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SAFETY AND OPERATIONAL EVALUATION OF DYNAMIC LANE MERGING IN WORK ZONES

by

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ABSTRACT

Traffic safety and mobility of roadway work zones have been considered to be one of the major concerns in highway traffic safety and operations in Florida. In intent to expose Florida's work zones crash characteristics, the Florida Traffic Crash Records Database for years 2002, 2003 and 2004 were explored. Statistical models were estimated and Florida's work zone crash traits for single vehicle crashes and two-vehicle crashes were drawn. For the single-vehicle crashes, trucks were found more likely to be involved in single vehicle crashes in freeway work zones compared to freeways without work zones. Straight level roadways are significantly affected by the presence of work zones. The lighting condition is also one of the risk factors associated with work zone single-vehicle crashes. In fact, at work areas with poor or no lighting during dark conditions, motor vehicles are more prone for crashes compared to non-work zone locations with poor or no lighting during dark. The weather condition is positively associated with single-vehicle work zone crashes. Results showed that during rainy weather, drivers are less likely to be involved in work zone crashes compared to the same weather conditions in non-work zone locations. This fact may be due to the vigilant driving pattern during rain at work zones. For the two-vehicle work zone crashes, results showed that drivers younger than 25 years of age and drivers older than 75 years old have the highest risk to be the at-fault driver in a work zone crash. Male drivers have significantly higher risk than female drivers to be the at-fault driver. The model conspicuously shows that drivers under the influence of narcotics/alcohol are more likely to cause crashes (i.e. at-fault driver) at work zones. Road geometry and the lighting condition were significant risk factors

associated with two-vehicle work zone crashes. Freeways straight segments are more susceptible to crashes in work zone areas. Poor lighting or no lighting at all during dark can lead to significantly higher crash hazard at work zones. Foggy weather causes a significant mount in work zone crash risk compared to non-work zone locations. In addition to that, work zones located in rural areas have higher crash potential than work zones located in urban areas.

After examining the current Florida work zone Maintenance of Traffic (MOT) plans, known as the Motorist Awareness System (MAS), it was realized that this system is static hence does not react to changing traffic conditions. An ITS-based dynamic lane management system, known as dynamic lane merging system, was explored to supplement the existing MAS plans. Two forms of dynamic lane management were recognized as dynamic lane merging namely the early merge and the late merge. These two systems were designed to advise drivers on definite merging locations. Previously deployed dynamic lane merging systems comprise several Portable Changeable Message Signs (PCMS) and traffic sensors. The addition of multiple PCMSs to the current MAS plans may encumber the latter and usually requires relatively extensive equipment installation and relocation which could be inefficient for short term movable work zones. Therefore, two Simplified Dynamic Lane Merging Systems (SDLMS) were designed, deployed, and tested on Florida's short term movables work zones. The first SDLMS was a simplified dynamic early merge system (early SDLMS) and the second SDLMS was a simplified dynamic late merge system (late SDLMS). Both SDLMS consisted of supplementing the MAS plans used in Florida work zones with an ITS-based lane management system.

From the two-to-one work zone configuration (first site), it was noted that the ratio of the work zone throughput at the onset of congestion over the demand volume was significantly the highest for the early SDLMS compared to the MAS and late SDLMS. Travel time through the work was the lowest for the early SDLMS, followed by the late SDLMS, and then MAS. However, the differences in mean travel times were not statistically significant. It was also concluded that the early SDLMS resulted in higher early merging compared to the MAS and that the late SDLMS in higher late merging compared to the MAS. The first site was used as a pilot for testing the system since data collection was limited to two days for each MOT type. Hence, operational measures of effectiveness (MOEs) could not be evaluated under different demand volumes. It should also be noted that the RTMS was not available during the MAS data collection which disabled us from collecting speed data.

From the three-to-two work zone configuration site, data was collected extensively relative to the first site. The RTMS was available for all three MOT types tested which enabled the collection of the speed data that are used as a safety surrogate measure. The mean speed fluctuation in the closed lane was the highest under the MAS system for all demand volumes and in all three lanes. Comparing the dynamic early merge and the dynamic late merge mean speed fluctuations in the closed lane and the middle lane, results showed that the mean speed fluctuation for the early merge are lower than those of

the late merge under all demand volumes. However, the difference in the mean speed fluctuation is only statistically significant under demand volume ranging between 1 and 500 veh/hr. As for the shoulder lane, it was noted that the speed mean speed fluctuation is significantly the lowest for demand volumes ranging between 1500 veh/hr and 2000 veh/hr under the late SDLMS compared to the early SDLMS and the MAS. The ratio of the throughput over demand volume was taken as the operational MOE. Results showed that the Dynamic early merge performs significantly better than the regular MAS under demand volume ranging between 500 veh/hr and 2000 veh/hr. Results also showed that the dynamic late merge perform better than the MAS under volumes ranging between 1500 veh/hr and 2000 veh/hr and significantly poorer than the MAS under low volumes. Therefore, the late SDLMS is not recommended for implementation under low volumes. Results also showed that the late SDLMS performs better than the early SDLMS under higher volume (ranging between 1500 veh/hr to 2000 veh/hr).

A simulated work zone with a two-to-one lane closure configuration was coded in VISSIM and operational and safety MOEs under MAS, early SDLMS, and late SDLMS were compared under different drivers’ adherence rate to the merging instructions, truck percentage in the traffic composition, and traffic demand volumes. Results indicated that throughputs are higher in general under the early SDLMS, travel times are lower under the early SDLMS. However, overall, the early SDLMS resulted in the highest speed variance among MOT types. The MAS resulted in the lowest speed variances overall.

ACKNOWLEDGMENTS

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It is my greatest pleasure to dedicate this small achievement to my parents in Lebanon and my uncle and his family in Orlando for their continuous and unconditional love and support.

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CHAPTER 1 INTRODUCTION

1.1. Work Zone Issues

Traffic safety and efficiency of roadway work zones have been considered to be one of the major concerns in highway traffic operations in Florida. Due to the capacity diminution resulting from the lane closure, congestion will occur with a high traffic demand. Moreover, the mandatory merging to the open lane increases number and severity of traffic conflicts which raises the potential for accidents. Consequently work zones became a challenge for traffic safety and operations engineers.

1.2. Work Zone Lane Management Schemes

To improve traffic safety and mobility in work zone areas, dynamic lane management systems also known as the dynamic lane merging (DLM) system, intelligent work zone traffic control system, have been introduced in several states of the U.S. The DLM can take two forms; dynamic early merge and dynamic late merge. The dynamic aspect of the DLM systems allow them to respond to real-time traffic changes via traffic sensors. The idea behind the dynamic early merge is to create a dynamic no-passing zone to encourage drivers to merge into the open lane before reaching the end of a queue and to prohibit them from using the closed lane to pass vehicles in the queue and merge into the open lane ahead of them (Tarko and Vegopal, 2001). A typical early merge DLM system consists of queue detectors and “DO NOT PASS WHEN FLASHING” signs that would be triggered by the queue detectors. When a queue is detected next to a sign, the next closest sign's flashing strobes, upstream, are activated creating the no-passing zone (Tarko et al., 1998).

The concept behind late merge is to make more efficient use of roadway storage space by allowing drivers to use all available traffic lanes to the merge point. Once the merge point is reached, the drivers in each lane take turns proceeding through the work zone (McCoy and Pesti, 2001). A typical dynamic late merge system consists of several PCMSs that would be activated under certain traffic conditions to display "USE BOTH LANES TO MERGE POINT" and a PCMS at the taper advising drivers to "TAKE TURNS / MERGE HERE". In contrast to the static lane merging, the DLM systems respond to real-time traffic changes via traffic sensors. The real-time traffic data acquired by the sensors are communicated to a central controller in a time-stamped manner. Appropriate algorithms determine whether to activate real-time lane merging messages to drivers based on preset traffic characteristics thresholds.

1.3. Research Motivation

After investigating Fatality and Analysis Reporting System (FARS), it was found that Florida's work zones fatalities are rising significantly compared to other states. Subsequently a Florida freeway work zone crash data analysis was conducted and crash traits were exposed. Results indicated the majority of freeways work zone crashes resulted from merging conflicts leading to rear-end and sideswipe crashes. After examining the current Florida work zone Maintenance of Traffic (MOT) plans, known as the Motorist Awareness System (MAS), it was realized that this system is static hence does not react to changing traffic conditions, and does not incorporate a lane management system. Therefore, an ITS-based lane management system, primarily designed to advise drivers on definite merging locations was explored to supplement the existing Florida MOT plans (i.e. MAS) for short term work zones. Previously deployed dynamic lane merging systems comprise several PCMS (or other forms of dynamic message signs) and traffic sensors. The addition of multiple PCMSs to the current FDOT MOT plans may encumber the latter. Moreover, previously deployed DLM systems (dynamic early merge systems and dynamic late merge systems) may require relatively extensive equipment installation and relocation which could be inefficient for short term movable work zones (moving on average every 7 to 10 hours). Therefore, two simplified dynamic lane merging systems (SDLMS) are suggested for deployment and testing on short term work zones. The first SDLMS is a simplified dynamic early merge system (early SDLMS) and the second SDLMS is a simplified dynamic late merge system (late SDLMS). The following chapters elaborate further on the two suggested forms of the SDLMS. This study aims at comparing the effectiveness of both forms of SDLMS to the conventional MAS plans.

1.4. Research Objectives

The main objective of this research is to evaluate the safety and operational effectiveness of the two proposed SDLMS systems. The objectives of this research can be summarized as the following:

1. Explore Florida's work zones crashes characteristics.
2. Investigate current practices and countermeasures used in work zones.
3. Propose a scheme for the field test including the simplified dynamic lane merging system configuration and the approach for data collection.
4. Compare safety and operational MOEs between with and without SDLMS (early and late) system in work zone areas for various traffic settings.
5. Provide field observations and recommendations regarding the system implementation.
6. Simulate a two-to-one work zone configuration in VISSIM and generalize the effectiveness of these recommendations to various traffic demands and motorists' adherence level.

1.5 Organization of the Dissertation

The dissertation is organized into eight chapters. The description of these chapters is given below:

Chapter 1 provides the motivation, background and objectives of the research for this dissertation and the need for dynamic lane management in wok zone. Chapter 2 lists the literature review in the field of work zone safety and operational concerns and the countermeasures and practices. Chapter 3 provides an insight on the Florida specific work zone crashes characteristics for single and two-vehicle accidents. Chapter 4 provides a description of the designed modified MOT plans, the SLDMS system's equipment, the systems' requirements, the equipment installation and relocation, the system's operation, and the entities involved in the deployment. Chapter 5 provides results of a deployment on a two-to-one work zone lane closure configuration. Chapter 6 provides results of a deployment on a three-to-two work zone lane closure configuration. Chapter 7 provides a simulation of a two-to-one work zone lane closure configuration and the resulting recommendation under various traffic settings and motorists' compliance rates. Chapter 8 summarizes the contribution of this dissertation and lists the conclusions and directions for future research.

CHAPTER 2 LITERATURE REVIEW

The first section of the literature review presents a synopsis of work zones safety aspects including crash rates, crash severity, contributing factors, crash types, and traditional safety countermeasures deployed in work zones. This section also exposes the road geometry, environment, and vehicle factors affecting work zone capacity. The second section provides a summary on ITS applications in work zones known as “smart work zone”. The third section explores previous dynamic lane management in work zones.

2.1 Safety Concerns at Work Zones

2.1.1 Crash rates at work zones

According to the Fatality and Analysis Reporting System (FARS), Florida fatal work zone crashes have risen over 300% since 1999 (See Figure 2.1.1), ranking Florida the second highest state in fatal work zone crashes after the state of Texas (Fatality Analysis Reporting System (FARS), 2006). Several studies were undertaken to assess the safety of highway construction zones in numerous states of the United States. These studies corroborate that work zones produce a significantly higher rate of crashes under certain conditions when compared to non-work zone locations. In particular, Hall et al. (1989) stated that work zones are responsible for a 26% increase in motor vehicle crashes during construction or roadway maintenance. Moreover, Rouphail et al. (1988), Garber and Woo (1990), Nemeth and Migletz (1978), Pigman and Agent (1990), Zhao (2001) Pal and Sinha (1996), Garber and Zhao (2002), Khattak et al. (2002) investigated crash rates at work zone and concluded that under certain conditions work zones generate significantly higher rates of crashes compared to non-work zone locations. Pratt et al. (2001) analyzed workers fatalities in American highway work zones between 1992 and 1998 and underlined the need to mitigate workers risk at work zones. Gundy (1998) presented a review of existing empirical studies and literature concerning work zone traffic accidents, and concluded that accident rates in work zones are higher than similar non-work zone locations. Table 2.1.1 summarizes the studies' results concerning crash rates.

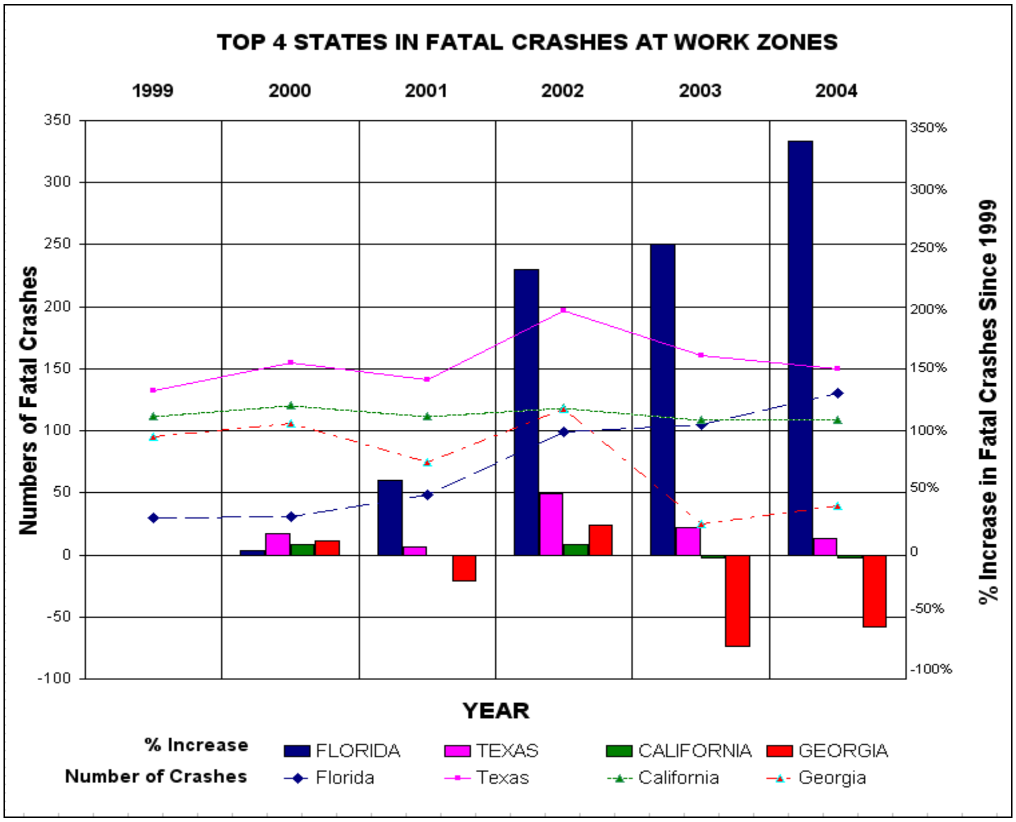


Figure 2.1.1: Top four States in Fatal Crashes in the U.S.

Table 2.1.1: Summary of studies' results concerning crash rates

SUBJECT	STUDIES	RESULTS
Crash Rates	Hall et al. (1989) Rouphail et al. (1988) Garber and Woo (1990) Nemeth and Migletz (1978) Pigman and Agent (1990) Gundy (1998) Pratt et al. (2001) Zhao (2001) Garber and Zhao (2002) Khattak et al. (2002)	Work zones produce significantly more crashes than non-work zones

2.1.2 Crash severity at work zones

The severity of crashes at work zone locations was compared to the severity of crashes at non-work zone locations by several studies. However, the findings of these studies were inconsistent. For instance, Ha and Nemeth (1995), Nemeth and Migletz (1978), Nemeth and Rathi (1983), and Rouphail et al. (1988) stated that work zone crashes were "to some extent" less severe than non-work zone crashes. On the other hand, Pigman and Agent (1987) and "Summary Report on Work Zone Accidents" (1987) reported that work zone crashes are more severe than non-work zone crashes. Moreover, Hall and Lorenz (1989) and Garber and Woo (1990) stated that there is no significant statistical difference between the crash severity at work zone and non work zone locations. Another study by Hargroves (1981) indicated that the average work zone crash was slightly more severe than non-work zone crashes in terms of the average property damage and the number vehicles involved in the crash. This study also concluded that the average work zone crash was slightly less severe than non-work zone crashes in terms of property damage only (PDO) crashes and the number of people injured or killed in the accident. Zhao et al. (2001) specified that 1% of the work zone crashes are fatal, 38% result in injuries and 61% in PDO (property damage only). Table 2.1.2 summarizes the studies' results concerning crash severity.

Table 2.1.2: Summary of study results concerning crash severity

SUBJECT	STUDIES	RESULTS
Crash Severity	Ha and Nemeth (1995) Nemeth and Migletz (1978) Hargroves (1981) Nemeth and Rathi (1983) Rouphail et al. (1988)	Work zone crashes are slightly less severe than non-work zone crash.
	Pigman and Agent (1987) "Summary Report on Work Zone Accidents" (1987)	Work zone crashes are more severe than non-work zone crash.
	Hall and Lorenz (1989) Garber and Woo (1990)	No difference between work zone and non-work-zone crash severity.

2.1.3 Crash types at work zones

Several studies indicated that rear-end collisions are the predominant type of collision at work zones (Garber and Woo (1990), Goddin (1999), Ha and Nemeth (1995), Hall and Lorenz (1989), Hargroves (1981), Nemeth and Migletz (1978), Nemeth and Rathi (1983), Pigman and Agent (1987), Rouphail et al. (1988), "Summary Report on Work Zone Accidents" (1987)). Zhao (2001) determined that rear-end is the predominant crash type at work zones (See Figure 2.1.2). Lervag and Fjerdingen (2003) indicated that in addition to rear-end collisions at work zones that sideswipe and same directions crashes are over-represented compared to road sections without work zones. Khattak et al. (2002) also found that rear-end collisions and sideswipe accidents are overrepresented in work zone areas compared to non-work zone areas.

