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Simulation Exploration of the Potential of Connected Vehicles in Mitigating Secondary Crashes

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SIMULATION EXPLORATION OF THE POTENTIAL OF CONNECTED VEHICLES IN MITIGATING SECONDARY CRASHES

by

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The thesis titled "Simulation Exploration of the Potential of Connected Vehicles in Mitigating Secondary Crashes on Freeways" submitted by Mike Alvin Soloka in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering has been:

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Dedicated to

My Creator, The Almighty God.

My first educators, my Father and Mother, Paul and Rose.

And my brother, Kevin.

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LIST OF ACRONYMS/ABBREVIATIONS

AID	Automatic Incident Detection
ATDM	Active Transportation and Demand Management
ATIS	Advanced Traveler Information Systems
AVI	Automatic Vehicle Identification
API	Application Programming Interface
CCTV	Closed-circuit Television
CME	Certificate Management Entities
CMS	Changeable Message Sign
COM	Component Object Model
CV	Connected Vehicles
DOT	Department of Transportation
DMS	Dynamic Message Signs
DR	Detection Rate
DSRC	Dedicated Short Range Communication
FAR	False Alarm Rate
FDOT	Florida Department of Transportation
FHWA	Federal Highway Administration
GPS	Global Positioning System
HCM	Highway Capacity Manual
ICT	Information and Communication Technology
ITS	Intelligent Transportation System

LOS	Level of Service
LTE	Long-Term Evolution
MOE	Measure of Effectiveness
MTTD	Mean Time to Detect
NHTSA	National Highway Traffic Safety Administration
OBU	On-Board Unit
PET	Post Encroachment Time
PKI	Public Key Infrastructure
PM	Performance Measure
RMSNE	Root Mean Square Normalized Error
RSU	Roadside Unit
SC	Secondary Crash
SCDOT	South Carolina Department of Transportation
SCMS	Security Credential Management Systems
SSAM	Surrogate Safety Assessment Model
THEA	Tampa-Hillsborough Expressway Authority
TIM	Traffic Incident Management
TMC	Traffic Management Center
TRANSYT-7F	Traffic Network Study Tool, Version 7F
TTC	Time-to-Collision
USDOT	United States Department of Transportation

VANET	Vehicle Ad-hoc Network
VB	Visual Basic
VISSIM	Verkehr In Städten Simulationsmodell
VMS	Variable Message Sign
VSL	Variable Speed Limit
V2I	Vehicle-to-Infrastructure
V2V	Vehicle-to-Vehicle
V2X	Vehicles to Everything
3GPP	3rd Generation Partnership Project

ABSTRACT

Secondary crashes (SCs) on freeways are a major concern for traffic incident management systems. Studies have shown that their occurrence is significant and can lead to deterioration of traffic flow conditions on freeways in addition to injury and fatalities, albeit their magnitudes are relatively low when compared to primary crashes. Due to the limited nature of crash data in analyzing freeway SCs, surrogate measures provide an alternative for safety analysis for freeway analysis using conflict analysis.

Connected Vehicles (CVs) have seen compelling technological advancements since the concept was introduced in the 1990s. In recent years, CVs have emerged as a feasible application with many safety benefits especially in the urban areas, that can be deployed in masses imminently. This study used a freeway model of a road segment in Florida's Turnpike system in VISSIM microscopic simulation software to generate trajectory files for conflict analysis in SSAM software, to analyze potential benefits of CVs in mitigating SCs.

The results showed how SCs could potentially be reduced with traffic conflicts being decreased by up to 90% at full 100% composition of CVs in the traffic stream. The results also portrayed how at only 25% CV composition, there was a significant reduction of conflicts up to 70% in low traffic volumes and up to 50% in higher traffic volumes. The statistical analysis showed that the difference in average time-to-collision surrogate measure used in deriving conflicts was significant at all levels of CV composition.

Keywords: Secondary Crashes, Safety Surrogate Measures, Connected Vehicles, Conflicts

CHAPTER 1 INTRODUCTION

Background

Freeway crashes that occur as a result of prior incidents (or primary incidents), also termed as secondary crashes (SCs), are a major concern for traffic incident management systems. Researchers and professionals have therefore been trying to study SCs for more than two decades, while relentlessly looking for new ways of predicting and preventing their occurrences with limited success. In addition to injury, fatalities, and loss of property, SCs can also result in additional traffic congestion and delay by speed reduction, queue formation, driver distraction and blocking of lanes. About three decades ago, incident delay was attributed to 61% of all urban freeway delay and was projected to be approximately 70% by 2005 (Lindley, 1987). Reports have stated that SCs range from 14 to 30% of all crashes while an estimated 18% of fatalities on freeways are caused by SCs (Owens et al., 2010).

Connected Vehicle (CV) technology has significantly advanced since the concept was introduced in the 1990s through the Automated Highway Systems (AHS) research and later developed under the Vehicle Infrastructure Initiative (VII) in 2003 (Harding et al., 2014). Currently, CV deployments are being tested and carried out in some areas in the nation and is showing promising results in helping mitigate not only SCs but all crashes in general.

Until recently, limited studies have explored the benefits that come with the implementation of the CV technology in mitigating SCs on freeways through microscopic simulation studies. Existing studies on SCs mainly focus on identification, analysis of characteristics and risk modeling (Yang, Wang, & Xie, 2017). This study, therefore, explores how CV technologies can be utilized to lower the SC risk through indirect safety analysis by using microscopic simulation software and conflict analysis software packages.

Study Objective

The objective of this study is to evaluate the potential benefits associated with the presence of CVs in the traffic stream in the reduction of SCs. In particular, the reduction of safety surrogate measures in the CV environment with both vehicle-to-vehicle (V2V) and vehicle-to-infrastructure (V2I) communications. Thus, this study presents an evaluation of traffic flow conditions in the event of an incident that causes partial blockage of the travel lanes on a freeway segment in a CV environment. Traffic simulation was done in VISSIM microscopic simulation software and the safety evaluation was conducted using the SSAM software.

Thesis Organization

This thesis contains 5 chapters. Chapter 1 provides a general background of the study and an overview of the research problem, as well as the objective of the study. Chapter 2 provides a review of literature relevant to the study including a description of SCs and a background of CVs and their applications. Chapter 3 discusses the approach and methodology used in the study including the study site and the simulation test bed adopted for the study. It also gives a description of the tools used in the study. Chapter 4 goes on to describe the preliminary results obtained from the study and gives a discussion of the results. Finally, Chapter 5 gives a conclusion based on the results and comments on further work warranted by the preliminary results from the study.

CHAPTER 2 LITERATURE SYNTHESIS

Literature Review

Secondary Crashes

The safety of a freeway facility is defined by the Highway Safety Manual (AASHTO, 2010) as the number of crashes, by severity, expected to occur on the entity per unit of time. Also in line with Gettman and Head (Gettman & Head, 2003), highway safety is quantified by using the expected number of crashes by type, that are expected to occur on an entity in a certain time interval, per time unit. A secondary crash (SC) is described as an incident that occurs within two hours from the onset of a primary incident and also within two miles downstream of the primary incident location (Chang & Rochon, 2011; Hirunyanitiwattana & Mattingly P, 2006; Jalayer, Baratian-Ghorghi, & Zhou, 2015; Kopitch & Saphores, 2011; Moore, Giuliano, & Cho, 2004; Tian, 2015).

Since secondary crashes (SCs) account for nearly 20% of all crashes and about 18% of fatalities occurring on freeways (Owens et al., 2010), they have significant occurrences especially on freeways across the nation. In mitigating the risk of SCs, therefore, a key element of traffic incident management is achieved. However, only a few studies have focused on utilizing specific measures to mitigate SCs (Karlaftis, Latoski, Richards, & Sinha, 1999; Kopitch & Saphores, 2011; Hyoshin Park & Haghani, 2016; Hyoshin Park, Haghani, Samuel, & Knodler, 2018; Yang et al., 2017). Prior to establishing methods to mitigate SCs, it is essential to identify the various factors that contribute to their occurrence.

From previous studies, the various attributes that have been found to significantly influence the risk of SCs include primary incident characteristics, real-time traffic characteristics and weather conditions, and roadway geometrics as shown in Figure 2-1.

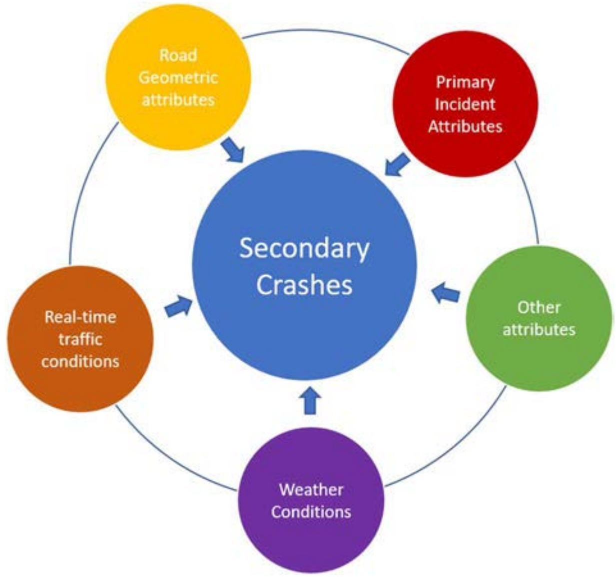


Figure 2-1: Factors Contributing to Secondary Crashes

Studies have further stated that severity and clearance durations of a primary incident are the major contributors to the occurrence of SCs (Sando et al., 2018; Yang, Wang, Xie, Ozbay, & Imprialou, 2018). In one study, the likelihood of SCs was observed to increase by 2.8% for each additional minute required to clear the initial crash (Owens et al., 2010). An increase of 2 to 3 minutes of incident duration was also shown to potentially lead to a 1 percentage point increase in the likelihood of a secondary crash by a different study (Goodall, 2017). A study in Maryland that assumed a linear correlation between secondary crashes and incident duration, the reduction of secondary incidents was estimated to stand at 41.35 percent with a count of 495 potentially reduced incidents (Chang & Rochon, 2011).

To prevent the risk of SC occurrence, the impact of the primary incident must be mitigated in a timely manner (Kitali et al., 2018). Previous studies have largely focused on exploring the potential of using effective incident management and advance warning messages in mitigating the risk of

SCs (Yang et al., 2018). Incident responding agencies, such as highway patrol, emergency medical services, towing agencies, etc., could be better prepared to respond to potential SCs when conditions associated with a high likelihood of occurrence exist. Nonetheless, the implementation of this countermeasure is challenging due to the limited resources available, e.g., patrol vehicles, personnel, traffic surveillance systems, etc. Moreover, each primary incident may occur during different conditions, resulting in different impacts. For example, an incident responder may be hindered by a long queue, thus delaying the process of incident clearance (Yang et al., 2018).

In addition to the optimal allocation of emergency response units, another approach explored by previous studies is the use of advance warning messages. The warning messages may include speed advisory, lane change advisory, and possible detour messages, among others, which could help drivers recognize traffic conditions in advance and thus act accordingly to improve both safety conditions and traffic flow.

Identification of SCs

The two commonly used methods of identification of SCs are the static method and the dynamic method. The static method uses fixed spatiotemporal thresholds whereas the dynamic method uses spatial and/or temporal thresholds that change depending on queue lengths, roadway types and other relevant factors.

Many studies have used the static methods which are determined using either a fixed duration or clearance time plus selected additional recovery time as the temporal background (Asad Khattak, Wang, & Zhang, 2009; Kopitch & Saphores, 2011; Pigman, Green, & Walton, 2011; Tian, 2015; Zhan, Gan, & Hadi, 2009). Some of these studies have spatiotemporal thresholds that extend in both directions of traffic (Zhan et al., 2009; Zhang & Khattak, 2010), whereas others exclude the opposite direction of traffic. Many studies have also used dynamic methods to classify crashes as SCs, using mostly queue lengths and incident duration (A. Khattak, Wang, Hongbing, & Mecit, 2011; Kitali et al., 2018; Yang, Bartin, & Ozbay, 2013; Zhan et al., 2009; Zheng, Chitturi, Bill, & Noyce, 2014).

Although it is usually difficult to accurately classify an incident as a secondary crash, and link an initial incident to the secondary incident (Moore et al., 2004), most of the previous studies use reasonable methods which provide a near accurate identification of the SCs. Table 1-1 shows a summary of previous studies on identification of SCs using the static and/or dynamic methods.

Table 2-1: Previous Studies on SC Identification

Study	**Area**	**Spatiotemporal Thresholds**	**Method**
Kitali et al. 2018	Freeway	Primary incident impact duration + location	Bayesian C-log-log Model (Dynamic)
Zheng et al., 2014	Freeway	Queue length; Incident Duration	Linear Referencing System, Crash Pairing (Dynamic)
Yang et al., 2013	Major Highway	Queue length; Incident duration	Speed Contour Maps (Dynamic)
Kopitch et al., 2011	Freeway	1 mile; Incident duration + 15 minutes	Fixed thresholds (static)
Pigman et al., 2011	Freeway	3 miles; 80 minutes	3rd Order Polynomial Models (Dynamic)
Zhang et al., 2010	Freeway	2 miles; 2 hours	Fixed thresholds (static)
Khattak et al., 2009	Freeway, Highway, State route	2 miles; 1 hour	Programming (static)
Zhan et al., 2009	Freeway	2 miles, 2 hours (same direction); 0.5 miles, 0.5 hour (opposite direction)	Database (static)
Khattak et al., 2009	Freeway, Highway, State route	Queue length; Incident Duration + Clearance Time	Queue-based software (Dynamic)
Moore et al., 2004	Freeway	2 miles: 2 hours in both directions	Fixed thresholds (Static)
Raub et al., 1997	State Highway	1000 ft; 80 minutes	GPS (static)
Raub et al., 1997	Urban Arterials	2 miles; Clearance time + 15 minutes	Programming (static)

Since this study is focused on evaluation of CVs in mitigation of secondary crashes, it was important to examine how the different static and dynamic methods from different studies have been used to identify and classify crashes for the sake of creating a threshold for measuring conflicts that can be associated with a likely secondary crash. In this sense, a spatial threshold of 2 miles and a temporal threshold of 30 minutes plus the incident duration were chosen for the analysis. This selection was done through visual inspection of the maximum queue length due to the modeled incident as well as the queue dissipation time observed in the microscopic simulation process.

Connected Vehicles (CVs)

CVs are vehicles equipped with technologies that facilitate communication with their environment. This connected environment helps the CVs to send messages to other vehicles in what is termed as Vehicle-to-Vehicle (V2V) communication, as well as to the infrastructure in Vehicle-to-Infrastructure (V2I) communication. These communications use onboard devices to convey information about a vehicle's status such as speed, heading, brake status, and other information to other vehicles and receive similar information from other CVs. The messages exchanged between vehicles have range and line-of-sight capabilities that exceed current stand-alone vehicle sensing technologies (Harding et al., 2014).

CVs have the potential of improving transportation incident management (Iqbal, Khazraeian, & Hadi, 2018), given their capability to communicate important information between themselves and the surrounding infrastructure. This potential coupled with some roadside equipment can subsequently provide the ability to alert drivers of downstream incidents, which can lead to enhanced safety and mobility. One of the benefits of such communication could be the prevention of SCs. Not only can CVs be used to detect incidents, but the messages sent, such as speed advisory, lane change advisory and detour messages can be highly effective in reducing the risk of SCs.

Intervehicle Communication (IVC)

The FCC reported in 1999, its decision to use the 5.850-5.925 GHz band for a variety of Dedicated Short-Range Communications (DSRC) uses, including traffic light control, traffic monitoring, travelers' alerts, automatic toll collection, traffic congestion detection, emergency vehicle signal preemption of traffic lights and electronic inspection of moving trucks through data transmissions with roadside inspection facilities (Federal Communications Commission, 1999). The addition of the spectrum was provided to further national goals including those of the Department of Transportation, ITS industry and Congress in the improvement of efficiency of the U.S. transportation system while also facilitating the growth of ITS.

The fostering of global research, technological innovations, industry standards-setting activities which could lead to the production of less expensive DSRC equipment are among key expectations that were speculated to arise with the allocation of the 75 megahertz for DSRC. Further, interoperability and perpetual development of the DSRC technology at a nationwide or even global level was expected to be encouraged with a significant allocation of DSRC.

In 2013, the FCC proposed the possibility and began the proceeding for the potential use of portions of the dedicated spectrum 5.9GHz band for unlicensed use on a share basis with DSRC, owing to the high demand of wireless broadband services and the future growth expectation, and the slow evolution of DSRC utilization of the spectrum.

In November 2018, the FCC released a notice of proposed rulemaking to repurpose the lower 45 megahertz of the band for unlicensed operations to support faster broadband applications. This would leave only 30 megahertz dedicated for the use of DSRC applications. Further, the revision of the current ITS rules for the 5.9 GHz band was proposed to allow C-V2X (Cellular Vehicle to Everything) operations in the upper 20 megahertz of the band (5.905-5.925 GHz). The proposed rulemaking would also seek comment on whether to retain the remaining 10 megahertz for DSRC or dedicate it to C-V2X.

The DSRC system has been designed to provide a short-range wireless link for information transfer between vehicles and the infrastructure. The links are essential to ITS services that can improve travelers' safety, improve traffic mobility and operations, and minimize pollution through emissions reduction. The FCC stated in its report that the spectral environment and propagation of characteristic of the 5.9 GHz band are appropriate for DSRC applications, supporting enough signal coverage and considerable frequency reuse.

The main functional characteristics of DSRC include: a low latency which reduces the delay in opening and closing of connections in the order of 0.02 seconds; limited interference owing to the protection by the FCC for transportation applications and a shorth range of about 3000 ft inhibits interference from further communication signals. DSRC also has a high performance in all weather conditions which makes it more relevant for transportation applications.

The DSRC service is composed of On-Board Units (OBUs) and Roadside Units (RSUs). An OBU is a device that is normally mounted in a vehicle to act as a transceiver, and sometimes it may even be a portable unit. Part 95 of the Rules as given by the FCC, describes the license under which OBUs mounted in vehicles and portable units are operated. On the other hand, an RSUs is a transceiver that is mounted on infrastructure, along a road or pedestrian passageway. An RSU may also be mounted on a vehicle or hand carried but is required to operate only when the vehicle or hand-carried unit is stationary. An RSU broadcasts data to OBUs or exchanges data with OBUs in its communications zone. The operation of RSUs is governed by Part 90 of the Rules.

CVs also use the Vehicle Ad-hoc Network (VANET) to convey and process signals to and from roadside units (RSUs) as well as other vehicles in the stream (Ghori, Zamli, Quosthoni, Hisyam, & Montaser, 2018).

Alternative Communication Technologies

In 2015, a reported stated how congress showed the desire to learn how CV implementations would not preference the use of any particular communications technology for use in CV operations (Bettisworth et al., 2015). Through the years, regular comparative analyses led by USDOT have been done to ensure that multiple choices in the communication technology are considered for use in CV applications. Through the analyses conducted by USDOT, it has been continually agreed that DSRC is still, to date, the best viable option for safety-critical applications due to its low latency properties. Through the same research, however, opportunities for use of other commercially available technologies such as cellular, satellite, radio, fiber and Wi-Fi, have been reported. In support of applications that do not necessarily require extremely low signal latency, as provided by DSRC, such technologies can be used. These applications include but are not limited to mobility and logistics, traveler and road weather information, security credential management, field equipment-to-center (backhaul) communications and agency communications or decision support systems.

Although the USDOT still considers the 5.9 GHz spectrum in DSRC to be a foundational requirement for safety-critical CV applications, in recent times, the FCC has not shown full support for the continued dedication of the spectrum for sole transportation uses. In November 2018, the FCC proposed a rulemaking that will possibly replace DSRC with Cellular to Everything (C-V2X). This development has spurred the transportation industry leading to divided attitudes towards the use of DSRC or other technologies, from both automakers and transportation agencies.

The new Cellular Vehicle to Everything (C-V2X) technology, uses the cellular Long-Term Evolution (LTE) protocol and its variates to provide wireless communication between vehicles and other devices. A recent development in the LTE standard known as LTE Sidelink, defined by 3GPP is a promising technology that allows for a more efficient conveyance of communication signals in comparison to the conventional cellular communications that transmits through cellular towers or infrastructure (Molina-Masegosa, Gozalvez, & Sepulcre, 2018). The 3rd Generation Partnership Project (3GPP) is a union of seven telecommunications standard development organizations that provides an environment for production of reports and specifications that define the 3GPP technologies.

In Release 14 of the C-V2X standard (also known as LTE-V or LTE-V2X), the 3GPP includes two modes of operation namely Mode 3 and Mode 4. In both modes, vehicles communicate to each other directly between them however, in Mode 3, communications still rely on cellular infrastructure to manage the communications and select sub-channels (Molina-Masegosa et al., 2018). Mode 4, however, is considered the baseline mode which represents an alternative to DSRC, eliminating the need for a cellular network through infrastructure. In this mode, vehicles autonomously select radio sources for their direct V2V communications, and include a distributed radio selection scheme for vehicles to communicate as well as the support for distributed congestion control (Molina-Masegosa & Gozalvez, 2017).

Since safety applications cannot rely on the availability of a cellular network, the development of the LTE standards by Sidelink may come as the most viable alternative to DSRC and other technologies for CV applications in the near future. For more in depth review, the reader is recommended to view an article by (Molina-Masegosa & Gozalvez, 2017), which has provided a comprehensive overview of the LTE standard for Sidelink 5G V2X vehicle communications.

Incident Detection

Automatic Incident Detection (AID)

Since incident management systems deal with detection and removing of incidents in road networks, incident detection can be termed as the most important part of an incident management system (Deniz, Celikoglu, & Gurcanli, 2012). An efficient incident detection method is thus crucial in any successful incident management system due to the reduction of congestion, possibility of secondary crashes, and fuel consumption and emissions, stemming from quicker response (Cambridge Systematics Inc., 2005). Incident detection may be accomplished by driver-based methods in the form of a report from an observer through a cellular device to an authority or Automatic Incident Detection (AID) using either probe data or roadway-based sensors. With the advancement and proliferation of sophisticated mobile devices or “smartphones”, incident detection can now also be achieved through mobile applications in the driver-based method. Although incident detection by road users reporting via cellular phones has become common, there exist some limitations such as data redundancy and reliability (Walters, Wiles, & Cooner, 1998;

C. Xie & Parkany, 2002). Given these limitations, AID is arguably a more preferred method in the connected vehicle environment.

