not serve the demand at all with 3-to-1 closure with any of the traffic control strategies considered. Therefore, it is evident that neither late merge nor early merge can accommodate the 3-to-1 closure with 1000 ft length or more during peak period.

4.3.2 Short-term work zone analysis (Non-peak period) for 1000 ft work zone

The short-term 3-to-1 lane closure with 500 ft work zone during non-peak does not exhibit as severe of an impact as the peak-period. Therefore, longer work zone lengths are analyzed starting with a length of 1000 ft. Flow, density, travel time and speed for the base case and late and early merge control strategies are shown in Figures 2-21 through 2-24 below.

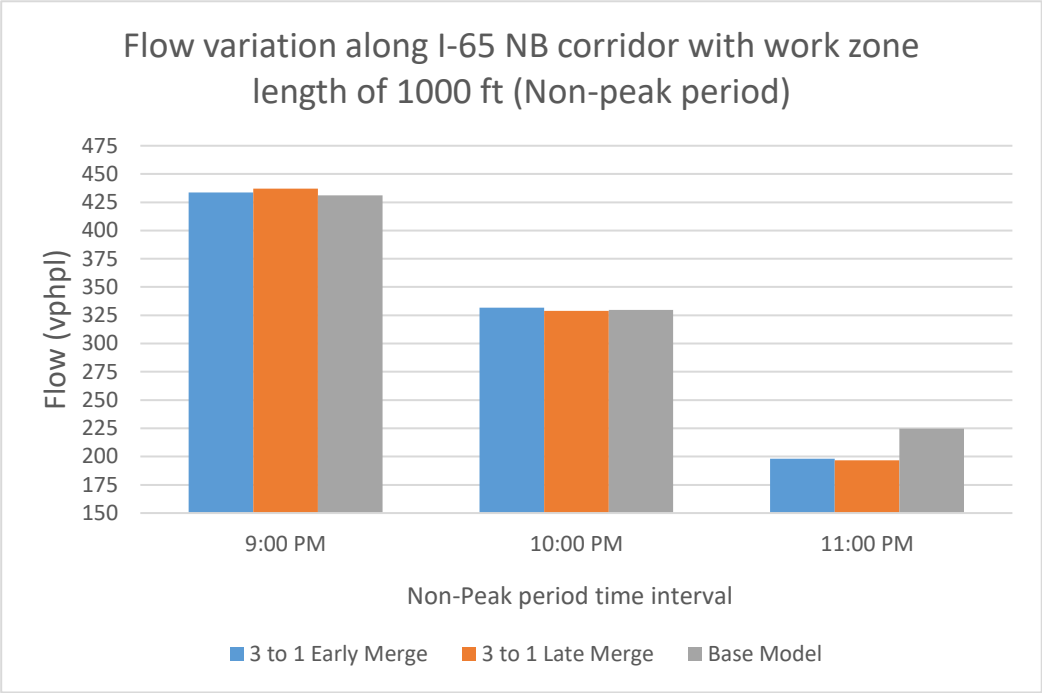


Figure 2-21: Flow variation along I-65 NB corridor (WZ length: 1000 ft; Non-peak period; 3-to-1 closure)

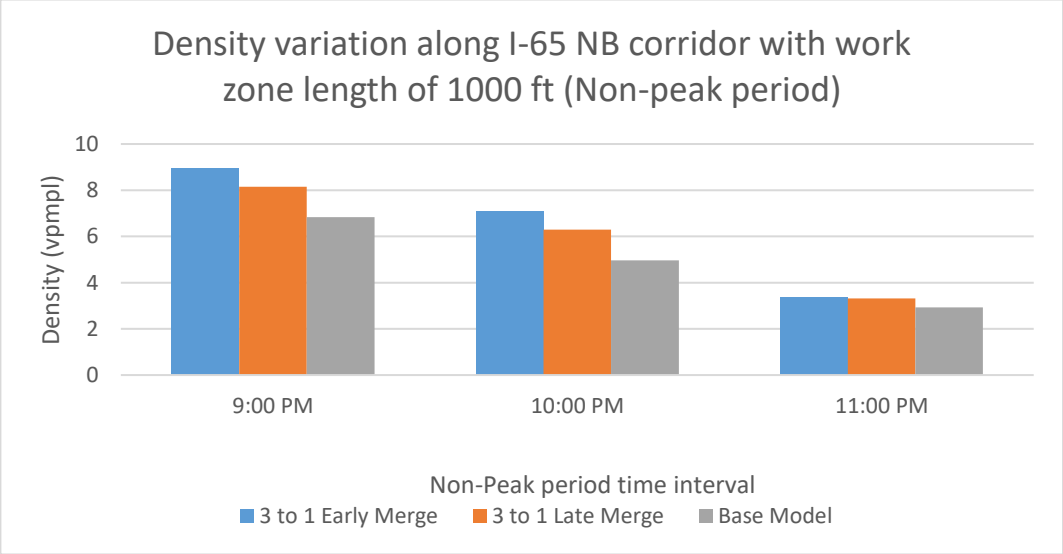


Figure 2-22: Density variation along I-65 NB corridor (WZ length: 1000 ft, Non-peak period; 3-to-1 closure)

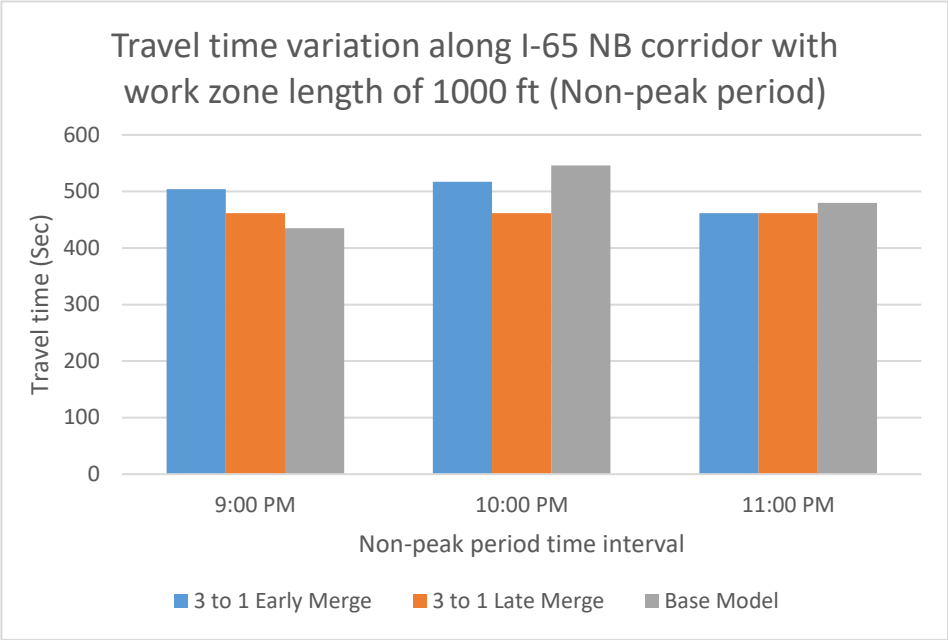


Figure 2-23: Travel time variation along I-65 NB corridor (WZ length: 1000 ft, Non-peak period; 3-to-1 closure)

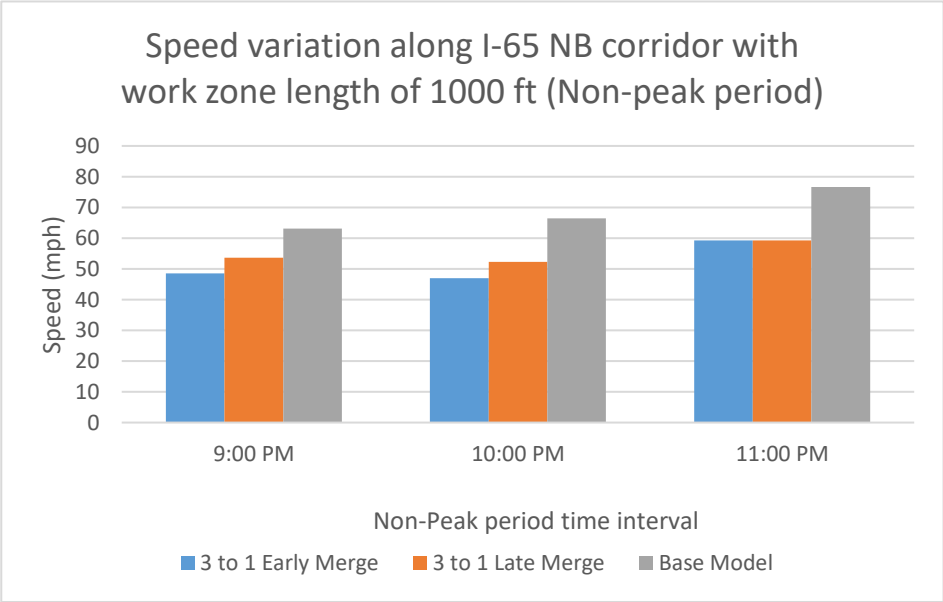


Figure 2-24: Speed variation along I-65 NB corridor (WZ length: 1000 ft, Non-peak period; 3-to-1 closure)

As shown in Figure 2-21, flow under the late and early merge when work zone length increased to 1000 ft was close to base flow during non-peak hour. This indicates that there is minimal impact of the closure on the lower volume during

night. Speed is slightly higher with late merge control at the start of the observation period but soon both TTC strategies show similar results. At some point in time, base flow has higher travel time than early or late merge. This might be attributable to the closure of a ramp (exit 254) during lane closure.

4.4 Comparison of 3-to-1 Merge Control for 1500 ft Work Zone Length

Figures 2-25 through 2-28 show a comparison of flow, density, speed and travel time between base model and late- and early merge control with a 3-to-1 lane drop and for a work zone length of 1500ft under non-peak traffic conditions.