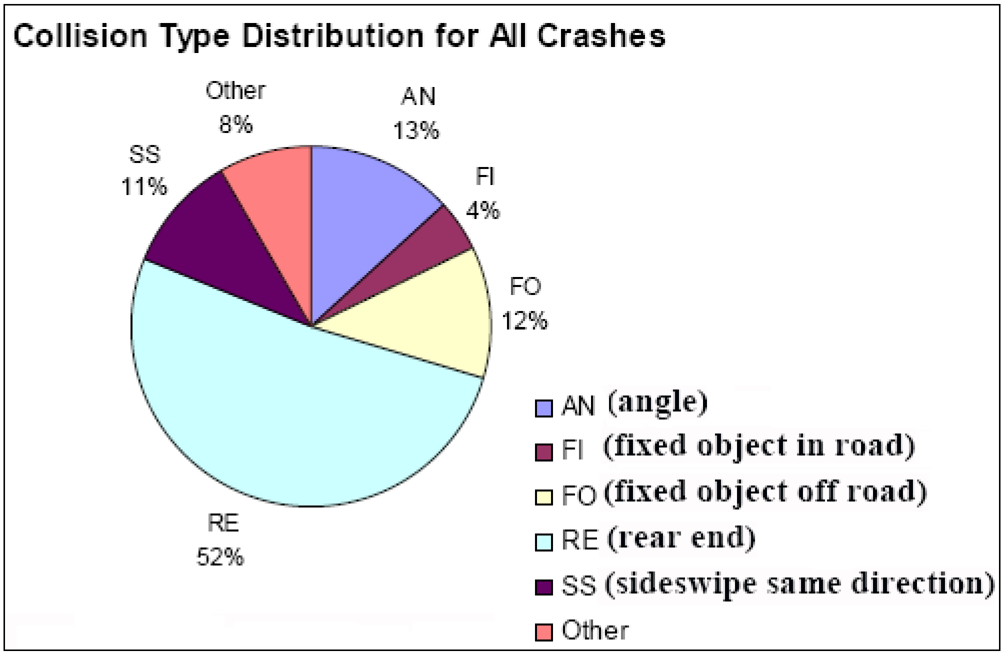


Figure 2.1.2: Collision type distribution at work zones (Zhao, 2001)

2.1.4 Contributing factors

2.1.4.1 Vehicles and drivers characteristics of work zone crashes

Several studies (Hall and Lorenz (1989), Rouphail et al. (1988), Garber and Woo (1990), Pigman and Agent (1987)) indicated that multi-vehicle crashes are over-represented at work zone areas. Moreover, some studies showed that heavy vehicles were overrepresented in work zone areas (Hall and Lorenz (1989), Pigman and Agent (1987), Nemeth and Rathi (1983)). Furthermore, Pigman and Agent (1987) stated that work zone crashes involving heavy vehicles were more severe than work zone accident not involving heavy vehicles. Benekohal et al. (1995) found that 90 % of truck drivers in a survey conducted in Illinois felt that driving through work zones was more hazardous than driving in other areas. Chambless et al (2001) presented several drivers' behavior parameters that contribute work zone crashes:

- Misjudging stopping distance
- Following too closely
- Improper lane change

Garber and Zhao (2002[a], 2002[b]) suggested that a major causal factor for work zone crashes is speed related. The accidents are mainly caused by speed differentials resulting in a speed variance. Raub et al. (2001) indicated that distraction from work in progress, failure to yield at the taper point, and excessive speed are over-represented causes for work zone crashes.

2.1.4.2 Environment characteristics at work zone crashes

Night time (or during darkness) crashes are more severe than day time crashes (Pigman and Agent, 1987). However, Nemeth and Migletz (1978) indicated that day light or day time crashes at work zones are more severe than night time work zone crashes. Chambless et al. (2001) indicated that road defects and vision obstruction are overrepresented parameters in work zone crashes. Raub et al. (2001) indicated that narrower lanes and concrete barriers make it hard for drivers to maneuver and avoid accidents. Several studies were carried out to study the crash location distribution within work zones. Raub et al. (2001) studied the location of crashes within work zones in Illinois. They divided work zones into four areas; the approach area, the taper area (transition area), the construction area, and the exit area. They found that:

- Almost 40% of the work zone accident occurred in the approach and transition area, and that more than 30% of this crashes involved injury and two vehicles.
- Crashes in the working area usually involved more than two vehicles, most commonly resulting in property damage only.

Garber and Zhao (2002) also studied the location of crashes within work zones in Virginia by splitting the work zone into five areas; advance warning area, transition area, longitudinal buffer area, activity area and buffer area. Their results indicate that the activity area was the predominant location for crashes both in total number of accidents and in number of fatal accidents.

2.1.5 Traditional safety countermeasures at work zones

- **Warning lights:** Ullman et al. (1998) stated that more colorful warning lights imply greater sense of urgency and they recommended the use of more colors, especially blue, for special flashing warning signs. A study conducted by Finley et al. (1999, 2001) suggested that sequential warning light systems improve traffic safety by encouraging drivers to exit the closure lane farther upstream.

- **Fluorescent signs:** Fluorescent sheeting is different from ordinary sheeting because it absorbs short wavelength solar energy and then reemits the energy as longer wavelength visible lights. This increases the luminance of the sign. The increased luminance in turn provides greater contrast to the surroundings and hence, a more conspicuous sign (Lervag et al., 2003). Carlson et al. (2000) Fontaine et al. (2000), and Eccles and Hummer (2000) studied the benefits of fluorescent signs in work zones and concluded that the latter give some modest benefits.

- **Speed limit:** Speed differential at work zones is one of the most significant contributing factors to crashes. Several studies were undertaken to assess speed related enhancement methods that would reduce traffic speed in work areas. Sakshaug (2002) and Maze et al. (2000) indicated that work zone speed limit should be combined with other regulatory signs. Hall and Wrage (1997) evaluated methods for enhancing motorist compliance with regulatory and advisory speeds in highway work zones and suggested that they might be improved by increasing the device's size and conspicuity. Several studies suggested the use of passive radars which are electronic radars that transmit in the microwave frequency band. Most studies concluded that passive radars have limited, if any, impacts on drivers' behavior in work zones (Hall and Wrage, 1997; Fontaine and Hawkins Jr., 2001; Carlson et al., 2000; Fontaine et al., 2000; Maze et al., 2000).Several studies examined the effect of speed monitoring displays on reducing speeds at work zones. Studies by Hall and Wrage (1997), Fontaine and Hawkins (2001), Pesti and McCoy (2001) and Maze et al. (2000) confirm that these SMDs reduce the average speeds and improve speed compliance. Several Studies tested the effect of using speed cameras on speed reductions at work zones. Elvik et al.

(1997) and Bolling and Nilsson (2001) stated that the use of speed cameras can reduce speeds significantly at work zone.

- **Dynamic message signs:** Dynamic Message Signs (DMS) also termed Changeable Message Signs (CMS) or Variable Message Signs (VMS) are commonly used in work zones. Fontaine et al. (2000), Fontaine and Hawkins (2001), Garber and Srinivasan (1998), Andrew and Bryden (2001), Dudek et al. (2000) conducted studies to explore the effectiveness of DMSs. Their results are consistent in terms of the positive effectiveness of the signs both in giving guidance and information during lane closure and somewhat in reducing speeds. Walton et al. (2001) evaluated the Kentucky's DMS in an effort to draw recommendations for better effectiveness of these DMSs. Authors found that DMSs should not be used to:
    - Replacement of static signs, regulatory signs, pavement markings, standard traffic control devices, conventional warning or guide signs.
    - Replacement of lighted arrow board
    - Advertising
    - Generic messages (e.g. welcome to our state)
    - Test messages
    - Weather related activities
    - Describing recurrent congestions
    - Time and temperature

    - Public service announcement (general traffic safety and non-traffic-related announcements)

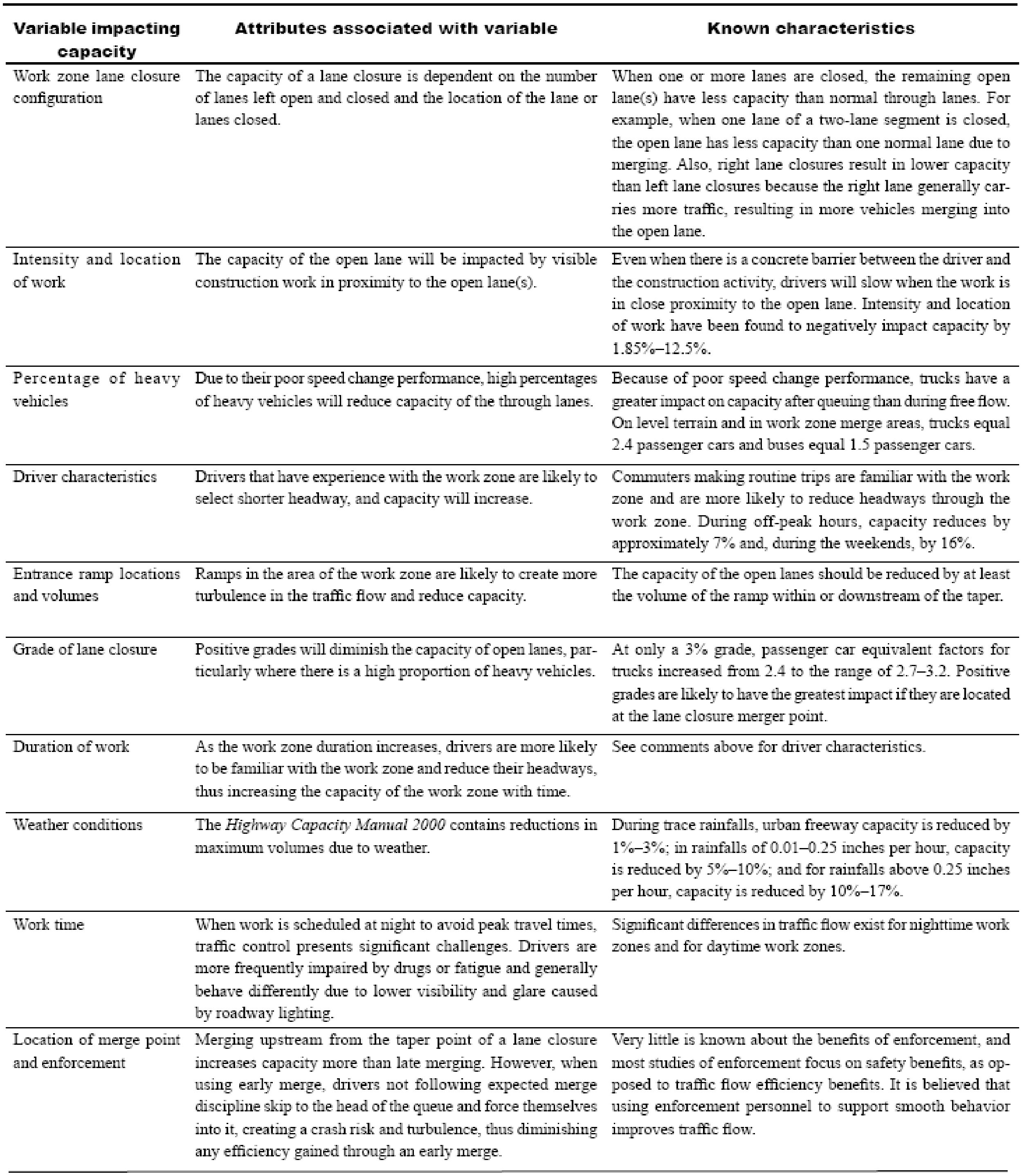
- **Pavement markings and rumble strips:** According to several studies (Noel et al., 1989; Perrillo, 1998; Fontaine at al., 2000; Fontaine and Carlson, 2001; Fontaine and Hawkins, 2001) rumble strips can reduce work- zone accident rates significantly. Berndhardt et al. (2001) showed the importance of pavement markings at work zones especially in guiding the drivers through the work area.

- **Arrow panels**: Arrow panels are commonly used in with work zones guiding the drivers to merge to the open lane (Noel et al., 1989). The Oregon department of transportation studied the effectiveness of a "sequentially flashing diamond" arrow panel display as an advance warning caution warning in temporary work zones and the results show that the diamond display mitigated speeds significantly (Griffith and Reid, 2002).

2.1.6 Factors Affecting Work Zone Capacity

In addition to creating safety issues, work zones are responsible for almost 24% of the non recurring congestions on the United States highway system (Oak Ridge National Laboratory, 2002) and are ranked second to cause drivers dissatisfaction (Keever et al., 2001). Maze and Bortle (2005) published a report titled "Synthesis and Procedures to Forecast and Monitor Work Zone Safety and Mobility Impacts" where they summarized the variables known to affect work zone operations (i.e. capacity). Table 2.3 below is borrowed from the report and exposes these variables. According to Maze and Bortle (2005), work zone lane closure configuration (i.e. number of the lanes left open and the location of the closed lane) affects the work zone capacity significantly. Another factor is the intensity and location of work. For instance, the capacity of a lane closure decreases when work is more intense under the same work zone settings. An increase in the percentage of heavy vehicles in the traffic composition was also found to reduce capacity. Also according to Maze and Bortle (2005), an increase in the drivers' familiarity with a certain work zone increases its capacity. Entrance ramps in the area of work zones diminish their capacity due to amplified turbulence in the traffic. Positive grades reduce the capacity of work zones especially with high proportions of heavy vehicles. Adverse weather conditions diminish the capacity of work zones. The time of work zone is also a significant factor negatively associated with work zone capacity. In fact, Maze and Bortle (2005) stated that during night time, work zone capacity is reduced due to the fact that drivers are often impaired by alcohol and/or fatigue and the fact that visibility may be limited. The location of the merge point is also significantly correlated with the work zone capacity. In fact, early merging increases the capacity of the work zone compared to the late merging, however, incompliance with the merge discipline increases the turbulence in traffic (Maze and Bortle, 2005).

Table 2.3: Variables affecting work zone capacity (Source: Maze and Bortle, 2005)



Variable impacting capacity	Attributes associated with variable	Known characteristics
Work zone lane closure configuration	The capacity of a lane closure is dependent on the number of lanes left open and closed and the location of the lane or lanes closed.	When one or more lanes are closed, the remaining open lane(s) have less capacity than normal through lanes. For example, when one lane of a two-lane segment is closed, the open lane has less capacity than one normal lane due to merging. Also, right lane closures result in lower capacity than left lane closures because the right lane generally carries more traffic, resulting in more vehicles merging into the open lane.
Intensity and location of work	The capacity of the open lane will be impacted by visible construction work in proximity to the open lane(s).	Even when there is a concrete barrier between the driver and the construction activity, drivers will slow when the work is in close proximity to the open lane. Intensity and location of work have been found to negatively impact capacity by 1.85%–12.5%.
Percentage of heavy vehicles	Due to their poor speed change performance, high percentages of heavy vehicles will reduce capacity of the through lanes.	Because of poor speed change performance, trucks have a greater impact on capacity after queuing than during free flow. On level terrain and in work zone merge areas, trucks equal 2.4 passenger cars and buses equal 1.5 passenger cars.
Driver characteristics	Drivers that have experience with the work zone are likely to select shorter headway, and capacity will increase.	Commuters making routine trips are familiar with the work zone and are more likely to reduce headways through the work zone. During off-peak hours, capacity reduces by approximately 7% and, during the weekends, by 16%.
Entrance ramp locations and volumes	Ramps in the area of the work zone are likely to create more turbulence in the traffic flow and reduce capacity.	The capacity of the open lanes should be reduced by at least the volume of the ramp within or downstream of the taper.
Grade of lane closure	Positive grades will diminish the capacity of open lanes, particularly where there is a high proportion of heavy vehicles.	At only a 3% grade, passenger car equivalent factors for trucks increased from 2.4 to the range of 2.7–3.2. Positive grades are likely to have the greatest impact if they are located at the lane closure merger point.
Duration of work	As the work zone duration increases, drivers are more likely to be familiar with the work zone and reduce their headways, thus increasing the capacity of the work zone with time.	See comments above for driver characteristics.
Weather conditions	The *Highway Capacity Manual 2000* contains reductions in maximum volumes due to weather.	During trace rainfalls, urban freeway capacity is reduced by 1%–3%; in rainfalls of 0.01–0.25 inches per hour, capacity is reduced by 5%–10%; and for rainfalls above 0.25 inches per hour, capacity is reduced by 10%–17%.
Work time	When work is scheduled at night to avoid peak travel times, traffic control presents significant challenges. Drivers are more frequently impaired by drugs or fatigue and generally behave differently due to lower visibility and glare caused by roadway lighting.	Significant differences in traffic flow exist for nighttime work zones and for daytime work zones.
Location of merge point and enforcement	Merging upstream from the taper point of a lane closure increases capacity more than late merging. However, when using early merge, drivers not following expected merge discipline skip to the head of the queue and force themselves into it, creating a crash risk and turbulence, thus diminishing any efficiency gained through an early merge.	Very little is known about the benefits of enforcement, and most studies of enforcement focus on safety benefits, as opposed to traffic flow efficiency benefits. It is believed that using enforcement personnel to support smooth behavior improves traffic flow.

2.2 ITS applications in work zones

Several states in the U.S., in an effort to enhance safety and mobility at work zones, deployed ITS technologies in work areas commonly referred as Smart Work Zones. The Smart Work Zone usually provides advanced traveler information to drivers to advise of delay and assist them in deciding whether to use alternate routes. Other types of Smart Work Zone were designed to address concerns with speed management and lane merging conflicts in work zones (lane merging is discussed in section 2.3). Several factors are associated with the success of these systems such as age, gender, trip purpose, network familiarity, education, and trust in the messages content. According to Peeta et al. (2000) the responsiveness of the drivers to these messages increased when at least two pieces of information are provided together.