With advancement in sensor technologies, AID has developed as a promising incident detection method over the years. Many algorithms have also been developed to facilitate AID, all of which have advantages and disadvantages. Some freeway incident detection algorithms include time-series algorithms, comparative algorithms, the McMaster algorithm, artificial intelligence algorithms, macroscopic algorithms, and wavelet algorithms (Teng & Qi, 2003). Each of these algorithm groups further contains multiple algorithms developed by studies over the years. Some of the algorithms, e.g. the wavelet algorithm, have shown excellent results in incident detection rates using data denoising and clustering methods (Adeli & Karim, 2000).

Some measures of effectiveness that have been used to compare and evaluate algorithms include the false alarm rate (FAR), detection rate (DR) and mean time to detect (MTTD). It has generally been shown that the performance of the algorithms is related to the incident location and traffic volume conditions, where higher false alarm rates occur in higher traffic volumes and a lower MTTD occurs when the incident occurs closer to the upstream detector (Deniz et al., 2012).

Table 2-2 provides a summary of some real-time incident detection algorithms that have been developed over the years along with their data needs.

Table 2-2: Some incident detection algorithms and data needs (Deniz et al., 2012)

Algorithm	Occupancy	Volume	Speed
California Algorithm (Basic)	+	-	-
California Algorithm #7	+	-	-
California Algorithm #8	+	-	-
All Purpose Incident Detection(APID)	+	+	-
Pattern Recognition	+	-	-
Standard Normal Deviates (SND)	+	+	-
Bayesian	+	-	-
Time Series	+	+	-
Exponential Smoothing	+	-	-
Neural Networks	+	+	+
Fuzzy Sets	+	+	+
Modified McMaster	+	+	-

According to the FHWA (Owens et al., 2010), some of the commonly used methods used in the detecting incidents include:

- Wireless telephone calls from motorists
- CCTV cameras viewed by operators
- Automatic vehicle identification (AVI) combined with detection software
- Electronic traffic measuring devices (e.g. video imaging, loop or radar detectors) and algorithms detecting traffic abnormalities
- Motorist aid telephones or call boxes
- Police patrols
- Aerial surveillance
- Department of transportation or public works crews via two-way radio
- Traffic reporting services

- Fleet vehicles (transit and trucking)
- Roaming service patrols

Among the mentioned methods, CCTV cameras, automatic vehicle identification (AVI) and Electronic traffic measuring devices (e.g. video imaging, loop or radar detectors), could be adopted for automated incident detection.

Automatic vehicle identification (AVI), such as that used by tolling systems has also been tested and shown to have similar results in detecting incidents when compared to vehicle sensors such as loop detectors (Pearce & Subramaniam, 1998). Inductive loop sensors also have limitations when distinguishing high-speed vehicles, low headways, and tall vehicles. Video surveillance methods have the best success so far in remote incident detection and verification. However, their limited distribution hinders complete coverage of the roadway networks (Klein, Mills, & Gibson, 2006).

With further advancements in technology, incident detection has become possible using non-intrusive methods such as probe-based data and cellular data. For instance, it has been reported that only a 5% population of probe vehicles in traffic can provide adequate information given that they are well distributed in the traffic (Pearce & Subramaniam, 1998).

CVs in Incident Detection

Through on-board dedicated short-range communications (DSRC) devices, Vehicle Ad-hoc Network (VANET), or Cellular communications, connected vehicles (CVs) are capable of V2V as well as V2I communications. This capability enables them to communicate continuously in real-time in a connected environment helps by sending and receiving messages to other vehicles and road-side units (RSUs) (Ghori et al., 2018; Harding et al., 2014).

Since CVs continuously broadcast information that describes their own speed, direction, GPS position, and acceleration or braking status, as well as that of other capable vehicles surrounding them, stopped vehicles are expected to be identified with ease from their speed data and verified to capture incidents. Although data from non-connected vehicles may not be captured directly, data from CVs at an incident location may be used to detect incidents that have occurred, by checking traffic measures such as slow-downs, high deceleration rates or hard-braking or stopped vehicles due to lane blockages.

Incident Information Dissemination

Existing Approaches in Communicating Incidents

Advanced Traveler Information Systems

Advanced traveler information systems (ATIS) is regarded as an important part of traffic management operations. It aids travelers in making decisions regarding their travels either pre-trip or en-route using information and communication technology (ICT) (Ackaah, Bogenberger, Bertini, & Huber, 2016). Although ATIS has been reportedly tested for use as early as in the 1960s in some regions including the USA, Europe and Japan, its advancement was not realized until the mid to late 1990s after the introduction of the Internet (Skabardonis, n.d.) when the focus shifted to real-time travel information delivery.

Data sources for traveler information systems include: fixed sensors along the road (either intrusive or non-intrusive), incident management teams or highway police patrol, closed-circuit television (CCTV) cameras, and eye witnesses of events reporting by mobile phone while travelling. Probe data can also be obtained as a source from toll tags and cellular phones as well as Bluetooth sensors. With the increase of market penetration of smartphone users, global positioning system (GPS) has also become important in the accuracy of data from mobile devices. Private vendors have also taken advantage of these sources and are continuously working on obtaining data using mobile device probes (Skabardonis, n.d.). Incident information has been among key road characteristics that is communicated in ATIS since its establishment.

Various methods have been used in ATIS over the years that vary from older ones such as telephone systems and highway advisory radio to DMS, web-based services such as 511 and smartphones. Figure 2 shows the distribution of information dissemination among different platforms based on the USDOT 2011 ITS deployment survey.

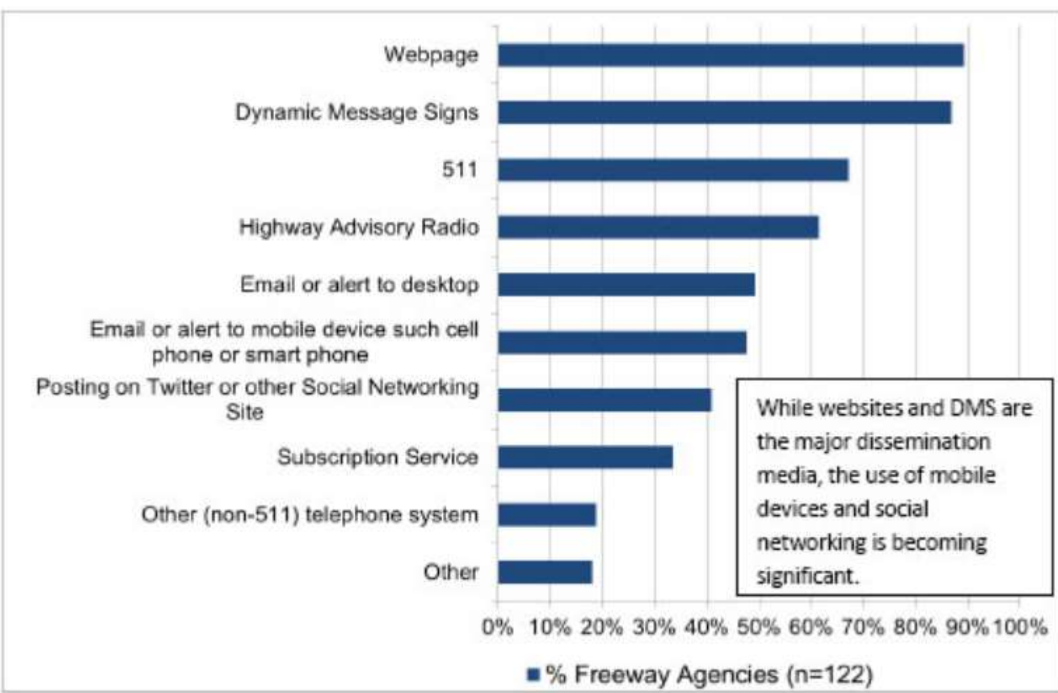


Figure 2-2: Incident Detection and Verification Approaches

The Use of CVs in Information Dissemination

The use of DSRC, VANET and other intervehicle communications enable CVs to communicate with other vehicles and roadside infrastructure by sending messages continuously between devices in a connected environment. This connected environment allows the CVs to share messages at low latencies with other vehicles in vehicle-to-vehicle (V2V) communication, and the infrastructure, through vehicle-to-infrastructure (V2I) communication (Harding et al., 2014).

V2V and V2I communications allows for vehicle status information including, speed, direction/heading, acceleration/deceleration, and other relevant information, to be shared between equipped vehicles and roadside units (RSUs) (Ghori et al., 2018). These messages enable drivers to be aware of traffic conditions within 1000 feet of other CVs and even miles ahead with assistance from roadside units and higher compositions of CVs in the traffic stream.

Authentication of CV messages

Like other wireless communication devices, vehicular communication in the CV environment is also susceptible to attacks and privacy issues. Ghori et al. (2018) conducted a review of VANET technology and identified and discussed multiple types of system attacks. Attack types mentioned in the study include GPS and tunneling attack, replay attack, Sybil attack, masquerading attack, identity disclosure attack, and wormhole attack (Ghori et al., 2018). Many of these types of attacks can affect both the authenticity and privacy of signals coming from the vehicles, while some, such as the Sybil attack can compromise the integrity of the signal, resulting in false alarms or information, such as traffic jams or accidents.

To control the issue of authentication of connected vehicle signals, multiple studies have initiated methods that can secure the communications and prevent compromising of the integrity and privacy of the messages. For example, a two-factor lightweight authentication scheme, developed by Wang et al. (2016), prevents the tracing of vehicles in the connected environment, even in the scenario when all RSUs have been compromised. This authentication method has also shown almost no network delay or packet loss ratio, which can be especially useful in safety applications.

Another proposed method of securing communications uses dual authentication and key management techniques to securely transmit data in VANETs (Vijayakumar et al., 2016). This method provides a high level of security in the CVs by preventing unauthorized vehicles from entering the network. Other methods have also been developed by various researchers (Jiang et al., 2016; Malik & Panday, 2016; Xie et al., 2016). However, most of these methods have been disputed as showing some sort of limitation in the authentication process (Ghori et al., 2018).

The use of a Public Key Infrastructure (PKI) security system has been suggested as a solution to security issues regarding CV messages (Hamilton, 2015). This system allows for digital certificates to be attached to messages coming to and from vehicles in the connected environment, thus preventing malicious behavior in the communications. Certificate management entities (CMEs) perform the functions behind administering a PKI security system, such as registering users and issuing or revoking certificates, and are what form the system referred to as a Security Credential Management System (SCMS). Research has proven that this system provides a framework that enables secure communications in the CV environment (Hamilton, 2015).

CVs and Speed Advisory

Variable speed advisory messages can be used to achieve the desired speed reduction to minimize hard-braking and high deceleration conditions that can lead to SCs. Driver compliance, as well as improved performance of the network, have been reported when advisory speeds are only slightly lowered, compared to higher reductions, from posted speeds (Riggins, Bertini, Ackaah, & Margreiter, 2016). The upstream communication approach often involves an incident warning in addition to the speed advisory, which may increase the likelihood of driver compliance and minimize SCs.

CVs and Lane Advisory

Lane-change advisory messages inform drivers of lane blockages resulting from traffic incidents downstream. The distance between the downstream incident and the upstream lane change message varies, depending on the method of dissemination. Due to the fixed nature of most DMS signs, advisory messages may be displayed well upstream of an incident. CV messages, however, can be delivered to vehicles at variable distances within the range of a vehicle's signal. The algorithm should, therefore, vary the advisory information to be disseminated based on incident characteristics and traffic flow parameters, such as queue formation, traffic flow, and density.

Safety Surrogate Measures in Safety Evaluation

Safety surrogate measures serve as an alternative method of evaluating the crash risk in a highway facility where crash data are lacking or are insufficient for the task. Several surrogate measures have been proposed and used in traffic safety engineering. It is also pointed out by one study that it has been widely accepted that three major conditions should be met to qualify as a good surrogate measure (Tarko, 2018). Firstly, a surrogate measure properly captures the effect of road and traffic changes. Second, the surrogate method correlates with the crashes affected by these changes. And finally, a surrogate measure is practical.

Alternative methods of analyzing safety without relying solely on crash data are found in a study by Perkins and Harris (Perkins & Harris, 1968) who first proposed the concept of traffic conflicts.

Surrogate measures have therefore been in use for the past 5 decades to supplement or even sometimes used a substitute for crash data in transportation safety evaluation. The Federal Highway Administration has described several safety surrogate measures used in modeling safety including Time-to-Collision (TTC), Post-Encroachment Time (PET), Deceleration Rate (DR), Gap Time (GT), and Proportion of Stopping Distance (PSD) (Gettman & Head, 2003).

Surrogate measures have seen limited use in freeway segments while studies have mainly been using deterministic and experimental queue theories along with incident durations and secondary crash data to perform safety analysis with some success (Chimba & Kutela, 2014; C. Wang & Stamatiadis, 2014). However, crash data are only representative of past events and changes in traffic flows and volumes can affect the expected number of crashes in unpredictable ways (Dijkstra et al., 2010). Safety surrogate measures provide a means for possibly a much more effective way of analyzing the safety of a traffic measure.

Conflicts Validity and Crash Prediction

The relationship between traffic conflicts from simulation models and real-world traffic conflicts and crashes has been addressed by a few researchers without a clear understanding. With crashes being impossible to simulate, given the programmed nature of vehicles in a microscopic simulation model, coupled with the lack of a real behavioral component in simulation (Dijkstra et al., 2010), the relationship becomes even more difficult. Some research has been done to investigate the relationship between conflicts generated from simulation models and real-world traffic conflicts (Dijkstra et al., 2010; Huang, Liu, & Li, 2011; Huang, Liu, Yu, & Wang, 2013) and even relationship to actual crashes (Dijkstra et al., 2010; Gettman, Pu, Sayed, & Shelby, 2008). This provides a foundation for prediction and quantification of conflicts into crashes, for more accurate safety evaluation.

In a study in Nanjing, China, the researchers compared simulated conflicts generated from SSAM using VISSIM trajectory files to field-measured conflicts at ten signalized intersections (Huang et al., 2013). Statistically significant relationships between simulated conflicts and the field-measured conflicts were observed. Overall, the results suggested a significant relationship exists between rear-end and crossing conflicts from the simulated models and the field measurements, with a less pronounced relationship for lane-change conflicts. Also, simulated conflicts were not found to be indicative of traffic conflicts from unexpected driving maneuvers like illegal lane-changes in the real world (Huang et al., 2013).

In a similar study also from Nanjing, China, at 6 yield-controlled freeway terminals, results showed a strong positive linear relationship between simulated and real-world conflicts. The results also suggested a close relationship exists between severe conflicts when compared to normal traffic conflicts. Further, it was shown that conflict types were not statistically different between simulated and field-measured conflicts (Huang et al., 2011).

In the Safety Surrogate Assessment Model (SSAM) validation report (Gettman et al., 2008), conflicts generated from traffic simulation models were compared among different software and actual crashes selected from 83 intersections from British Columbia, Canada. Four microscopic simulation software packages were used and results from each analyzed. Of most interest to this study among the validation tests carried out in the report, was one that compared conflicts generated from simulation models to actual crashes through a developed conflicts-based crash-prediction model. Despite having lower values than the volume-based crash-prediction model also developed in the report, the goodness-of-fit had a range similar to that found in traditional crash prediction models in previous studies with similar conditions (Bauer & Harwood, 2000). The difference in results, however, might have also been attributed to the difference in volumes used between the two models.

Objective

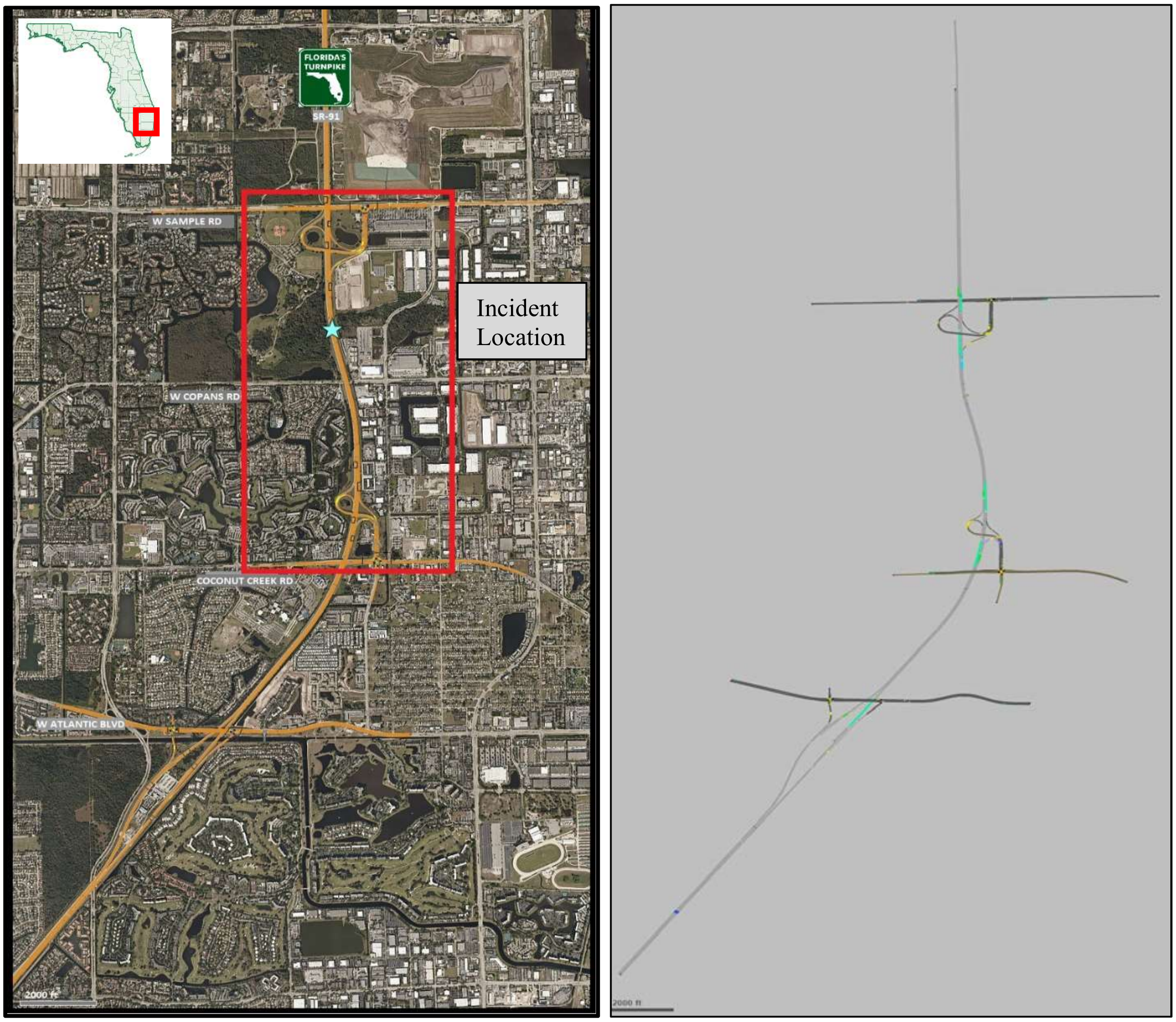
The objective of this study is to evaluate the potential benefits associated with the presence of CVs in the traffic stream in the reduction of SCs. In particular, the reduction of safety surrogate measures in the CV environment with both vehicle-to-vehicle (V2V) and vehicle-to-infrastructure (V2I) communications. Thus, this study presents an evaluation of traffic flow conditions in the event of an incident on a freeway segment in a CV environment. Traffic simulation was done in VISSIM microscopic simulation software and the safety evaluation conducted in the SSAM software.

CHAPTER 3 METHODOLOGY

Study Area

The study was conducted on Florida's system of toll roads, also known as Florida's Turnpike. The system consists of the Mainline from Miami to Central Florida, the Homestead Extension (HEFT), Sawgrass Expressway, Seminole Expressway, Beachline Expressway, Southern Connector Extension, Veteran's Expressway, Suncoast Parkway, Polk Parkway, Western Beltway and the I-4 Connector. The study area is located on the Turnpike Mainline, which serves traffic from the central part of the state in Orlando all the way to the southern part in Miami, partially running almost parallel to the I-95 Interstate Route.

The study model is a 7.8-mile road segment on Florida's Turnpike Mainline also known as SR-91. The freeway segment is in Broward County and currently has 3 lanes in one direction and intersects 4 roads namely, Sawgrass Expressway, Sample Road, Copans Road, and Atlantic Boulevard at intervals ranging from 1 to 2 miles, with interchanges at each of the crossings except at the Copans Road crossing. The site was chosen due to its relatively high number of crashes in the past year, 2018, compared to other segments along Florida's Turnpike, based on Signal Four Analytics data. Figure 2-2 shows the map of the study location and the simulation model for the study, with the simulated incident location in the marked region.



(a) (b)

Figure 3-1 (a) Study area along Florida's Turnpike Mainline (SR-91); (b) VISSIM Model

VISSIM Microscopic Simulation

Simulations conducted for traffic analysis are either macroscopic or microscopic in nature. In macroscopic models, traffic processes are described with aggregate quantities like flow and density, whereas microscopic models analyze the behavior of individual entities as they react to surrounding traffic and environment in general. Microscopic analysis entails the use of computer models to reproduce stochastically real traffic flow from transportation facilities (FDOT Systems Planning Office, 2014). Microscopic models use input information such as traffic volume, facility type, and vehicle-driver characteristics, to move vehicles in a split-second or time-step basis through simple gap acceptance, acceleration, and lane change rules. The models cannot optimize traffic signals as macroscopic models but rather focus on analyzing the complex congested traffic conditions, especially in urban areas, giving outputs per individual vehicle performances.

VISSIM microscopic simulation tool is a powerful multi-modal modeling software with capabilities in each of the traffic modes including cars, transit, heavy vehicles, and even pedestrians. It can also be used to model toll lanes, freeway merge/diverge and weaving segments as well as exclusive lanes. VISSIM was selected in this study due to its strong capabilities in modeling incidents or blocked lanes which enables the creation of a secondary crash environment. It is also possible to model complex traffic conditions in VISSIM such as a CV environment, using external modules such as the component object model (COM). Further, VISSIM satisfies all requirements for CV communications as specified in SAEJ2735 (Hyungjun Park et al., 2011). A model of the freeway segment was created in VISSIM using roadway characteristics and calibrated to represent real-world freeway operations.

The Calibration process was achieved by following guidelines presented in the Traffic Analysis Handbook (FDOT Systems Planning Office, 2014) provided by the Florida Department of Transportation Systems Planning Office, which is described in the model calibration segment of this thesis. The handbook also supports the selection of VISSIM as an analysis tool for operations on freeways or limited access highway facilities with higher detail and accuracy with contrast other tools as shown in Figure 2-1.