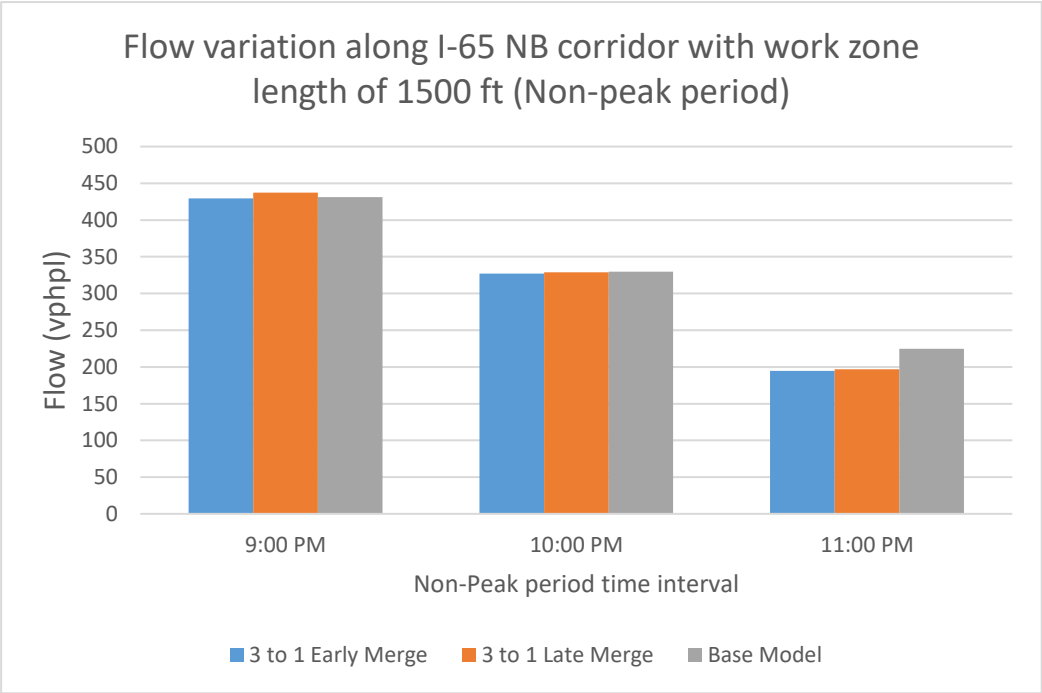


Figure 2-25: Flow variation along I-65 NB corridor (WZ length: 1500 ft; Non-peak period; 3-to-1 closure)

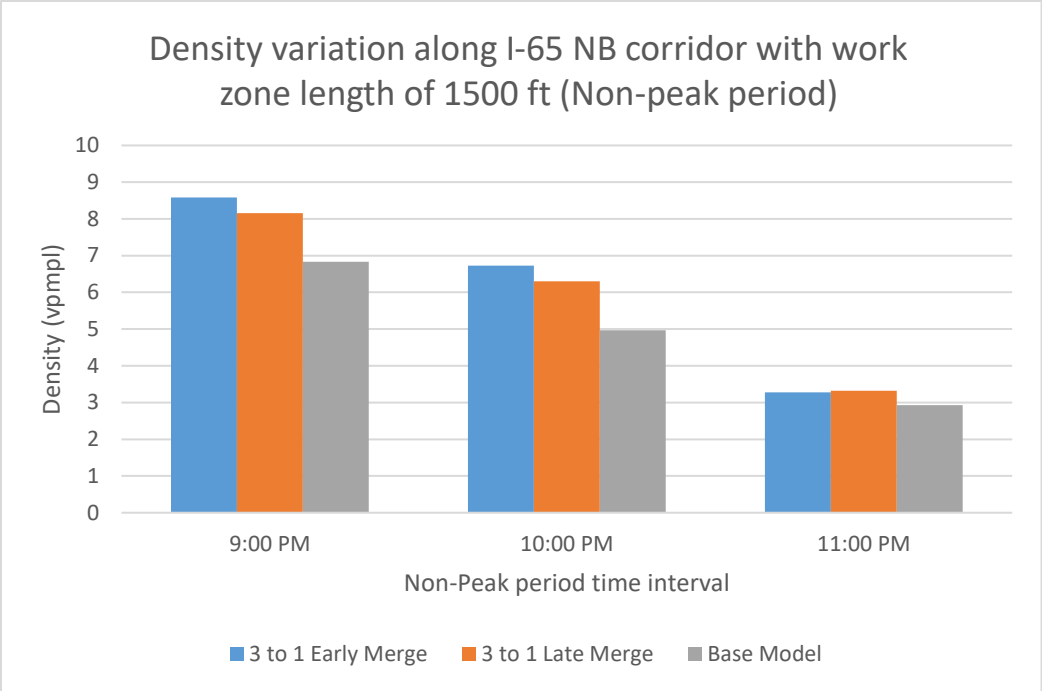


Figure 2-26: Density variation along I-65 NB corridor (WZ length: 1500 ft; Non-peak period

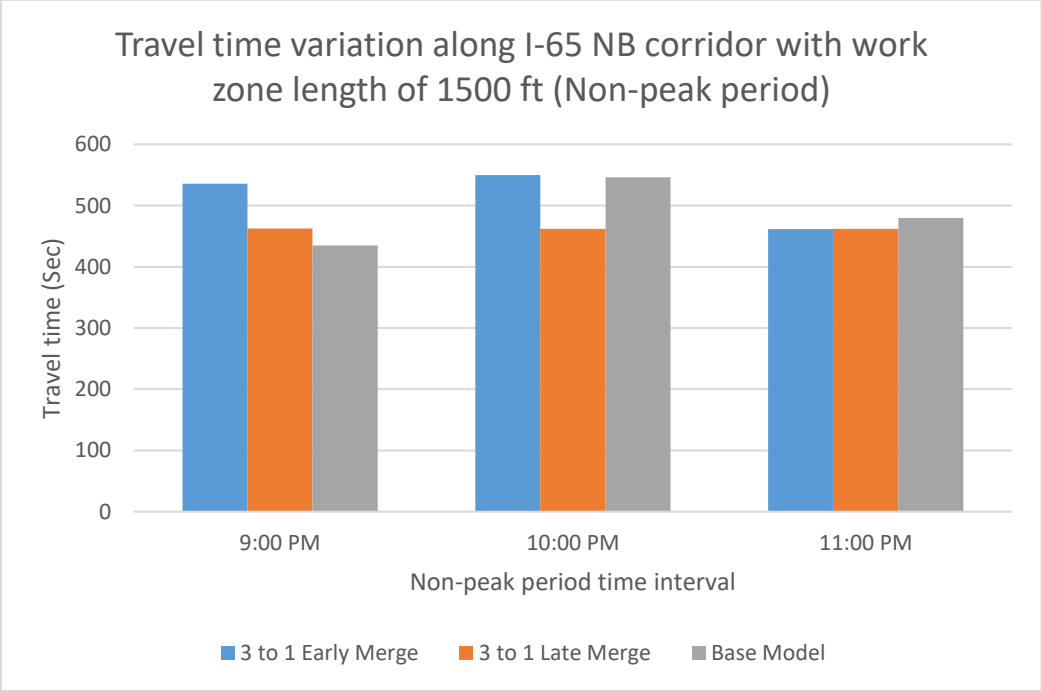


Figure 2-27: Travel time variation along I-65 NB corridor (WZ length: 1500 ft; Non-peak period; 3-to-1 closure)

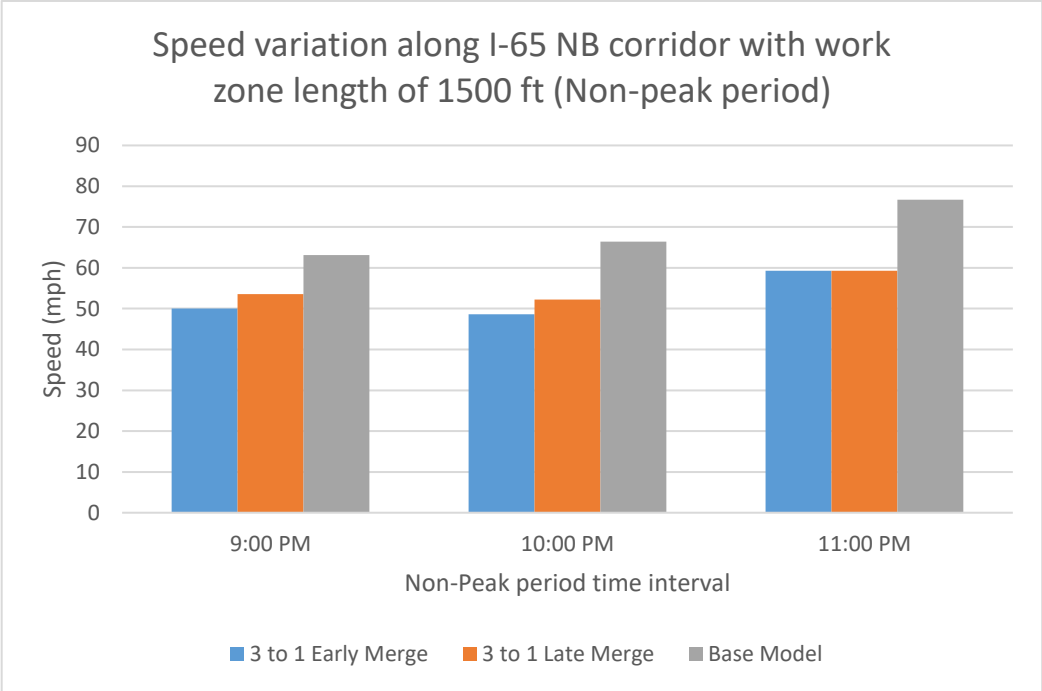


Figure 2-28: Speed variation along I-65 NB corridor (WZ length: 1500 ft; Non-peak period; 3-to-1 closure)

The performance of late merge and early merge for the 1500 ft work zone remains almost similar to their performance with 1000 ft non-peak hour closure. Due to the low demand, flow remains similar for base, early, and late merge. Early merge causes more density in the roadway, resulting in higher travel time compared to late merge. The base model has slightly higher travel time at one point, perhaps due to ramp closure and elimination of a queue from that exit point. Overall, the late merge scenario performs slightly better than early merge during the non-peak period with 3-to-1 closure for work zone length of 1500 ft, a finding that is consistent with those observed for work zones of 500 and 1000 ft as well.

4.5 Comparison of 3-to-2 Merge Control for 1000 ft Work Zone Length

Performance of the early and late merge control was studied for 3-to-2 lane closure scenario as well. For demonstration purposes, only one length (1000 feet) of work zone was considered for inclusion in this report. **Error! Reference source not found.** and **Error! Reference source not found.** show the comparison of performance for the measure of effectiveness of density and travel time. Though there is not much difference in the performance of late and early merge, longer lengths of work zones may reveal more insights.