2.2.1 Minnesota Smart Work Zone

In 1996, the Minnesota Department of Transportation was one the first state departments of transportation to deploy and begin experimenting the smart work zone concept. Their system used several semi-portable field units that transmit traffic data to the Traffic Management Center (TMC). The data is reviewed by an operator at the TMC and messages were displayed on the permanent and portable message signs in the vicinity of the work zone accordingly (SRF Consulting Group, 1997).

2.2.3 Wisconsin Smart Work Zone

A field study was conducted in Wisconsin to investigate the drivers' response to the messages displayed by the Smart Work Zone signs in a rural area. The messages displayed by the signs included the distance to the work zone taper and the travel time to the end of the work zone. Alternate route advisories were not provided to drivers on the dynamic message signs. However, alternate routes were marked on static signs should motorists choose to use alternate routes. The results indicated that alternate route selection increased by 7 to 10 per cent during peak hours (Horowitz et al., 2003).

2.2.4 Nebraska Smart Work Zone

A field study was conducted in Nebraska to explore the response of drivers to advanced advisory information approaching a work zone. In this application of the Smart Work Zone concept, when delay exceeded 5 minutes' delays advisories are provided. When delays exceed 30 minutes a message "CONSIDER ALT ROUTE" is displayed without specific alternate route advisory. Alternate route use increased from 7% when the signs were blank to 11% of freeway traffic when an alternate route advisory was provided (Fontaine, 2003).

2.2.5 Arkansas Smart Work Zone

A Smart Work System, similar to the Nebraska and Wisconsin system, was deployed in Arkansas. Tudor et al. (2003) conducted a study where they compared the crash rates of the Smart Work Zone to two other control sites with similar characteristics with no Smart

Work Zone. Using the number of crashes per million vehicle miles traveled as a measure of effectiveness, the fatality rate decreased from 3.2 and 3.4 at the sites without the Smart Work Zone system to 2.2 at the sites with the Smart Work System. The average overall crash rate reduction was 33%. The average rear-end crash reduction was 7%. Traffic counts also showed that the alternate route use increased when back-up advisory message without identifying alternate route was displayed.

2.2.6 Missouri Smart Work Zone

Another Smart Work System was deployed and explored in Missouri. King et al. (2004) examined the use of this system that consisted of an automated system which advises drivers when delays and speed reductions were occurring at work zone sites. The analysis showed that this system had a positive effect on the safety of work zone. In fact, there was a positive effect on the reduction of the mean speed and the speed variance as the traffic neared the work zone.

2.2.7 Michigan Smart Work Zone

A different type of Smart Work Zone was deployed in Michigan. A variable speed limit (VSL) system was deployed in Michigan to manage speeds through work zones under different traffic and environmental conditions. The system monitors traffic flow and the surface condition to detect the presence of water, ice, or snow. Based on these conditions speed limits are determined and posted for drivers. As a conclusion, Lyles et al. (2004) stated that “the VSL system can present far more credible information (realistic speed limits) to the motorist , responding to both day-to-day changes in congestion as well as significant changes in congestion and geometry as motorists go through a given zone".

2.2.8 North Carolina Smart Work Zone

The North Carolina Department of Transportation was concerned about the safety and mobility of drivers on I-95 since it was undergoing major rehabilitation and resurfacing. To address their concerns the NCDOT begun deploying advanced technology to enhance safety and mobility of their work zones. A system that consisted of portable changeable message signs located along the approach of the work zone site providing motorists with advisory information of delays and suggesting alternate routes when necessary. The results showed that alternate route use increased from 10 to 15 per cent. Moreover, a survey conducted showed that 80% of the drivers were pleased with the information given by the dynamic signs. As for the safety improvements the authors indicated that there were not enough data to draw conclusions concerning the safety of drivers in work zones with the deployment of the Smart Work Zone System (Bushman et al., 2004).

2.3 Previous DYNAMIC LANE MERGING (DLM) Applications

When traffic demand exceeds the capacity of a work zone, queues expand beyond the advance warning signs, often surprising the oncoming vehicles thus increasing the crash potential. The early and late merge routines are two strategies that were designed with the intent to resolve these problems. The early merge and late merge strategies take two forms: static and dynamic. The following sections elaborate on these systems.

2.3.1 Early Merge Strategy

The early merge strategy encourages earlier merging in advance of work zone lane closures to lower the potential for merging friction at the merge point of a lane closure. A disadvantage of this strategy is that it requires additional signage and supplementary control measures further upstream of a lane closures which can make the maintenance of traffic control more difficult (Beacher et al., 2004). The early lane merge strategy can take two forms: static and dynamic. These two concepts will be further explained.

2.3.1.1 Static Form

The static form of lane merging does not change in real time in response to traffic conditions. The static form typically includes additional "LANES CLOSED" sings placed upstream of lane closure on average at 1-mile intervals (McCoy and Pesti, 2001). The static early merge strategy is intended to mitigate rear-end collisions by forewarning drivers of latent slowing traffic. Other static methods for promoting early merging comprise the use of supplementary control measures (Beacher et al., 2004). Bernhardt et al. (2001) studied numerous supplementary traffic control measures to encourage early merging at work zones. Bernhardt et al. (2001) evaluated several supplementary traffic control measures including the following:

- White lane drop Arrows:

This method led to a 4.2% increase in the number of vehicles in the open lane at the work zone taper. Mean speeds decreased by 6.1 mph under congested conditions. The number of vehicles below the speed limit under uncongested conditions increased by 14.8%. A decrease of 10.3 mph in the mean speeds of the fastest 15 % of vehicles occurred under congested conditions.

- The Wizard Work Zone Alert and Information by TAFCON:

This method led to an increase in the number of vehicles in the open lane by 12.4% under congested conditions. The number of vehicles traveling below speed limit increased by 11.7% under uncongested conditions.

- Orange rumble strips as a supplement to the standard lane merge configuration:

This method increased the number of vehicles in the open lane at the start of the work-zone taper during congested conditions by 10.2%. For uncongested conditions, the means speeds in the closed lane decreased by 16.1 mph. Uncongested 85th percentile speeds decreased by 6.9 mph and the mean speed of the fastest 15% of vehicles decreased between 6.7 mph and 15.1 mph.

According to Datta et al. (2004) the static lane merge system may confuse drivers, especially under uncongested conditions where the travel speed is high, and the volume is low. Nemeth and Rouphail (1982) found through a simulation study that the early merge strategy significantly reduced the frequency of forced merges, especially at higher traffic volumes. Another simulation study by Mousa et al. (1990) determined that the early merge strategy increased the travel times through the work zone because the vehicles are more likely to be delayed over greater distances by slower vehicles ahead of them in the open lane.

2.3.1.2 Dynamic Form

The dynamic early merge system creates a NO-PASSING zone upstream of a work zone taper based on real-time measurements of traffic conditions (Tarko and Venugopal, 2001). The system consists of queue detectors and “DO NOT PASS WHEN FLASHING” signs that would be triggered by the queue detectors. When a queue is detected next to a sign, the next closest sign’s flashing strobes, upstream, are activated creating the NO-PASSING zone. This system makes queues jumping an illegal task. Figures 2.3.1.1 and 2.3.1.2 illustrate this system**.**

The Indiana Lane Merge System (ILMS) was tested in the field in the 1997 construction season by the Indiana Department of Transportation. It was found that the system smoothes the merging operations in advance of the lane closures. Drivers merged when they were supposed to merge, the flow in the open lane was uniform, and rear-end accident rates decreased. However, this system did not increase the throughput and the results of a simulation study conducted by Purdue University indicated that travel times through work zones with ILMS are larger (Tarko, 1998).

In 1999, the University of Nebraska conducted a study of the Indiana Lane Merge System (ILMS) on I-65 in the vicinity of Remington, Indiana. This study was limited to a four day data collection exclusively under uncongested conditions. In this project, the right lane was closed and the data collected (by video cameras and laser speed gun) and extracted included traffic volumes, speeds, conflicts, lane distributions, flows, and time headways. Comparing the ILMS with the standard MUTCD merge control, the results showed that the ILMS increased the capacity to some extent (from 1,460 to 1,540 vphpl).

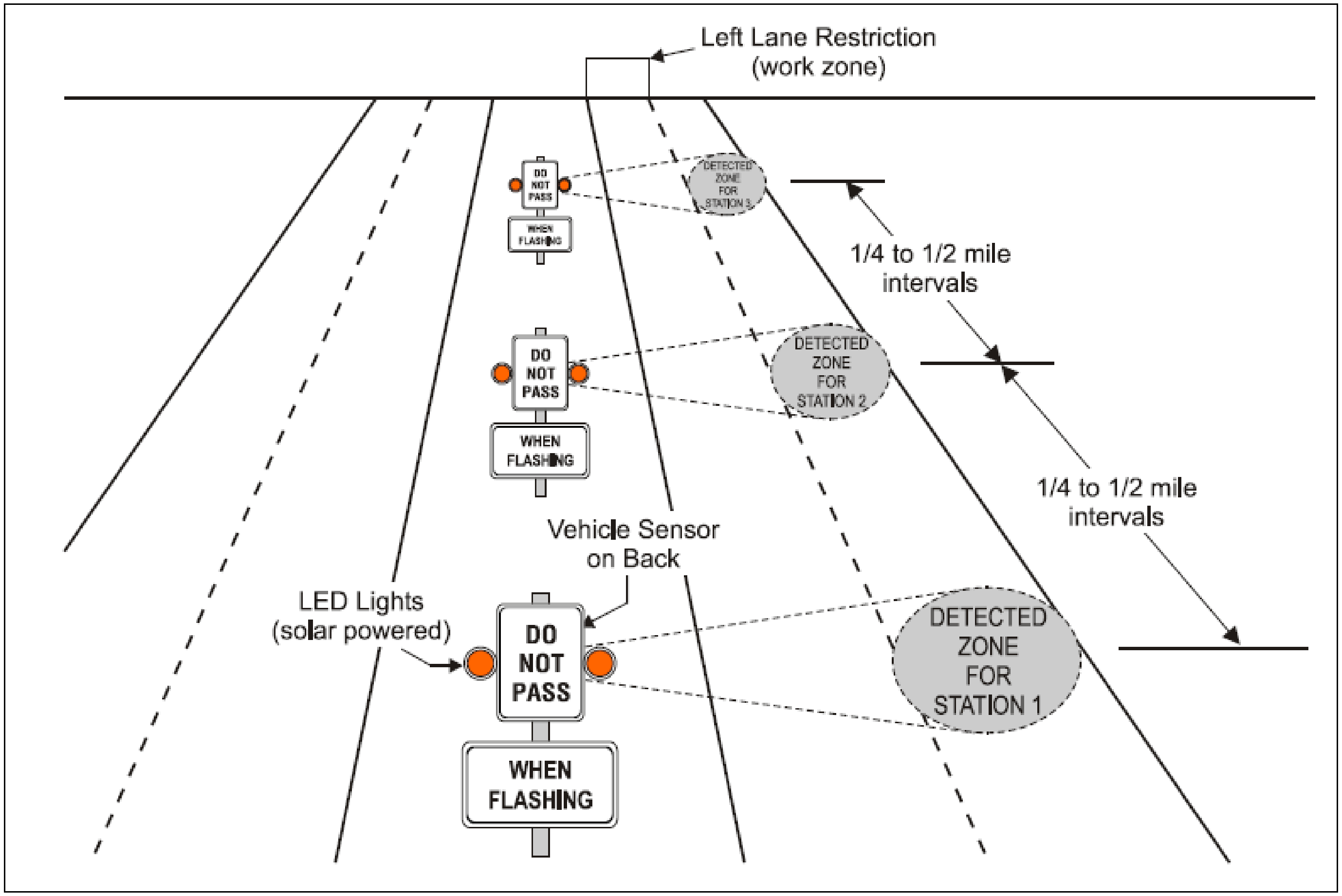


Figure 2.3.1.1: Indiana Lane Merge System (Source: Beacher et al., 2004)

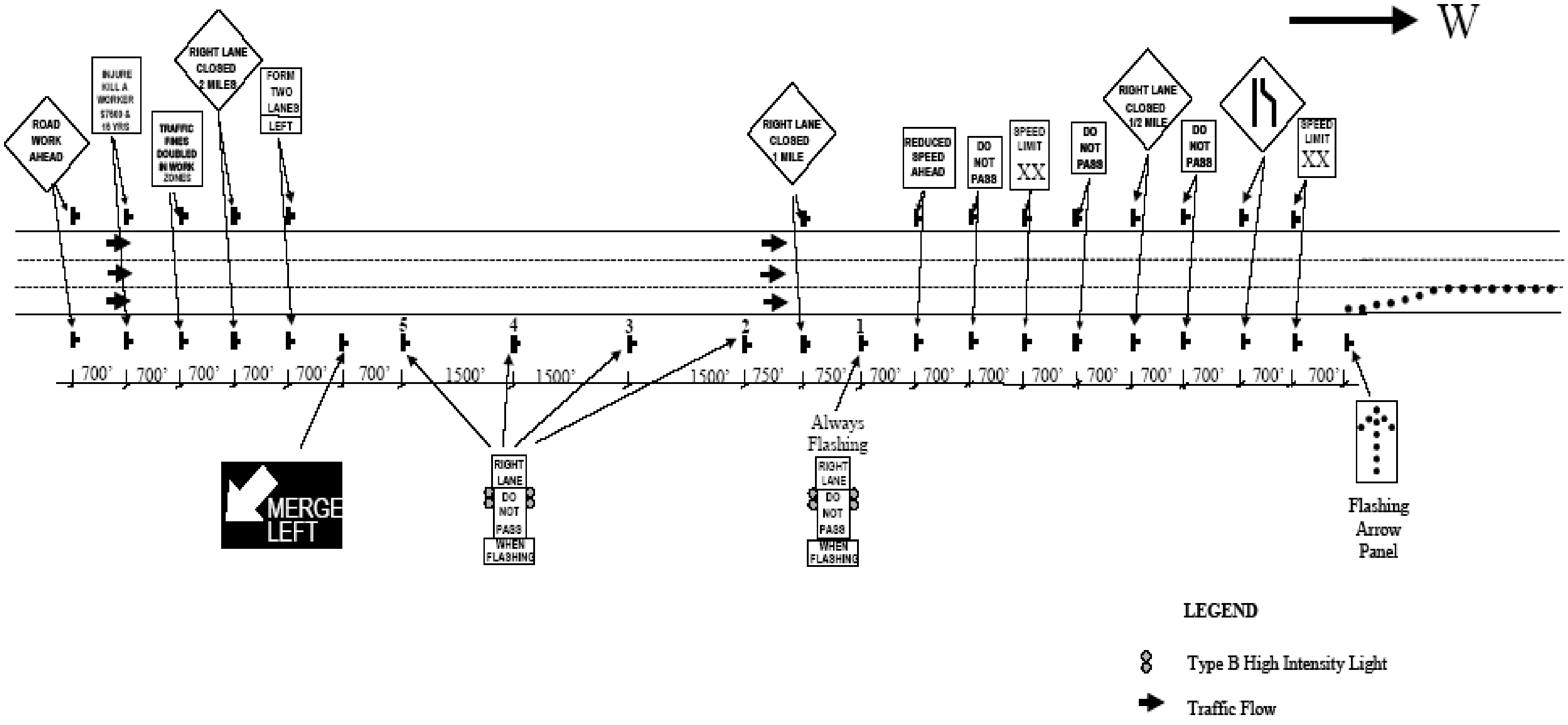


Figure 2.3.1.2: Dynamic Early Lane Merge Traffic Control System Used in Michigan (Source: Datta et al., 2004)

As for the safety aspect of the ILMS, since the data collected was limited to uncongested conditions and to 16 hours of video data, it was not clear whether the ILMS improve safety in terms of number of forced merges (McCoy et al., 1999).

The ILMS was also studied by Purdue University and the results were detailed in a report published in 2001. This system was studied on I-65 near West Lafayette, Indiana. This project entailed extensive data collection under both congestion and uncongested conditions for a duration of four months in 1999. Multiple loop detectors and two cameras were used for data collection. Purdue University studied both the safety effects of the ILMS by developing conflict frequency models as well as capacity effects of the ILMS. The results of the analyses showed that the ILMS decreases the capacity by 5%. The Authors mentioned that the decline in the capacity may be due to the unfamiliarity of the drivers with the system (Tarko and Venugopal, 2001).

The Wayne State University conducted a study to assess the ILMS commonly referred to as Michigan Lane Merge Traffic Control System (LMTCS). This study compared four sites where the system was installed to four control sites where traditional MUTCD merge was implemented. The “DO NOT PASS WHEN FLASHING” signs were activated manually by personnel on the four sites. The lane closure configuration and geometry of freeway sections were homogeneous in the test and control sites for consistency. The data collected included aggressive driver behavior, location of merging, presence of law enforcement. In addition to that, the floating car method was utilized to record travel times and delays. According to their results the ILMS (or LMTCS) increased the average operating speed, decreased the delays (49 vehicle hours of delay per hour), decreased the number of aggressive driving maneuvers during peak hours (from 73 to 33) (Wayne State University, 2001).

The results of the studies on dynamic early merging are mixed. The Wayne State study showed an increase in average operating speeds, a decrease in average delay, no difference in capacity, and a decrease in the number of aggressive driving maneuver during the peak hour (Wayne State University, 2001). The Nebraska study showed few forced merges with the ILMS, however, it was unclear whether this was a result of the ILMS or it was due to the lack of congested conditions during the study. The Nebraska study estimated that the ILMS increases the capacity from 1,460 to 1,540 vphpl (McCoy et al., 1999). The Purdue University study showed that the dynamic early merging decreased capacity by 5% (Tarko and Venugopal, 2001). Table 2.3.1 summarizes the advantages and disadvantages of the dynamic early merge strategy. It should be noted that Table 2.3.1 is not a cross-comparison between each study as each was implemented on different facilities and under different conditions.