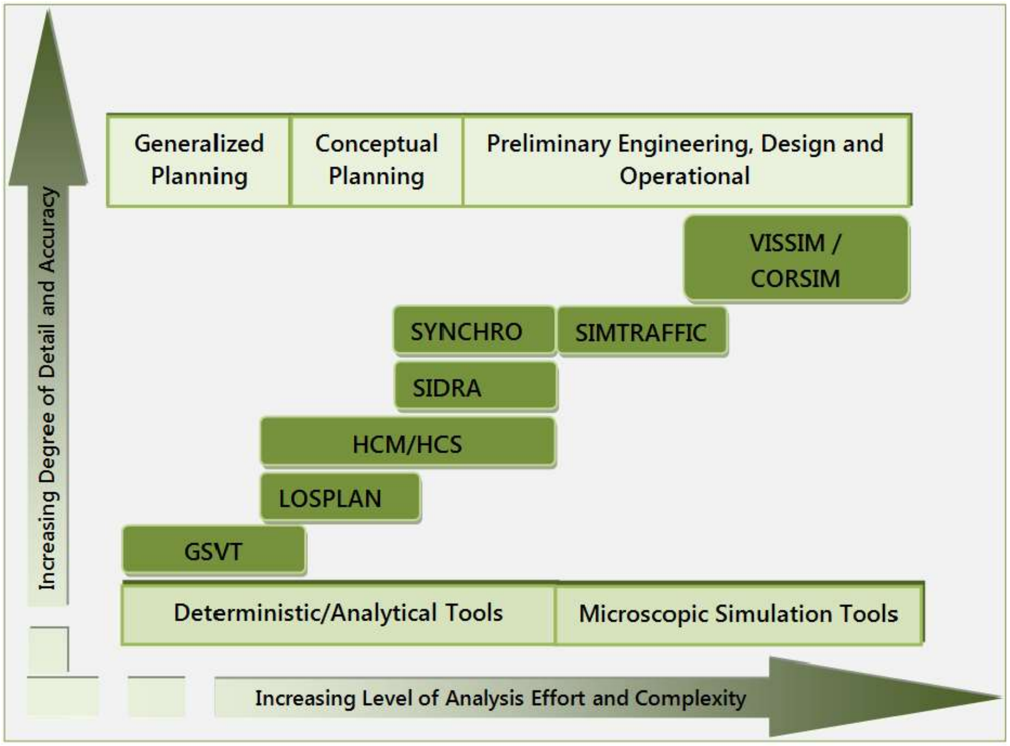


Figure 3-2: Traffic Analysis Tools (FDOT Systems Planning Office, 2014)

Simulation Test Bed and Data Inputs

VISSIM Model Calibration and Validation

This segment documents the model development and calibration efforts of the VISSIM microsimulation model used in this study. The VISSIM model was obtained from Florida's Turnpike authority as part of the report for model development and calibration process for the existing 2016 AM and PM peak conditions for the SW 10th Street project in Broward County. The process covered the general steps as depicted in Figure 3-3, which was extracted from the guide developed by the FHWA (Dowling, Skabardonis, & Alexiadis, 2004).

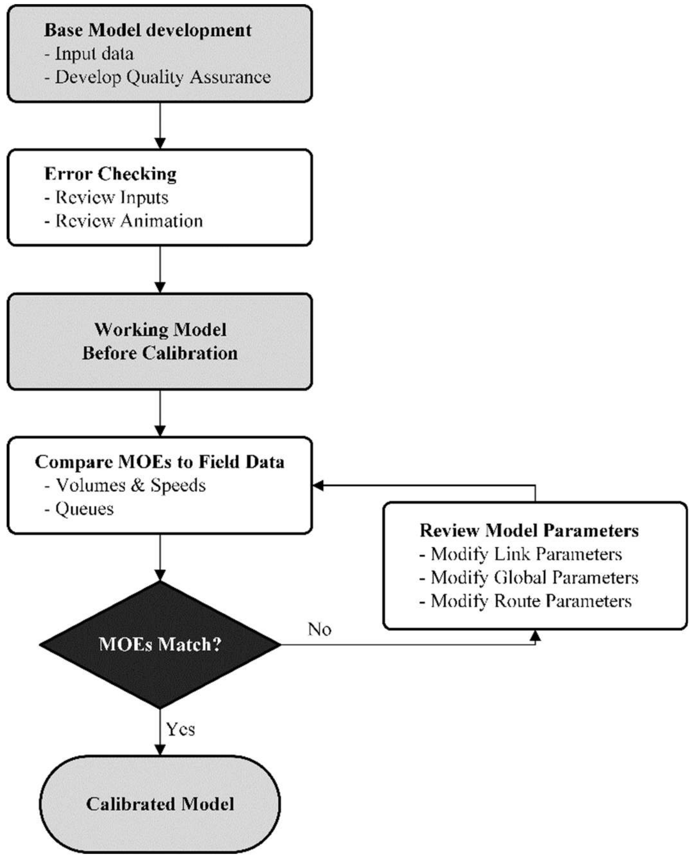


Figure 3-3: FHWA Initial Modeling & Calibration Process (Dowling et al., 2004)

The calibration process also followed the guidelines provided in the Florida Department of Transportation (FDOT) Traffic Analysis Handbook: A Reference for Planning and Operations. The calibration was generally achieved by changing model parameters to replicate results obtained in the report provided by FDOT.

Data Collection

The data collection process can be described as one of the most time and resource-consuming components of an analytical study. As such, it is important to identify the essential data that will be needed for the study and plan for resources accordingly. The following are some basic requirements in data collection for an effective microscopic analytical study as proposed by the FHWA (Dowling et al., 2004):

- Use of data that are measurable in the field.
- Quality and quantity of data influence analysis.
- Required analytical accuracy drives the quantity collected.
- Use data that are relatively recent.
- Use data that are time-variant.
- Use contemporaneous data.

For this study, the data used were taken from the report by the FDOT during the model development and calibration report for the existing 2016 AM and PM peak conditions for the SW 10^{th} Street project. All data collected in the report were gathered in accordance with the FHWA's Traffic Analysis Toolbox Volume III: Guidelines for Applying Traffic Microsimulation Modeling Software.

Base Model Development and Verification

The existing VISSIM network that was used to develop the model in this study was developed by The Florida Department of Transportation, from previously developed models for Interstate 95 (I-95) and the Sawgrass Expressway. The SW 10^{th} Street of the segment was then added to the model using 2016 aerial imagery. Roadway features and corresponding dimensions were also extracted from the aerial imagery and verified on site. The modeled limits of the arterials extended 0.5 miles outside of the construction project limits to capture the extent of real-world queues in the modeled network. Figure 3-4 shows the overall project model as well as the study segment selected for this study.

VISUM software was used in the original model development to transfer and refine origin-destination information into VISSIM. Bluetooth data was then used to validate the AM and PM peak conditions estimated for the existing conditions. The original model is as shown in Figure 3-4 with an outline showing the area that was selected for this study.

The morning and evening peak hour periods that were analyzed in the study were from 6:30-9:30 AM and 4:00-7:00 PM. The morning peak hour was from 7:30-8:30 AM while the evening peak hour was from 5:00-6:00 PM. To develop the buildup and dissipation of congestion during the peak period, an hour of simulation time was added prior to the peak hour and after the peak hour respectively. In addition to the buildup and dissipation times, additional 30 minutes were added as seeding time which loaded the network to equilibrium between entering and exiting vehicles. The total simulation time was, therefore, 6:00-9:30 AM and 3:30-7:00 PM for the morning and evening peak hour periods respectively. Table 3-1 shows the splits in total simulation time as well as the hourly conversion factor in percent for the period splits.

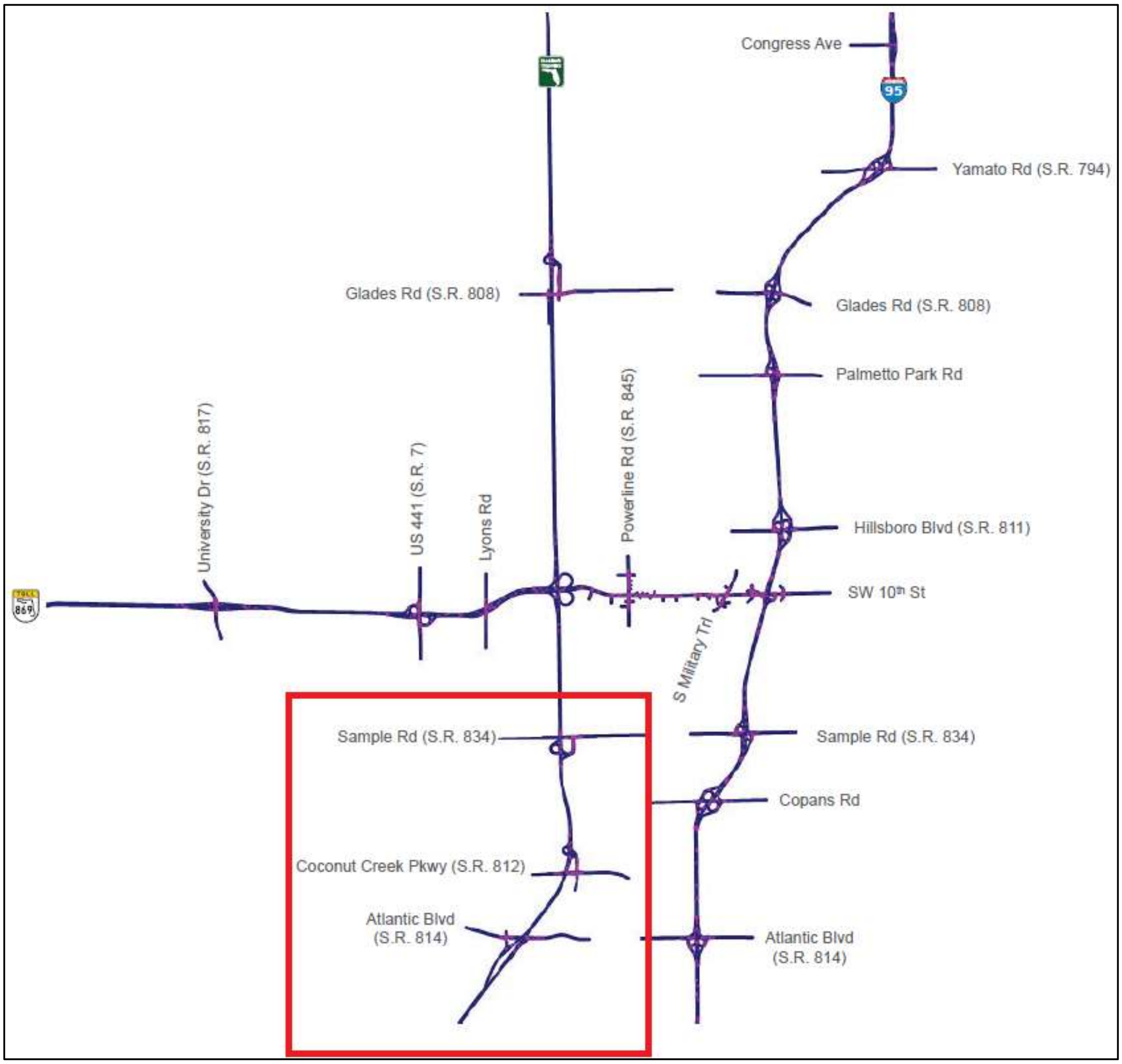


Figure 3-4: VISSIM Model Study Area

Table 3-1: Hourly Volume Conversion Factor

	Simulation Time (Seconds)	AM Condition		PM Condition	
		15 minutes	Hourly	15 minutes	Hourly
Seed Time	0 - 900	9.38%	22.07%	22.08%	45.34%
	900 - 1800	12.69%		23.26%	
Pre-Peak Hour	1800 - 2700	16.57%	81.55%	22.37%	92.31%
	2700 - 3600	19.38%		22.92%	
	3600 - 4500	21.29%		23.20%	
	4500 - 5400	24.31%		23.82%	
Peak Hour	5400 - 6300	25.50%	100.00%	24.25%	100.00%
	6300 - 7200	25.32%		25.20%	
	7200 - 8100	24.74%		25.39%	
	8100 - 9000	24.44%		25.17%	
Post-Peak Hour	9000 - 9900	23.60%	87.19%	24.44%	92.82%
	9900 - 10800	22.38%		24.07%	
	10800 - 11700	20.74%		22.83%	
	11700 - 12600	20.47%		21.48%	

Number of Simulation Runs

VISSIM uses random seed numbers in performing simulation runs, to reflect the stochastic nature of traffic flow. The random seed value initiates a random number generator that assigns a unique seed number to a simulation run. The random seeds helps vary properties assigned to individual vehicles entering the network such as: the decision on the vehicle type entering the network, the time a vehicle enters a network, the lane assigned to a vehicle entering the network, the aggressiveness of the driving behavior, and the type of interaction once the vehicle is in the network (Russo, 2008). Random seeding facilitates the replication of stochastic behaviors and patterns that are observed in the real-world traffic flow in the VISSIM simulation model. This results in the variation of results from simulation results in which VISSIM calculates additional meaningful values for result attributes in evaluations such as minimum and maximum values and means.

Although 10 simulation runs are considered adequate by the FDOT Traffic Analysis Handbook (FDOT Systems Planning Office, 2014), it is almost impossible to determine the number of simulation runs to be performed for meaningful statistical analysis and conclusions of the results without some kind of test. The following formula, recommended by the Traffic Analysis Handbook, was thus used to determine the number of simulation runs to be carried out for the microscopic study.

$$n = \left[\frac{s * t_{\alpha/2}}{\mu * \varepsilon}\right]^2 \qquad \text{(Eq. 1)}$$

Where:

n - the required number of simulation runs,

s - the standard deviation of the system performance measure (based on previously conducted simulation runs),

$t_{\alpha/2}$ - the critical value of a two-sided Student's t-statistic, at the level of confidence α and n-1 degrees of freedom,

μ - the mean of the system performance measure, and

ε - the tolerable error, specified as a fraction of the μ.

To estimate the sample standard deviation for use in determining the required number of simulation runs, preliminary simulation runs were carried out. The selected performance measure for the estimation of the standard deviation was the speed of vehicles in the network. Preliminary simulation runs were thus carried out for 10 repetitions and the speeds, as well as the standard deviation, were determined. In the estimation of the standard deviation, the 95% confidence level was used as recommended in the study by Russo (Russo, 2008).

A different seed number was used for performing a total of 10 simulation runs with 10 total corresponding seed numbers and the average speeds on the Turnpike mainline freeway and arterial routes were evaluated as shown in Table 3-2.

Table 3-2: Average Performance Measures from Preliminary Simulation Runs

Simulation Run	Seed Number	Average		
		Speed (mph)	Volume (veh/h)	Density (veh/mi/ln)
1	10	51.91	2469	46.02
2	15	52.00	2470	45.94
3	20	51.99	2474	45.91
4	25	51.91	2475	46.17
5	30	52.02	2463	45.70
6	35	51.99	2373	46.12
7	40	51.94	2479	46.20
8	45	51.85	2469	46.19
9	50	51.98	2474	46.00
10	55	52.01	2478	46.11
Average		51.96	2462	46.04
Standard deviation		0.05	30	0.15
Maximum		52.02	2479	46.2
Minimum		51.85	2373	45.7

A confidence level was determined at the 95% confidence level ($\alpha = 0.05$). With 9 degrees of freedom, the standard deviation S for the average speeds of vehicles along the mainline route, and from the statistic table $t_{\alpha/2}$ was obtained as 2.26. Using the value of 10% as the error tolerance, as recommended by the Traffic Analysis Handbook, the number of simulation runs computed using Equation 1 was found to be less than 5. Since the computed number of required simulation runs were quite low, the value used was chosen to correspond with that recommended in the Traffic Analysis Handbook by the Florida Department of Transportation (FDOT).

Error Checking

This step of the microsimulation analysis process is important in developing a working model to ensure that the calibration process that will follow does not result into distorted model parameters that compensate for the unaccounted-for coding errors. The calibration heavily relies on the elimination of all errors in model network coding and demand coding.

According to the FHWA (Dowling et al., 2004), the error checking process follows the checklist in Table 3-3, involving various reviews of the coded network, demand, and default parameters, in the following three stages:

1. Review of software errors
2. Review input coding errors
3. View animation to spot less obvious errors

It is also recommended that residual errors be checked when the simulation model still does not perform to the analyst's satisfaction with respect to the field conditions. The residual errors may sometimes be a result of analyst's expectations surpassing the capabilities of the software or an existing software error.

Table 3-3: Model Verification (Error Checking) Process Checklist

Error Type	Description	Check
Software	Verify no runtime or syntax error occurs in the Protocol Window	
	Review the error file (.err) for any errors or runtime warnings that affect simulation results	
	Review RBC errors or warnings	
Model run parameters	Review the temporal boundary limit to confirm it matches the approved methodology	
	Verify initialization period is at least equal to twice the time to travel the entire network	
Network	Verify the spatial boundary limit against the approved methodology	
	Check basic network connectivity.	
	Verify the background image has been properly scaled	
	Verify link geometry matches lane schematics	
	Check link types for appropriate behavior parameters	
	Check for prohibited turns, lane closures and lane restrictions at intersections and on links	
	Check and verify traffic characteristics on special use lanes against general use lanes	
Demand and routing	Verify coded volume and vehicle mix/traffic composition	
	Check HOV vehicle type and occupancy distribution as appropriate	
	Check routing decision including connector look back distances	
	Verify O-D matrices and their placement in the network	
Control	Check and verify the intersection control type and data are properly coded. Verify vehicles are reacting properly to the controls	
	Check ramp meter control type and data	
	Check conflict area settings	
Traffic operations and management data	Verify bus operations—routes, dwell time	
	Check parking operations	
	Verify pedestrian operations and delays	
Driver and Vehicle characteristics	Check if driver behavior adjustments are necessary in saturated conditions	
	Verify no lane changes occur in unrealistic locations and vehicles make necessary lane changes upstream in the appropriate location	
	Verify average travel speed reasonably match field conditions	
Animation	Review network animation with the model run at low demand levels—check for unrealistic operational characteristics such as congestion and erratic vehicle behaviors	
	Review reasonableness of the model against data coding, route assignment, and lane utilization	
	Compare model animation to field characteristics	
	Verify all turn bays are fully utilized and they are not blocked by through vehicles	
	Verify there are no vehicles turning at inappropriate time or locations	

The software errors may be identified with careful review of the software documentation. An alternative software can be used in place of the current erroneous software or with advanced skills, the analyst may develop their own application programming interface to produce the desired software performance.

After successful error checking the analyst can then proceed to the model calibration after what is termed as a key decision point, where all input data and parameter values are checked for correctness and the animation performing as expected based on the analyst's judgement.

The following were the results that were yield after carrying out the recommended three stages of error checking:

Software errors

Th latest software was used for the analysis and no software errors were found after review of the VISSIM software documentation and other material from the user groups. No known errors or bugs were reported related to the study network and the scenarios in the analysis.

Review of Input Data and Parameters

First, basic network objects were checked for consistency with the original model used for the base model development, as well as the current site from google earth pro. All coded geometry and turning movements were checked and errors corrected. The major errors corrected were misalignment of turning movement links and sharp transitions between a few links.

Static network displays were reviewed including lane numbers, lane behavior type displays and lane drop locations. The consistency of link attributes including freeway and arterial behaviors and speed decisions, was checked and confirmed to be as coded in the report.

Traffic demand was thereafter checked to ensure the input demand volumes at each link entrance was as defined in the field values provided in the traffic volume tables given in the original model validation report. Traffic Signals were also reviewed carefully including the signal timing and phases used. These were found to be correct and all signal timing files (.vissig) were correctly referenced to the correct signal controllers. Further, all vehicle parameters were reviewed including vehicle types, classes and inputs with some 3d model of vehicles found to be obsolete or missing and hence the standard 3d models were adopted for these.

Review Animation

The simulation animation was first run with reduced vehicle inputs to ensure vehicles traveled smoothly over the network and to check for any unrealistic or unexpected movements of vehicles. Minor alignment errors were detected in this step, which were adjusted accordingly.

The traffic demands were then increased to 50% of the volume inputs as the animation was being reviewed to check for any errors. In reviewing the animation in this stage some coding errors in the cash toll lanes were uncovered, where vehicles were unrealistic lane changes were being made by vehicles from the Sunpass™ only lanes (electronic tolling) to the cash only lanes at undesirable locations and thus causing bottlenecks. This was corrected by increasing the lane change distance for the desired route as well as the emergency stop distance for the said segments and another animation review did not result in this error.

Key Decision Point

The revised model was finally run with actual vehicle input data and default model parameters and the output and animations reviewed. Review results were compared to reported outputs and a conclusion was made that the model was working as expected.

Model Calibration and Validation

Model calibration was done in accordance with the guidelines in the Traffic Analysis Handbook. The model calibration targets were taken as defined in Table 2-4, from the handbook, and were achieved by varying the working model parameters that are described in the following subsections.

Table 3-4: Model Calibration Targets

Calibration item	Calibration Target/Goal
Capacity	Simulated capacity to be within 10% of the field measurements.
Traffic Volume	Simulated and measured link volumes for more than 85% of the links to be: Within 100 vph for volumes less than 700 vph Within 15% for volumes between 700 vph and 2700 vph Within 400 vph for volumes greater than 2700 vph.
	Simulated and measured link volumes for more than 85% of links to have a GEH* statistic value of five (5) or lower.
	Sum of link volumes within the calibration area to be within 5%.
	Sum of link volumes to have a GEH* statistic value of 5 or lower.
Travel Time (includes Transit)	Simulated travel time within ±1 minute for routes with observed travel times less than seven (7) minutes for the routes identified in the data collection plan.
	Simulated travel time within ±15% for routes with observed travel times greater than seven (7) minutes for the routes identified in the data collection plan.
Speed	Modeled average link speeds to be within the ±10 mph of field-measured speeds on at least 85% of the network links.
Intersection Delay	Simulated and field-measured link delay times to be within 15% for more than 85% of cases.
Queue Length	Difference between simulated and observed queue lengths to be within 20%.
Visualization	Check consistency with field conditions of the following: on-ramp and off-ramp queuing; weaving maneuvers; patterns and extent of queue at intersection and congested links; lane utilization/choice; locations of bottlenecks; etc.
	Verify there are no unrealistic U-turns or vehicles exiting and reentering the network.

*GEH is an empirical formula expressed as $\sqrt{2*(M-C)^2/(M+C)}$ where M is the simulation model volume and C is the field counted volume.

It is also essential that the calibration process focus more on adjusting model parameters that are pertinent to the study objective, and thus more likely to influence the performance measures of effectiveness (FDOT Systems Planning Office, 2014). Focusing on adjusting such parameters reduces the overall time required to calibrate the model.