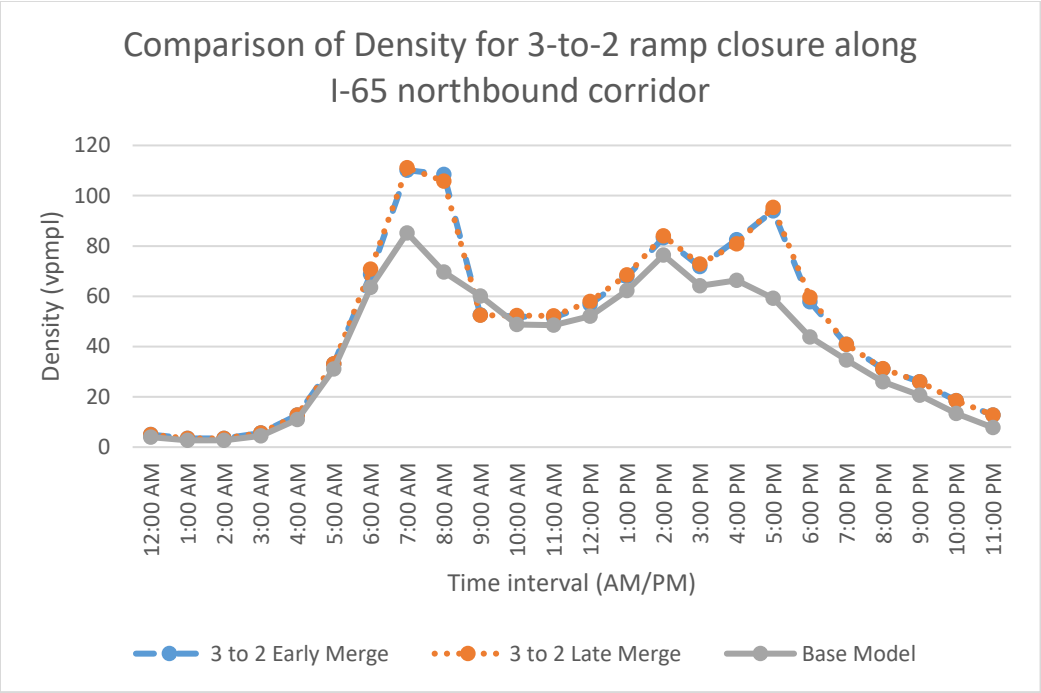


Figure 2-29: Density variation for 3-to-2 ramp closure along I-65 NB northbound corridor (WZ length: 1000 ft; 24-hr)

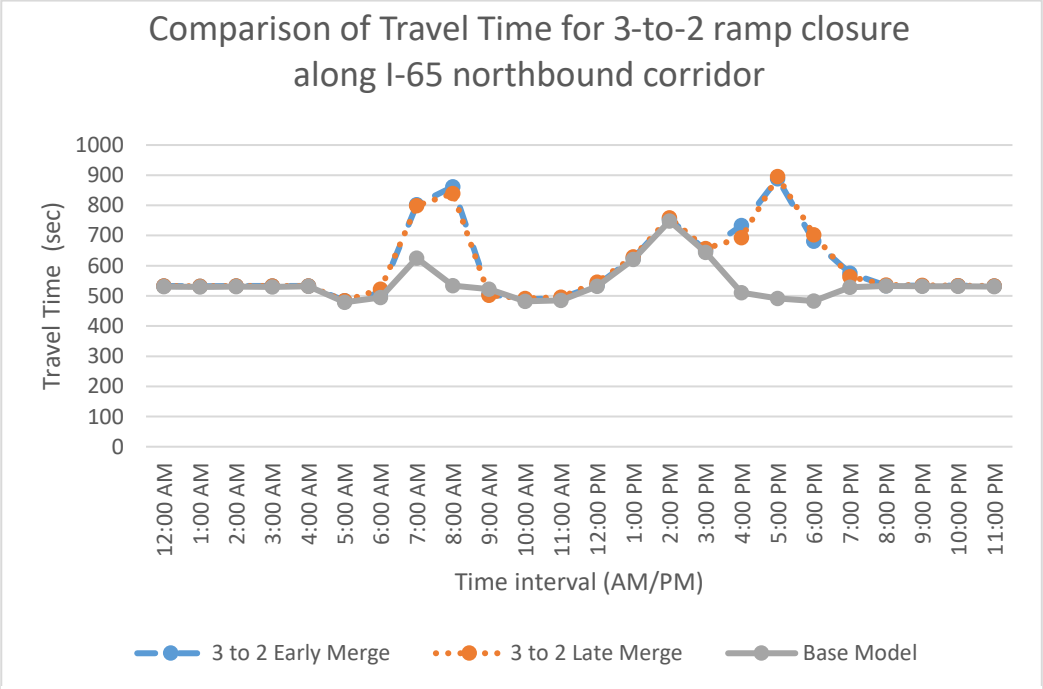


Figure 2-30: Travel time variation for 3-to-2 ramp closure along I-65 NB northbound corridor (WZ length: 1000 ft; 24-hr)

4.6 Statistical Significance Analysis

The comparisons performed so far focused on visual inspections of MOEs estimated values. Still, there is a need to study if the observed difference in the performance between early and late merge control is statistically significant. Therefore, a t-test is conducted to identify the t-score for each of the MOE for each combination. The t-score has to be more than 2.132 to ensure with 90% confidence that the MOEs are significantly different between early and late merge for two-tailed test. Two-tailed test means that one set of values can be significantly greater or smaller than another set of values.

The results from the comparison based on a two-tailed t-test and for a 90% confidence interval are shown in Table 2-8. All t-test scores are below 2.132, which confirms that for 3-to-1 lane closures, early merge and late merge strategies do not have statistically significant differences in their performance. In other words, any of these two TTC strategies will result in similar impacts in the presence of a 3-to-1 lane closure for work zone lengths varying from 500 to 1500ft under both peak and non-peak traffic conditions.

Table 2-8: T-score for statistical significance analysis for 3-to-1 lane closure

Length	Time period of day	MOEs	T-score (Two-tailed test)	Significance
500 ft	Peak	Density	0.200241444	Not significant
		Speed	0.17525763	Not significant
		Travel time	0.144133745	Not significant
		Volume	0.394332176	Not significant
	Non-Peak	Density	0.100127412	Not significant
		Speed	0.036932124	Not significant
		Travel time	0.183719004	Not significant
		Volume	0.886465803	Not significant
1000 ft	Peak	Density	0.258916873	Not significant
		Speed	0.234173895	Not significant
		Travel time	0.060329507	Not significant
		Volume	0.023461693	Not significant
	Non-Peak	Density	0.169950664	Not significant
		Speed	0.182707404	Not significant
		Travel time	0.188432782	Not significant
		Volume	0.89066417	Not significant
1500 ft	Non-Peak	Density	0.219960903	Not significant
		Speed	0.182552745	Not significant
		Travel time	0.18759073	Not significant
		Volume	0.179880748	Not significant

5.0 SUMMARY AND CONCLUSIONS

Work zones involving lane closures cause disruption on freeway operations leading in congestion. Almost one-fourth of the non-recurring delay is attributable to lane closures along roadways. Large amount of economic loss is also incurred due to congestion created by closing lanes. The literature review suggested that there are some available traffic control options, but most of the state DOTs do not have formal guidelines guiding the proper selection of traffic control strategies at work zones (Ramadan and Sisiopiku, 2015).

Earlier researches focused mostly on 3-to-2 lane closure configuration for short-term work zones. The objective of this task was to investigate the operational impacts of two temporary traffic control (TTC) strategies, namely static late and early merge control with 3-to-1 lane-drop configurations for a hypothetical work zone with various work zone lengths (500 ft, 1000 ft, 1500 ft) at a corridor along I-65 in Birmingham, AL. 3-to-2 lane closure was also investigated for a 1000ft lane closure for 24-hr. The study employed the VISSIM simulation platform for modeling the corridor and generating MOEs under different control scenarios, including flow, density, speed, and travel time.

The study considered long-term work zone placement, as well as short-term work zone placement during AM peak period and PM non-peak period. A total of 12 different combinations of TTC types, work zone lengths, and work zone types were examined. Some major findings from this study are summarized below:

- Between 12:00 PM and 5:00 AM, a time period that corresponds to very low demand, 3-to-1 lane closures are feasible under any type of configuration studied and have minimal impact on mobility.
- During the morning peak, under the 3-to-1 lane closure traffic conditions deteriorate quickly, the network gets overwhelmed by the excess demand and unable to cope. Thus long-term 3-to-1 lane closures are not recommended, unless other provisions are taken including traffic diversion to direct excess traffic volume away from the facility affected by the traffic lane closures.
- When considering short-term work zones during the morning peak, the late merge strategy slightly outperforms the early merge with 500 ft short-term work zone when volume-to-capacity ratio is still below 1. However, both TTC strategies are unable to accommodate the demand and eventually the system breaks down during the AM peak period.
- As the work zone length increases to 1000 ft, the short-term work zone during the AM peak period shows no significant difference in the performance of late and early merge control. Both strategies completely fail to serve the demand with 3-to-1 closure. Thus, it is recommended avoiding scheduling 3-to-1 lane closures of any control type during the AM peak period.

- Late merge and early merge for a 3-to-1 lane drop perform quite similarly during the non-peak period with short-term work zones of 500, 1000, and 1500 ft length. The length of the work zone appears to have minimal impact on the performance measures considered.
- While late merge control outperforms early merge control when any noticeable differences between the two strategies are observed, the differences are not statistically significant for any of the comparisons performed in this study. Thus, there is no evidence that one or the other TTC strategy studied yields better results under the 3-to-1 scenario and either may be used when demand is low, such as during non-peak times.
- For short length of work zone, 3-to-2 lane closure scenario did not show much difference in performance with respect to merge control in the Birmingham study.

6.0 RECOMMENDATIONS

6.1 Recommendations

From the results from investigation, it is recommended that long-term work zones with 3-to-1 lane closures should be avoided. Instead, short duration closures should be considered, preferably during non-peak periods in order to minimize the impact on mobility. When non-peak hour work zones are scheduled, both late merge control and early merge control strategies can be used, with late merge control showing slight advantages with respect to operational performance.

6.2 Suggestions for Future Study

This study did not consider traffic diversion during the 3-to-1 lane closures at the worksite. As a result, the traffic network quickly become oversaturated and failed to serve the demand. A sensitivity analysis is recommended to determine the percentage of traffic that needs to be diverted in order to provide an acceptable level of service to users of the facility during the 3-to-1 operation. Future study can also look into the impact of 3-to-1 closure during other time intervals and consider the impacts of placement of the lane closure(s) on the left side, rather than the right side of the roadway.