Table 2.3.1: Summary of Early Merge Strategy

Static Early Merge		**Dynamic Early Merge**	
Advantages	**Disadvantages**	**Advantages**	**Disadvantages**
Reduces the frequency of forced merges especially at higher traffic volume (*Nemeth and Rouphail, 1982*).	Requires additional signage and supplementary control measures which makes maintenance more difficult (*Beacher et al., 2004*)	Smoothes the merging operations in advance of a lane closure (*Tarko, 1998*)	Travel times through work zones are larger (*Tarko,1998*)
	May confuse drivers under uncongested condition (*Datta et al., 2004*)	Rear-end Accident rates decreased (*Tarko,1998*)	Decrease capacity by 5% (*Tarko and Venugopal, 2001)*
	Increase travel time through the work zone (*Mousa et al. 1990*)	Increase the capacity of work zones under UNCONGESTED conditions (McCoy et al., 1999)	Unfamiliarity of confusion of the drivers with the systems (*Tarko and Venugopal, 2001*)
		Decrease delays (*Wayne State University, 2001*)	
		Decrease in number of forced merges (*Wayne State University, 2001*)	

2.3.2 Late Merge Strategy

The concept behind late merge is to make more efficient use of roadway storage space by allowing drivers to use all available traffic lanes to the merge point. Once the merge point is reached, the drivers in each lane take turns proceeding through the work zone. The combined effect of maximized storage and orderly merging operations may have the potential to increase throughput, reduce queue lengths, shorten travel times, and discourage aggressive driving (Beacher et al., 2004).

2.3.2.1 Static Form

The Pennsylvania Department of Transportation (PennDOT) introduced the static form of the late merge to mitigate aggressive driving and road rage at merge points (McCoy and Pesti, 2001). The PennDOT's late merge strategy's traffic control plan comprises signs calling for "USE BOTH LANES TO MERGE POINT" 1.5 miles upstream of the work zone and "MERGE HERE TAKE YOUR TURN" near the beginning of the taper (See Figure 2.3.2.1). The static late merge strategy was examined by a study conducted in Nebraska and another study conducted by the Texas Transportation Institute (TTI). The Nebraska's research was limited to a 2-to-1 lane reduction scenario. Comparing this static late merge strategy to the standard MUTCD lane merge strategy, the results showed 75% fewer forced merges and an increase from 1,460 to 1,730 pcph in capacity. This study also suggested that an effective signing plan be made available to optimize the potential of the concept. This study also showed that trucks had more difficulty merging from left to right than right to left (McCoy et al., 1999).

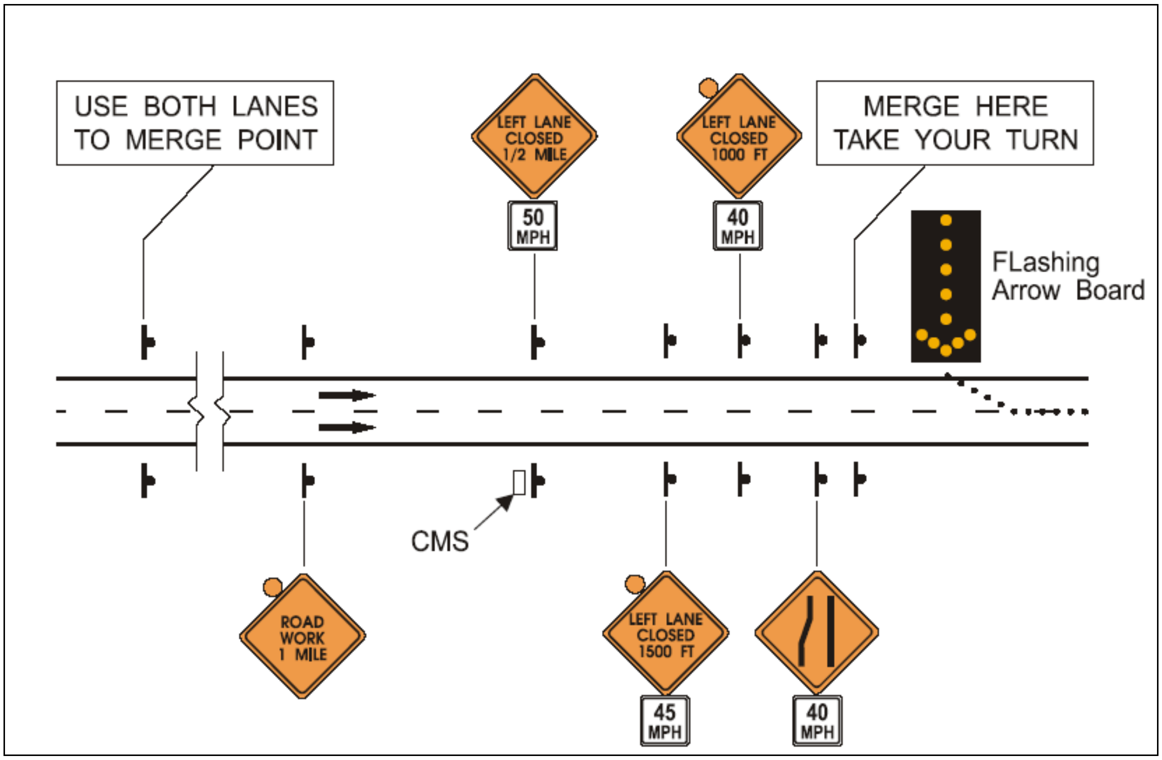


Figure 2.3.2.1 PennDOT's Late Merge Concept (Beacher et al., 2004)

The (Texas Transportation Institute) TTI explored the late merge concept in a 3-to-2 lane closure scenario. The data collection was limited to 1 day under standard MUTCD lane closure and to 1 day under the static late merge strategy. The results of the comparison showed that the late merge strategy delayed the onset of the congestion by 14 minutes, reduced queue length from 7,800 to 6,000 feet. Moreover, an analysis of volumes by lane showed that a larger percentage of vehicles used the open lane with the late merge in place and that more vehicles were able to pass through the merge point (Walters et al., 2001). On the other hand, the University of Nebraska conducted a survey in Pennsylvania to explore the opinion of the drivers regarding the late merge system application. Sixty percent of the truck drivers versus 22 percent of the passenger car drivers stated that they experienced or observed other drivers having difficulty merging. This could be related to the fact that 73% of the truck drivers and 40% of the passenger car drivers did not believe that the signs worked (Byrd, 1999).

2.3.2.2 Dynamic Form

McCoy and Pesti (2001) expressed their concern about the confusion of drivers at the merge point with the late emerge in place. To resolve this issue, they proposed a dynamic late merge in which the late merge would be employed only at times of high congestion. McCoy and Pesti (2001) stated that the late merge can reduce congestions and delays, whereas the early merge increased congestions and delays. Beacher et al. (2004) applied the dynamic late merge system in Tappahannock, Virginia and conducted a before and after study to explore the benefits of the system. Figure 2.3.2.2.1 shows the site diagram with the dynamic late merge system. According to their results, the percentage of vehicles in the closed lane increased significantly from 33.7 to 38.8 percent when comparing the late merge to the MUTCD treatment. The throughput volumes showed no statistical difference between the MUTCD treatment and the late merge. Time in queue was not significantly different between the two types of traffic control. According to Beacher et al. (2004) the lack of improvement in throughput and time in queue may be attributable to the relatively low percentage of heavy vehicles. Beacher et al. (2004) proposed some guidelines for the application of the dynamic late merge system:

- Two-to-one lane closure: the late merge should be considered for 2-to-1 lane closure configurations to improve throughput when large numbers of heavy vehicles are present (>20%) for the majority of the time and congestion and queuing are often present.

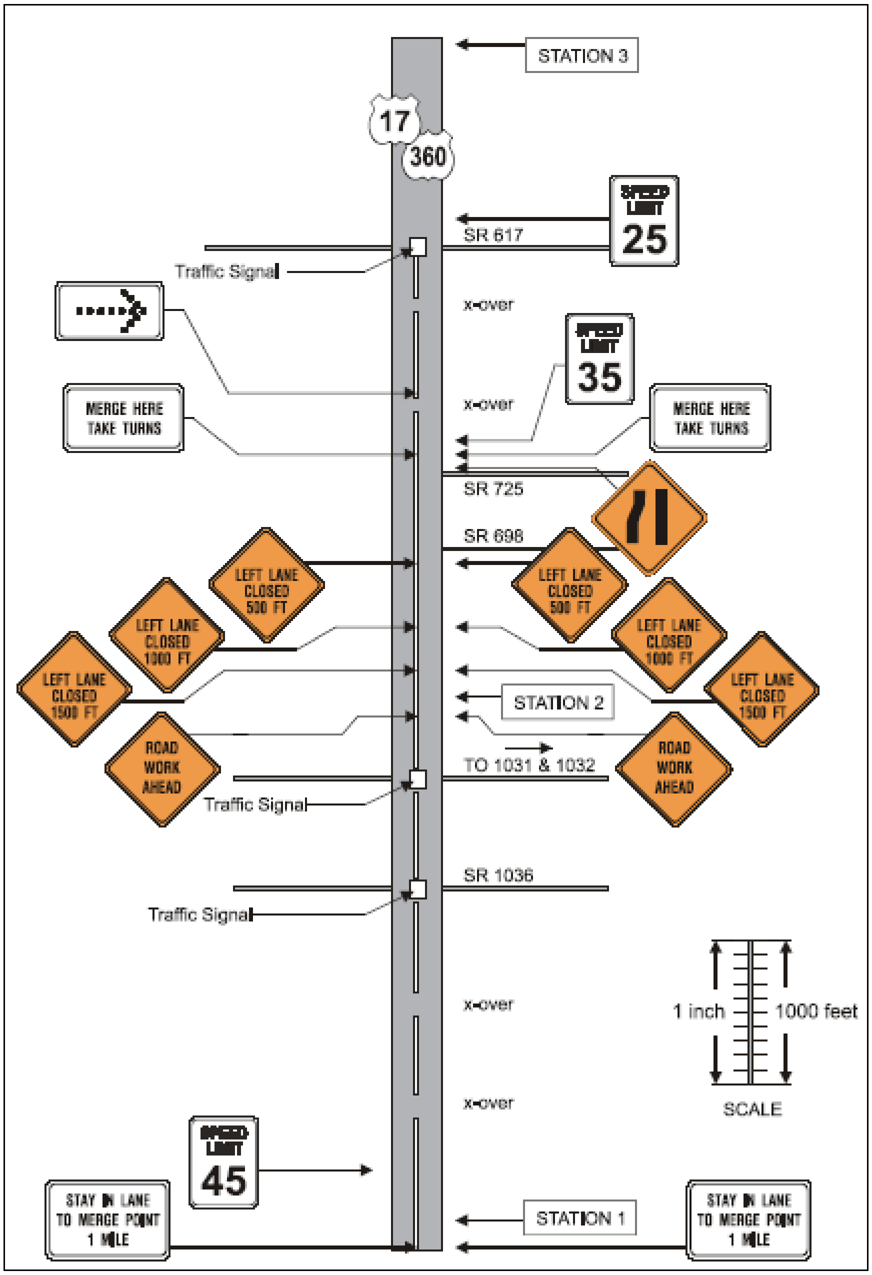


Figure 2.3.2.2.1: Tappahannock, Virginia site diagram (Beacher et al., 2004)

- Three-to-one lane closure: while the simulation results showed that the late merge significantly improved throughput for all situations, there are no documented evaluations of the deployment of the lat merge in this configuration. Further research is needed to determine how the late merge could be deployed in this type of configuration to ensure driver understanding of the signs.

- Three-to-two lane closure: The late merge should be considered in the 3-to-2 configuration as a possible means to improve flow when heavy vehicles represent more than 20 percent of the traffic stream and congestion and queuing are frequent.

In June 2003, the University of Kansas, in cooperation with the Kansas Department of Transportation and the Scientex Corporation deployed the Construction Area Late Merge (CALM) system in Kansas (Scientex; Meyer. 2002). This system is the dynamic version of the Late Merge Concept introduced by PennDOT (See Figure 2.3.2.2.2). This system employs traffic detectors to sense congestion upstream of a construction lane closure. The traffic data is communicated in real-time to a central controller where proprietary software algorithms determine the critical thresholds of traffic density and speed to activate real-time messages directing motorists to remain in their lanes until they approach the lane closure, where they merge alternately by taking turns. The CALM system provides real-time safety alerts to motorists. This system is configured to operate as an early merge system under light traffic loads and as a late merge system under heavier traffic loads (Meyer, 2002). Meyer (2002) reported that the compliance of the drivers with the system increased with time and recommended that drivers be familiarized and trained to the system to optimize the potential merit of the system. The average volume through the work zone was enhanced after the drivers were accustomed with the system.

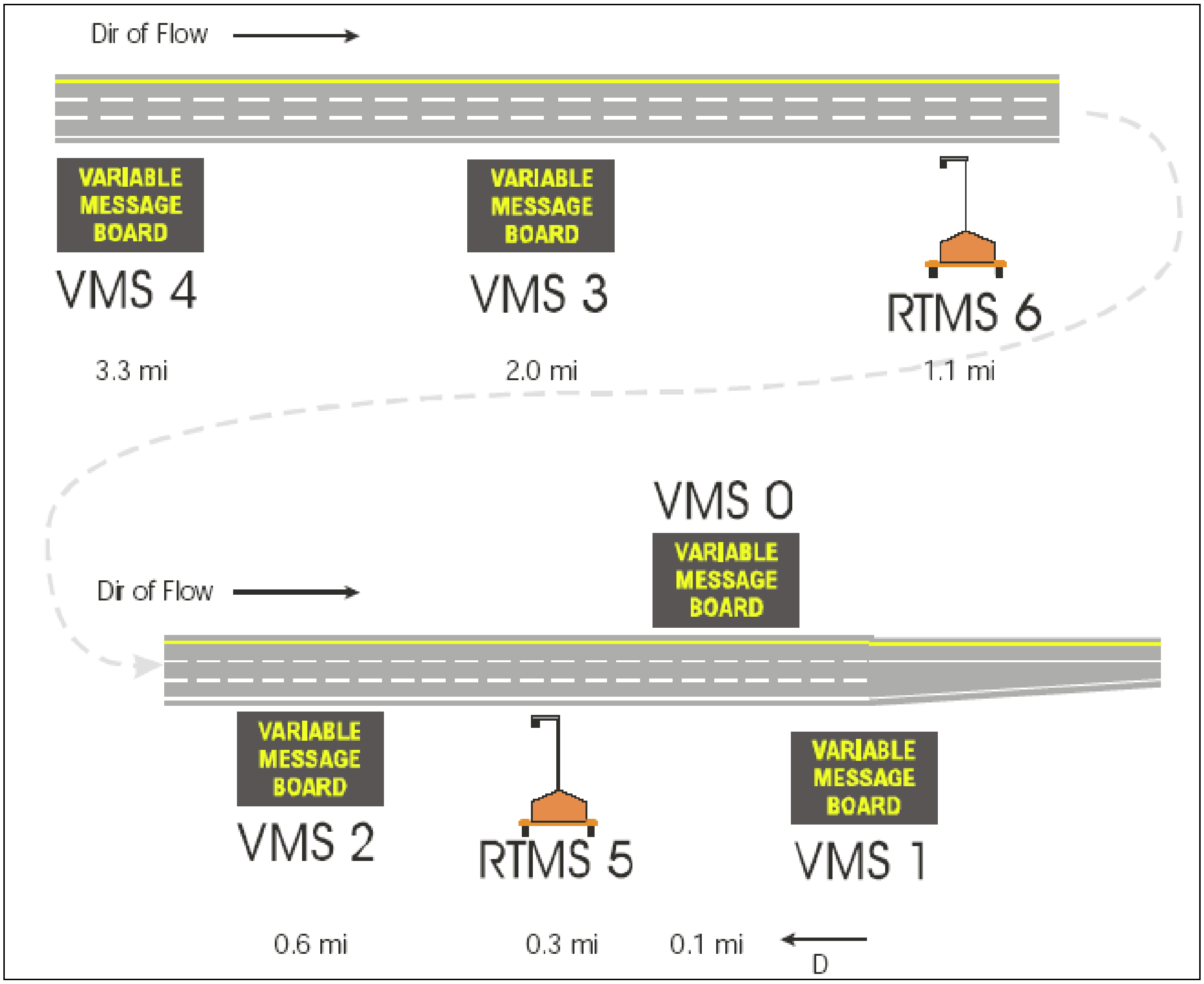


Figure 2.3.2.2.2: CALM System Field Components (Source: Meyer, 2002)

However, the net change in volume did not show a significant improvement over baseline values. Like others, this system also utilized wireless communication between RTMS detectors and portable CMS to display lane use instructions to drivers based on traffic conditions. This system was designed to operate in three distinct modes- Early merge, late merge, and incident. The incident category was a special case of the late merge strategy when traffic speeds were exceptionally low. Transitions between the modes occurred seamlessly based on the current traffic average operating speeds and transition thresholds between the three modes. According to the results, the late merge systems have the potential to improve freeway operations around construction lane closures. The evaluations also highlighted the importance considering the location of entrance and exit ramps when placing the signs and sensors.

Maryland's Dynamic Late Merge (DLM) System comprises a set of 4 portable CMS and 3 RTMS detectors that are added to the standard static traffic control devices utilized at construction lane closures. The CMS furthest upstream (~1.5 miles) from the taper alternated between the messages "USE BOTH LANES" and "TRAFFIC BACKUP". The next two CMS located at approximately ½ mile and ¼ mile from the taper itself, the final CMS alternated between messages "TAKE YOUR TURN" and "MERGE HERE". The location of the CMS and RTMS are shown in Figure 2.3.2.2.3. The University of Maryland, College Park conducted the evaluation of the system by utilizing one day of baseline (or control) data where the road closure utilized only the standard static traffic control signs. This was followed by 4 days with the DLM system activated. Four measures of effectiveness were evaluated; work zone throughput, lane volume distribution, maximum queue length, and simulation data analysis. According to the findings, the DLM increased the work zone throughputs when compared to the baseline conditions. Traffic volumes collected during 10-minute intervals during the 4 days of DLM system deployment were higher than under the baseline conditions. Another method of investigating traffic throughput utilized a calibrated computer simulation. Lane volume distribution was also compared under the baseline and DLM System conditions. The results showed that more vehicles were in the discontinuous lane. Many drivers were observed merging before the designated merge location during the evaluation period. These early merges resulted in multiple merging points and appeared to result in some confusion on the proper place to merge. The queue lengths were observed to be reduced between 8% and 33% during the 4 days evaluation with the activation of the DLM System.