It could also be beneficial to categorize adjustable parameters into those that directly affect capacity and those that affect route choice. The process was thus done by checking those parameters that showed observed changes in the performance measures, which were discussed in detail in the following sections. Since model calibration involves an iterative process, the Florida Analysis handbook recommends that a good practical strategy is to divide the calibration process into two basic categories that can be separately catered for, namely:

1. Parameters that the analyst is sure about and does not wish to change, and
2. Parameters that the analyst is less certain about and is willing to adjust.

The first category represents value parameters that are measured directly from the field and used as base model inputs, such as vehicle lengths. This also includes values that can be taken from previous analyses and are applicable to the study. Further, this category includes parameters that have little influence on calibrations measures of effectiveness.

The latter category includes only those parameters that have a medium to higher levels of sensitivity to the calibration measures of effectiveness.

Driver Behavior Parameters Calibration

The VISSIM model parameters that are used in the calibration process are grouped as either vehicle following, or lane change which describe the psycho-physical longitudinal movements and rulebased lateral vehicle movements respectively. As such, VISSIM uses two car-following models for freeways and arterials separately, the Wiedemann 99 and the Wiedemann 74, respectively. The behaviors were initially developed from the research done by Rainer Wiedemann in 1974 in Germany. The driver behaviors that were modified in the calibration process were the car following behavior and the lane change behavior. The lateral behaviors were found to be consistent with the real-world behavior at default settings, hence not changed.

Lane-Changing Behavior

VISSIM has been reported to generate several simulated crashes due to some modeling limitations in the lane-changing behavior especially for vehicles in queues (Gettman et al., 2008). During the simulations performed in this study such conflicts were also observed during visual inspections of queued vehicles. Vehicles were observed to make abrupt lane-changes, which led to a several observable simulated crashes. No clear justification was found for the abnormal behavior, however, the following measures suggested by PTV (PTV AG, 2018) and Gettman (Gettman et al., 2008) were taken to minimize the undesired behavior:

- First, the driver behavior parameter for lateral clearance was adjusted by an additional 0.5 s to improve the lane-change characteristics. This parameter represents the minimum distance for vehicles overtaking within the same lane. This change decreases the simulated conflicts due to lane changing but also decreases the capacity of road segment in simulation.

- Secondly, in VISSIM the two types of lane changes can either be necessary or free lane changes. Necessary lane changes can be limited by changing parameters by changing the maximum acceptable decelerations as well as the emergency stop distance for routes through link connectors. The lane change distance defined in the link connectors also have a great effect and were changed to a smaller value for the next upstream route to limit lane changes along the location where lane changes were not desired. Free lane changes in VISSIM are guided by the safety distance computed for a trailing vehicle on the own vehicle's desired new lane, which highly depends on the vehicle speeds. The aggressiveness of free lane changes cannot be currently changed however, by changing the following safety distance, the free lane changes can be slightly minimized where desired.

Car Following Parameters Calibration

In the following parameters, the major parameters include the look-ahead distance, number of interaction objects, number of interaction vehicles, look back distance and the temporary lack of attention. Other less predominant parameters are standstill distance for static objects, enforce absolute braking distance and implicit stochastics. For the purpose of keeping this document brief, only the major parameters are described, and the reader is referred to the VISSIM manual(PTV AG, 2018) for further detailed descriptions.

Look back distance: This is defined as the maximum and minimum distance that a vehicle can observe behind it so as to adjust its behavior accordingly. The minimum lookback distance plays a major role when modeling lateral behavior. The calibrated value for the maximum was left at the default for this parameter at 492.13 feet whereas the minimum was 0.00.

Look ahead distance: Like the lookback distance, the look-ahead distance defines how far a vehicle can see ahead in order to react to other vehicles ahead or adjacent to it on the same link. This parameter is taken into consideration along with the entered number of interaction vehicles. The look-ahead distance values that were found to be consistent with field conditions in the calibration process were 820.21 and 0.00 for maximum and minimum values respectively.

Number of interaction objects: This parameter depicts how many objects or vehicles are observed by a vehicle in conjunction with the minimum and maximum look-ahead distances. In VISSIM these interaction objects are modeled as a preceding vehicle to an observing vehicle. Interaction objects can be red signal heads, reduced speed areas, priority rules, stop signs, public transport stops and parking lots. The value for this parameter was set to 4 in the calibration efforts of the study.

Further, Table 3-5 gives calibration parameter ranges as given by the FHWA's Traffic Analysis Toolbox Volume III (Dowling et al., 2004).

Table 3-5: VISSIM Model Calibration Parameters

Calibration Parameter	**Default Value**	**Suggested Range**	
		Basic Segment	**Weaving/Merge/ Diverge**
Freeway Car Following (Wiedemann 99)			
CCO Standstill distance	4.92 ft	>4.00 ft	>4.92 ft
CC1 Headway time	0.9 s	0.70 to 3.00 s	0.9 to 3.0s
CC2 'Following' variation	13.12 ft	6.56 to 22.97 ft	13.12 to 39.37ft
CC3 Threshold for entering 'following'	-8	use default	
CC4 Negative 'following' threshold	-0.35	use default	
CC5 Positive 'following' threshold	0.35	use default	
CC6 Speed Dependency of oscillation	11.44	use default	
CC7 Oscillation acceleration	0.82 ft/s2	use default	
CC8 Standstill acceleration	11.48 ft/s2	use default	
CC9 Acceleration at 50 mph	4.92 ft/s2	use default	
Arterial Car Following (Wiedemann 74)			
Average standstill distance	6.56 ft	>3.28 ft	
Additive part of safety distance	2.00	1 to 3.5[i]	
Multiplicative part of safety distance	3.00	2.00 to 4.500[i]	
Lane Change			
Maximum deceleration	-13.12 ft/s^2 (Own) -9.84 ft/s^2 (Trail)	< -12 ft/s2 < -8 ft/s2	
-1 ft/s2 per distance	200 ft (Freeway) 100 ft (Arterial)	>100 ft >50 ft	
Accepted deceleration	-3.28 ft/s^2 (Own) -1.64 ft/s^2 (Trail)	<-2.5 ft/s2 <-1.5 ft/s2	
Waiting time before diffusion	60 s	Use default	
Min. headway (front/rear)	1.64 ft	1.5 to 6 ft	
Safety distance reduction factor	0.6	0.1 to 0.9	
Max. dec. for cooperative braking	-9.84 ft/s^2	-32.2 to -3 ft/s2	
Overtake reduced speed areas	Depends on field observations		
Advanced Merging	checked		
Emergency stop	16.4 ft	Depends on field observations	
Lane change	656.2 ft	>656.2 feet	
Reduction factor for changing lanes before signal	0.6	default	
Cooperative lane change	Unchecked	Checked especially for freeway merge/diverge areas	

[i]The relationship should be based on the User Manual i.e. Multiplicative = Additive+1

After the development of the working calibrated model, the measures of effectiveness were compared to those reported in the report by FDOT's calibration efforts of the SW 10th Street in order to perform model calibration. The selected measure of effectiveness was the traffic volume as recommended by FDOT (FDOT Systems Planning Office, 2014). The calibrated model was initially validated by using independent datasets, then used to create multiple scenarios of connected vehicles (CV) by varying vehicle compositions, driving behaviors, and link behavior types in the model to achieve the CV environment as described in CV behavior calibration section of this report.

Unmet Demand at Entry Links Check

An essential assessment is to identify whether the expected vehicular demand can be processed at the network entry links. If the model outputs indicated that substantial demand was not able to enter the network. the lengths of the entry links with unmet demand were extended to store more vehicles. If unmet demand was still reported, the driver behavior was adjusted for the link(s) that reported issues. Simulation results performed after calibration indicated there was an unmet demand of only one or two vehicles for both periods.

Mainline and Ramp vehicles processed

Calibration results for the AM peak period and the PM peak period for the mainline freeway are shown in Tables 3-6 and 3-7, respectively. Further, Tables 3-8 and 3-9 show calibration results for the AM peak period and the PM peak period for ramps, respectively. The existing conditions volume calibration results are summarized for the AM and PM peak period models as follows:

- *Calibrated Existing 2016 AM Model* – calibration target for the sum of the mainline and ramp link flows is achieved for 100.0 percent of cases.

- *Calibrated Existing 2016 AM Model* – GEH targets (<5) for individual mainline and ramp link flows are achieved for 100.0 percent of cases.

- *Calibrated Existing 2016 PM Model* – Calibration target for the sum of the mainline and ramp link flows is achieved for 100.0 percent of cases.

- *Calibrated Existing 2016 PM Model* – GEH targets (< 5) for individual mainline and ramp link flows are achieved for 100.0 percent of cases.

The results described above indicate that the existing VISSIM models satisfy the volume calibration criteria.

Table 3-6: Freeway Volumes – AM Peak Hour

Location	**Demand Volume**	**Model Volume**	**GEH**	**Location**	**Demand Volume**	**Model Volume**	**GEH**
Florida's Turnpike Northbound				Florida's Turnpike Southbound			
Before Atlantic Boulevard off-ramp	6,090	6,089	0.0	After on-ramp from Sawgrass Expressway	5,460	5,453	0.09
After Atlantic Boulevard off-ramp	4,860	4,856	0.1	After on-ramp from Sample Road	5,740	5,718	0.29
After on-ramp from Coconut Creek Road	4,810	4,802	0.1	After on-ramp from Coconut Creek Road	4,910	4,921	0.16
After on-ramp from Sample Road	4,160	4,108	0.8	After on-ramp from Atlantic Boulevard	5,840	5,749	1.2

Table 3-7: Freeway Volumes – PM Peak Hour

Location	**Demand Volume**	**Model Volume**	**GEH**	**Location**	**Demand Volume**	**Model Volume**	**GEH**
Florida's Turnpike Northbound				Florida's Turnpike Southbound			
Before Atlantic Boulevard off-ramp	5,720	5,609	1.5	After on-ramp from Sawgrass Expressway	3,980	3,975	0.08
After Atlantic Boulevard off-ramp	4,830	4,528	4.4	After on-ramp from Sample Road	4,610	4,563	0.69
After on-ramp from Coconut Creek Road	5,560	5,238	4.4	After on-ramp from Coconut Creek Road	4,660	4,501	2.35
After on-ramp from Sample Road	5,140	5,086	0.8	After on-ramp from Atlantic Boulevard	5,900	5,658	3.18

Table 3-8: Ramp Volumes – AM Peak Hour

Location	**Demand Volume**	**Model Volume**	**GEH**	**Location**	**Demand Volume**	**Model Volume**	**GEH**
Florida's Turnpike Northbound				Florida's Turnpike Southbound			
Off-ramp to Atlantic Boulevard	1,230	1,233	0.1	On-ramp from Atlantic Boulevard	930	904	0.86
Off-ramp to Coconut Creek Parkway	710	709	0.0	Off-ramp to Coconut Creek Parkway	1,230	1,198	0.92
On-ramp from Coconut Creek Parkway	660	650	0.4	On-ramp from Coconut Creek Parkway	400	392	0.4
Off-ramp to Sample Road	1,200	1,225	0.7	Off-ramp to Sample Road	690	706	0.61
On-ramp from Sample Road	550	539	0.5	On-ramp from Sample Road	970	958	0.39

Table 3-9: Ramp Volumes – PM Peak Hour

Location	**Demand Volume**	**Model Volume**	**GEH**	**Location**	**Demand Volume**	**Model Volume**	**GEH**
Florida's Turnpike Northbound				Florida's Turnpike Southbound			
Off-ramp to Atlantic Boulevard	890	851	1.3	On-ramp from Atlantic Boulevard	1,240	1,214	0.74
Off-ramp to Coconut Creek Parkway	400	349	2.6	Off-ramp to Coconut Creek Parkway	620	603	0.69
On-ramp from Coconut Creek Parkway	1,130	986	4.4	On-ramp from Coconut Creek Parkway	670	579	3.64
Off-ramp to Sample Road	1,060	1,004	1.7	Off-ramp to Sample Road	400	406	0.3
On-ramp from Sample Road	640	612	1.1	On-ramp from Sample Road	1,030	1,020	0.31

Model Calibration Results

The model calibration procedures carried out in the study resulted in calibration parameters that are summarized in Table 2-10 and Table 2-11. It is worth noting that since most of the parameters were adopted from the report for model development and calibration process for the existing 2016 AM and PM peak conditions for the SW 10th Street project, the values presented in Tables 3-6 to 3-9 are either exactly as seen in the report or closely matched. Table 3-10 presents the calibration parameters for the Freeway calibration, whereas Table 3-11 presents the arterial calibration parameters calibration range.

Table 3-10: Model Freeway Calibration Parameters

<table>
<tr><th colspan="2">Lane Change Parameters</th><th>Default</th><th>Freeway Calibration Parameters</th></tr>
<tr><td colspan="4">Necessary Lane Change (Route)</td></tr>
<tr><td colspan="2">Maximum deceleration</td><td>-13.12 ft/s2 (Own)
-9.84 ft/s2 (Trail)</td><td>-13.12 ft/s2
-9.84 ft/s2</td></tr>
<tr><td colspan="2">-1 ft/s2 per distance</td><td>200 ft (Freeway)</td><td>200 ft</td></tr>
<tr><td colspan="2">Accepted deceleration</td><td>-3.28 ft/s2 (Own)
-1.64 ft/s2 (Trail)</td><td>-3.28 ft/s2
-1.64 ft/s2</td></tr>
<tr><td colspan="2">Waiting time before diffusion</td><td>60 s</td><td>180</td></tr>
<tr><td colspan="2">Min. headway (front/rear)</td><td>1.64 ft</td><td>0.98 and 1.51 ft</td></tr>
<tr><td colspan="2">To Slower Lane if Collision Time Above (seconds)</td><td>0.00</td><td>0.00</td></tr>
<tr><td colspan="2">Safety distance reduction factor</td><td>0.6</td><td>0.25 and 0.40</td></tr>
<tr><td colspan="2">Max. decel. for cooperative braking</td><td>-9.84 ft/s2</td><td>-29.99 and -31.99 ft/s2</td></tr>
<tr><td colspan="2">Overtake reduced speed areas</td><td>Uncheck</td><td>Checked</td></tr>
<tr><td colspan="2">Advanced Merging</td><td>Checked</td><td>Checked</td></tr>
<tr><td colspan="2">Cooperative lane change</td><td>Unchecked</td><td>Checked especially for freeway merge/diverge areas</td></tr>
<tr><td rowspan="2">If Checked</td><td>Max Speed Difference</td><td>6.71 mph</td><td>6.71mph</td></tr>
<tr><td>Max Collision Time</td><td>10 sec</td><td>10 sec</td></tr>
</table>

Table 3-11: Arterial Calibration Parameters

<table>
<tr><th colspan="2">Lane Change Parameters</th><th>Default</th><th>Arterial Calibration Parameters</th></tr>
<tr><td colspan="4">Necessary Lane Change (Route)</td></tr>
<tr><td colspan="2">Maximum deceleration</td><td>-13.12 ft/s2 (Own)
-9.84 ft/s2 (Trail)</td><td>-13.12 ft/s2
-9.84 ft/s2</td></tr>
<tr><td colspan="2">-1 ft/s2 per distance</td><td>100 ft (Arterial)</td><td>100 ft</td></tr>
<tr><td colspan="2">Accepted deceleration</td><td>-3.28 ft/s2 (Own)
-3.28 ft/s2 (Trail)</td><td>-3.28 ft/s2
-3.28 ft/s2</td></tr>
<tr><td colspan="2">Waiting time before diffusion</td><td>60 s</td><td>180</td></tr>
<tr><td colspan="2">Min. headway (front/rear)</td><td>1.64 ft</td><td>1.51 ft</td></tr>
<tr><td colspan="2">To Slower Lane if Collision Time Above (seconds)</td><td>0.00</td><td>0.00</td></tr>
<tr><td colspan="2">Safety distance reduction factor</td><td>0.6</td><td>0.25, 0.40, 0.50</td></tr>
<tr><td colspan="2">Max. dec. for cooperative braking</td><td>-9.84 ft/s2</td><td>-29.99 and -31.99 ft/s2</td></tr>
<tr><td colspan="2">Overtake reduced speed areas</td><td>Uncheck</td><td>Checked</td></tr>
<tr><td colspan="2">Advanced Merging</td><td>Checked</td><td>Checked</td></tr>
<tr><td colspan="2">Cooperative lane change</td><td>Unchecked</td><td>Checked</td></tr>
<tr><td rowspan="2">If Checked</td><td>Max Speed Difference</td><td>6.71 mph</td><td>6.71mph</td></tr>
<tr><td>Max Collision Time</td><td>10 sec</td><td>10 sec</td></tr>
</table>

CV environment modeling

To achieve the CV environment in the simulation model, COM API was found to be a useful tool. COM API enables a user to model traffic behavior and conditions during simulation using an external programming language. Visual Basic scripting language (VBS) was selected as the primary language for modeling in VISSIM using event-based scripts. A script was written in VBS to simulate a connected vehicle environment in the following steps:

1. Introducing of a stopped vehicle to simulate an incident on one lane
2. Real-time collection of vehicle data including the performance measures speed and travel time
3. Tracking real-time deterioration of the collected traffic performance measures

4. Sending warning messages, including speed advisory and lane change messages to vehicles upstream when traffic performance measures deteriorate

5. Termination of messages when performance measures are improved or end of the incident.

The performance measures that were tracked in traffic upstream and downstream of the incident location included speed, travel time and density. Upstream and downstream traffic detectors about 300 ft from the incident were used to compare the values of the measures and warning messages were sent when upstream traffic flow measures deteriorated. The warning messages instructed the vehicles to either change lanes only or reduce speed as well as change lanes. The messages were also sent assuming no latency therefore instantly received by connected vehicles downstream and upstream of the incident.

Lane change messages:

These messages were sent to all CVs within the communications range of 2 miles after the incident had occurred. Once received, vehicles' desired lanes were set to those not blocked by the incident making the vehicles change lanes once they found gaps on adjacent lanes.

Speed advisory messages:

Vehicles were only advised to reduce speeds once the average speed, travel time or density within 300 ft of the incident location deteriorated to 10% less than the normal values or more. Speed reductions were advised at 20 mph less than the speed limit 1 mile before the incident and 10 mph below the speed limit at 2 miles before the incident or further.

Incident modeling

Through the COM API, incident modeling was done by writing and running a VBS script to create an incident environment that can be evaluated for conflicts and as a result, secondary crash risk prediction as shown in Figure 2-5. The incident was simulated at about 1500 ft north of the W Copans Road crossings. Since it was not possible to simulate a real incident in VISSIM without an external program, the application of COM API was crucial. A vehicle was thus added to the facility after the simulation warm-up period to represent a crash on the outside lane at the chosen location, and removed after the simulated incident duration of 30 minutes all through the code script. The incident duration selected was in range with the mean duration of a freeway incident with a closed shoulder as provided in Exhibit 11-22 of the 2016 Highway Capacity Manual (HCM, 2016) presented here as Table 3-12.

Table 3-12: Mean duration of Freeway Incidents (HCM, 2016)

	Incident Severity Type				
Parameter	Shoulder Closed	1 Lane Closed	2 Lanes Closed	3 Lanes Closed	4+ Lanes Closed
Distribution (%)	75.4	19.6	3.1	1.9	0
Duration (mean)	34	34.6	53.6	67.9	67.9
Duration (std. dev.)	15.1	13.8	13.9	21.9	21.9
Duration (min.)	8.7	16	30.5	36	36
Duration (max.)	58	58.2	66.9	93.3	93.3

Figure 3-5 Incident modeled in VISSIM using COM API

Traffic Volumes

The traffic volumes used in this study were consistent with volumes recorded in the calibration report of the simulation model of the SW 10th Street in 2017 by the Florida Department of Transportation. These volumes represented traffic conditions of three different one-hour periods including the pre-peak hour, peak hour and the post-peak hour. The traffic volumes were given as 15-minute traffic flows for each of the periods stated for both the AM and PM peak periods. The traffic conditions are therefore representative of the conditions in 2016 in the study area. Table 2-13 provides a breakdown of volumes along the Mainline Turnpike and the ramp volumes on each on and off-ramp used in the model.

Table 3-13: Traffic Volumes used in the simulation model

Location	Demand Volume	Location	Demand Volume
AM Period			
Florida's Turnpike Northbound		**Florida's Turnpike and Southbound**	
Mainline before Atlantic Boulevard off-ramp	6,090	Mainline after on-ramp from Sawgrass Expressway	5,460
Mainline after Atlantic Boulevard off-ramp	4,860	Mainline after on-ramp from Sample Road	5,740
Mainline after on-ramp from Coconut Creek Road	4,810	Mainline after on-ramp from Coconut Creek Road	4,910
Mainline after on-ramp from Sample Road	4,160	Mainline after on-ramp from Atlantic Boulevard	5,840
Off-ramp to Atlantic Boulevard	1,230	On-ramp from Atlantic Boulevard	930
Off-ramp to Coconut Creek Parkway	710	Off-ramp to Coconut Creek Parkway	1,230
On-ramp from Coconut Creek Parkway	660	On-ramp from Coconut Creek Parkway	400
Off-ramp to Sample Road	1,200	Off-ramp to Sample Road	690
On-ramp from Sample Road	550	On-ramp from Sample Road	970
PM Period			
Location	**Demand Volume**	**Location**	**Demand Volume**
Florida's Turnpike Northbound		**Florida's Turnpike Southbound**	
Mainline before Atlantic Boulevard off-ramp	5,720	Mainline after on-ramp from Sawgrass Expressway	3,980
Mainline after Atlantic Boulevard off-ramp	4,830	Mainline after on-ramp from Sample Road	4,610
Mainline after on-ramp from Coconut Creek Road	5,560	Mainline after on-ramp from Coconut Creek Road	4,660
Mainline after on-ramp from Sample Road	5,140	Mainline after on-ramp from Atlantic Boulevard	5,900
Off-ramp to Atlantic Boulevard	890	On-ramp from Atlantic Boulevard	1,240
Off-ramp to Coconut Creek Parkway	400	Off-ramp to Coconut Creek Parkway	620
On-ramp from Coconut Creek Parkway	1,130	On-ramp from Coconut Creek Parkway	670
Off-ramp to Sample Road	1,060	Off-ramp to Sample Road	400
On-ramp from Sample Road	640	On-ramp from Sample Road	1,030

Further, Table 3-14 shows the distribution of traffic throughout the peak period during the pre-peak hour, peak hour and the post-peak hour as percentages of the peak hour traffic flow. It is important to note that the incident was modeled in the northbound direction of traffic, therefore, the volumes in the right-hand side of Table 2-13 should be considered when relating to conflicts in the analysis.