Additionally, there are various merge control strategies that focuses on dynamic features. Evaluating the impact of the dynamic merge control for various lane closure scenarios both for peak and off-peak can be a valuable future contribution. Future study can also investigate performance of various traffic control strategies for lane closure on weekends. Finally, the study can be extended to document results from a sensitivity analysis considering impacts of varying heavy vehicle percentages, driver behaviors, and

traffic demand changes on study MOEs. By considering a variety of driver behaviors and traffic conditions future studies can provide results that are easily transferable to other freeway segments with different characteristics.

7.0 PART 2 REFERENCE LIST

1. Wunderlich, K. and D. Hardesty, *A Snapshot of Work Zone Activity Reported on State Road Closure and Construction Websites.* Mitretek Systems, Falls Church, Va, 2002.

2. Wolff, G. *Traffic Control Safety: Meeting the FHWA Minimums Is Not Enough*. in *ASSE Professional Development Conference*. 2007. American Society of Safety Engineers.

3. Wei, H., et al., *Synthesis of unconventional dynamic merge metering traffic control for work zones.* Open Transportation Journal, 2010. **4**: p. 52-60.

4. Jiang, Y., *Traffic capacity, speed, and queue-discharge rate of Indiana's four-lane freeway work zones.* Transportation Research Record, 1999. **1657**(1): p. 10-17.

5. Chitturi, M.V., R.F. Benekohal, and A.-Z. Kaja-Mohideen, *Methodology for computing delay and user costs in work zones.* Transportation Research Record, 2008(2055): p. 31-38.

6. Mallela, J. and S. Sadavisam, *Work Zone Road User Costs: Concepts and Applications*. 2011: US Department of Transportation, Federal Highway Administration.

7. McCoy, P.T. and G. Pesti, *Dynamic Late Merge–Control Concept for Work Zones on Rural Interstate Highways.* Transportation Research Record, 2001. **1745**(1): p. 20-26.

8. Chitturi, M. and R. Benekohal, *Work zone queue length and delay methodology.* Transportation Letters, 2010. **2**(4): p. 273-283.

9. Tang, Y. and S. Chien, *Scheduling work zones for highway maintenance projects: Considering a discrète time-cost relation.* Transportation Research Record: Journal of the Transportation Research Board, 2008(2055): p. 21-30.

10. Kim, T., D.J. Lovell, and J. Paracha. *A new methodology to estimate capacity for freeway work zones*. in *80th Annual Meeting of the Transportation Research Board, Washington, DC*. 2001.

11. Sisiopiku, V.P. and O.E. Ramadan, *Evaluation of Traffic Control Options in Work Zones*. 2017, Southeastern Transportation Research, Innovation, Development and Education ….

12. *Manual on Uniform Traffic Control Devices for Streets and Highways*, U.S.D.o. Transportation, Editor. 2009.

13. Benekohal, R.F., A.-Z. Kaja-Mohideen, and M.V. Chitturi. *Methodology for estimating operating speed and capacity in work zones*. 2004. National Research Council.

14. Pesti, G., et al., *Traffic control strategies for congested freeways and work zones*. 2008, Texas Transportation Institute.

15. McCoy, P.T., G. Pesti, and P.S. Byrd, *Alternative Driver Information to Alleviate Work-Zone-Related Delays*. 1999.

16. *Work Zone Operations Best Practices Guidebook (Third Edition)*, U.S.D.o. Transportation, Editor. 2013: Washington, D.C.

17. Nemeth, Z.A. and N.M. Rouphail, *Lane closures at freeway work zones: Simulation study*. 1982.

18. Mousa, R.M., N.M. Rouphail, and F. Azadivar, *Integrating microscopic simulation and optimization: Application to freeway work zone traffic control.* Transportation Research Record, 1990. **1254**: p. 14-25.

19. Datta, T.K., et al., *Development and evaluation of an advanced dynamic lane merge traffic control system for 3 to 2 lane transition areas in work zones*. 2004.

20. Beacher, A.G., M.D. Fontaine, and N.J. Garber, *Evaluation of the late merge work zone traffic control strategy*. 2004, Virginia Transportation Research Council.

21. Tarko, A.P., S.R. Kanipakapatnam, and J.S. Wasson, *Modeling and optimization of the Indiana Lane merge control system on approaches to freeway work zones*. 1998: Purdue University [Joint Transportation Research Program].

22. Pesti, G., et al., *Traffic flow characteristics of the late merge work zone control strategy.* Transportation Research Record, 1999. **1657**(1): p. 1-9.

23. Walters, C.H., et al., *Understanding road rage: Summary of first-year project activities*. 2000.

24. Idewu, W.I.A. and B. Wolshon, *Joint merge and its impact on merging speeds in lane reduction areas of construction zone.* Transportation Research Record, 2010(2169): p. 31-39.

25. Lentzakis, A.F., et al. *Real-time work zone management for throughput maximization*. in *Transportation Research Board 87th Annual Meeting*. 2008. Citeseer.

26. Tympakianaki, A., et al., *Real-time merging traffic control for throughput maximization at motorway work zones.* Transportation Research Part C: Emerging Technologies, 2014. **44**: p. 242-252.

27. Oner, E., *An Evaluation of Entrance Ramp Metering for Freeway Work Zones using Digital Simulation*. 2009, Ohio University.

28. Sun, C., P. Edara, and Z. Zhu, *Evaluation of temporary ramp metering for work zones.* Transportation Research Record: Journal of the Transportation Research Board, 2013(2337): p. 17-24.

29. Ramadan, O.E. and V.P. Sisiopiku, *Bottleneck merge control strategies for work zones: Available options and current practices.* Open Journal of Civil Engineering, 2015. **5**(04): p. 428.

30. Chatterjee, I., et al., *Replication of work zone capacity values in a simulation model.* Transportation Research Record, 2009(2130): p. 138-148.

31. Elghamrawy, T.M., *Optimizing work zone practices for highway construction projects*. 2011, University of Illinois at Urbana-Champaign.

32. Scriba, T., P. Sankar, and K. Jeannotte, *Implementing the Rule on Work Zone Safety and Mobility*. 2005.

33. Beacher, A.G., M.D. Fontaine, and N.J. Garber. *Field evaluation of late merge traffic control in work zones*. 2005. National Research Council.

34. Kurker, M., et al., *Minimizing User Delay and Crash Potential through Highway Work Zone Planning*. 2014.

35. Harb, R., E. Radwan, and V.V. Dixit, *Comparing Three Lane Merging Schemes for Short-Term Work Zones: A Simulation Study.* ISRN Civil Engineering, 2012. **2012**.

36. Harb, R., *Safety and Operational Evaluation of Dynamic Lane Merging in Work Zones.* 2009.

37. Vaughan, C., et al., *The Effects of Late Lane Merges on Travel Times*, N.C.D.o. Transportation, Editor. 2018: North Carolina.

38. Lammers, E., et al., *Applicability of Zipper Merge Versus Early Merge in Kentucky Work Zones*. 2017, University of Kentucky Transportation Center.

39. Kang, K.-P., G.-L. Chang, and J. Paracha, *Dynamic late merge control at highway work zones: evaluations, observations, and suggestions.* Transportation Research Record, 2006. **1948**(1): p. 86-95.

40. Finley, M.D., G.L. Ullman, and C.L. Dudek, *Sequential warning-light system for work-zone lane closures.* Transportation Research Record, 2001(1745): p. 39-45.

41. Grillo, L.F., T.K. Datta, and C. Hartner, *Dynamic late lane merge system at freeway construction work zones.* Transportation Research Record, 2008(2055): p. 3-10.

42. *VISSIM User Manual*. 2011.

43. Choa, F., R.T. Milam, and D. Stanek. *Corsim, paramics, and vissim: What the manuals never told you*. in *Ninth TRB Conference on the Application of Transportation Planning MethodsTransportation Research BoardLouisiana Transportation Research CenterLouisiana Department of Transportation and DevelopmentLouisiana Planning Council*. 2004.

44. *Advanced CORSIM Training Manual*, M.D.o. Transportation, Editor. 2008.

45. Bloomberg, L. and J. Dale, *Comparison of VISSIM and CORSIM traffic simulation models on a congested network.* Transportation Research Record, 2000. **1727**(1): p. 52-60.

46. Hardy, M. and K. Wunderlich, *Traffic analysis tools volume IX: work zone modeling and simulation: a guide for analysts.* 2009.

PART 3 USE OF VIDEO IMAGING AND ARTIFICIAL INTELLIGENCE TO QUANTIFY DRIVER BEHAVIOR IN FREEWAY WORK ZONES

The purpose of this part of the research was to develop information on driver merging behavior in advance of a freeway lane closure using video image processing. The study sites also allowed for determination of how that behavior varied between cars and heavy trucks, as well as between curved and straight approaches. The video log images collected for the preliminary driver behavior study on I-95 near Savannah, Georgia, demonstrated the feasibility of using commonly available images of work zones to extract and study driver behaviors (merging timing/locations) in varying roadway geometries (straight and curved sections). On a curved road, cars and trucks tend to merge closer to a work zone taper than they do when traveling on straight roads. On a curved road, cars are more likely to merge closer to the work zone taper than trucks. This is due to the available sight distance of a work zone taper. This suggests that the traffic control devices should be placed at a sufficient sight distances that drivers, especially on curved road, can have sufficient time to react to roadway conditions safely. Additional data and analysis are still needed to confirm the observations in this study. This study also recommends the use of images acquired from drones and traffic monitoring cameras and the use of artificial intelligence to automatically extract driver behaviors and traffic characteristics, data that is currently lacking. This will significantly enhance the development of reliable work zone traffic simulation models.

1.0 INTRODUCTION

This document is a part of the research project entitled "Improving Work Zone Mobility through Planning, Design, and Operations." It is sponsored by the Southeastern Transportation Research, Innovation, Development, and Education (STRIDE) and focuses on the task of "studying the impact of differing traffic conditions and roadway geometries on driving behaviors using existing data and video from a freeway work zone project." In support of developing reliable work zone traffic simulation models, which are very much needed, this research project recommends the use of widely available images to automatically and semi-automatically extract driver behaviors and traffic characteristics in work zones. The project's objectives and sub-tasks, and the organization of this report are presented below.

This project is directly related to the STRIDE theme of "reducing congestion." It has been estimated that 10 to 25% of traffic congestion and traveler delay is due to congestion in work zones. As travel on the highway system increases, so does the need for system maintenance and repair, which causes increased congestion. Therefore, it is important to understand driver behaviors and traffic characteristics in maintenance work zones to reduce congestion. This can be done through the use of video log images, which are commonly available and can be used to

study work zone characteristics a0nd driver behaviors so that essential data (e.g., merging timing/location, speed, etc.) can be gathered to support and refine the work zone traffic simulation models. Overall, this study is aimed at reducing work zone congestion and improving highway and driver safety.