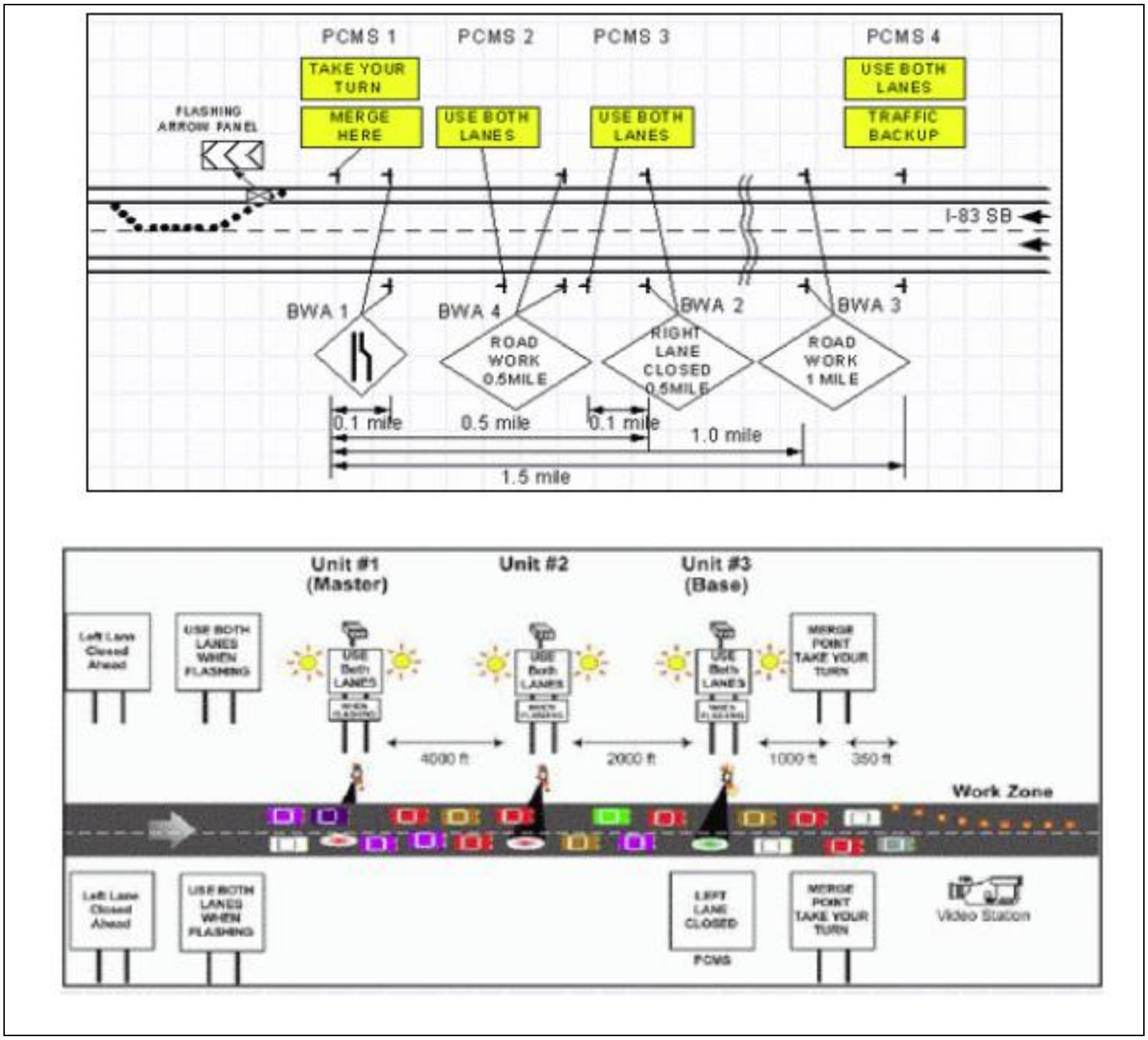


Figure 2.3.2.2.3: Maryland's DLM (An Applied Technology and Traffic Analysis Program)

Unfortunately, numerous traffic conflicts were observed between the two-lane traffic. Many vehicles were observed making forced merges at the taper point because they were not allowed to merge. These conflicts resulted in conditions of stop and go traffic. The authors finally stated that the advantages of the DLM system are increased throughput, shorter queue lengths, and more uniform distribution of lane use before the taper. The disadvantages were listed as increased stop and go conditions and multiple merging points. The authors recommended that future deployments could comprise variable speed limit signs, change the distance between the DLM system equipment based on perception/reaction time based on site-speed characteristics, and remove separate static merging signs for the DLM system to avoid confusion on the correct merging location (An Applied Technology and Traffic Analysis Program).

The Minnesota Department of Transportation (MnDOT) evaluated the Dynamic Late Merge System (DLMS) which consists, in addition to the standard orange and black warning signs placed in advance of the lane closure, of three Changeable Message Signs (CMS) and a Remote Traffic Microwave Sensor (RTMS) detector. When congestion begins to form, the signs are activated to provide lane use instructions to drivers. The CMS farthest from the work zone displays the message “STOPPED TRAFFIC AHEAD-USE BOTH LANES”. The next CMS sign reads “USE BOTH LANES-MERGES AHEAD”. The sign closest to the work zone will show alternating messages of “TAKE TURNS-MERGE HERE” (Figure 2.3.2.2.4). When traffic speeds increase as congestion dissipates, the signs will turn off and the system will return to the typical static work zone traffic control that encourages early merging (Tavoola et al., 2004)

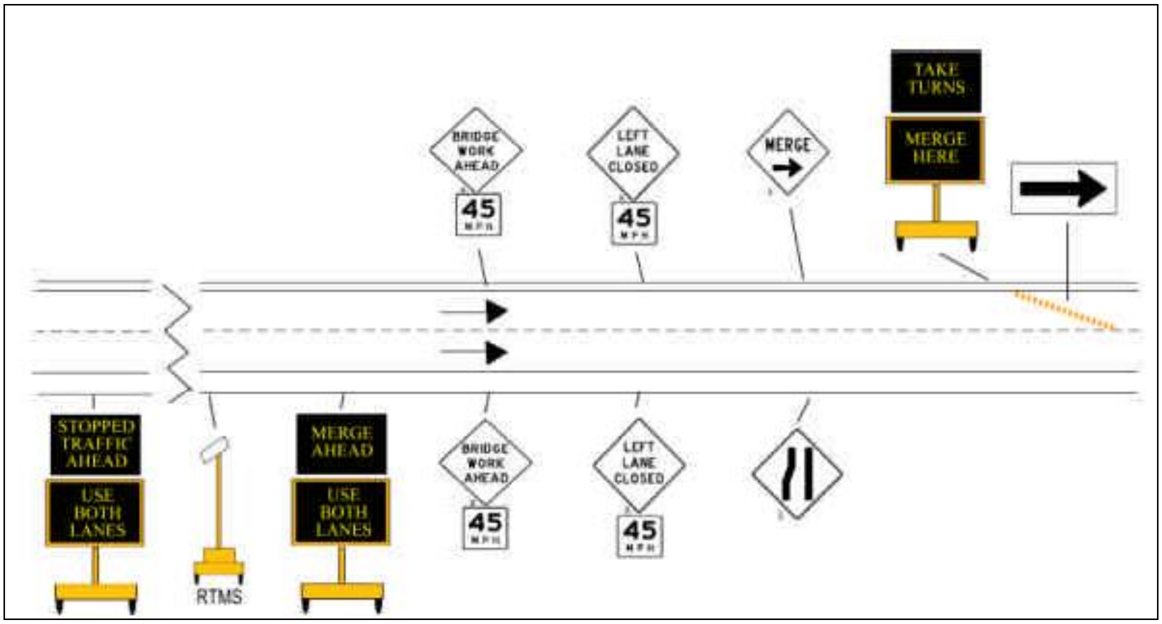


Figure 2.3.2.2.4: Minnesota's DLM (Dynamic Late Merge System Evaluation)

The results of the Minnesota 2004 study showed:

1) The use of the discontinuous lane increased dramatically when the CMS were activated. During the heaviest demand, the discontinuous lane use percentage increased to levels of almost 60% at locations approximately half-mile from the construction taper.
2) The queue lengths were observed to be relatively minimal. It was also observed that some drivers refused to use both lanes and wait in a long single queue.
3) The overall driving conditions were improved upstream of construction lane closures.

4) The maximum volume throughput within the single lane construction closure at deployment locations was nearly identical.

Table 2.3.2.1 summarizes the advantages and the disadvantages of the Late Merge strategy. It should be noted that Table 2.3.1 is not a cross-comparison between each study as each was implemented on a different work zone and under different conditions.

2.4 SUMMARY

The literature review demonstrated that work zones indeed deteriorate safety and operations of roadways. From the safety aspect, work zones produce significantly higher crash rates and result in higher crash severity under certain conditions. From the operations aspect, work zones reduce roadway capacity drastically. The magnitude of the capacity diminution varies under different drivers' characteristics, vehicles' characteristics, and environments' characteristics.

The literature review summarized current practices and countermeasures used in work zones, particularly ITS technologies. ITS technologies in work zone areas, commonly referred as "Smart Work Zones" were categorized in three groups: advanced traveler information systems advising on alternate routes; speed management systems; and lane management systems.

Table 2.3.2.1: Summary of Late Merge Strategy

Static Late Merge		**Dynamic Late Merge**	
Advantages	**Disadvantages**	**Advantages**	**Disadvantages**
75% fewer forced merges (*McCoy et al., 1999*)	Confusion of drivers at the merge point when the static form is employed during low congestions (*McCoy and Pesti, 2001*)	Work zone throughputs increased (*An applied technology…*)	No difference in time in queue when truck percentage is lower than 20% (*Beacher et al., 2004*)
Increase in capacity from 1,460pcph to 1730pcph (*McCoy et al., 1999*)		Queue lengths were reduced between 8% and 33% (*An applied technology…*)	No difference in the throughput volume when truck percentage is lower than 20% (*Beacher et al., 2004*)
Delayed the onset of congestion by 14 minutes (*Byrd, 1999*)		Reduced queue length (Tavoola et al., 2004)	Increased stop and go at the taper point (*An applied technology…*)
Reduced queue length from 7,800ft to 6,000ft (Byrd, 1999)		Enhance the overall driving condition upstream of the lane closure (Tavoola et al., 2004)	

This chapter mainly addressed lane management systems in work zones usually referred to as dynamic lane merging. The two forms of dynamic lane merging (early merge and late merge) were explored separately by several studies and results showed some promising advantages and some disadvantages. Moreover, it was not clear from the literature which dynamic lane merging scheme (early merge or late merge) performs better since all studies compared one form of the dynamic lane merging to a conventional maintenance of traffic plans. Therefore, to determine which lane merging scheme perform better, one should deploy and compare the dynamic early merge and the dynamic late merge under the identical work zone lane closure configuration, similar vehicular traffic, and matching environments' and geometric characteristics.

CHAPTER 3 FREEWAY WORK ZONE CRASH ANALYSES

3.1 Introduction

As mentioned in Chapter 2 work zone safety continues to be a priority and a concern for the FHWA as well as most state departments of transportation (DOTs). The increase in Florida work zone crash fatalities and the significantly higher rate of crashes in work zone areas when compared to non-work zone locations underscore the urgent need to develop a substantive understanding about how Florida's work zone crashes occur and their corresponding risk factors. This task was essential prior to exploring and deploying potential countermeasures in Florida's work zones.

Studies on work zone crashes have typically inspected a combination of injury, fatal, and property damage crashes to discover aspects that contribute to unsafe conditions within work zones. Daniel et al. (2000) focused only on the analysis of fatal crashes within work zones in Georgia since their database did not identify work zones unless there was a fatal injury. This study examined the difference between fatal crashes within work zones compared with fatal crashes in non-work zone locations. The overall findings of the study indicate that work zones influence the manner of collision, lighting conditions, truck involvement, and roadway functional classification under which fatal crashes occur. Ming and Garber (2001) conducted research to uncovera work zone crash attributes accounting for the location of each crash within the work zone and its surroundings in Virginia. However, their study strictly presented statistical summaries and basic inferential statistics of these crashes and their attributes without relating to interactions and confounding effects. This study concluded that work zone crashes are predominant in the activity area and that there is a higher rate of multi-vehicle accident in work zone locations compared to non-work zone locations. Benekohal et al. (1995) considered exclusively the effect of trucks and their involvement in work zone crashes. Their study indicated that the accident experiences were significantly related to the experience of bad driving situations but not other driver/truck characteristics. However, other studies showed that heavy vehicles were overrepresented in work zone areas (Hall and Lorenz (1989), Pigman and Agent (1987), Nemeth and Rathi (1983)). Garber and Zhao (2002^a, 2002^b) suggested that a major causal factor for work zone crashes is speed related. The accidents are mainly caused by speed differentials resulting in a speed variance. Raub et al. (2001) indicated that distraction from work in progress, failure to yield at the taper point, and excessive speed are over-represented causes for work zone crashes.

The lack of literature concerns the overall aspect of the crash traits at work zones such as environment, vehicle, and driver characteristics and their interactions. Therefore, this study aims at evaluating freeway single-vehicle and two-vehicle crashes in work zones to identify their drivers/vehicles/environment traits accounting for interactions and confounding factors. For that purpose, the Florida Traffic Crash Records Database for years 2002, 2003, and 2004 is employed. The first section of this Chapter describes in details the methodology used in conducting the analysis. The second section elaborates on the statistical modeling for the single and the two-vehicle crashes at work zones. The third part summarizes the findings of this analysis.

3.2 Methodology

3.2.1 Accident Database and Work Zone Risk Factors Identification

The Florida Traffic Crash Records Database for years 2002, 2003 and 2004, were utilized in this study and were obtained from the Office of Management Research and Development in Florida. The database consists of seven main files: events file, drivers file, passengers file, pedestrians file, property file, vehicles file, violation file, and DOT file. The events (containing information about the characteristics and environment of the crash), vehicles (containing the information about the vehicles' characteristics and vehicles actions in the traffic crash), and drivers (containing information about drivers' characteristics) files were subject of interest in this study. It should be mentioned that the work zone classification variable was first incorporated in the Florida database in year 2002. Table 3.2.1 lists the variables included in each model and the number of observations in each model in addition to the percentage of each level under each variable.

3.2.2 Comparison Methodology

The purpose of this study is to identify the characteristics and risk factors (drivers, vehicles, and environment) that classify work zone crashes solely on freeways. The first part of this study (model #1) focuses on single-vehicle crashes at work zones and the second part (models #2 and #3) spotlights on two-vehicle crashes at work zones. The single-vehicle crashes are defined as any vehicle that crashes with a fixed object (or pedestrian/worker) contained by the work zone or any vehicle that runs off the road within a work zone area.

For the single-vehicle crash analysis, freeway work zone single-vehicle crashes were compared to freeway non-work zone (exposure) single-vehicle crashes as shown in Figure 3.2.2.1. As for two-vehicle crashes and as shown in Figure 3.2.2.2, first (model #2) a comparison between at-fault drivers and not-at-fault drivers (quasi-induced exposure analysis) was conducted which exposed drivers/vehicles attributes using multiple logistic regression. Second (model #3), similarly to single vehicle analysis, a conditional multiple logistic regression revealed the two-vehicle work zone crash environments' characteristics. It should be mentioned that comparing freeway work zone and non-work zone crashes (exposure) could be problematic due to the non- homogeneity with the exposures distributions. To illustrate that, Figure 3.2.2.3 shows that the highest frequency for crashes in work zone occur at speed limit varying between 55 and 65 mph and non-work zone at speed limit varying between 65 and 70 mph. This is due to the reduced speed limit for the duration of the work zone. Therefore, a comparison between crashes with different speed distributions is erroneous and misleading. To overcome this issue, the within-stratum analysis (or stratified sampling) was implemented.