Table 3-14: Hourly volumes as a proportion of the peak hour volume

	% Hourly Volume	
	AM Period	PM Period
Pre-Peak Hour	81.55%	92.31%
Peak Hour	100.00%	100.00%
Post-Peak Hour	87.19%	92.82%

Safety Evaluation

Safety Surrogate Measures

The Surrogate Safety Assessment Model uses developed algorithms to identify conflicts from vehicle trajectory files developed in traffic microscopic simulation software. It is a computationally intense task that may require moderate to long processing periods, depending on the size of the trajectory file which is, in turn, a function of the number of vehicles in the network.

Thresholds for analyzed surrogate measures can be altered or changed in the software to match desired thresholds of analysis including TTC, PET, Max-D (Maximum deceleration), DeltaS (Speed difference) and DR and MaxDeltaV (Pu & Joshi, 2008). Figure 3-6 shows the operational concept of the SSAM software.

The surrogate measures used in the SSAM software, and thus in this study, to identify conflicts were the time-to-collision (TTC) and the post encroachment time (PET). A simulated conflict is recorded once the defined thresholds of TTC, and PET are exceeded in the trajectory files. The thresholds used for the evaluation were the same as those predefined in SSAM (1.5 and 5 seconds respectively). Traffic conflicts recorded were then analyzed, and the results are discussed in the data results segment.

Simulated conflicts analyzed in SSAM are categorized as shown in Figure 2-6 according to the degree of collision as either lane-change conflicts, rear-end conflicts or crossing conflicts.

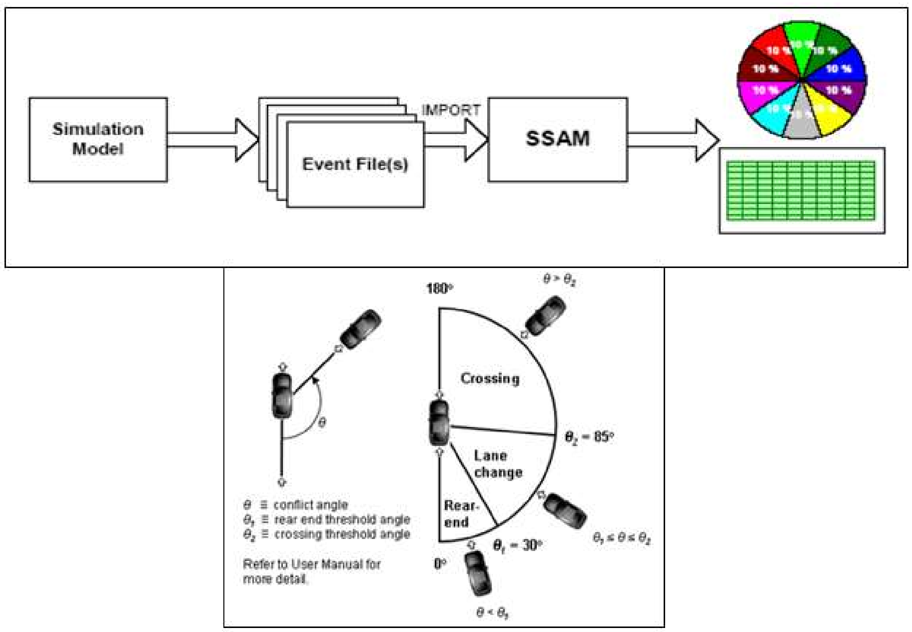


Figure 3-6: Operational concept of SSAM & SSAM conflict angle diagram (SSAM, 2008)

CHAPTER 4 RESULTS AND DISCUSSION

SSAM Conflict Results

The conflict analysis results from SSAM software are presented in this section. The analysis was done on the trajectory files extracted from VISSIM microscopic simulation software simulations for the AM and PM peak traffic conditions. There is a limitation that was observed when modeling real lane-change behavior of vehicles in a queue in VISSIM. An abrupt lane-changing behavior, different from what happens in real-world, was observed during simulation, which resulted in several simulated crashes with TTC = 0. The total conflicts were thus filtered out in SSAM into two sets of results. One set included all the conflict results with the TTC threshold of 1.5 seconds. The second set of results was composed of conflicts with TTC less than 1.5 seconds but greater than 0 seconds. The second filter was applied to obtain more accurate data, and account for the abovementioned modeling limitation, which results in conflicts with a TTC of 0 seconds, denoting vehicles colliding in their conflicting paths.

The conflicts were further categorized according to SSAM software as lane changing and rear-end conflicts, while crossing conflicts were not observed due to the nature of the simulation model, having no crossing points in freeway traffic.

AM Period Results

The conflicts found during the AM period are presented using histograms in Figure 4-1. A conflict change table is also presented as Table 4-1, showing the change in conflicts during each interval in the AM peak period, and at different CV compositions.

Total Conflicts

As shown in the results, during the AM period, total conflict results showed an overall gradual reduction of conflicts in all time periods, with a total reduction of conflicts amounting to approximately 76%, 98% and 31% in the pre-peak, peak and post-peak hour periods, respectively. The reduction in conflicts was not observed to follow any regular pattern at each 25% increment of CVs composition in traffic, however, most reductions were seen in the transition between the 50% and 75% compositions of CVs and from 75% to 100% composition of CVs in traffic. Also as expected, conflicts in the pre- and post-peak periods were considerably less compared to the peak hour period, due to the relatively less volume of traffic during those periods.

Rear-End Conflicts

The rear-end conflicts obtained in the analysis were observed to have more reductions compared to lane change conflicts. During the pre-peak period, for example, there was a total of 73% reduction in conflicts with just a 25% CV composition, and by 50% composition of CVs, the reductions were already at 91%. The high reductions seen have been attributed to the decrease in vehicle speeds due to advance warnings of the incident as well as the change in driver behavior due to the advance messages. The overall reduction of rear-end conflicts in the pre-peak period from 0% to 100% CVs composition were at 98%.

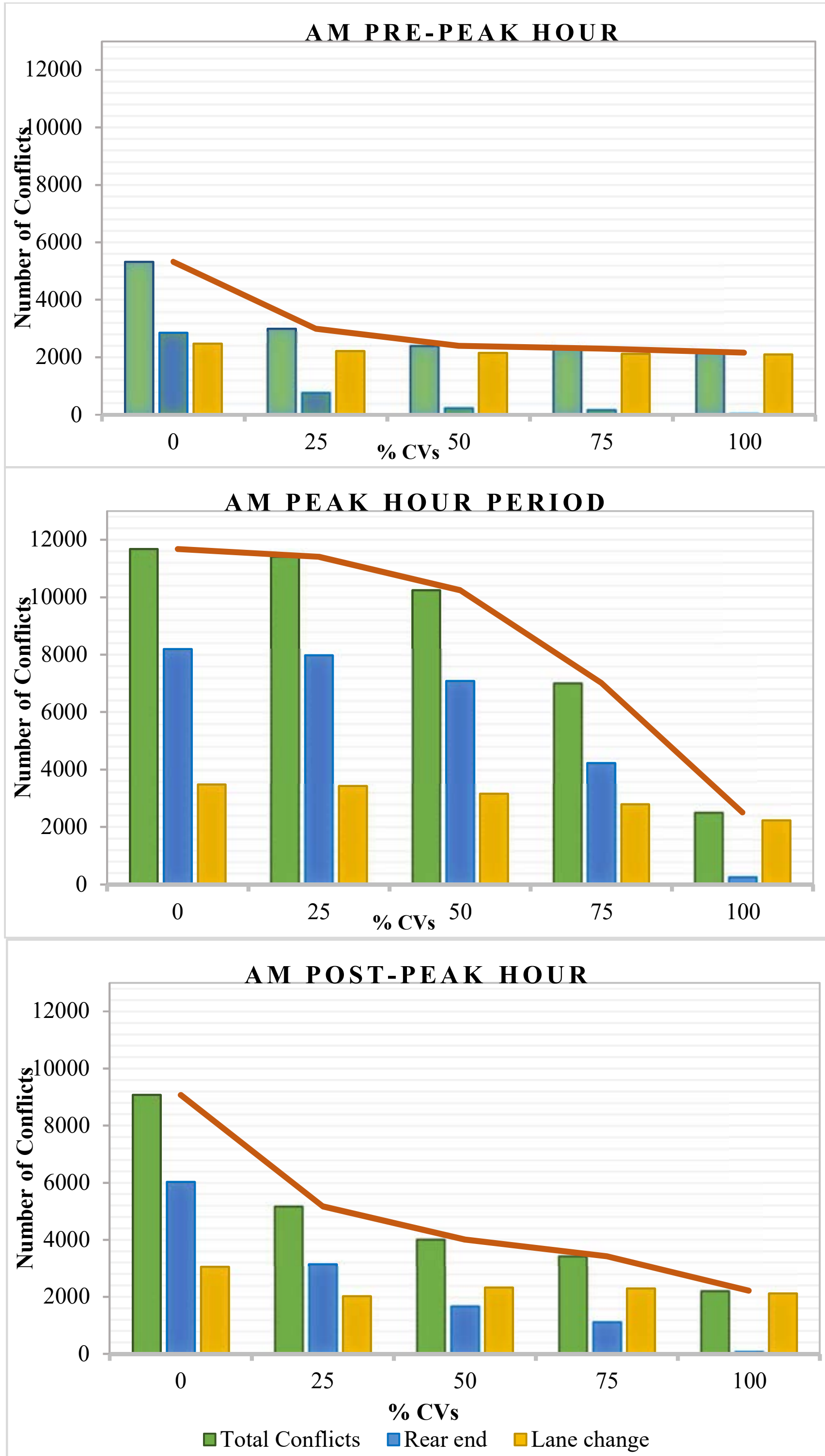


Figure 4-1: SSAM Total Conflicts during the AM Peak Period

Table 4-1: Percent change in total traffic conflicts during the AM period

	AM Pre-peak hour				AM Peak hour				AM Post-peak hour			
All Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-43.8%				-2.3%				-43.1%			
50	-54.9%	-67.4%			-12.3%	-10.2%			-55.9%	-22.5%		
75	-56.8%	-75.8%	-4.1%		-39.9%	-38.5%	-31.5%		-62.3%	-33.7%	-14.4%	
100	-59.5%	-92.5%	-10.1%	-6.3%	-78.5%	-78.0%	-75.5%	-64.2%	-75.6%	-57.1%	-44.6%	-35.3%
Rear-End Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-72.6%				-2.6%				-47.9%			
50	-91.1%	-67.4%			-13.5%	-11.2%			-72.1%	-46.4%		
75	-93.4%	-75.8%	-25.9%		-48.5%	-47.1%	-40.4%		-81.2%	-64.0%	-32.8%	
100	-97.9%	-92.5%	-77.0%	-69.0%	-96.6%	-96.5%	-96.0%	-93.4%	-98.4%	-97.0%	-94.4%	-91.7%
Lane-Change Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-10.6%				-1.6%				-33.7%			
50	-13.3%	-3.0%			-9.4%	-7.9%			-24.0%	14.7%		
75	-14.6%	-4.4%	-1.5%		-19.8%	-18.5%	-11.5%		-24.9%	13.4%	-1.1%	
100	-15.2%	-5.1%	-2.2%	-0.7%	-36.0%	-35.0%	-29.4%	-20.2%	-30.5%	5.0%	-8.5%	-7.5%

Conflict reduction   Conflict increase

Similarly, in the post-peak period, there was a total reduction of 98% of rear-end conflicts from no CVs in traffic to 100% CV composition, with 50% of conflict reductions occurring at only 25% CV composition. In the peak period, however, there was only a 3% reduction in rear-end conflicts at the 25% CV composition mark, however, the overall reductions were at 97% when 100% of vehicles were CVs.

Lane Change Conflicts

The reduction of lane-change conflicts with the increase of CVs in traffic was less pronounced than the rear-end conflict changes. In the pre- and post-peak hour periods, the reductions at 100% CV compositions were only at 15% and 31% respectively, whereas most of the reductions were observed in the peak hour at 36%. With only 25% composition of CVs however, only 2% of the conflicts were reduced in the peak hour as opposed to 11% and 34% in the pre- and post-peak hours. The changes in lane-change conflicts were seen to decrease possibly because with more CVs in traffic, more vehicles were getting lane change warnings and thus there was an overall increase in lane-change maneuvers.

Filtered Conflicts with TTC > 0

Due to limitations in simulation models, many simulated crashes (or conflicts with a TTC = 0) may result in vehicle trajectory file analysis in SSAM (Gettman et al., 2008). Most of these conflicts arise from the abnormal lane change behavior once vehicles are in a queue situation for a while. The conflicts that had TTC = 0 were thus removed from the conflict results to give a more accurate number of the conflicts that were produced as a result of the incident modeling. Table 4-2 gives a summary of the change in conflicts after removing conflicts with TTC = 0.

Table 4-2: Summary of conflicts with TTC = 0 and TTC > 0

AM Period									
	Pre-peak			Peak			Post-peak		
	Total Conflicts	Rear-end	Lane change	Total Conflicts	Rear-end	Lane change	Total Conflicts	Rear-end	Lane change
TTC ≥ 0	15178	4133	11045	42854	27761	15092	23888	12074	11815
TTC > 0	4217	3903	314	28324	26781	1543	12343	11559	784
% Change	-72.2	-5.6	-97.2	-33.9	-3.5	-89.8	-48.3	-4.3	-93.4
PM Period									
	Pre-peak			Peak			Post-peak		
	Total Conflicts	Rear-end	Lane change	Total Conflicts	Rear-end	Lane change	Total Conflicts	Rear-end	Lane change
TTC ≥ 0	111703	92619	19084	111946	92386	19560	105505	86579	18926
TTC > 0	94187	90872	3315	94429	90678	3751	88319	84880	3439
% Change	-15.7	-1.9	-82.6	-15.6	-1.8	-80.8	-16.3	-2.0	-81.8

After filtering out conflicts with TTC = 0, the results were analyzed once more to obtain more accurate conflict figures as shown in Figures 4-2 and 4-3. As illustrated in Figure 4-2, the conflict results with TTC > 0, had similar trends to those described in the total conflicts. However, the reduction in conflicts from 0% to 50% composition of CVs was less compared to when all conflicts are included in the results. At 75% and full deployment of CVs, only a small amount of conflicts was observed. Similar to when the total conflicts with TTC = 0 were included, the conflicts in the pre- and post-peak hours were significantly less compared to those found during the peak hour.

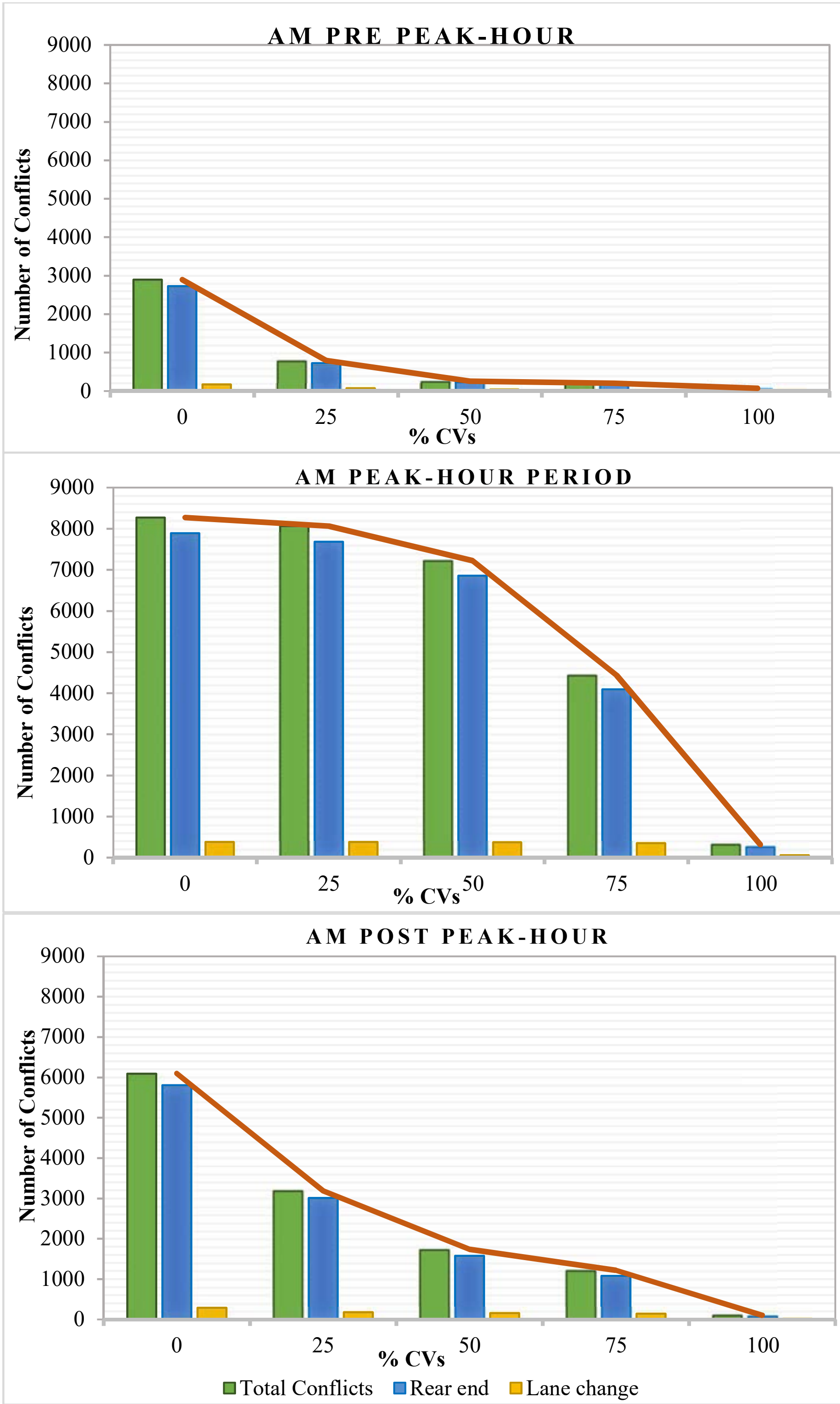


Figure 4-2: SSAM Conflicts during the AM Peak Period (TTC > 0)

Table 4-3: Percent change in total traffic conflicts during the AM period (TTC > 0)

	AM Pre-peak hour				**AM Peak hour**				**AM Post-peak hour**			
All Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-72.6%				-2.5%				-47.7%			
50	-91.1%	-69.3%			-12.6%	-10.4%			-71.5%	-45.5%		
75	-93.1%	-75.3%	-22.9%		-46.3%	-44.9%	-38.6%		-80.0%	-61.8%	-29.9%	
100	-97.6%	-92.3%	-72.9%	-64.9%	-96.2%	-96.1%	-95.6%	-92.9%	-98.3%	-96.7%	-93.9%	-91.4%
Rear-End Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-73.6%				-2.6%				-48.1%			
50	-91.9%	-69.3%			-13.1%	-10.8%			-72.9%	-47.7%		
75	-93.5%	-75.3%	-19.6%		-48.1%	-46.8%	-40.3%		-81.5%	-64.3%	-31.7%	
100	-98.0%	-92.3%	-74.9%	-68.7%	-96.7%	-96.7%	-96.2%	-93.7%	-98.5%	-97.2%	-94.6%	-92.1%
Lane-Change Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-57.6%				-0.8%				-38.6%			
50	-78.3%	-48.9%			-3.0%	-2.2%			-44.2%	-9.1%		
75	-87.5%	-70.6%	-42.4%		-8.8%	-8.1%	-6.0%		-51.1%	-20.3%	-12.4%	
100	-91.6%	-80.3%	-61.4%	-33.0%	-84.4%	-84.3%	-84.0%	-82.9%	-92.8%	-88.3%	-87.1%	-85.3%

Conflict reduction   Conflict increase

PM Period Results

The conflicts found during the AM period are presented in column charts in Figure 4-3. Conflict change results are also presented in Table 4-4, showing the change in conflicts during each sub-period in the PM peak period, and at different CV compositions.

Total Conflicts

As shown in Figure 4.3 and Table 4.4, the total conflict produced during the PM period was significantly more compared to those shown in the AM period. This comes as no surprise given higher traffic volumes during the PM simulation period. Despite the higher magnitude relative to the AM results, the results showed an overall reduction of conflicts in all time periods, similar to the AM periods. The most reduction in total conflicts, however, was observed with the change of CV composition from 0% to 25%, with reductions of 70%, 54% and 55% in the pre-peak, peak and post-peak hour periods respectively. As with the AM results, conflict reductions in the PM period did not conform to a regular pattern with each increase in CV composition in traffic. Overall conflicts were found to be reduced by an average of 87% through the different traffic composition.

Rear-End Conflicts

Rear-end conflict reductions accounted for the larger proportion of total conflict reductions during the PM period. With only 25% CVs in the traffic stream, reductions in rear-end conflicts of 72%, 56%, and 58% were seen in the pre-peak, peak hour and post-peak hour respectively. At 100 % CV composition, the reductions increased to a maximum of 98% and a minimum of 87% among those PM sub-periods. Two factors can be used to justify large reductions in rear-end conflicts. Firstly, with the advance speed reductions and lane-change warnings, vehicles perform less hard-braking events as well as fewer late lane changes which lead to a steadier flow of traffic through the incident area. Also, as more vehicles approach the incident area with caution due to upstream warnings, their driving behavior is more cautious, and their speeds reduced thus vehicles are expected to experience less rear-end conflicts.

Lane Change Conflicts

As with the AM conflict results, lane change conflicts did not exhibit many reductions during the PM peak period. With a maximum of 66% reduced conflicts at 100% CV composition during the pre-peak period, this was relatively small when compared to the 98% reduction observed in rear-end conflicts during the same period.

As illustrated in Figure 4-4, the conflict results with TTC > 0, had similar trends to those described in the total conflicts. However, the reduction in conflicts from 0% to 50% composition of CVs was less compared to when all conflicts are included in the results. Most conflicts were reduced between 75% and full deployment of CVs in traffic. Similar to when the total conflicts with TTC = 0 were included, the conflicts in the pre- and post-peak hours were significantly less compared to those found during the peak hour.