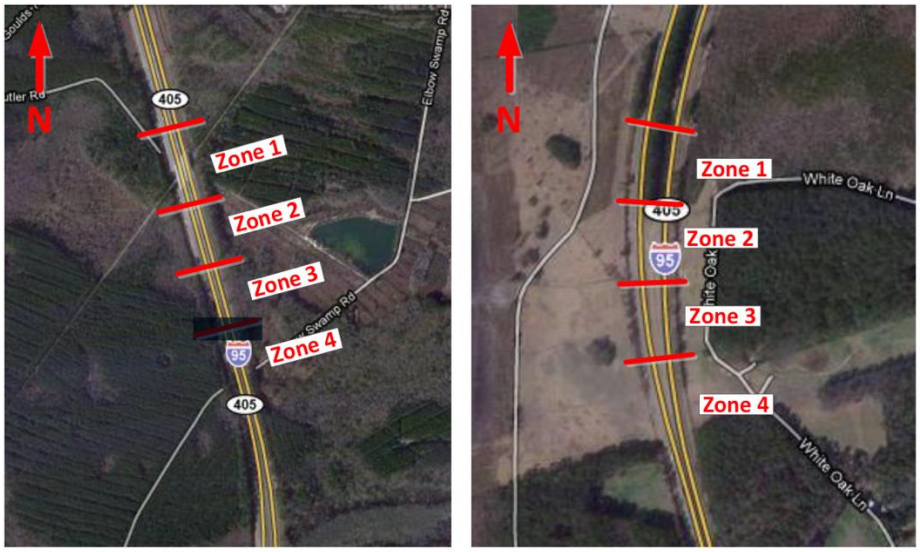
The objectives of this study's task are to study the "impact of differing traffic conditions and roadway geometries on driving behaviors using existing data and video from a freeway work zone project" and determine the feasibility of using video log images to obtain driver behaviors in work zone areas, especially near merging points. The sub-tasks conducted in this report include 1) acquiring, preparing, and processing existing video log images from a freeway work zone project, and 2) studying vehicle driving behaviors and different roadway geometries within work zone areas for the purpose of recommending refinements in the spatial and temporal characteristics of work zones.

Various traffic characteristics and driver behavior parameters in work zones can be estimated and extracted using different sensor technologies, such as radar, lidar, and vision-based ones. Through the different types of vehicle detection and tracking methods, traffic characteristics have been extracted, such as flow (Coifman et al., 1998; Bas et al., 2007; Ching-Po et al., 2003), velocity (Wu et al., 2006; Magee & Derek, 2004), density (Artimy et al., 2005; Kerner & Klenov, 2004), vehicle count (Kim & Malik, 2003; Rad & Mansour, 2005), and vehicle classification (Ha, D. M. et al., 2004; Gupte et al., 2002; Morris et al, 2008). In addition, various sensor technologies are recommended for studying driver behaviors, such as merge timings/location in work zones (Tsai et. al., 2011; Tsai, 2011; Hallmark et. al., 2011; Weng & Meng, 2011; Vaughan et. al., 2018).

This report is organized as follows. The objectives and tasks are first presented in this section. The second section presents the video log images collected from a work zone on I-95. The data processing is briefly introduced. The third section presents the work zone driver behavior section. Finally, the fourth section presents conclusions and recommendations.

2.0 Video Log Image Data Collection on I-95 Work Zone

This section presents the video log images collected on an I-95 work zone project near Savannah, Georgia, to support a vision-based driver behavior study. Two roadway sections with different upstream geometry (curved and straight sections preceding a work zone) along Interstate-95 near Savannah, Georgia, were used for this demonstration. The location selected was south of Savannah. Figure 3-1 shows the two roadway sections. Two cameras mounted on a 30-foot tower were used to collect the videos; the tower was placed on the highway shoulder, and the cameras faced away from the work zone towards the upstream traffic. Figure 3-2 shows the 30-foot tower, which consists of the sensing system that houses multiple cameras.



(a) Four zones on the straight road (b) Four zones on the curved road

Figure 3-1: Upstream geometry and the demarcated zones (Tsai, et al. 2011)

For the straight road section, two different camera views were used to observe Zones 1 and 2 (the sections closest to the camera) and Zones 3 and 4 (the sections furthest from the camera); for the curved section, a single camera was able to capture all 4 zones.

Figure 3-2: Camera mounted on the tower (Tsai, Y., et al. 2011)

The videos were collected at noon for outer lane closure in both the cases. The videos were processed for 30 minutes in this case study; the count of the different classes of vehicles in this time period is shown in Figure 3-3.

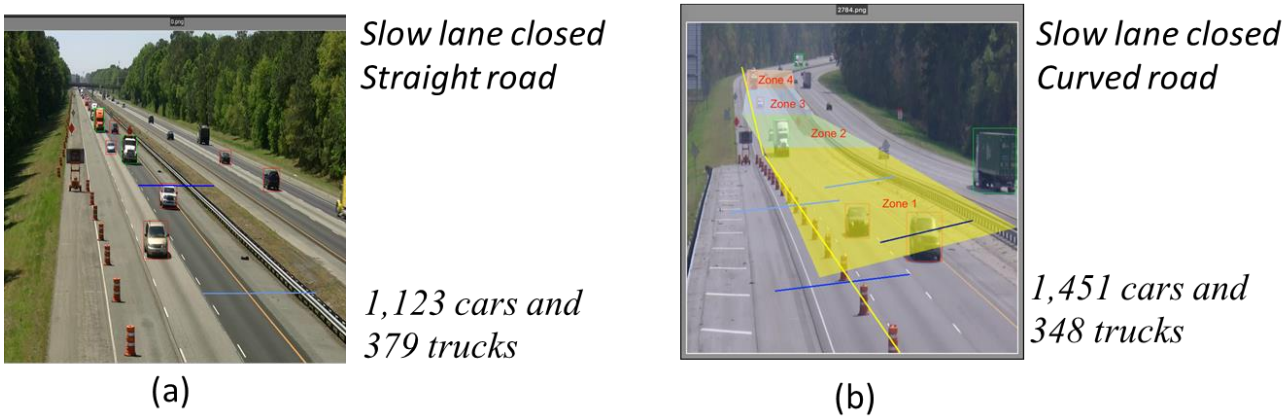


Figure 3-3: Count-data from the developed tool for the two sections, (a) straight section and (b) curved section

Four zones were defined to study driver behavior (merge location and average headway space) based on placement of advanced warning sign locations as shown in Figure 3-4.

Zone 1: Covers the transition area

Zone 2: 500 ft. upstream from the beginning of transition area

Zone 3: 500 ft. to 1000 ft. upstream from the beginning of transition area

Zone 4: 1000 ft. and beyond upstream from the beginning of transition area

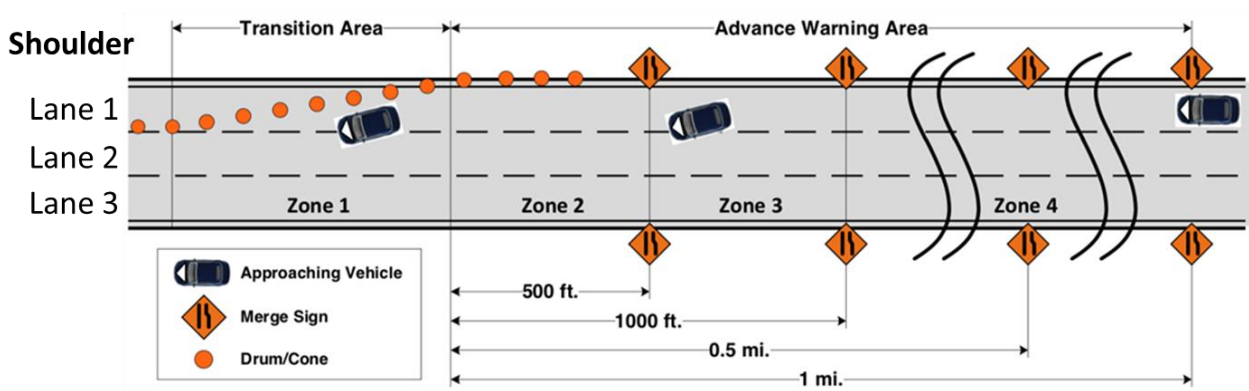


Figure 3-4: Zone demarcation driver behavior analysis

A tool was designed to manually extract the driver behavior using video log images. This tool allows the researchers to set the zone boundaries and the reference lines to count the vehicles. The primary output from the tool is the lane-based count of classified vehicles and their corresponding frame numbers when the count increments. Each lane consisted of two user-defined reference lines (called the flow lanes) of known distances to allow measurement of average speed. An illustration of the expected lane-wise output is shown in Table 3-1. Whenever a vehicle crosses the flow line, the count is incremented by 1, and the corresponding frame number is recorded. We can see from the table that the first car crosses Lane 1, Flow line 1 in Frame # 2, and same car crosses Lane 1, Flow line 2 in Frame #4.

Table 3-1: Lane-wise vehicle count and class

	Lane 1, Flow Line 1		Lane 1, Flow Line 2		Lane 2, Flow Line 1		Lane 2, Flow Line 2	
Frame #	Cars	Trucks	Cars	Trucks	Cars	Trucks	Cars	Trucks
0	0	0	0	0	0	0	0	0
2	1	0	0	0	0	0	1	0
4	1	0	1	0	1	0	1	0
7	2	0	1	0	1	0	1	0
8	2	0	1	0	2	0	1	0

Similarly, we also extracted the zone-based counts to monitor the merge behavior of each class of vehicles in each zone. Each zone consists of a merge line passing through the pavement marking separating the closed lane (Lane 1 in Figure 3-4) and the neighboring open lane (Lane 2 in Figure 3-4). When any vehicle crosses this merge line from a closed lane to an open lane, the corresponding zone number and vehicle class is incremented. Table 3-2 shows an illustration of the expected zone-wise output. The merge behavior can be inferred from these tables as follows. In frame #47, the first truck to change lanes from the closed lane to the open lane in Zone 2 is detected. Similarly, the first car merges lanes in Frame #57 in Zone 2, and the second truck merges lanes in Zone 3 in Frame #67. When the next car merges lanes in Zone 2, the count will be incremented to 2 in the corresponding frame. The total number of vehicles that merged lanes at the end of 67 frames is 3. From this table, we can infer when and where each class of vehicle changes lanes for merging.