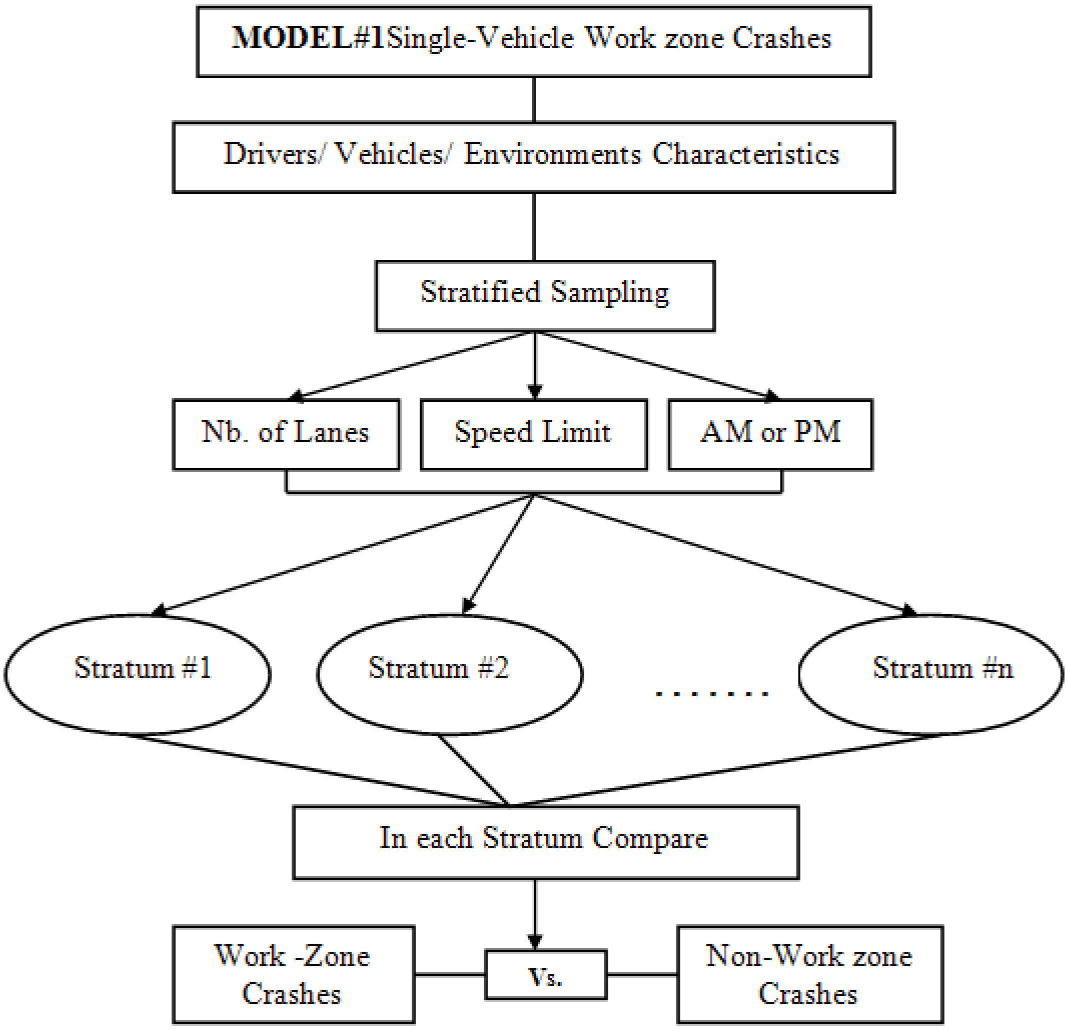


Figure 3.2.2.1: Single vehicle work zone crashes comparison methodology

Table 3.2.1: Variables description

			Model 1 (Single Vehicle)		Model 2 (2-vehicle)		Model 3 (2 vehicle)	
Type	**Variables**	**Categories**	Work Zone	Non-Work-Zone	W.Z. At-Fault	W.Z. Not At-Fault	W.Z. At-Fault	N.W.Z. At-Fault
			% of each level	% of each level	% of each level	% of each level	% of each level	% of each level
Driver Characteristics	Age	<25 years old	32.35	36.41	29.72	19.51	29.72	32.11
		26-35 years old	23.02	23.21	24.35	23.60	24.35	25.37
		36-45 years old	20.97	18.45	19.71	24.44	19.71	16.20
		46-55 years old	12.66	11.43	13.06	17.65	13.06	5.21
		56-65 years old	6.27	6.11	7.15	9.68	7.15	11.10
		66-75 years old	3.20	3.00	4.81	3.72	4.81	1.33
		>75 years old	1.53	1.39	1.20	1.40	1.20	8.68
	Gender	Male	68.09	65.89	50.03	64.32	50.03	62.71
		Female	31.91	34.11	49.97	35.67	49.97	37.29
	Driving Under the Influence	Not Under the Influence	84.11	87.29	91.34	98.80	91.34	74.58
		Alcohol/Drugs/Both	15.89	12.71	3.57	1.20	3.57	25.42
	Residence Code	Live in the State of the Accident	86.84	88.67	88.26	86.33	88.26	86.30
		Live outside the State of the Accident	13.16	11.33	11.74	13.67	11.74	13.70
Vehicle Characteristics	Speed	<25 mph	2.22	2.75	3.22	4.21	3.22	3.14
		26-35 mph	0.14	2.25	2.10	1.99	2.10	1.90
		36-45 mph	3.83	5.20	4.26	5.20	4.26	3.40
		46-55 mph	15.20	9.60	31.01	27.88	31.01	31.22
		56-65 mph	50.31	20.93	40.23	42.04	40.23	39.89
		66-75 mph	22.50	49.42	18.20	17.89	18.20	16.50
		>75 mph	5.80	9.85	0.98	0.79	0.98	3.95
	Vehicle Type	Passenger Car/ Light Trucks (SUV)	86.21	93.11	82.85	84.57	82.85	86.32
		Trucks/Large Truck	13.79	6.89	17.15	15.43	17.15	13.68
Environment Characteristics	Speed Limit	<35 mph	1.20	2.50	2.00	2.00	2.00	1.90
		45 mph	3.56	9.56	10.31	10.31	10.31	7.89
		55 mph	51.62	14.84	60.05	60.05	60.05	65.22
		65 mph	36.43	17.24	22.72	22.72	22.72	21.10
		70 mph	7.19	45.57	4.91	4.91	4.91	3.89
	Road Surface Condition	Normal Surface Condition	72.74	66.41	65.37	65.37	65.37	71.20
		Wet/Slippery Surface Condition	27.26	33.59	34.63	34.63	34.63	28.80
	Rural/Urban	Rural Area	50.70	62.12	37.36	37.36	37.36	44.48
		Urban Area	49.30	37.88	62.64	62.64	62.64	55.52
	Road Characteristics	Straight-Level	69.95	63.25	75.36	75.36	75.36	74.97
		Straight- Upgrade/Downgrade	14.62	15.73	14.89	14.89	14.89	16.81
		Curve-Level	7.08	10.48	5.38	5.38	5.38	4.50
		Curve-Upgrade/Downgrade	8.35	10.53	4.37	4.37	4.37	3.72
	Event Location	Bridge	83.65	79.41	88.79	88.79	88.79	86.39
		Entrance Ramp	5.97	4.70	3.11	3.11	3.11	3.08
		Exit Ramp	3.46	6.45	3.88	3.88	3.88	4.28
		Straight Segment	6.92	9.44	4.22	4.22	4.22	6.26
	Weather	Clear	53.49	55.27	62.53	62.53	62.53	65.30
		Cloudy/Rainy/Foggy	46.51	44.73	37.47	37.47	37.47	34.70
	Lighting Condition	Dark with Lighting	50.76	56.60	63.61	63.61	63.61	66.45
		Dark without Lighting	3.85	3.97	3.39	3.39	3.39	3.90
		Dusk/Dawn	23.22	21.56	19.99	19.99	19.99	19.59
		Day Light	22.17	17.87	13.00	13.00	13.00	10.06
	Number of lanes	1 Lane- 2 Lanes- 3Lanes	7.23	15.41	43.14	43.14	43.14	35.40
		4 Lanes- > 4Lanes	92.77	84.60	56.86	56.86	56.86	64.60
Number of Observations			950.00	7100.00	3353.00	3353.00	8300.00	28500.00

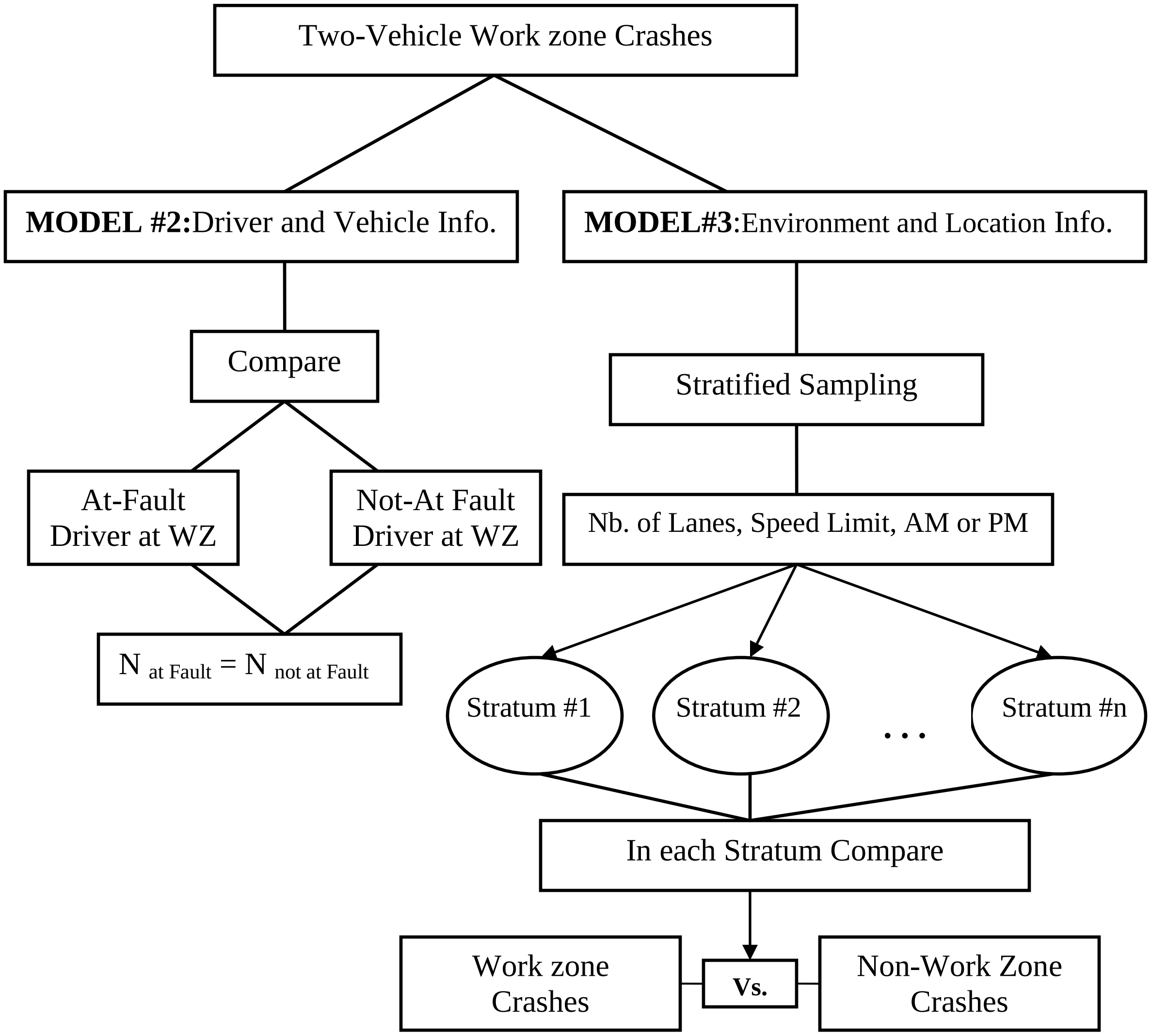


Figure 3.2.2.2: Two-vehicle work zone crashes comparison methodology

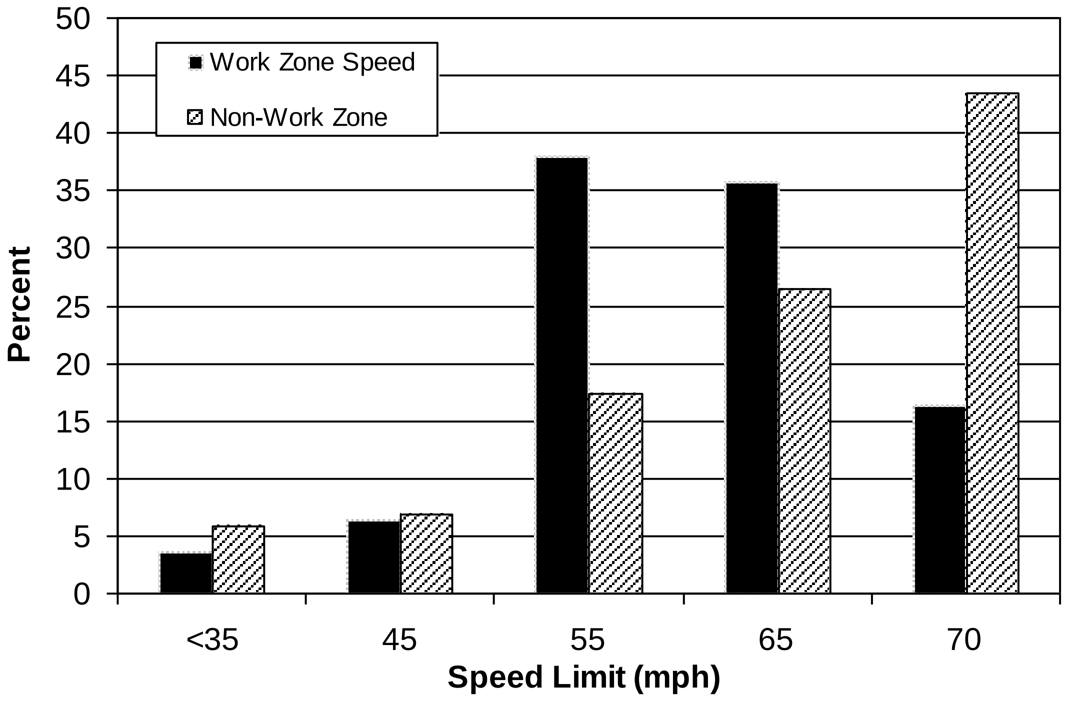


Figure 3.2.2.3: Speed limit comparison work zone versus non-work zone

As mentioned previously and as shown in Figure 3.2.2.1 (model #1) and Figure 3.2.2.2 (model #3), the stratification criteria for these models were speed limit, number of lanes and time of day (AM or PM). For example, a within stratum analysis characterized by 55mph speed limit, 3 lanes, and AM time, will be performed to classify the risk factors associated with work zone crashes.

3.2.3 Quasi-Induced Exposure Technique

The quasi-induced exposure technique (Carr, 1970; Haight, 1973; Stamatiadis and Deacon, 1997) is used in traffic safety research to explore traffic crash databases by comparing at-fault drivers' characteristics to not-at-fault drivers (exposure) traits. The at-fault drivers are those who are blamed by the police officer for the crash occurrence and the not-at-fault drivers are those found not responsible for the crash occurrence. The fundamental conjecture of this method is that the distribution of the not-at-fault drivers characterizes (or pseudo-duplicates) the distribution of all drivers (drivers' population) exposed to crash hazards. Several studies (Stamatiadis and Deacon, 1997; Albridge et al., 1999) applied the quasi-induced exposure technique where the determination of at-fault drivers strictly depended upon whether the driver was issued a citation. According to Jiang and Lyles (2007), this could be problematic. Jiang and Lyles (2007) stated that a police officer may be likely to assign responsibility and issue a ticket to a driver once he determines an indication of another violation (e.g. drinking and driving, revoked license, etc.) regardless of the hazardous driving related to the accident itself. According to De Young et al. (1997) this would inflate the involvement ratio of these groups and result in biased data and results. To overcome this issue in our analysis, the at-fault driers were selected if they match two criteria; they were issued a citation, and they contributed (e.g. careless driving, speeding, etc.) to the crash occurrence.

Yan et al. (2005) focused on the investigation of non-driver/vehicle-related (road environment) factors as exclusive main effects on the traffic safety. To introduce the road environment factors into the statistical model and test their exclusive main effects on crashes, Yan et al. extended the application of the quasi-induced exposure technique. In their study, they modeled rear-end collisions at signalized intersections. First, two-vehicle crashes occurring at signalized intersections were identified. Then, they were categorized into two groups: rear-end crashes and non-rear-end crashes (exposure) instead of at-fault and not-at-fault (exposure) drivers. By doing so, Yan et al. were able to compare the environment distributions in the rear-end group and the non-rear-end group to investigate crash propensities, which indicate whether specific traffic conditions increase rear-end crashes likelihoods at signalized intersections. Similarly to Yan et al.'s approach, this research extends the quasi-induced exposure technique to examine work zone traffic crash susceptibility. For the single-vehicle crash analysis, a comparison between work zone single-vehicle crashes and non-work zone (exposure) single vehicle crashes is conducted. This comparison is explained in detail in the next section. As for two-vehicle work zone freeway crashes, first, we categorize vehicles/drivers into at-fault and not-at-fault drivers. Second, comparing at-fault and not-at-fault drivers exposes drivers/ vehicles attributes. To extend the quasi-induced exposure technique into exploring the environment characteristics for work zone two-vehicle crashes, we compare at-fault work zone drivers and at-fault non-work zone drivers. This comparison is further explained in the next section.

Based on the above categorization, three types of Relative Accident Involvement Ratios (*RAIRs*) are calculated to test the main effect of driver, vehicle, and environment factors related to work zone crashes for each of the three models. Using the *RAIR* formula developed by Stamatiadis and Deacon (1997), the relative crash involvement ratio is defined as Equation 3.1:

$$RAIR_i = \frac{\dfrac{D1_i}{\sum D1_i}}{\dfrac{D2i}{\sum D2_i}} \text{ or } RAIR_i = \frac{\dfrac{V1_i}{\sum V1_i}}{\dfrac{V2i}{\sum V2_i}} \text{ or } RAIR_i = \frac{\dfrac{E1_i}{\sum E1_i}}{\dfrac{E2i}{\sum E2_i}} \tag{3.1}$$

$RAIR_i$ is the relative accident involvement ratio for type *i* drivers/ vehicles/ environments. For instance, in the comparison of work zone at fault drivers and non-work zone at fault drivers, *D1i* is the number of at-fault drivers of type *i* in work zone crashes, *D2i* the number of at-fault drivers in non-work zone crashes, *V1i* the number of at-fault vehicles of type *i* in work zone crashes, *V2i* the number of at-fault vehicles of type *i* in non-work zone crashes, *E1i* the number of work zone crashes involving environment type *i*, and *E2i* is the number of non-work zone crashes involving environment type *i*. Furthermore, to test the interaction between type *i* drivers/vehicles/environments and type *j* drivers/vehicles/environments, the *RAIR* can be defined as Equation (3.2)

$$RAIR_{i,j} = \frac{\dfrac{N1_{i,j}}{\sum\sum N1_{i,j}}}{\dfrac{N2_{i,j}}{\sum\sum N2_{i,j}}} \qquad (3.2)$$

The $RAIR_{i,\,j}$ is the relative accident involvement ratio types *i* and *j* drivers/vehicles/ environments. For example, in the comparison of work zone at fault drivers and non-work zone at fault drivers, $N1_{i,\,j}$ is the number of work zone crash drivers, vehicles, or the related environments of type *i* and *j* in work zone collisions, and $N2_{i,\,j}$ is the number of at-fault drivers, vehicles, or the related environments of type *i* and *j* in non-work zone crashes.