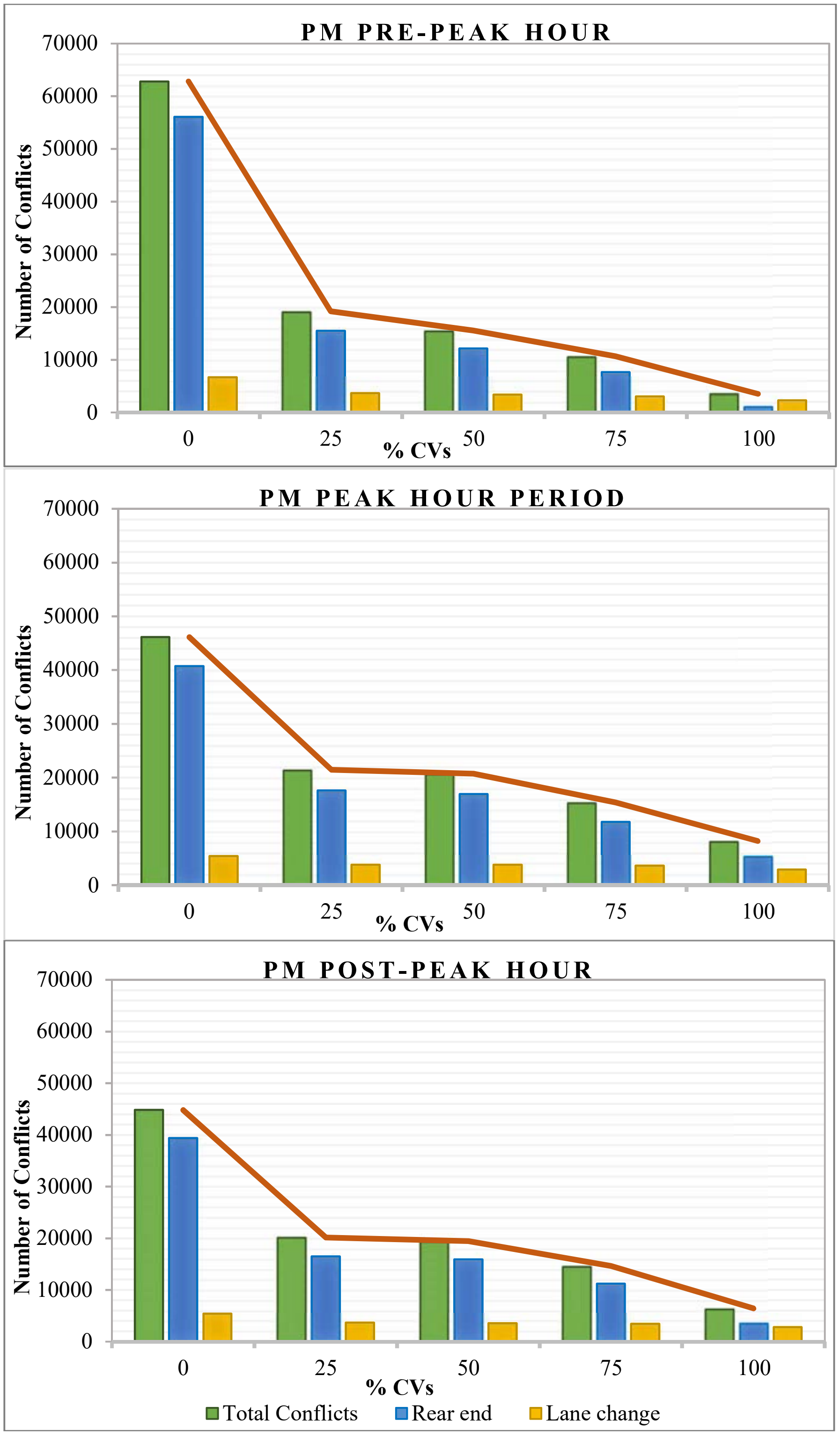


Figure 4-3: SSAM Total conflicts during the PM Peak Period

Table 4-4: Percent change in total traffic conflicts during the PM Period

	PM Pre-peak hour				PM Peak hour				PM Post-peak hour			
All Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-69.5%				-53.5%				-55.0%			
50	-75.3%	-19.0%			-55.0%	-3.3%			-56.6%	-3.6%		
75	-83.0%	-44.4%	-31.3%		-66.6%	-28.2%	-25.8%		-67.3%	-27.4%	-24.7%	
100	-94.4%	-81.5%	-77.2%	-66.8%	-82.2%	-61.7%	-60.4%	-46.7%	-85.7%	-68.2%	-67.0%	-56.1%
Rear-End Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-72.4%				-56.7%				-58.2%			
50	-78.4%	-21.9%			-58.4%	-4.0%			-59.6%	-3.5%		
75	-86.4%	-50.8%	-37.0%		-71.1%	-33.3%	-30.5%		-71.6%	-32.1%	-29.7%	
100	-97.8%	-92.1%	-89.9%	-83.9%	-87.0%	-69.9%	-68.7%	-54.9%	-90.9%	-78.2%	-77.4%	-67.9%
Lane-Change Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-45.5%				-29.1%				-31.7%			
50	-49.2%	-6.7%			-29.0%	0.2%			-34.5%	-4.1%		
75	-54.8%	-17.2%	-11.2%		-32.7%	-5.0%	-5.1%		-36.2%	-6.6%	-2.5%	
100	-65.5%	-36.7%	-32.1%	-23.6%	-45.9%	-23.7%	-23.8%	-19.7%	-47.6%	-23.3%	-20.0%	-17.9%

Conflict reduction   Conflict increase

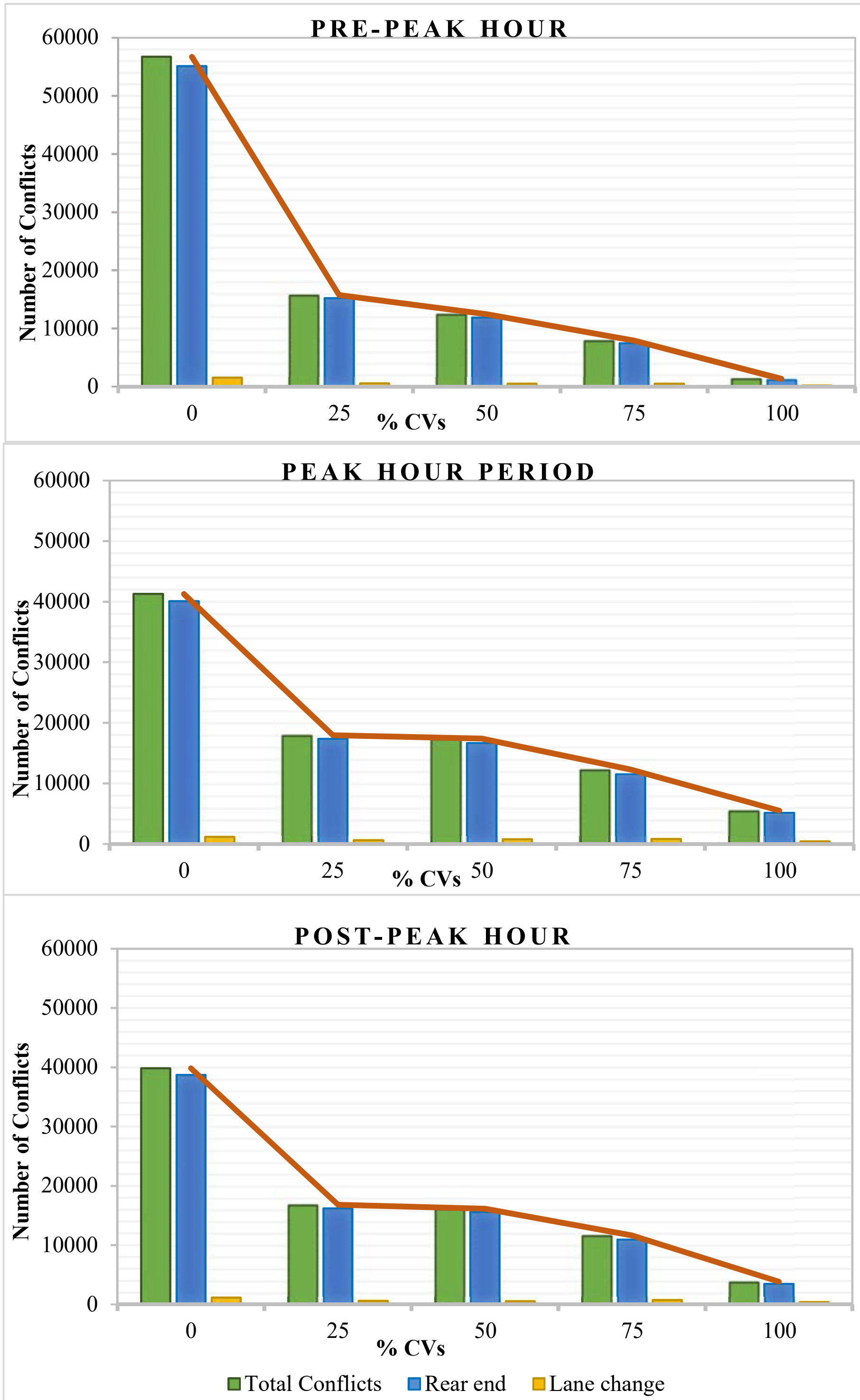


Figure 4-4: SSAM Conflicts during the PM Peak Period (TTC > 0)

Table 4-5: Percent change in conflicts during the PM period (TTC > 0)

	PM Pre-peak hour				PM Peak hour				PM Post-peak hour			
All Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-72.2%				-56.5%				-57.7%			
50	-78.1%	-21.1%			-57.8%	-2.9%			-59.4%	-3.9%		
75	-86.0%	-49.8%	-36.3%		-70.2%	-31.4%	-29.4%		-70.8%	-30.9%	-28.0%	
100	-97.6%	-91.5%	-89.2%	-83.1%	-86.6%	-69.2%	-68.3%	-55.1%	-90.4%	-77.2%	-76.2%	-67.0%
Rear-End Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-72.4%				-56.7%				-58.1%			
50	-78.4%	-21.8%			-58.4%	-3.9%			-59.7%	-3.7%		
75	-86.5%	-51.2%	-37.5%		-71.3%	-33.7%	-31.0%		-71.8%	-32.7%	-30.1%	
100	-97.9%	-92.4%	-90.3%	-84.4%	-87.2%	-70.5%	-69.3%	-55.5%	-91.1%	-78.7%	-77.9%	-68.3%
Lane-Change Conflicts												
	Initial Composition				Initial Composition				Initial Composition			
%CVs	0	25	50	75	0	25	50	75	0	25	50	75
25	-65.7%				-50.0%				-46.0%			
50	-66.3%	-1.8%			-36.9%	26.2%			-51.0%	-9.2%		
75	-69.1%	-10.2%	-8.5%		-32.4%	35.2%	7.2%		-36.5%	17.6%	29.6%	
100	-88.4%	-66.4%	-65.7%	-62.6%	-66.0%	-32.0%	-46.1%	-49.8%	-66.0%	-37.1%	-30.7%	-46.5%

Conflict reduction Conflict increase

Speed Profiles

One of the strategies of reducing conflicts and mitigating secondary crashes near incidents is to reduce the speed differential among vehicles approaching the incident location by enabling vehicles to decelerate smoothly and minimize hard-braking situations, which could lead to potential crashes. Therefore, the development of speed profiles after the introduction of CVs in the traffic stream can be useful in demonstrating how the speeds of vehicles are affected near the incident location. These speed profiles can also show the effect of CVs in the speeds of vehicles upstream of the incident.

Average vehicle speeds were extracted from VISSIM on a 2.5-mile section upstream of the incident location. Speeds were collected at 32.8 ft (10 m) intervals along the length of the segment to create speed profiles after modeling of the incident. The speeds were recorded at a 5-minute resolution from incident occurrence and speed profiles created for every 10-minute intervals during the 30-minute incident durations as shown in Figures 4-4 through 4-9 at different compositions of CVs in the traffic stream. The speed profiles are presented for both the AM and PM periods and further subdivided into the pre-peak, peak, and post-peak hour subperiods.

AM Period Speed Profiles

During the AM period, the introduction of CVs was observed to reduce the vehicle speeds near the incident area. Although traffic speed was reduced further upstream from the incident area due to the advance messages, vehicle speeds generally were not observed to drop as much as without CVs in traffic. Most benefits were observed at the 75% and 100% CV compositions, which generally kept the speeds at the advisory speeds of 50 mph.

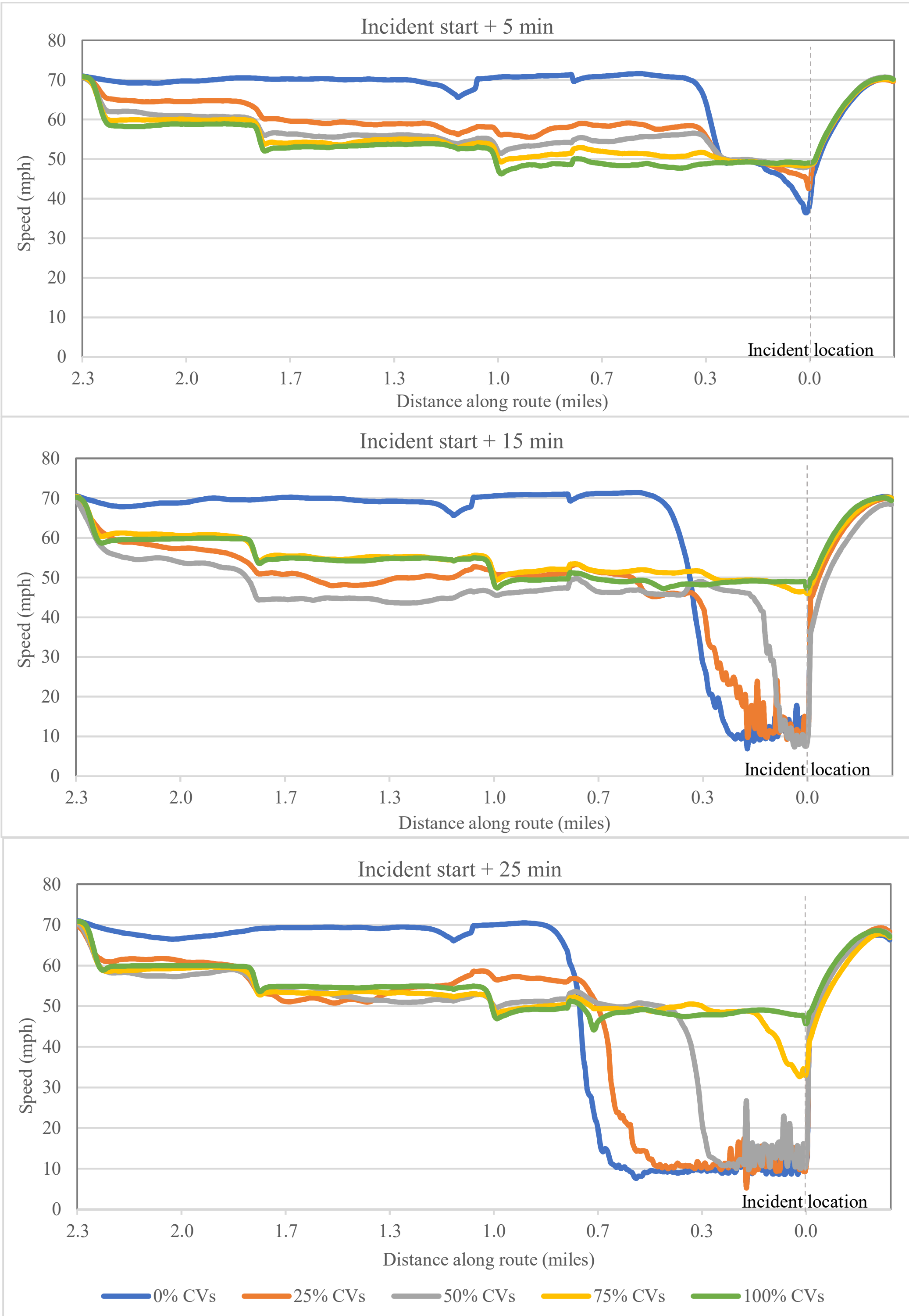


Figure 4-5: Speed profiles during the incident (AM pre-peak hour)

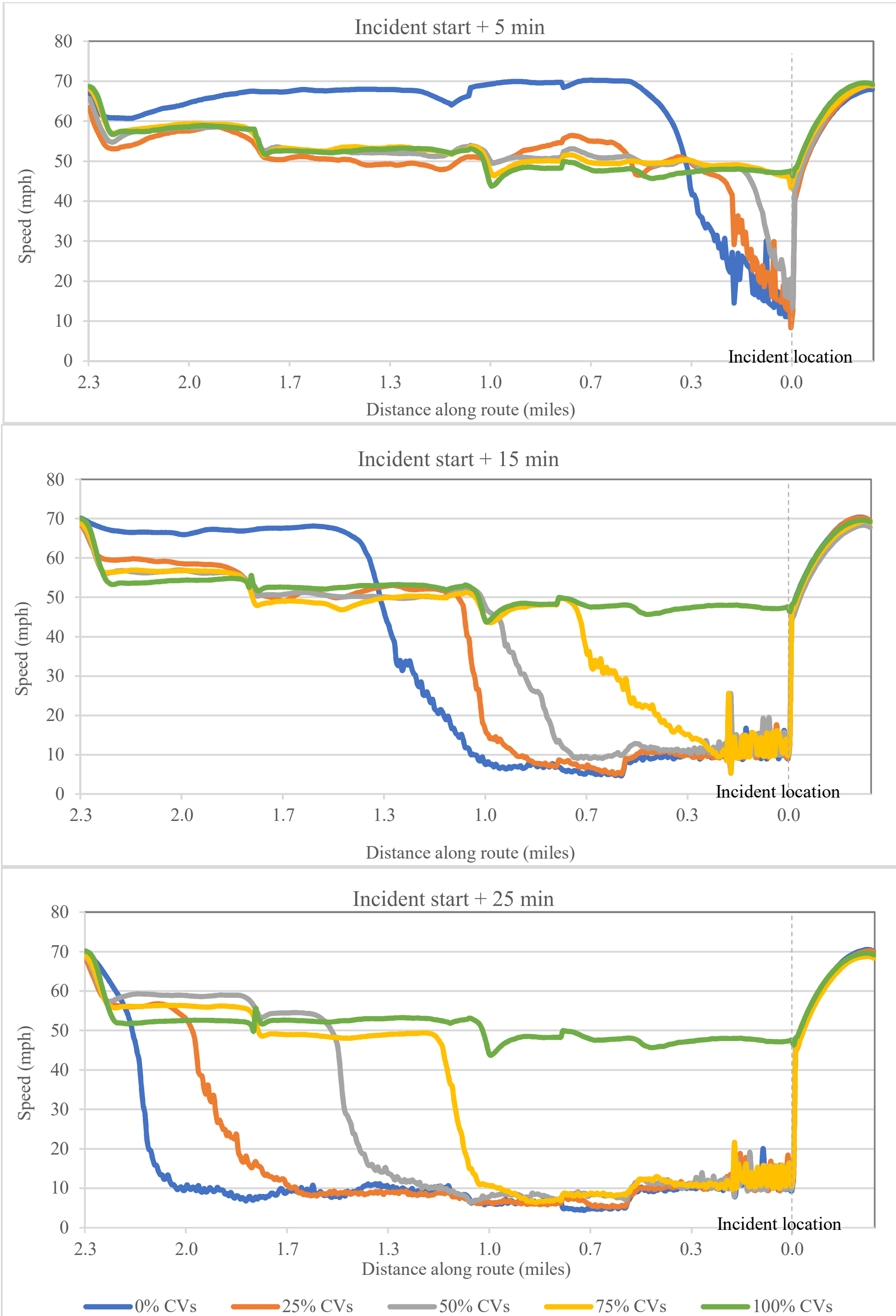


Figure 4-6: Speed profiles during the incident (AM peak hour)

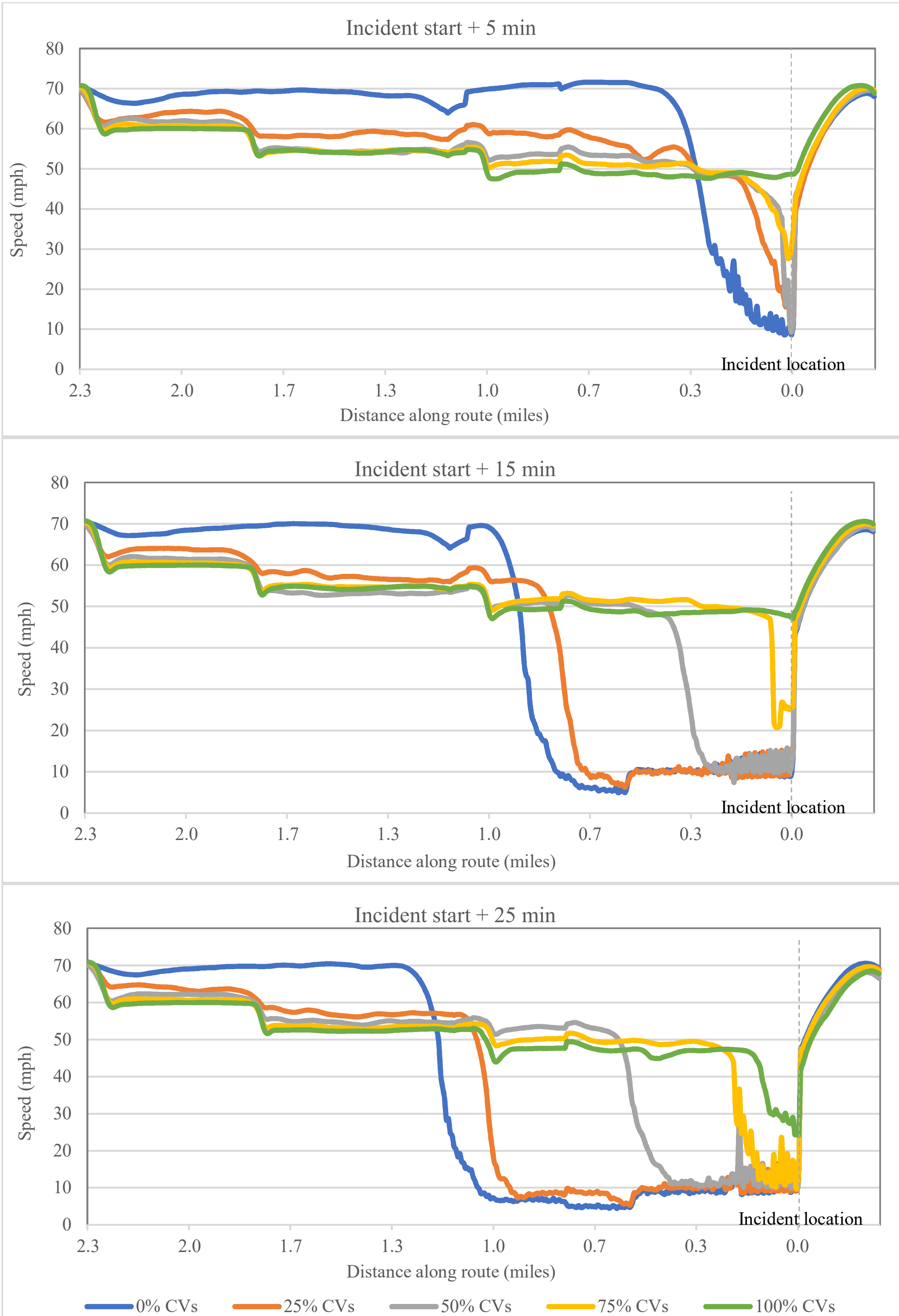


Figure 4-7: Speed profiles during the incident (AM post-peak hour)

Further, CVs during the pre-peak and post-peak hours showed the most benefits in reducing speeds near the incident. During the peak hour, however, speeds deteriorated more, but at 100% CV composition, the speeds did not drop further below the advisory speed. At 25% CV composition, there was the least improvement in speeds along the route, whereby speeds started to drop at about 0.1 miles later downstream than at 0% CV market penetration.