Table 3-2: Zone-wise vehicle merge count and class

	Zone 1		Zone 2		Zone 3		Zone 4	
Frame #	Cars	Trucks	Cars	Trucks	Cars	Trucks	Cars	Trucks
47	0	0	0	0	0	1	0	0
57	0	0	1	0	0	1	0	0
67	0	0	1	0	0	1	0	1

3.0 Work Zone Driver Behavior Study

The merge frequencies in each of the 4 zones were analyzed during a 30-minute period. Figure 3-5 shows the results of the merge frequencies in each zone for both a straight and curved road. Zone 1 represents the area closest to the work zone; Zone 4 represents the area furthest from the work zone. In Figure 3-5(a), the straight road is analyzed where 10% of the vehicles merged in Zone 1, 3% of the vehicles merged in Zone 2, 16% of the vehicles merged in Zone 3, and 71% of the vehicles merged in Zone 4. Results for the curved road are shown in Figure 3-5(b) where 10% of the vehicles merged in Zone 1, 22% of the vehicles merged in Zone 2, 35% of the vehicles merged in Zone 3, and 33% of the vehicles merged in Zone 4.

(b) Straight road (a) Curved road

Figure 3-5: Merge frequency for all vehicles at different zones under different roadway geometry

On a straight road, 87% of vehicles merge early in Zones 3 and 4. Merging from such a distance may be associated with a longer sight distance, so the drivers can see what is ahead. On a curved road, 68% of the vehicles merge early in Zones 3 and 4. When comparing the percentages of vehicles merging in Zone 2, it can be seen that the curved road has a larger percentage. When on the curved road, the sight distance is shorter, so the drivers of the

vehicles may have less time to react to the oncoming end of the lane and, by default, the drivers of the vehicles will merge as soon as they can, thus merging within Zone 2.

It is interesting to see that regardless of the geometry of the road, 10% of the vehicles merged on both the straight and curved roadways. In both cases, reckless drivers may be present, regardless of how much sight distance is provided. For these vehicles, further study would need to be conducted to determine their individual driving behaviors and characteristics, such as speed and the average distance from the beginning of the fully closed off work zone in Zone 1 in which they merged.

The results of different merge frequencies on different road geometries and with different vehicle types are shown in Figure 3-6. When comparing cars to trucks on a straight road, it is interesting to see that 88% of cars and 86% of trucks all merge within Zones 3 and 4. It was previously expected that the cars would primarily merge within Zones 2 and 3, but we can see that this is not the case. When there is enough sight distance and the flow conditions relate to what was exhibited within this study, it seems that vehicles prefer to merge out of the work zone earlier.

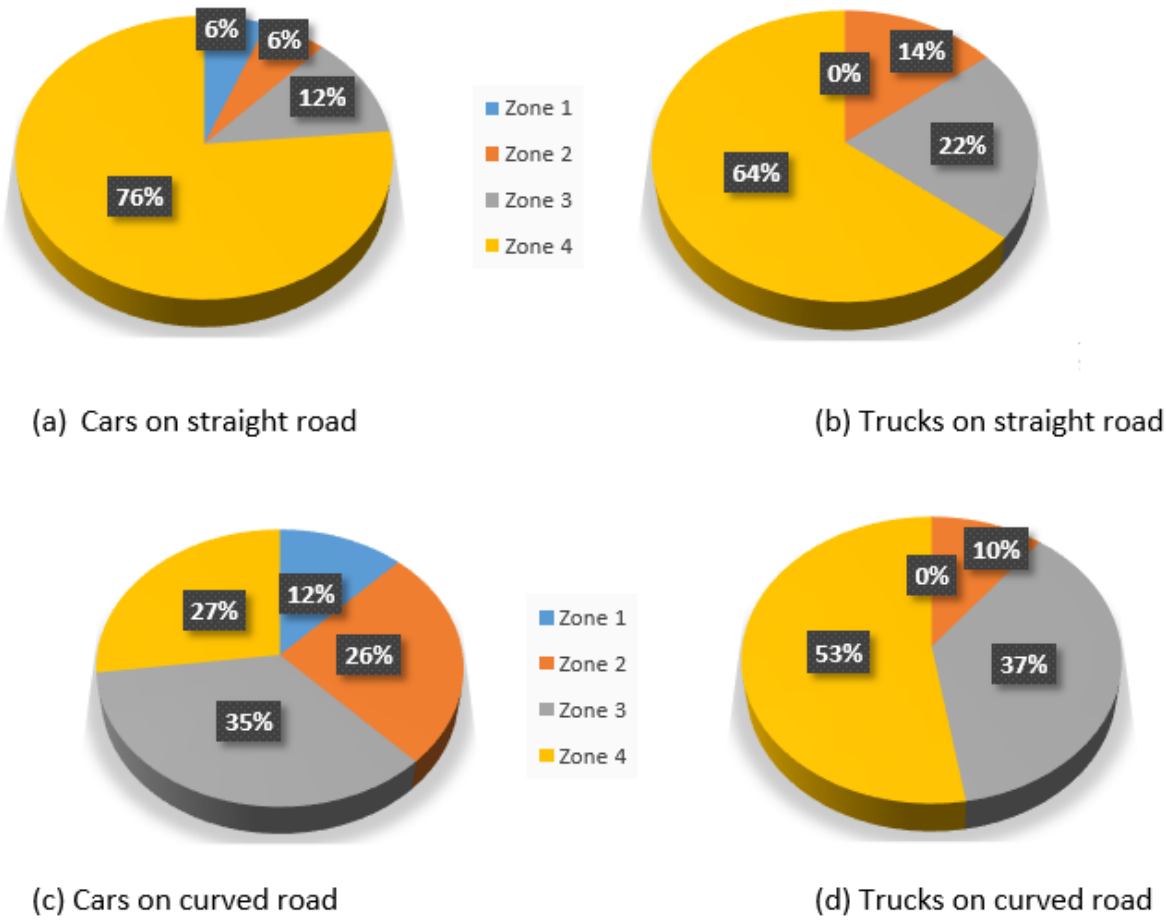


Figure 3-6: Merge timing/location based on different vehicle types

Continuing to observe the straight section (Figures 3-6 a and b), it is interesting to see that within Zone 4, cars merged at a 76% rate while trucks merged at a 64%. There may be two possible reasons that explain this. With the height advantage that the trucks have and because the road is straight, the trucks could have seen the construction zone further away from Zone 4, so the trucks could already be out of the closed lane when they drive into Zone 4. It can also be reasoned that truck drivers usually drive relatively slower than cars, and the driver's field of vision is wider, so the truck driver may notice caution signs that a work zone is ahead before a car driver of can.

When comparing a car on a straight road to a car on a curved road (Figure 3-6 a, and c), it is clear that a 38% of drivers tend to merge in Zones 1 and 2 on a curved road but only 12% merge in Zones 1 and 2 on a straight road . This difference may be related to drivers on a curved road might not perceive that a work-zone is nearing because the sight distance is less

than on a straight road. Also, the flow may be higher on the curved road, so cars may have had to delay merging in order to merge safely into other traffic.

Regarding trucks on a straight road versus a curved road (Figure 3-6 b, and d), it is reasonable to see that in both cases, 0% of drivers merged within Zone 1. As far as merging within Zones 3 and 4 on a straight road (86%) and on a curved road (90%), these similar percentages are expected again due to the sight distances of the trucks.

When comparing cars to trucks on a curved road (Figure 3-6 c, and d), it is interesting to see that 88% of cars and 100% of trucks merge prior to Zone 1. The reason so many cars merge prior to Zone 1 may be due to car drivers noticing that 100% of the trucks are merging out of the work zone lane. After the drivers of the cars notice this, they may try to look further ahead to see that the lane may close, or they just may follow the trend of merging out of the soon-to-be closed lane.

4.0 CONCLUSIONS AND RECOMMENDATIONS

The preliminary work zone driver behavior study using the video log images taken on I-95 near Savannah, Georgia, demonstrated the feasibility of using commonly available video log images to extract driver behaviors (merging timing/locations) to study drivers' work zone driving behaviors as responses to varying roadway geometries (straight line and curve sections). The driver behaviors, including those exhibited in merge areas was analyzed in this study using video log image data. We compared the merge behavior on a straight road and a curved road containing a work zone. The following are the findings from the analyses:

- On a straight road, cars and trucks tend to merge in Zones 3 and 4, which are furthest away from the work zone taper. On a curved road, cars are more likely to merge closer to the work zone taper than trucks. This is due the available sight distance to work zone taper. This observation suggests that the traffic control devices should be placed at sufficient sight distances to give drivers, especially on curves sections, the space they need to make safe decisions. Certainly, more study is needed to confirm this behavior.
- With the height advantage that the trucks have over cars, truck drivers could have seen the construction zone further away than car drivers, so the trucks could already be out of the work zone when they drive into Zone 4. Along with this height advantage, truck drivers also tend to drive slower than cars. By driving slower, the driver's field of vision is wider, so the driver may better notice caution signs that warn a work zone is ahead before a car driver can.

The following recommendations are made:

- Additional video image data is required to confirm the findings on work zone driver behavior, although this study has demonstrated that it is feasible to use video log images to extract work zone driver behavior.
- A comprehensive set of research questions and hypotheses involving the merge behavior needs to be further studied. Such questions may be with what the impact of roadway configuration on the queue length is, what the acceptable headway space for different classes of vehicles before merging is, what the impact of the open lane's vehicle density on the merge behavior is, etc.
- It is recommended that a field validation be performed using speed radars to ensure the extracted speed in the case study was realistic.
- It is recommended that the image data collection quality be improved. In this study, the camera's location configuration was not good enough to extract a 100% accurate count and complete headway space information. In addition, camera shake was a problem that needs to be controlled and improved.
- Using drones having a better view of the highway situation for automatic data processing to capture work zone driver behavior is recommended. Also, leveraging camera images already widely available to transportation agencies for extracting work zone driver behavior is recommended. Using artificial intelligence, like deep learning, is recommended to improve data-extraction efficiency. It is also recommended for performing preprocessing and post-processing of before and after deep learning.