3.2.4 Multiple Logistic Regression Modeling

Previous studies had properly applied logistic regression analysis to test the significance of traffic crash risk factors based on the technique of induced exposure (Hing, 2003; Stamatiadis and Deacon, 1995). Logistic regression belongs to the group of regression methods for describing the relationship between explanatory variables and a discrete response variable. It is a powerful alternative to classical discrimination and regression methods and it applies to a large family of parametric distributions, involving both discrete and continuous variables (Cox, 1966; Day and Kerridge, 1967; Anderson, 1972). A binary logistic regression is proper to use when the dependent variable is dichotomous (i.e. the dependent variable is binary) and can be applied to test association between a dependent variable and the related potential risk factors. Binary logistic regression is used to model at-fault and not-at-fault drivers at work zone. The dependent variable Y (crash classification) can only take two values: Y=1 for at-fault drivers, and Y=0 for not-at-fault drivers. The probability that a driver is at-fault or not is modeled as logistic distribution in Equation 3.3:

$$\pi(x) = \frac{e^{g(x)}}{1+e^{g(x)}} \tag{3.3}$$

The logit of the multiple logistic regression model (Link Function) is given by Equation 3.4:

$$g(x) = \ln\left[\frac{\pi(x)}{1-\pi(x)}\right] = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \beta_3 x_3 + \ldots + \beta_n x_n \tag{3.4}$$

Where $\pi(x)$ is conditional probability of at-fault work zone drivers, which is equal to the number of at-fault drivers divided by the total number of drivers. x_n are the independent variables (driver/vehicle/environment factors). The independent variables can be either categorical or continuous, or a mixture of both. Both main effects and interactions can be accommodated. β_n is model coefficient, which directly determines the odds ratio involved in the at-fault drivers. The odds of an event are defined as the probability of the outcome event occurring divided by the probability of the event not occurring. The odds ratio is equal to exp (β_n) and tells the relative amount by which the odds of the outcome increase (OR greater than 1.0) or decrease (OR less than 1.0) when the value of the predictor is increased by 1.0 units (David and Lemeshow, 1989). Previous studies (Stamatiadis and Deacon, 1995; Hing et al. 2000) clearly expressed the relationship between logistic regression and *RAIR* in the quasi-induced exposure analysis. In fact, for a specific type of drivers/vehicles/environments, the odds generated from the logistic regression model are analogous to the corresponding *RAIRs*, and the odds ratio from the model are equivalent to the comparisons among those *RAIRs*. In this study, the *RAIRs* were based on the univariate analysis rather than the network analysis which clarifies the small differences between the models' odds ratios and the *RAIRs*. Furthermore, a significant *p*-value (e.g. $P \leq 0.05$) for a Wald χ^2 statistic is evidence that a regression coefficient in the model is nonzero, which also indicates the statistical importance of those *RAIRs*' comparisons between different types of drivers/vehicles/environments. The SAS program procedure, LOGISTIC, was used for the model development and the hypothesis testing was based on 0.05 significance level.

3.2.5 Conditional Logistic Regression Modeling (Matched Work zone Non-Work zone Crashes)

For modeling at-fault work zone drivers and at-fault non-work zone drivers, a matched work zone non-work zone analysis was implemented. The purpose of the proposed matched work zone non-work zone analysis was to explore the effects of traffic characteristics variables while controlling for the effects of other confounding variables through the design of the study. This modeling is called conditional logistic regression. It was used in this study to model single-vehicle work zone crashes against single vehicle non-work-work zone crashes and two-vehicle work zone at-fault-drivers versus two-vehicle non-work zone at-fault drivers.

In a matched work zone non-work zone crash study, first crashes were selected. For each selected crash, some non-environment variables such as number of lanes, time of day, speed limit etc., associated with each crash were selected as matching factors. A subpopulation of work zone crashes was then identified using these matching factors. For example, for freeways work zone crashes, with specific number of lanes, speed limit, and time of day, a subpopulation of work zone crashes was identified based on the matching criteria. A total of m non-work zone crashes were then selected at random from each subpopulation of work zone crashes. Within stratum differences between work zone and non-work zone characteristics were utilized in the development of statistical model. This was done under the conditional likelihood principle of statistical theory.

Abdel-Aty et al. (2004) employed this modeling technique to predict freeway crashes based on loop detector data. Similarly to them, we assumed that there were N strata with n work zone crashes and m non-work zone crashes in stratum j, $j = 1, 2, \ldots\ldots N$. We also assumed that $p_j(x_{ij})$ was the probability that the i^{th} observation in the j^{th} stratum is a crash where $x_{ij} = (x_{1ij}, x_{2ij}, \ldots\ldots x_{kij})$ was the vector of k traffic characteristics variables $x_1, x_2, \ldots\ldots x_k$; $i = 0, 1, 2, \ldots\ldots m+n-1$; and $j = 1, 2, \ldots\ldots N$. This crash probability $p_j(x_{ij})$ may be modeled using a linear logistic model as follows:

$$logit\ (p_j(x_{ij})) = \alpha_j + \beta_1 x_{1ij} + \beta_2 x_{2ij} + \ldots\ldots\ldots + \beta_k x_{kij} \tag{3.5}$$

The intercept term α is different for different strata. It summarizes the effect of variables used to form strata on the probability of crash. In order to take account of the stratification in the analysis of the observed data, one constructs a conditional likelihood. This conditional likelihood function is the product of *N* terms, each of which is the conditional probability that the crash in a particular stratum says the j^{th} strata, is the one with explanatory variables x_{0j}, conditional on x_{0j}, $x_{1j}, \ldots$ x_{mj} being the vectors of explanatory variables in the j^{th} stratum. The mathematical derivation of the relevant likelihood function is quite complex and is neglected here. The reader may consult Collett (1991) for full derivation of the conditional likelihood function that can be expressed as (Abdel-Aty et al. (2004)):

$$L(\beta) = \prod_{j=1}^{N} [1 + \sum_{i=1}^{m} \exp\{\sum_{u=1}^{k} \beta_u (x_{uij} - x_{u0j})\}]^{-1} \tag{3.6}$$

Where, β's are the same as in Equation 3.5. The likelihood function $L(\beta)$ is independent of the intercept terms α_1, α_2,…….. α_N. So the effects of matching variables cannot be estimated and hence Equation 5 cannot be used to estimate crash probabilities. However, the values of the β parameters that maximize the likelihood function given by Equation 3.6 are also estimates of β coefficients in Equation 3.6. These estimates are log odds ratios and can be used to approximate the relative risk of a crash.

SAS procedure *PHREG* gives these relative risks (termed as hazard ratio under *PHREG*). The log odds ratios can also be used to develop a prediction model under this matched crash-non-crash analysis.

3.3 Data Analysis

3.3.1 Statistical modeling for single-vehicle work zone crashes.

Based on the model for single-vehicle work zone crash analysis, the conditional logistic regression identified the risk factors associated with work zone crashes. As shown in Table 3.2.1, the numbers of observations for work zone and non-work zone crashes were 950 and 7100 respectively. The reader should be cautious that the identified risk factors imply that these factors have higher sensitivity to work zones than to non-work zones locations. The Hazard ratio is analogous to the odds ratio. A hazard ratio (odds ratio) of one implies that the event is equally likely in both groups. A Hazard ratio (odds ratio) greater than one implies that the event is more likely in the first group. A hazard ratio (odds ratio) less than one implies that the event is less likely in the first group. Figure 3.3.1.1 illustrates the univariate comparisons of relative crash involvement ratios between different conditions for drivers/vehicles/environment characteristics prior to the application of the stratified sampling. The listed graphs in Figure 3.3.1.1 stand for the variables found significant at 0.05 significance level in the univariate analysis. The RAIRs show a trend for each of the drivers/ vehicles/ environment factors. For instance, the RAIR of trucks is clearly higher than the RAIR of passenger cars / SUVs / Vans. The weather graph shows that the RAIR of cloudy weather is higher than RAIR of clear weather and the RAIR of rainy weather is undoubtedly lower than the RAIR of clear weather.

The conditional logistic regression aforementioned compares drivers/vehicles/ environment characteristics associated with work zone versus non-work zone crashes. The final model's results shown in Table 3.3.1 illustrate the model's significant variables and goodness of fit. The Log likelihood, AIC, and SBC criteria show that the final model has a good fit. This statistical modeling accounts for the confounding effects and interactions between the factors from the univariate analysis. The model shows that large trucks have additional risk at work- zone locations compared to non-work zone locations (*p*-value=0.0005). Trucks and large trucks are 44.6% more likely to be involved in a work zone single-vehicle crash compared non-work zone locations. According to the model, roadway geometry including vertical and horizontal alignment is a significant risk factor.

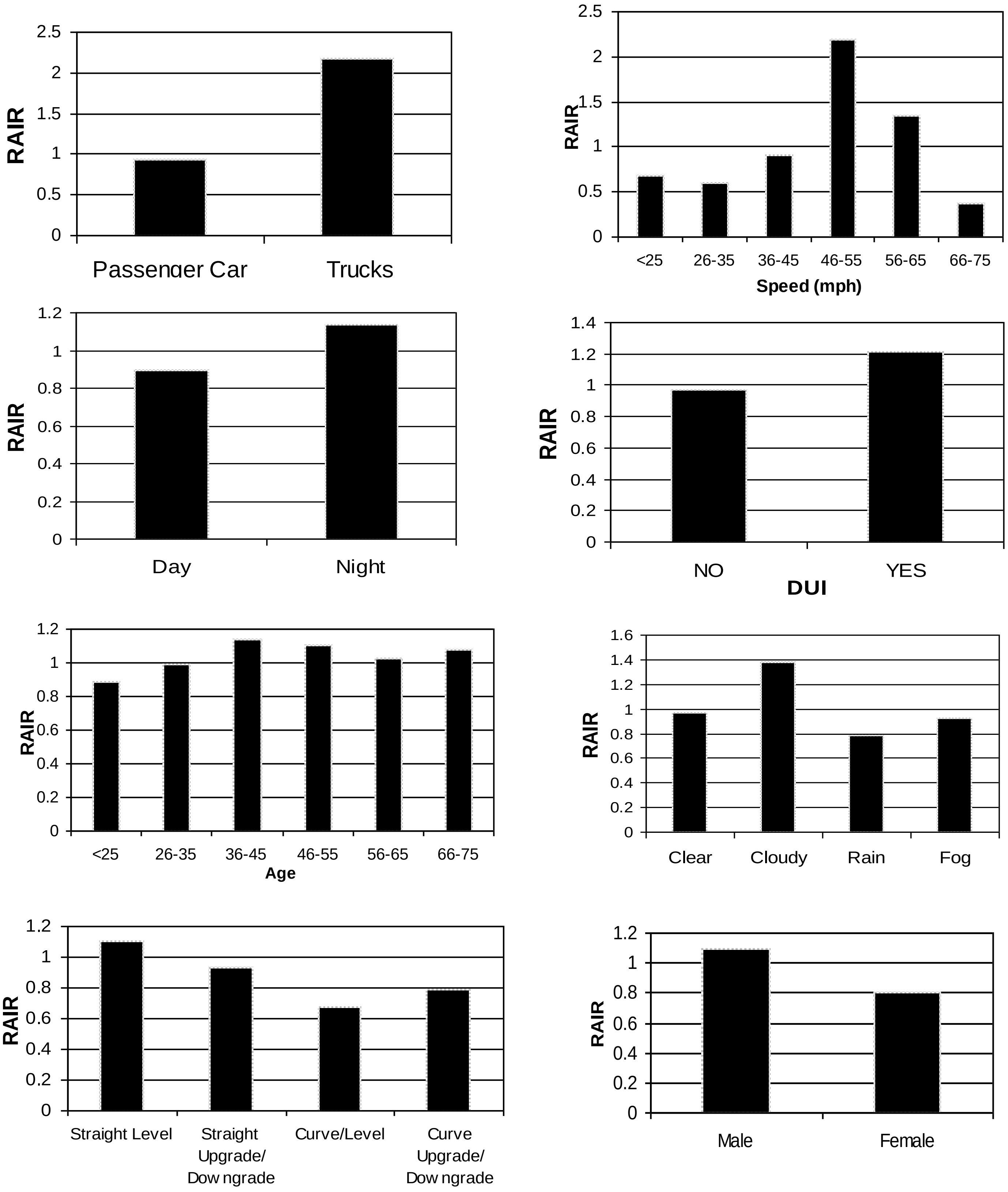


Figure 3.3.1.1: Relative accident involvement ratios by road environment factors for single vehicle crashes

Within a work zone straight-level segments have increased likelihood of single-vehicle crashes compared to straight-upgrade/downgrade, curve-level, and curve-upgrade/ downgrade. The hazard ratios (or odds ratios) are 0.749, 0.728, and 0.718 respectively when compared to straight-level. The corresponding *p*-values are 0.0037, 0.0239, and 0.017 in that order (See Table 3.3.1). An explanation of this is that drivers are more likely to drive cautiously on vertical and horizontal curves. The lighting condition is also one of the risk factors associated with work zone single-vehicle crashes. The model shows that with poor or no lighting during dark at work zones, motor vehicles are more prone (23.5%) for crashes compared to non-work zone locations (*p*-value=0.0151). The weather condition is also one of the statistically significant risk factors. In fact, the model results illustrate that during rainy weather, drivers are less likely to be involved in work zone single vehicle crashes (*p*-value= 0.0476). This fact may be due to the vigilant driving pattern during rain especially at work zones. Finally it should be mentioned that the work zone presence was found to have no statistically significant effect on the gender and age factors.

Table 3.3.1: Single-Vehicle Conditional Logistic Regression Model Estimation

Variable	Parameter Estimate	Standard Error	Chi-Square	P-Value	Hazard Ratio
Large Truck Vs. Passenger Car/ SUV/ Vans	0.36895	0.10573	12.17610	0.00050	1.44600
Straight- Upgrade/Downgrade Vs. Straight-Level	-0.28886	0.09955	8.41940	0.00370	0.74900
Curve-Level Vs. Straight-Level	-0.31689	0.14034	5.09850	0.02390	0.72800
Curve-Upgrade/Downgrade Vs. Straight-Level	-0.33089	0.13865	5.69590	0.01700	0.71800
Dark with Poor or no Lighting Vs. Day Light	0.21098	0.08683	5.90440	0.01510	1.23500
Rainy Weather Vs. Clear Weather	-0.17571	0.08869	3.92500	0.04760	0.83900

Model Fit Statistics

Criterion	Without Covariates	With Covariates
Log Likelihood	-4650.88000	-4640.58000
AIC	9301.77500	9293.60000
SBC	9301.77500	9298.22800

3.3.2 Statistical modeling for two-vehicle work zone crashes

3.3.2.1 Drivers and vehicles characteristics

For two-vehicle crash analysis, the first multiple logistic regression model compares work zone at-fault drivers versus work zone not-at-fault drivers and exposes drivers/ vehicles attributes. Table 3.2.1 shows the number of observations in this model (3353 observation for at-fault drivers and 3353 observations for not-at-fault drivers). Figure 3.3.2.1 illustrates the univariate comparisons of relative crash involvement ratios (RAIRs) between different conditions for each drivers/vehicles characteristics. The listed graphs in Figure 3.2.2.1 show the variables found significant at 0.05 significance level in the univariate analysis. The driving under influence (DUI) graph clearly shows that drivers under the influence of narcotics are more prone to accidents. The age graph illustrates that drivers at age 25 or less and 75 or more are the most sensitive to crashes at work zones. The graph also confirms that males are more at risk than females to be at-fault in a work zone crash and that trucks are more sensitive to crashes than regular passenger cars at work zones. The last two graphs in Figure 3.2.2.1 illustrate that local drivers have a higher relative crash involvement ratio than out-of-state drivers and that speeding (at > 65 mph) in work zone produces a high crash hazard at work zones.

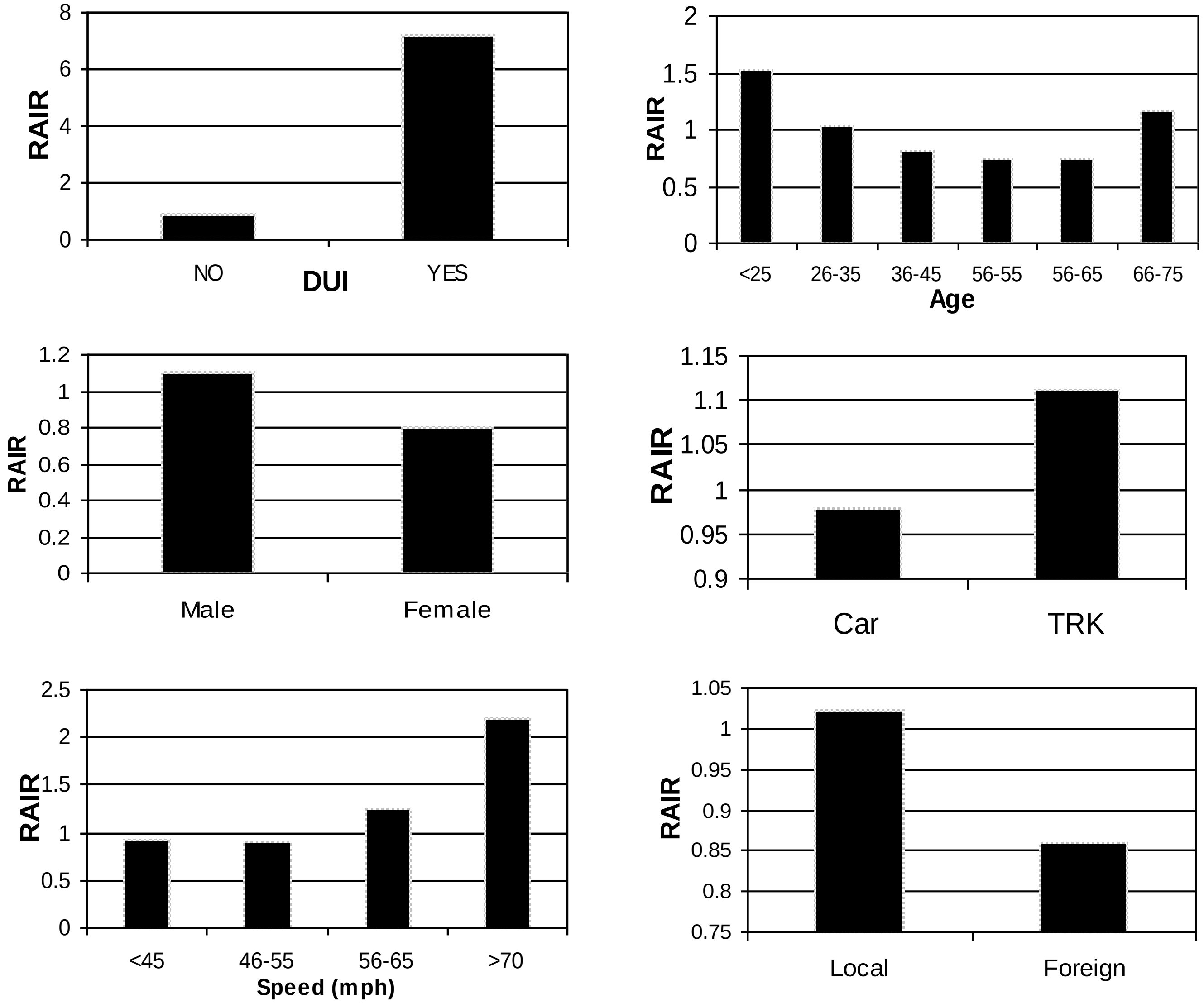


Figure 3.3.2.1: Relative accident involvement ratios by drivers/vehicles factors for two-vehicle crashes

Figure 3.3.2.2 shows the interaction between age and gender. As illustrated by the graph males of 25 years old and younger and females older than 75 years old have the highest relative crash involvement ratio.