At 0% CV market penetration, the worst drop in vehicle speeds was observed at 0.7 miles, 2 miles and 1 mile from the incident, during the pre-peak, peak, and post-peak, respectively, upstream of the incident after 25 minutes of the incident occurrence. While not much improvement was observed with 25% CV composition, at 50% composition, the distance was reduced to about half. Overall, traffic speeds were seen to improve, and with every 25% increase of CVs, the largest drop in speeds was observed at an average distance of about 0.3 miles further downstream. Significant improvements here are thus expected to occur when 50% of CVs are present in traffic.

PM Period Speed Profiles

During the PM period, speeds were observed to have only little improvements as compared to the AM period. As with the AM period, at 25% CV composition, only minimal improvements were observed. Significant improvements in the flow of traffic were observed from 50% CV composition as shown in Figure 4-7, 4-8 and 4-9. Further, even at 100% CV composition in traffic, there was still a significant deterioration in traffic speeds along the 2-mile segment upstream of the incident. The smaller improvements during the PM period were speculated to be attributed to a relatively higher demand traffic volume during the PM period in comparison to traffic in the AM period.

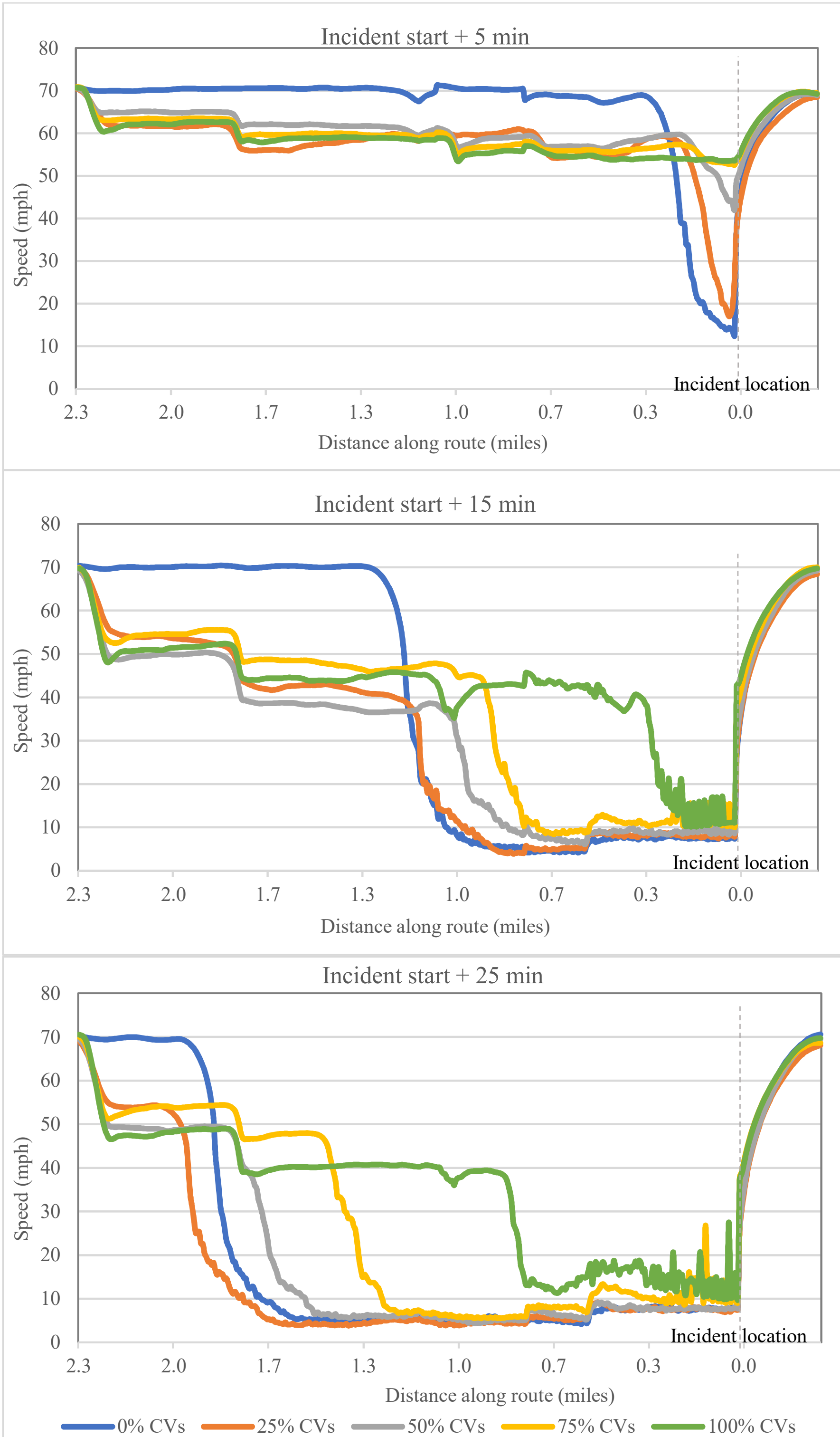


Figure 4-8: Speed profiles during the incident (PM pre-peak hour)

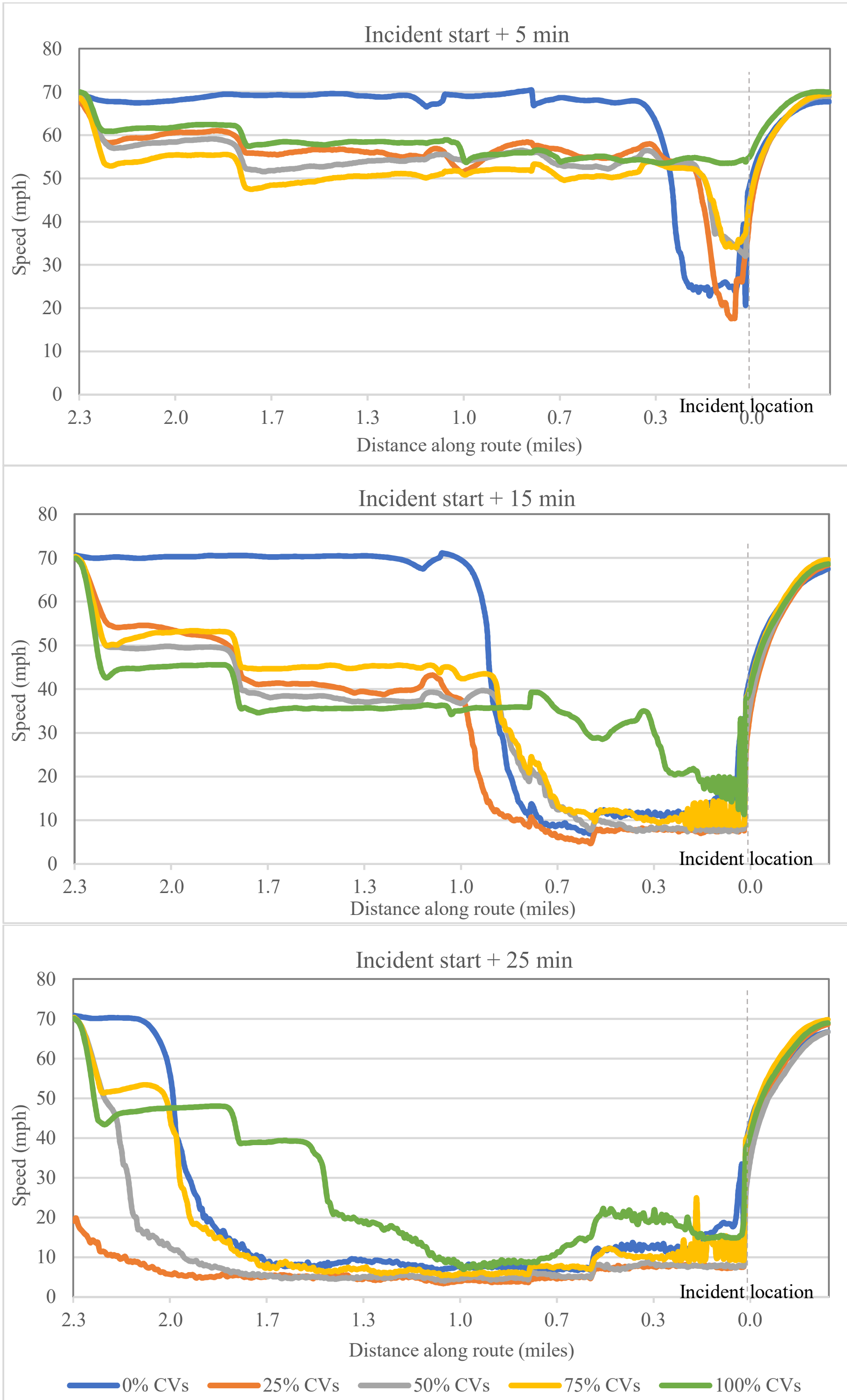


Figure 4-9: Speed profiles during the incident (PM peak hour)

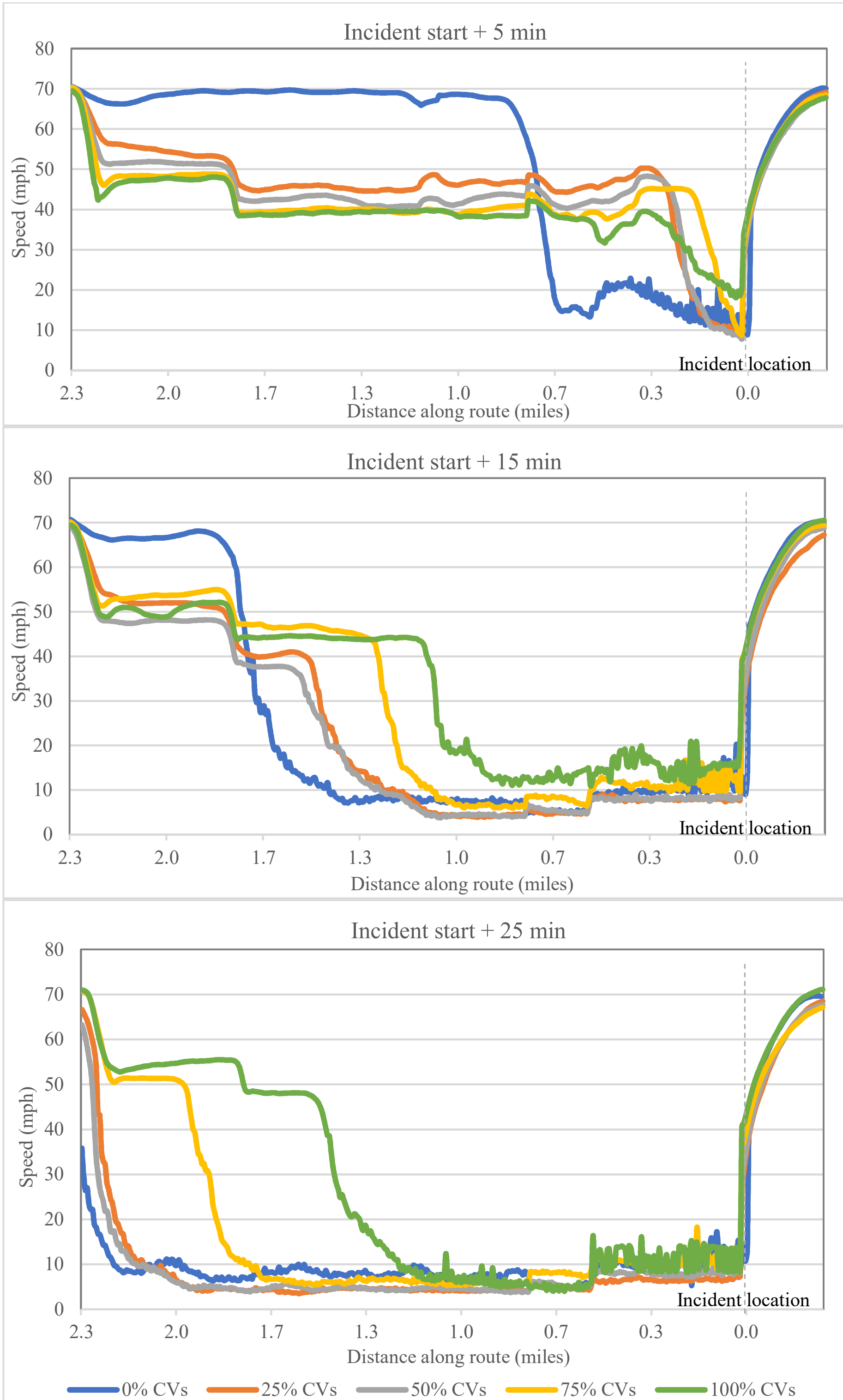


Figure 4-10: Speed profiles during the incident (PM post-peak hour)

Overall, the speeds of vehicles upstream of the incident were observed to deteriorate less at higher CV market penetration rates (above 50%), at which vehicle speeds were reduced to 50 mph or lower and maintained steadily through the incident area. However, when traffic demand volume is relatively higher, the results suggest that minimal speed improvements can be expected even at a high CV market penetration rate in the traffic stream. This may be attributed to the lower level of service and fewer gaps in traffic for vehicles to perform lane-change maneuvers, leading to traffic being trapped in the blocked lane. Consequently, this may lead to the formation of bottlenecks at the incident area which may further lead to queues and low speeds further downstream.

Statistical Comparison of Conflicts

This study analyzed the effect of CVs in the mitigation of SCs on freeways by checking time-to-collision (TTC) as a measure of effectiveness in examining vehicle conflicts. Using a one-tailed t-test, statistical analysis of the average TTC values was performed with two hypotheses tests.

The null hypothesis was that the mean difference between the average TTC values between 0% and a subsequent percentage of CV compositions is zero. This was tested versus an alternate hypothesis that the mean difference between the average TTC values of the two scenarios is less than zero.

- Null Hypothesis, H_0: $\mu_1 - \mu_2 = 0$, OR $\mu_1 = \mu_2$
- Alternate Hypothesis, H_A: $\mu_1 - \mu_2 < 0$, OR $\mu_1 < \mu_2$

Where:

μ_1 = mean TTC value at 0% CV composition

μ_2 = mean TTC value at i% CV composition

At the 95% confidence level, the results showed that significant differences exist between the average TTC values between 0% CV composition and each 25% increment in CV composition as shown in Table 4-6. Further, the differences were checked during all three periods i.e. pre-peak hour, peak hour and post-peak hour, for both AM and PM demand flows, and significant differences between the values were observed.

The results shown in Table 4-6 indicated that the mean TTC values from each simulation run were significantly reduced as the composition of CVs increased. This trend was expected due to the conflict reductions as the number of CVs was increased. in simulation. Large t-values were obtained from the analysis, similar to those obtained in a study by Gettman et. al (Gettman et al., 2008), which indicated high significant differences between the TTC values. Also, the mean TTC values suggest that as CVs were increased in simulation, the severity of the fewer conflicts yielded was slightly increased albeit the observed overall reduction of conflicts.

Also, as expected in the PM peak hour, the significance of the reduction was lower when compared to the pre and post-peak hours and the AM peak period. This follows the trend that was observed in the conflict reductions shown in Figure 4-1 to Figure 4-4.

Table 4-6: Summary of paired t-test results for TTC values based on the time period

	CV Composition	N	Mean	St Dev	SE Mean	t-value	p-value	Significant
AM Pre-peak hour	0%	10	0.67	0.632	0.316			
	25%	10	0.32	0.548	0.274	58.57	<.001	YES
	50%	10	0.13	0.387	0.194	103.45	<.001	YES
	75%	10	0.11	0.361	0.180	110.02	<.001	YES
	100%	10	0.04	0.224	0.112	143.33	<.001	YES
	CV Composition	**N**	**Mean**	**St Dev**	**SE Mean**	**t-value**	**p-value**	**Significant**
AM Peak hour	0%	10	0.16	0.424	0.212			
	25%	10	0.93	0.566	0.283	-174.87	<.001	YES
	50%	10	0.87	0.592	0.296	-155.65	<.001	YES
	75%	10	0.79	0.624	0.312	-125.51	<.001	YES
	100%	10	0.04	0.224	0.112	143.33	<.001	YES
	CV Composition	**N**	**Mean**	**St Dev**	**SE Mean**	**t-value**	**p-value**	**Significant**
AM Post-peak hour	0%	10	0.82	0.600	0.300			
	25%	10	0.76	0.624	0.312	12.58	<.001	YES
	50%	10	0.54	0.632	0.316	54.47	<.001	YES
	75%	10	0.44	0.608	0.304	70.02	<.001	YES
	100%	10	0.06	0.265	0.132	201.15	<.001	YES
	CV Composition	**N**	**Mean**	**St Dev**	**SE Mean**	**t-value**	**p-value**	**Significant**
PM Pre-peak hour	0%	10	1.12	0.412	0.187			
	25%	10	1.02	0.510	0.182	24.09	<.001	YES
	50%	10	0.99	0.529	0.141	27.13	<.001	YES
	75%	10	0.93	0.574	0.182	32.73	<.001	YES
	100%	10	0.48	0.624	0.190	59.82	<.001	YES
	CV Composition	**N**	**Mean**	**St Dev**	**SE Mean**	**t-value**	**p-value**	**Significant**
PM Peak hour	0%	10	1.03	0.510	0.255			
	25%	10	1.03	0.500	0.250	-1.69	0.083	NO
	50%	10	1.04	0.500	0.250	-3.16	0.017	YES
	75%	10	1.00	0.539	0.269	5.19	0.003	YES
	100%	10	0.84	0.616	0.308	23.95	0.001	YES
	CV Composition	**N**	**Mean**	**St Dev**	**SE Mean**	**t-value**	**p-value**	**Significant**
PM Post-peak hour	0%	10	1.1	0.436	0.218			
	25%	10	1.03	0.500	0.250	16.65	<.001	YES
	50%	10	1.03	0.500	0.250	15.81	<.001	YES
	75%	10	0.99	0.539	0.182	21.18	<.001	YES
	100%	10	0.75	0.640	0.320	42.6	<.001	YES

CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

This study presented a simulation exploration of the safety benefits associated with the implementation of CV technology on freeways. In the study, traffic on a freeway segment from Florida's Turnpike was simulated during the AM and PM peak periods. The study was composed of three main components namely: the creation of a CV environment in VISSIM using a code script written using the COM API, modeling of an incident blocking one lane of traffic also using COM API, and finally extracting trajectory files from VISSIM and performing a conflict analysis in SSAM.

The safety benefits evaluated in this study are specifically targeted towards the reduction of SCs or incidents, which represent a significant proportion of all crashes on US freeways. The benefits were measured in terms of changes in the simulated conflicts, specifically in the reduction of the conflicts simulated from the SSAM software. Conflicts have been stated to have proportional relationships to collisions in some studies (El-Basyouny & Sayed, 2013; Gettman & Head, 2003), however, the results from previous studies have not been conclusive on the relationship.

It was evident from the results in this study that with the adoption of the CV technology, there is a sizeable decline in the number of conflicts arising from early lane changes and speed reduction of vehicles approaching the incident area. A steady decline of conflicts up to 90% during pre and post-peak hour simulation periods and up to 60% reduction during the peak hour. With literature stating that a relationship between conflicts and crashes exists, although not direct, this reduction potentially represents reduction in SCs.

Also, with a full market penetration of CVs not expected soon, a sensitivity analysis of different composition rates of CVs in the freeway stream was carried out which showed how safety conditions were improved consistently from lower to higher CV compositions in traffic during both the AM and PM peak periods. In the AM period, improvements were seen with conflict reductions up to 94%, while in the PM period, conflict reductions went up to 84% between different CV compositions.

Recommendations for Future Work

There are several limitations of this study that could be addressed in future research. This study assumes that communication in the CV environment remain unchanged at different traffic densities. In a future study, the effect of traffic density on signal transmission on the freeways could be evaluated using network simulation software coupled with traffic simulation software in a CV environment, if found to have a significant effect on the propagation of messages.