5.0 PART 3 REFERENCE LIST

1. Artimy, Maen M., Robertson, William, and Phillips, William J., 2005. Assignment of dynamic transmission range based on estimation of vehicle density. In Proceedings of the 2nd ACM international workshop on Vehicular ad hoc networks (VANET '05). ACM, New York, NY, USA, 40-48. DOI=http://dx.doi.org/10.1145/1080754.1080761

2. Bas, E., Tekalp, A. M., Salman, F. S., "Automatic Vehicle Counting from Video for Traffic Flow Analysis," 2007 IEEE Intelligent Vehicles Symposium, Istanbul, 2007, pp. 392-397. doi: 10.1109/IVS.2007.4290146

3. Ching-Po, Lin, Jen-Chao, Tai, and Kai-Tai, Song, "Traffic monitoring based on real-time image tracking," 2003 IEEE International Conference on Robotics and

Automation (Cat. No.03CH37422), Taipei, Taiwan, 2003, pp. 2091-2096 vol.2. doi: 10.1109/ROBOT.2003.1241902

4. Coifman, B., Beymer, D., McLauchlan, P., Malik, J. (1998), A real-time computer vision system for vehicle tracking and traffic surveillance, Transportation Research Part C: Emerging Technologies, Volume 6, Issue 4, 1998, Pages 271-288.

5. Gupte, S., Masoud, O., Papanikolopoulos, P., "Vision-based vehicle classification," ITSC2000. 2000 IEEE Intelligent Transportation Systems. Proceedings (Cat. No.00TH8493), Dearborn, MI, USA, 2000, pp. 46-51. doi: 10.1109/ITSC.2000.881016

6. Ha, D.M., Lee, J.M., Kim, Y.D., Neural-edge-based vehicle detection and traffic parameter extraction, Image and Vision Computing, Volume 22, Issue 11, 2004, Pages 899-907, ISSN 0262-8856

7. Hallmark, S., et al., “Behavior Study of Merge Practices for Drivers at Work Zone Closures,” InTrans Project 09-359, 2011, pp. 1-62.

8. Kerner, Boris S., Klenov, Sergey L., “Spatial–Temporal Patterns in Heterogeneous Traffic Flow with a Variety of Driver Behavioural Characteristics and Vehicle Parameters.” Journal of Physics A: Mathematical and General, vol. 37, no. 37, 2004, pp. 8753–8788., doi:10.1088/0305-4470/37/37/001

9. Kim and Malik, "Fast vehicle detection with probabilistic feature grouping and its application to vehicle tracking," Proceedings Ninth IEEE International Conference on Computer Vision, Nice, France, 2003, pp. 524-531 vol.1.

10. Magee, Derek R., Tracking multiple vehicles using foreground, background and motion models, Image and Vision Computing, Volume 22, Issue 2, 2004, Pages 143-155, ISSN 0262-8856, https://doi.org/10.1016/S0262-8856(03)00145-8.

11. Malinovskiy, Yegor, et al. “Video-Based Vehicle Detection and Tracking Using Spatiotemporal Maps.” Transportation Research Record, vol. 2121, no. 1, Jan. 2009, pp. 81–89, doi:10.3141/2121-09.

12. Morris, B. T., Trivedi, M. M., "Learning, Modeling, and Classification of Vehicle Track Patterns from Live Video," in IEEE Transactions on Intelligent Transportation Systems, vol. 9, no. 3, pp. 425-437, Sept. 2008. doi: 10.1109/TITS.2008.922970

13. Rad, R., Jamzad, M., Real time classification and tracking of multiple vehicles in highways, Pattern Recognition Letters, Volume 26, Issue 10, 2005, Pages 1597-1607, ISSN 0167-8655, https://doi.org/10.1016/j.patrec.2005.01.010.

14. Robert, K., "Night-Time Traffic Surveillance: A Robust Framework for Multi-vehicle Detection, Classification and Tracking," 2009 Sixth IEEE International Conference on Advanced Video and Signal Based Surveillance, Genova, 2009, pp. 1-6.

15. Tsai, Y., Wang, C., Wu, Y. “A Vision-based Approach to Study Driver Behavior in Work Zone Areas” Proceedings of the 3rd International Conference on Road Safety and Simulation, Indianapolis, U.S., September 14-16, 2011.

16. Tsai, Y. “Development of a Sensing Methodology for Intelligent and Reliable Work-Zone Hazard Awareness”, National Academy of Sciences NCHRP IDEA-139 Final Report, 2011.

17. Vaughan, C., et al. “The Effects of Late Lane Merges on Travel Times.” Research and Development, Apr. 2018, connect.ncdot.gov/projects/research/RNAProjDocs/2015-08 Final Report.pdf.

18. Weng, J., Meng, Q., Modeling speed-flow relationship and merging behavior in work zone merging areas, Transportation Research Part C: Emerging

Technologies, Volume 19, Issue 6, 2011, Pages 985-996, ISSN 0968-090X, https://doi.org/10.1016/j.trc.2011.05.001.

19. Wu, Y., Lian F., Chang, T., "Traffic Monitoring and Vehicle Tracking using Roadside Cameras," 2006 IEEE International Conference on Systems, Man and Cybernetics, Taipei, 2006, pp. 4631-4636. (https://ieeexplore.ieee.org/abstract/document/4274643)

PART 4 DATA COLLECTION PRACTICES AND MODELING TECHNIQUES

1.0 INTRODUCTION

Along with the original research and analysis completed for this project, the STRIDE research team included a task to describe and document key findings and lessons learned from a series of NCDOT and NCHRP work zone-related research projects. Three such research projects were deemed relevant to the STRIDE effort. These projects are summarized in the remainder of this report section. The key feature of the first project was the use of a mesoscopic network modeling tool, DTALite, to model the impact of various lane closure and work scheduling scenarios. The results of this project motivated a change by NCDOT and the construction contractor to the planned work zone design and schedule which significantly mitigated the negative work zone impacts. The second project coincided with the execution of the work zone plans that were informed by the first project. This project also used DTALite in an ongoing modeling effort that incorporated real-time traffic information and supplemented the network modeling with targeted FREEVAL modeling of key multi-segment facilities within the overall work zone. The final project was funded under the National Cooperative Highway Research Program. This project provided a summary of work zone capacity analysis methods. The project synthesized and extended previous research to provide analytical methods for freeway, urban arterial, and two-lane highway work zones.

2.0 NCDOT PROJECT 2012-36 WORK ZONE TRAFFIC ANALYSIS & IMPACT ASSESSMENT

The project was tasked with assessing the estimated traffic impacts of the proposed NCDOT TIP Project I-5311/I-5338, a pavement rehabilitation project on interstates I-40 and I-440 from Exit 293 to I-40 Exit 301 and I-440 Exit 14. The project aimed to predict corridor and network-wide impacts of the work zone during construction, including routes along the work zone corridor, as well as key alternative routes. The primary focus of this study was the development and calibration of a network-wide mesoscopic simulation model of the Triangle region, as well as a macroscopic representation of the work-zone corridor.

The geographical coverage of the mesoscopic simulation model included the entire triangle region and additional sections of US264, I-40, and I-95 east of the triangle. The model was calibrated using field estimated spot volume and speed data, as well as key route travel times obtained from INRIX. The model was initially developed and tested in the DynusT software tool and was then transferred to the DTALite software tool. Both tools gave reasonable results for the baseline scenario, and when modeling higher-capacity work zone scenarios. However, for lower capacity (more severe) work zone scenarios DTALite performed more reasonably, while

the DynusT tool yielded unrealistically high traffic densities in segments upstream of the work zone. Therefore, while both tools proved useful in this project, the DTALite results were thought to provide a more realistic assessment of the expected work zone impacts at this time.

In addition to the two mesoscopic tools, the macroscopic analysis tool FREEVAL was used to explore the estimated impacts on the work zone corridor. While FREEVAL is not able to predict diversion rates and network-wide performance, it has been proven to be a useful tool for assessing work zone impacts based on previous research conducted for NCDOT.

Multiple work zone scenarios were modeled in all three tools to test the relative impacts of different lane closure configurations on route and network performance. Scenarios included a reduction of the overall cross-section to only two travel lanes, from a base of three to five lanes per direction. Additional scenarios maintained more travel lanes at key bottleneck sections during construction, as well as a three-lane scenario that was thought to offer significant congestion relief during construction. The analysis further differentiates between no diversion (drivers not taking alternative routes) and with diversion (drivers taking alternative routes to travel through or around the construction) results.

FREEVAL results suggested that 30-40% of drivers must select alternative routes in the AM Peak hour to keep average travel speeds through the work zone above 20 mph with two open lane work zone pattern. With three open lanes, FREEVAL estimated that if 10-20% of drivers select alternative routes the average travel speeds will be over 40 mph. For the PM Peak, the FREEVAL analysis suggests that if 40- 50% of drivers select alternative routes, the average speed will be over 10 mph with at least two travel lanes open. For the three-lane open case, it is estimated that a 40-mph average travel speed can be maintained only if 30-40% of drivers select alternative routes.

The network model estimates I-40 westbound travel times to increase through the work zone from 8.6 to 15.1 min for the two-lane pattern, and to 12.7 minutes for the three-lane pattern in DTALite model. In the more constrained two-lane open pattern, the model estimates travel time increases over 30% for I-440 eastbound (+32%), US70 northbound (+92%), Hammond Rd. northbound (+115%), and Rock Quarry Road westbound (+43%) in the AM Peak for the two-lane. For the three-lane open case, network-wide impacts are mitigated to some extent because fewer drivers select alternative routes. The model still estimated travel time increases over 30% for US70 northbound (+44%), Hammond Road northbound (+54%), and Rock Quarry Road westbound (+32%), and other significant impacts on I-440 EB (+10%), NC55 NB (+17%) and Hammond Road northbound (+22%).

In the PM Peak, the network was generally more congested due to higher traffic volumes. DTALite estimates I-40 eastbound travel time to increase through the work zone from 8.6 to 18.5 minutes for the two-lane open pattern, and to 13.9 minutes for the three-lane open pattern. These increases are determined by the model calculating an idealized 62% traffic

volume reduction and 36% traffic reduction for the two-lane open and three-lane open pattern, respectively. Driver-selected diversion to alternative routes resulted in travel time increases over 30% for Wade Ave. EB (+59%) and Davis Drive SB (+33%), as well as significant impacts to I-440 EB (+20%), I-540 EB (+20%), and US64 WB (+30%). Similar to the AM Peak, many of these impacts are mitigated with the three-lane open option, and a travel time increase over 30% is estimated only for US64 westbound (+32%), with other significant impacts on I-440 EB (+17%), Wade Avenue EB (+21%), NC55 EB (+14%), and Davis Drive SB (=10%).