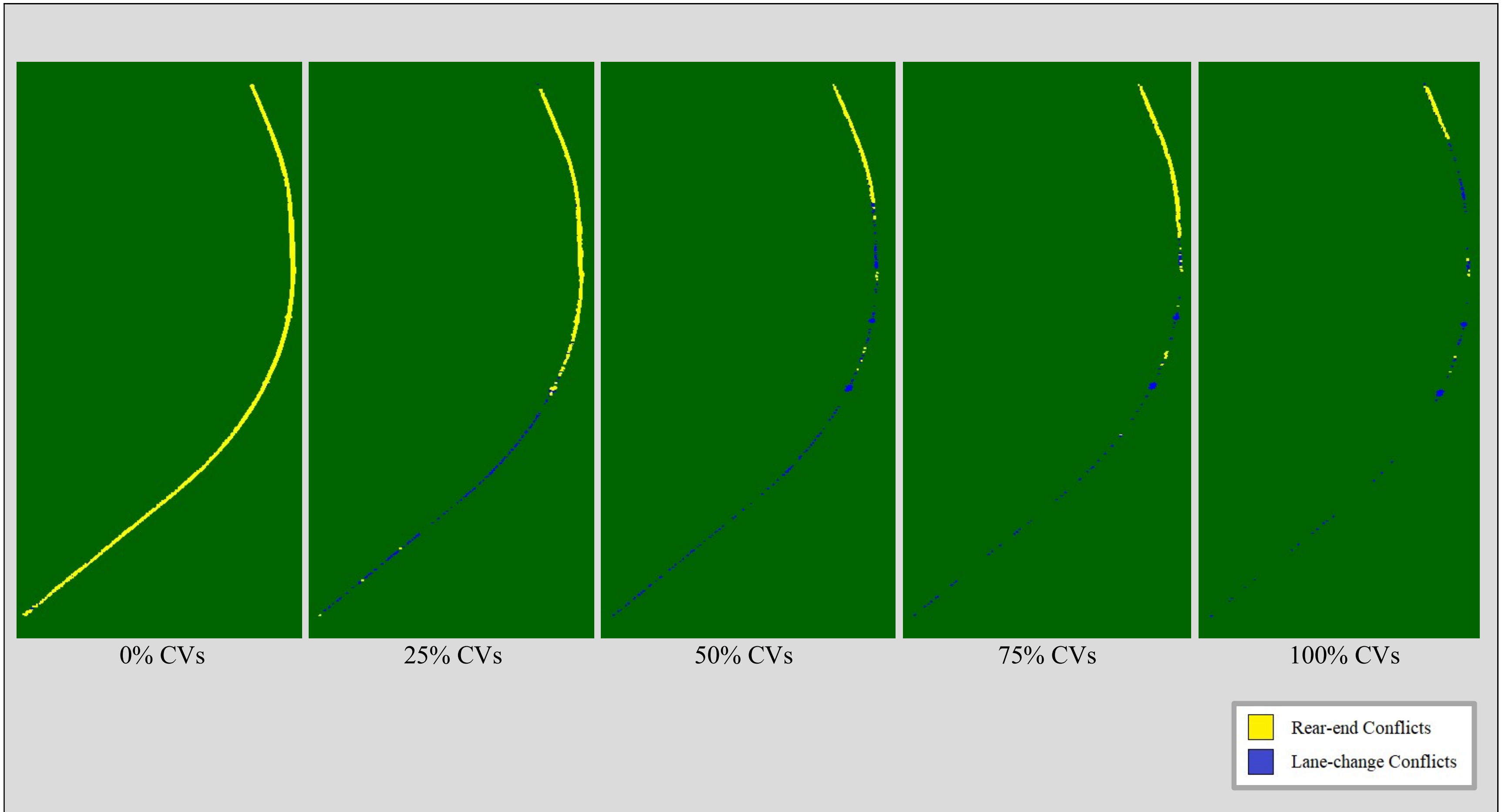
This study also evaluated only one crash scenario where only the outermost travel lane (right lane) is blocked during the incident duration. Plans are underway to add several scenarios including blocking the left lane, middle lane or even multiple lanes during an incident. Further, the consideration of the use of detours or diversion of traffic to alternate routes to bypass an incident location could be implemented in a future study to evaluate benefits of detour advisory to the safety and operation of traffic on the freeway.

Due to the difficulty in modeling other human driving behaviors or factors such as rubbernecking, or driver temporary inattention due to an incident, conflicts derived in this study do not represent conflicts that may be caused by such factors, which could be another cause of conflicts leading to SCs. This factor also hinders the modeling of conflicts in the opposite direction of traffic since their causes can only be humanistic. This may warrant the need for future research into how such humanistic factors can be included in a microscopic simulation model to account for humanistic causes for crashes.

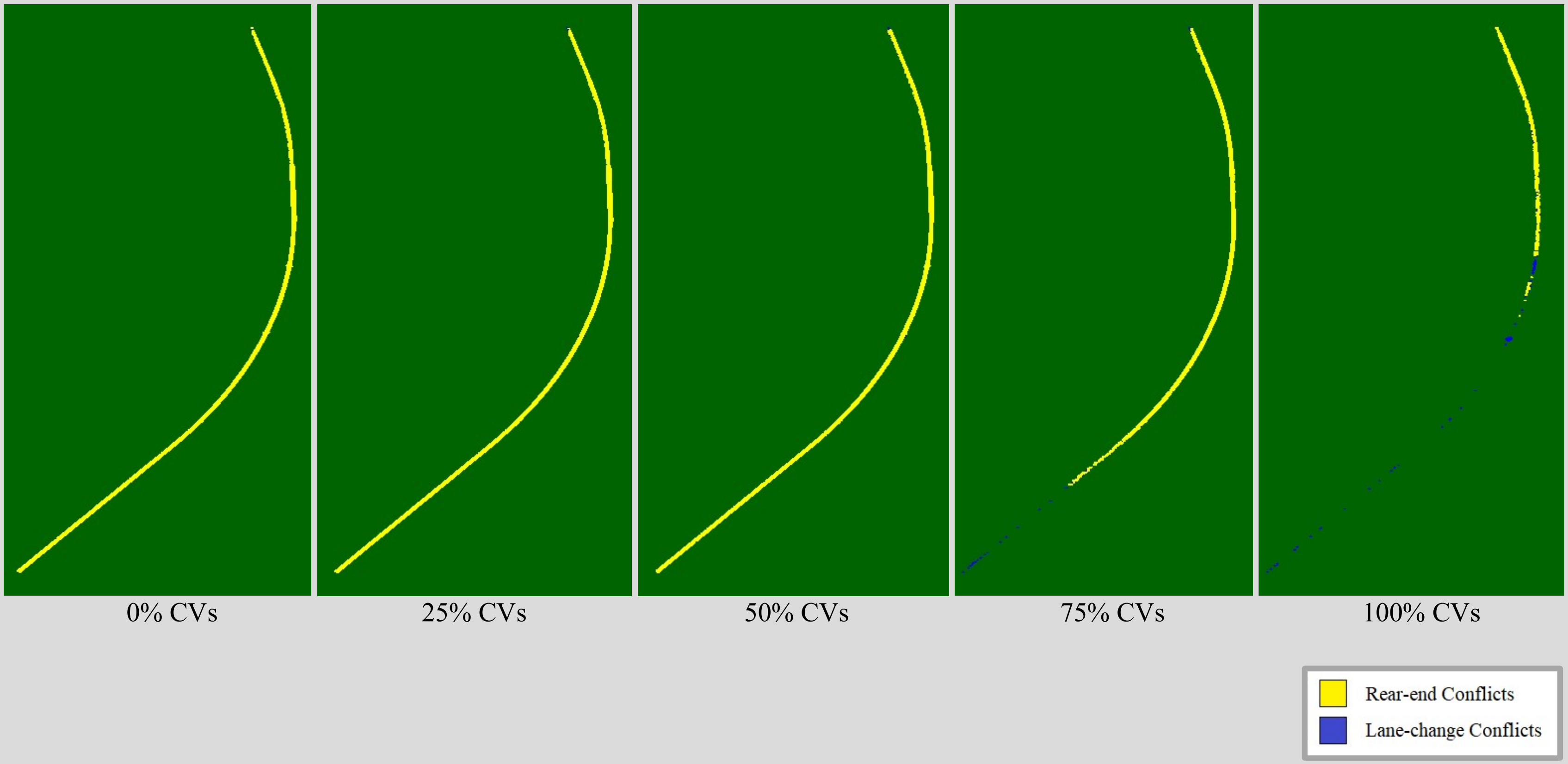
Finally, the level of compliance of drivers receiving warning messages in vehicles could be evaluated in a future study to obtain the expected compliance level for basic safety messages and other CV advisory messages. While microscopic simulation models have some limitations in analyzing driver behavior, compliance and other factors such as driver reaction times can be investigated by use of driver simulator studies.

APPENDIX

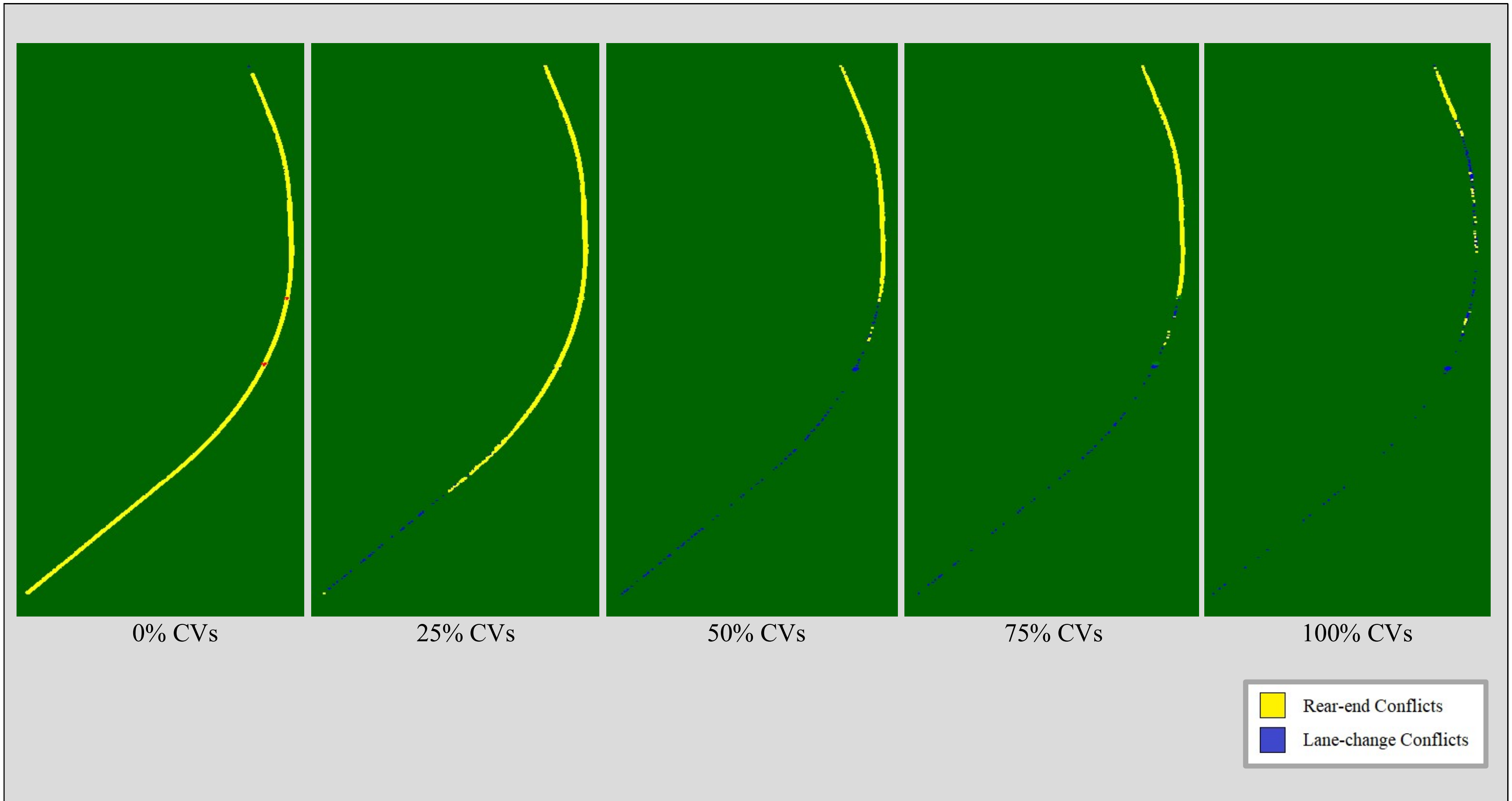
Conflict Map Diagrams from SSAM



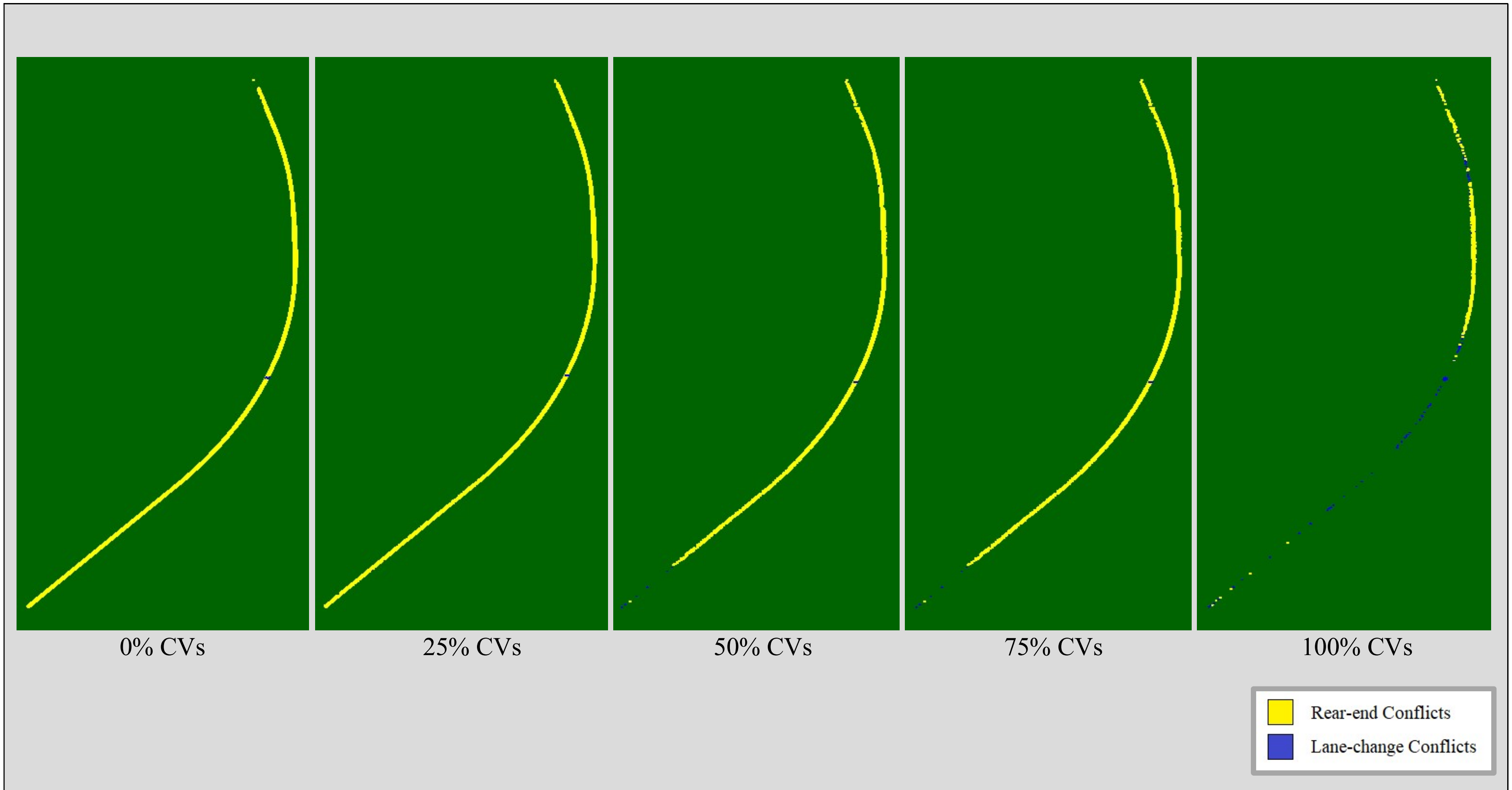
Am Pre-Peak Hour Conflict Diagram (2 Miles Upstream Incident)	Figure A-1



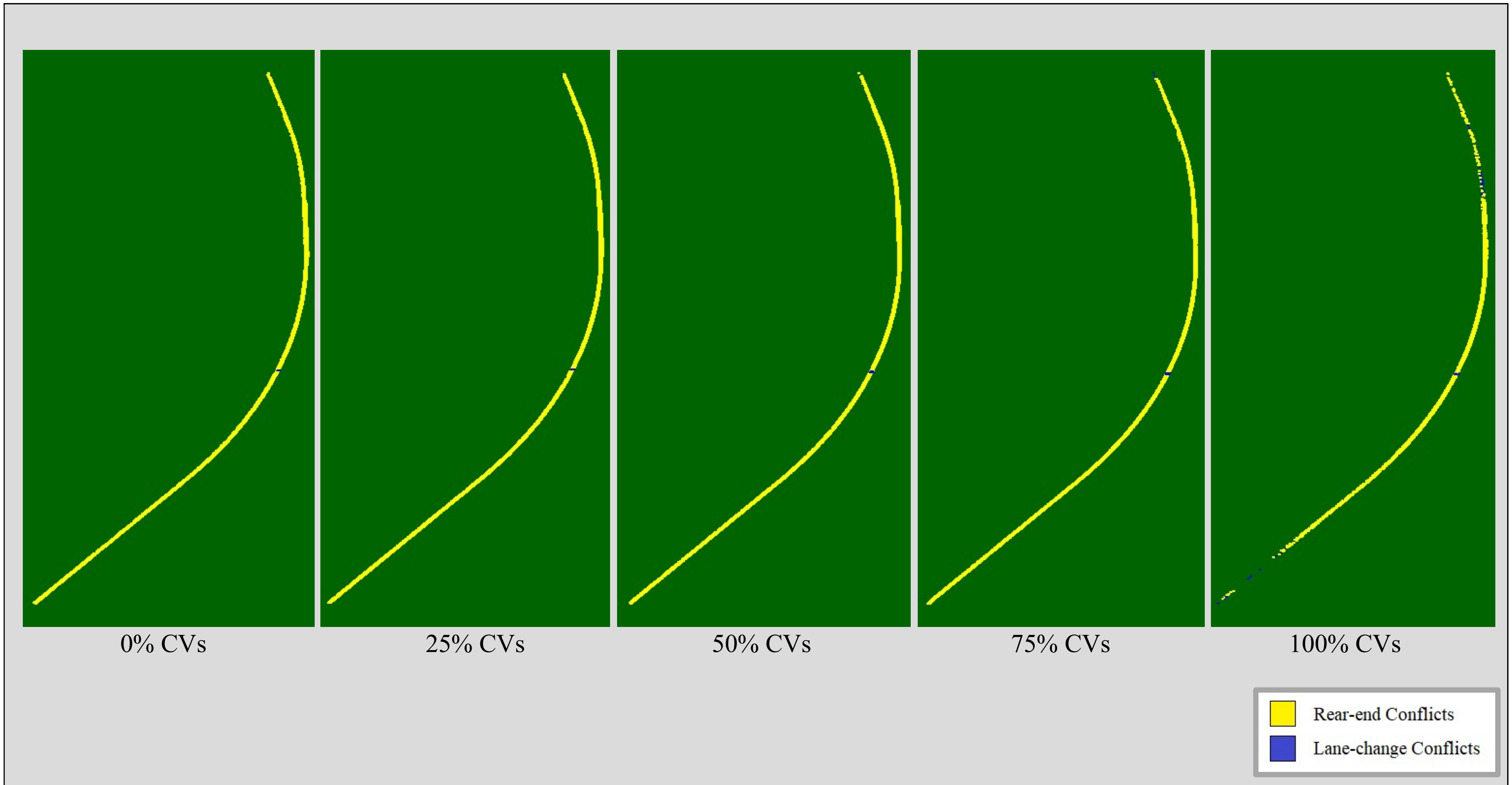
Am Peak Hour Conflict Map (2 Miles Upstream Incident)	Figure A-2



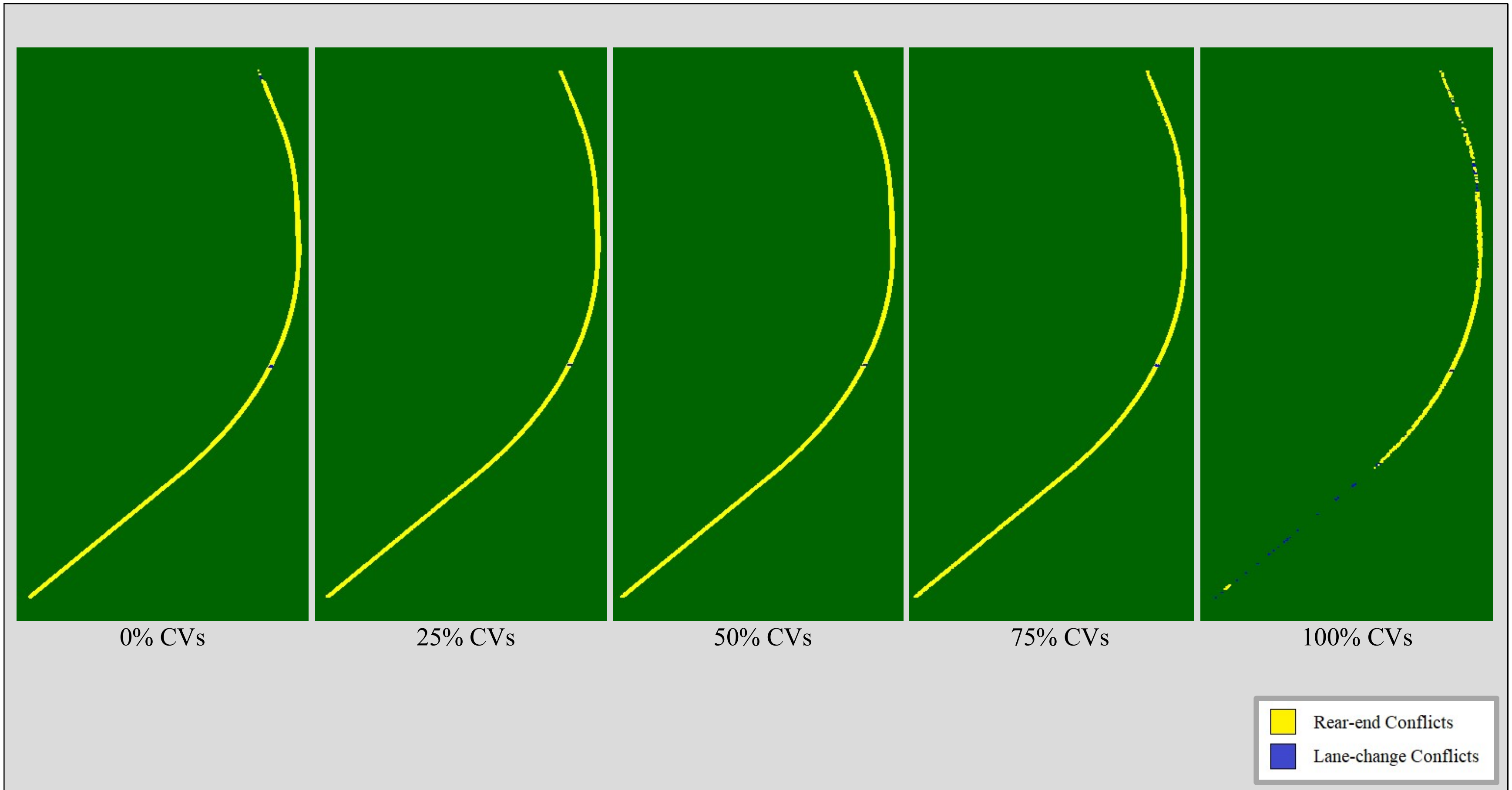
Am Post-peak Hour Conflict Diagram (2 Miles Upstream Incident)	Figure A-3



PM Pre-peak Hour Conflict Diagram (2 Miles Upstream Incident)	Figure A-4



PM Peak Hour Conflict Diagram (2 Miles Upstream Incident)	Figure A-5



PM Post-Peak Hour Conflict Diagram (2 Miles Upstream Incident)	FIGURE A-6

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