A large amount of data was deemed necessary to support the calibration and validation of the baseline models. Four different sources of data developed the calibration and validation datasets:

- Sensor-based speed and volume data from Traffic.Com side-fire radar stations across the triangle region to support volume calibration and speed validation of model results;
- Probe-based travel time data from INRIX.com to support validation of modeled route travel time to field observations;
- Custom point volume estimates requested from NCDOT at key locations outside of the Traffic.Com sensor coverage in the triangle; and
- Arterial traffic counts on key non-freeway routes in the triangle, which are likely to serve as key diversion routes to the proposed work zone.

In conclusion, this project provided an in-depth and comprehensive comparison and application of three software tools for evaluating corridor and network impacts of a major urban freeway work zone. All three models were calibrated and validated with a significant amount of field-measured data and local work zone capacity estimates from prior research. Further work on monitoring, measuring, and validating the actual impacts was carried out in a follow up research project.

3.0 NCDOT PROJECT 2014-33 – WORK ZONE MONITORING AND ASSESSMENT FOR TIP I-5311/I-5338

This project focused on enhanced modeling, continued monitoring, and assessment of NCDOT's TIP numbers I-5311/I-5338: I-40 and I-440 Re-Construction Work from Exit 293 to I-40 Exit 301 and I-440 Exit 14 work zone project in Raleigh, North Carolina. The project encompassed a significant amount of predictive modeling work of expected work zone impacts, using a variety of software tools. The project also included a broad data monitoring effort, capturing operational performance data of the work zone using a variety of data sources. In addition to these modeling and monitoring activities, the project produced a work zone operation guide for NCDOT.

The ITRE team was previously involved in a prior NCDOT research project (NCDOT Project 2012-36 Work Zone Traffic Analysis & Impact Assessment) to predict operational impacts of this work zone using network-wide and corridor-level evaluations tools to estimate the congestion and traffic diversion impacts of the eleven-mile work zone. This project expanded on that work and included a significant data monitoring component. This project was constructed in two primary operation stages called as Area 3 and Areas 1&2 by the contractor. Area 3 is the section of the work zone on I-440 from the I-40/440 split at exit 301 to the US 264 interchange on I-440 (roughly 3 miles in length). Areas 1&2 is the section referred to as the construction between I-40/440 split at exit 301 and the US1/64 interchange at exit 293 on I-40.

Three primary methods of analysis were employed for stage 1 (Area 3) of the project: field data obtained during construction, macroscopic modeling data from the FREEVAL tool, and mesoscopic modeling data from the DTALite tool. Monitoring of the project was done using three different types of sensor technology for the same state.

- HERE® (previously known as Traffic.com) side-fire radar sensors deployed throughout the Triangle region, which provided traffic volume and (spot) speed estimates on the freeway network.
- INRIX probe-based data that is available for all freeways and major arterials in North Carolina and provides travel time and (segment) speed estimates. By looking at speed estimates over multiple segments, it is further possible to estimate queue lengths. It is noted that INRIX data can be unreliable for arterial performance, especially over short segments.
- Video observations from overhead mounted NCDOT traffic cameras. These video streams are used to provide a visual of work zone performance, as well as confirm traffic volumes and queue lengths if needed.

FREEVAL tool was used to evaluate the impact of stage 1 on all the routes for this stage. Results of only of the routes is presented in this paragraph - a long commuter route from Exit 312 in Clayton, NC to Exit 284 in Cary, NC. The FREEVAL analysis included AM peak period (6:00am to 10:00am) in the westbound direction and PM peak period (3:30pm to 7:30pm) in the eastbound direction. FREEVAL results showed that minimal queue was present under the base case but increased to an 8 miles queue with the presence of work zone. The associated increase in travel time went from 34.7 minutes to 92.3 minutes, in the peak fifteen-minute period – a travel time index of 3.38. Under consideration of 20% traffic diversion, the AM peak queue is estimated at 5.4 miles and a maximum travel time of 75.0 minutes and a TTI of 2.75. PM peak results show a 1-mile queue in the base condition that increased to 6.6 miles with the work zone. The travel time accordingly increased from a maximum of 35.0 minutes to a maximum of 74.4 minutes. With a 20% diversion, the queue is reduced to 2.8 miles and the travel time to 52.8 (a TTI of 1.93). The finding of the remaining three routes can be looked up in the final report for this project.

The results of DTALite in terms of work zone effects suggested that in the AM peak period, the Area 3 activity resulted in a 15% increase in network travel time for the no diversion case, but a 27.8% reduction in travel time in the with-diversion case as drivers found alternate routes. For the PM peak, Area 3 estimates a 105.9% increase in travel time in the no diversion scenario, but a reduction of 11.8% in the with-diversion scenario. The reduction in the UE case is attributed to drivers avoiding the overall work zone and finding quicker alternate routes.

Stage 2 (Areas 1&2) of the project employed three methods of analysis: 1) field data obtained during construction, 2) macroscopic modeling data from the FREEVAL tool, and 3) mesoscopic modeling data from the DTALite tool. In addition, four different types of sensor and technologies were used to monitor this stage of the work zone.

- HERE.Com (previously Traffic.com) side-fire radar sensors deployed throughout the triangle, which provide traffic volume and (spot) speed estimates on the freeway network.
- INRIX probe-based data that is available for all freeways and major arterials in North Carolina and provides travel time and (segment) speed estimates. By looking at speed estimates over multiple segments, it is further possible to estimate queue lengths. It is noted that INRIX data can be unreliable for arterial performance, especially over short segments.
- Video observations from overhead mounted NCDOT traffic cameras. These video streams are used to provide a visual of work zone performance, as well as confirm traffic volumes and queue lengths if needed.
- The team also deployed Bluetooth sensors across the network to test actual route diversion patterns as a result of the work zone. Even though Bluetooth sensors can capture a fraction of the overall traffic volume, the station counts can serve as a surrogate for volume.

A comprehensive analysis of travel times for both stages of the project was executed for almost the entire year of 2015 (January 1 to December 21) using INRIX data. The analysis was done both at daily level for peak periods and monthly level for the peak periods. Furthermore, Bluetooth sensors were deployed for short periods to collected data that could be used for travel time analysis and surrogate for volume. In addition, the project did an analysis of traffic diversion resulting from the Fortify work zone on I-40 and I-440. This analysis included observations during the work in Area 3 as well as Area 1 & 2. The diversion analysis is performed on point sensor data collected on sections of I-40 and I-440 approaching active work zones. First, the team looked at four-hour peak period volumes at the three-primary entry-points to the construction area. These volumes were collected for the months of February, August, and September for the years of 2009 through 2015. Second, the team looked at peak-hour volumes for different sequences of sensors leading up to and into the work zone for the years 2013, 2014, and 2015 using the same three primary approaches into the work zone for

both AM and PM peak periods. Detailed information on how the travel times change daily and monthly and how the work zone impacted the traffic diversion can be found on the final report for the project.

The results of DTALite for the Area 1 and 2 analyses for the AM peak showed an increase of 177.7% in the no diversion scenario, which is mitigated to a 46.3% increase in the with-diversion scenario. It is emphasized that local increases in travel time are expected to be much higher, but that these increases are offset when calculating an average of the entire model. In the PM peak, the no-diversion scenario shows a very drastic increase in travel time of about 481% from 16.9 to 98-minute average travel time. But with the with-diversion, the increase drops to 106% and an average travel time in the network of 34.8 minutes.

Macroscopic analysis tool, VISUM software, was used to do an analysis of the work zone in order to get a quick overview of the volume shift that is brought about by the work zone and help reduce the network complexity and extent of the model to avoid unrealistic diversion. The software modeled a reduced network of TRM (triangle regional model) to identify key alternate routes that may be used for in depth analysis. They key routes identified by this tool were: Tryon Road, US 70, I-440, Ten Ten Road and I-540.

The last outcome of the project is a guidance document for work zone operational modeling and monitoring. This document presents guidance for modeling and monitoring work zone operations and mobility performance in North Carolina. Although the document was prepared with the aim to help NCDOT congestion management groups prepare and train their staff to properly scope work zone studies, it can be used by any organization anywhere in the United States. The main focus of the guide is on operational analysis and monitoring, which is one of the critical aspects of work zone performance assessment.

In the context of the developed guidance, Modeling refers to prediction of the operational effect and impacts of a work zone before start of construction. Modeling is done to predict work zone operations, compare and contrast alternatives, and support decision-making for work zone staging. The document discusses model types and presents guidance for application of different levels of analysis tools to determine work zone effects.

Once the work zone is in effect, monitoring should be conducted to evaluate and track its performance (monitoring is done during active construction). The guidance discusses types of sensing and data collection approaches to quantify operational characteristics of a work zone and makes recommendations for what level of monitoring is appropriate for a given work zone.

The guidance has the form of key questions that an analyst or work zone designer may have for a specific project. From the answers to these key questions, the guidance then proposes different levels of work zone monitoring and modeling, depending on the desired level of detail and analysis outcomes.

4.0 NCHRP 3-107 – Work Zone Capacity Methods for The Highway Capacity Manual

This project developed analytical models to predict work-zone capacity for freeways, urban streets, and two-lane highways. The developed models were calibrated and validated using data collected through focused field studies and simulation studies, and data available from previous research. Furthermore, the developed models can be used to estimate work-zone capacity, as well as operating speed and queuing across a range of work-zone conditions and traffic volumes. The goal of the methodologies was to provide transportation agencies with tools to quickly evaluate maintenance of traffic plans in order to minimize impacts to mobility and reduce road-user costs associated with roadway construction and maintenance projects. In addition, guidance on the application of microscopic simulation to model work zones, based on the application of simulation for this research, was prepared and included in the final report.

Freeways Findings Database: In the context of freeways two types of models were developed. First, using measured queue discharge rates collected for freeway work zones across the United States, a regression-based capacity prediction model was developed and validated. Second, using free-flow speeds collected for freeway work zones across the United States, a regression based free-flow speed prediction model was developed and validated. Data used for this part of the project included field collected data using videos at 13 work zones in six states (to obtain queue discharge rates), and sensor data (mainly free-flow speed) acquired from RITIS, PeMS, and Traffic.com databases.

Urban-Streets Findings and Database: work zone presence on an urban street drastically impacts both the operation and capacity of the street. The severity of the impact depends on the location the work zone, work-zone length, number of open lanes, presence of shoulder, and signal timing. Free-flow speed data were collected at five urban-street segments collectively located in two states. Saturation flow rate data were collected for three intersection approaches collectively located in two states.

Two-Lane Highways Findings: lane-closure work zones on a two-lane highway, typically controlled with either flaggers or signals on each end, operates similarly to a signalized intersection operating under two-phased control. As a result, the capacity for this type of streets can be determined based on the saturation flow rate at the control points and the cycle length.