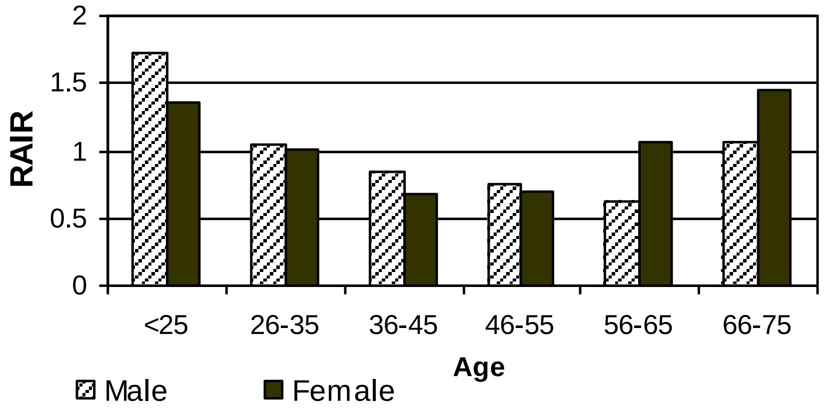


Figure 3.3.2.2: Relative accident involvement ratios: drivers' age and gender interaction for two-vehicle crashes

The multiple logistic regression model accounting for interactions between terms and confounding effects is summarized in Table 3.3.2.1. The Log likelihood, AIC, and SC criteria show that the model has a good fit. According to the model, age constitute a risk factor for work zone crashes. Comparing 56-65, 46-55, 36-45, and 26-35 years old drivers groups to <25 years old drivers group shows that drivers of 25 years old or younger comprise the highest risk factor for work zone crashes (Wald chi-square *p*-values: <0.0001). The odds ratios are 0.477, 0.444, 0.526, and 0.669 respectively. The model also shows that the crash likelihood for male drivers is significantly higher than female drivers (*p*-value < 0.0001). The odds ratio for females to be involved in a two-vehicle crash at work zone is 0.714 compared to male drivers. This can be explained by the fact that male drivers are usually more aggressive in driving. The Driving under influence (DUI) factor is significant in the final model. The model clearly shows that drivers under the influence of narcotics are more 10.526 time more likely to cause crashes (*p*-value<0.0001). The Rescode variable defines whether the driver lives in the state of where he was involved in the crash or not. The Final model shows that out-of-state drivers are less likely to be involved in work zone crash compared to local drivers (*p*-value = 0.0283). The model also illustrates that the odds ratio for foreign drivers to be involved in work zone crashes is 0.979 compared to local drivers. This can be explained by the fact that foreign drivers are usually more careful on unfamiliar roads.

Table 3.3.2.1: Two-Vehicle Logistic Regression Model Estimation

Parameter		Estimate	Standard Error	Wald Chi-Square	P-Value	Odds Ratio	95% Wald Confidence Limits	
Intercept		-1.1544	0.2554	20.4345	<.0001			
Age	**75 vs. 25**	-0.1744	0.1381	1.5952	0.2066	0.8400	0.6410	1.1010
	65 vs. 25	-0.7405	0.1210	37.4426	<.0001	0.4770	0.3760	0.6050
	55 vs. 25	-0.8123	0.1005	65.3300	<.0001	0.4440	0.3640	0.5400
	45 vs. 25	-0.6420	0.0892	51.7665	<.0001	0.5260	0.4420	0.6270
	35 vs. 25	-0.4020	0.0860	21.8696	<.0001	0.6690	0.5650	0.7920
Sex	**Female vs. Male**	-0.3384	0.0662	26.1291	<.0001	0.7130	0.6260	0.8120
DUI	**Yes vs. No**	1.9723	0.1947	102.6544	<.0001	7.1870	4.9070	10.5260
Rescode	**Foreign vs. Local**	-0.2011	0.0917	4.8118	0.0283	0.8180	0.6830	0.9790
Interactions								
Sex*Age	**75 vs. 25**	0.5472	0.3144	3.0301	0.0817			
	65 vs. 25	0.6324	0.2591	5.9552	0.0147			
	55 vs. 25	0.0316	0.2187	0.0209	0.8852			
	45 vs. 25	0.0111	0.1917	0.0033	0.9540			

Model Fit Statistics

Criterion	Intercept Only	Intercept and Covariates
Log Likelihood	-3346.5800	-3198.3130
AIC	6695.1610	6430.6260
SC	6701.6630	6541.1540

3.3.2.2 Environment characteristics

The second model (model #3) conditional logistic regression aforementioned compares the environments' characteristics associated with work zone. In this model the strata had number of lanes, speed limit, and time of day (AM or PM), and driver gender and age as matching criteria. Table 3.2.1 shows that the numbers of observations for work zone and non-work zone are 8,300 and 285,000 respectively. Figure 3.3.2.2 demonstrates the univariate comparisons conducted prior to the statistical modeling of relative crash involvement ratios between different conditions for each drivers/vehicles/environment characteristics before applying the stratified sampling technique. The listed graphs in Figure 3.3.2.2 display the variables found significant at 0.05 significance level in the univariate analysis for two-vehicle crashes. The weather graph in Figure 3.3.2.2 clearly shows that the RAIR for cloudy weather is higher than the RAIR for clear weather. The Rural-Urban graph confirms that the relative crash involvement ratio is higher for urban locations compared to rural locations. The lighting condition graph demonstrates that at night with poor or no lights could be a serious crash threat at work zones compared to non-work zone locations. The Roadway characteristics graph shows that straight-upgrades and straight downgrades have lower likelihood for crash at work zones compared to non-work zone settings.

A conditional logistic regression model identified the environmental factors associated with work zone crashes. Table 4 recapitulates the final model parameter estimates. The Log Likelihood, AIC, and SBC criteria show that the model has a good fit (See Table 4). Similarly to the single-vehicle model, the road geometry (upgrade/downgrade) had a negative effect on the crash likelihood on work zones compared to non-work zone locations. Similarly to the preceding model (single-vehicle crash), this fact can be clarified by the alertness of drivers on upgrades/downgrades compared to straight-level sections. The lighting condition factor is analogous to the previous model. Poor lighting or no lighting at all can cause significantly (*p*-value <0.0001) higher crash hazard (35.2% increase, hazard ratio 1.352) on work zones compared to non-work zones. The weather condition affects positively the work zone crash likelihood. This model shows that foggy weather causes a significant (*p*-value=0.0017) rise in work zone crash risk (hazard ratio =1.161) compared to non-work zone locations. In addition to that, work zones located in rural areas have higher crashes potential than work zones located in urban areas.

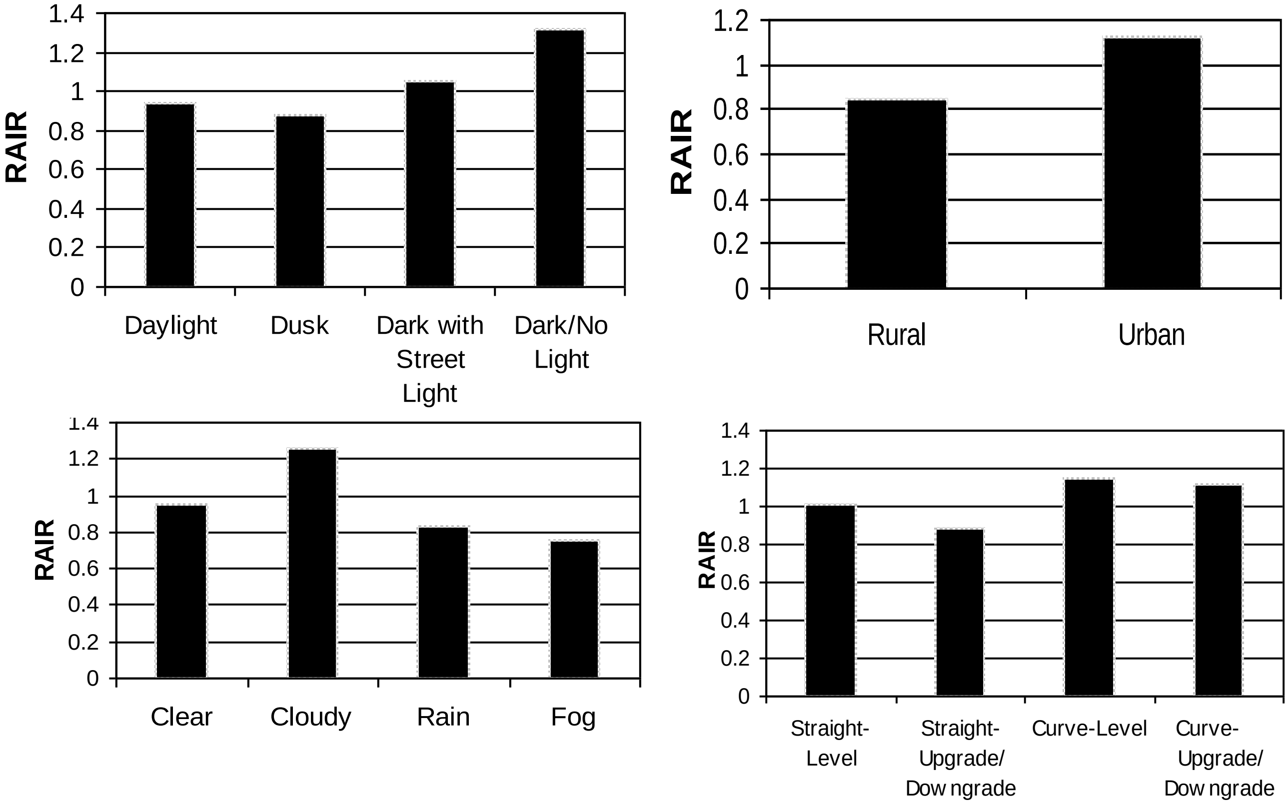


Figure 3.3.2.2: Relative accident involvement ratios by environment factors for two-vehicle crashes

Table 3.3.2.2: Two-Vehicle Logistic Conditional Regression Model Estimation

Variable	Parameter Estimate	Standard Error	Chi-Square	P-value	Hazard Ratio
Straight-Upgrade/Downgrade vs. Straight-Level	-0.26589	0.05725	21.56750	<.00010	0.76700
Poor or No Street Light vs. Day Light	0.30193	0.06567	21.14040	<.00010	1.35200
Foggy Weather vs. Clear Weather	0.14943	0.04765	9.83450	0.00170	1.16100
Rural vs. Urban	0.25776	0.05014	26.42730	<.00010	1.29400

Model Fit Statistics

Criterion	Without Covariates	With Covariates
Log Likelihood	-2524.04700	-2519.67500
AIC	5048.09400	5040.53500
SBC	5048.09400	5047.36300

3.4 Conclusions and Discussions

The main objective of this study was to conduct a statistical analysis to expose work zone crash characteristics while accounting for confounding parameters. The Florida Traffic Crash Records Database for years 2002, 2003 and 2004 were employed and statistical models were assembled to draw drivers/vehicles/ environment traits of work zone crashes. Three models were developed to analyze single-vehicle and two-vehicle freeway work zone crashes. The first model (conditional logistic regression model) compared work zone versus non-work zone single-vehicle crashes and exposed the vehicles/drivers/ environment attributes. The second model (multiple logistic regression model) compared two-vehicle work zone at-fault versus not-at-fault drivers. This model revealed the drivers/vehicles characteristics. The third model (conditional logistic regression) compared at-fault work zone versus at-fault non-work zone drivers for two-vehicle crashes and retrieved work zone environment attributes. The hypotheses of models #1 and #3 investigate whether the attributes (parameters included in the models) are significantly affected by the presence of work zones. The hypothesis of model #2 assesses whether at-fault drivers' attribute is significantly different from the not-at-fault drivers' attributes at work zones.

For the single-vehicle crashes, results showed that trucks are 44.6% more likely to be involved in a work zone single-vehicle crash compared to trucks in non-work zone locations. This fact may be due to narrower lanes during maintenance or construction. Several studies agree that heavy vehicles are overrepresented in work zone areas (Hall and Lorenz, 1989; Pigman and Agent, 1990; Nemeth and Rathi 1983). However, the main reason behind this issue is still obscure and subject for future investigations. Results also showed that roadway geometry is also a significant risk factor associated with freeway single-vehicle work zone crashes. Straight-level has increased likelihood compared to straight-upgrade /downgrade, curve-level, and curve-upgrade/ downgrade. In other words, straight level roadways are significantly affected by the presence of work zones compared to non-work zone locations. An explanation of this could be related to the fact that drivers may be more likely to drive cautiously on vertical and horizontal curves. In this context, Daniel et al. (2000) stated that fatal work zone crashes are less influenced by horizontal and vertical alignment compared to non-work zone locations. The lighting condition is also one of the risk factors associated with work zone single-vehicle crashes. The model shows that in work areas with poor or no lighting during dark, motor vehicles are more prone (23.5%) for crashes compared to non-work zone locations with poor or no lighting during dark. This fact may be due to the invisibility of the work zone equipment during poor or no lighting which may lead to single-vehicle crashes. The weather condition is also associated with single-vehicle work zone crashes. In fact, the first model shows that during rainy weather, drivers are less likely to be involved in work zone crashes compared to the same weather conditions in non-work zone locations. This fact may be due to the vigilant driving pattern during rain at work zones.

For the two-vehicle crashes, the second model's results illustrate that drivers younger than 25 years old and drivers older than 75 years old have the highest risk to be the at-fault driver in a work zone crash. Male drivers have significantly higher risk (approximately 40% higher) than female drivers to be the at-fault driver. The interaction between age and gender confirmed that younger (≤ 25 years old) male drivers and older (≥ 75 years old) female drivers are prone to be the at-fault driver in a work zone crash. The age and gender trends in work zone crashes are consistent with the general trend of age and gender in the overall crashes (National Highway Traffic Safety Administration, 2000). This can be explained by the fact that young male drivers are usually more aggressive in driving and older females’ alertness and reaction time decreases with age. The model noticeably shows that drivers under the influence of narcotics/alcohol are 10.526 times more likely to cause crashes (i.e. at-fault driver) at work zones. The second model finally shows that out-of-state drivers are slightly less likely to be the source (i.e. at-fault driver) of a work zone crash compared to local drivers. This can be explained by the fact that foreign drivers are usually more careful on unfamiliar roads. The third model revealed the environment characteristics for two-vehicle work zone crashes. Similarly to the single-vehicle model (first model), the road geometry and the lighting condition were significant risk factors for two-vehicle work zone crashes. Freeways straight segments are more susceptible to crashes in work zone areas. As explained before, this fact may be due to the alertness of drivers on non-straight segment. This finding is consistent with previous studies (Milton and Mannering, 1998; Chang, 2005). Poor lighting or no lighting at all during dark can lead to significantly higher crash hazard (35.2% increase, hazard ratio 1.352) on work zones compared to non-work zones. Analogously to this finding, Daniel et al. (2000) also concluded that poor or no lighting at night affects increase the likelihood of a fatal crash in work zone compared to non-work zones. This third model shows that for two-vehicle crashes, foggy weather causes a significant mount in work zone crash risk compared to non-work zone locations. In addition to that, work zones located in rural areas have higher crashes potential than work zones located in urban areas.

It should be noted that there exist some consistency in the environment factors associated with single-vehicle and two-vehicle crashes at work zones. For instance, Straight-level segments in which work zones are located are more prone to single-vehicle and two-vehicle crashes than “curves-upgrades” segment compared to non-work-zone locations. Poor lighting during dark results in higher single-vehicle and two-vehicle crash risk in work zone locations compared to non-work-zone locations.

Some recommendations can be drawn based on the findings of the work zone crash analysis. First, for both single-vehicle and two-vehicle crashes, good lighting should be provided in the work areas and around them so drivers can be alerted ahead of time and to facilitate the driving maneuver during work zone hazards at night. Trucks should be granted extra care in the work zones especially with lane closures and narrowing. A reduced speed limit could help the trucks better maneuver in work zones. The drivers’ inattentiveness and hostile driving are overrepresented in work zones. This fact was illustrated by the age and gender factor, the road geometry factors, residence, and inclement weather. For that purpose, additional enforcement is recommended such as police cars and/or flashing signs for double fining in work-areas.

ITS lane management systems could also be potential countermeasures worthy of implementation and testing on Florida’s work zones. For instance, previous studies showed that dynamic early merging can smoothen the merging operation in advance of a lane closure (Tarko, 1998), decrease the rear-end accident rate (Tarko, 1998), and reduce the number of forced merges (Wayne State University, 2001). The dynamic late merging can reduce conflict points (or locations) to one single location at the taper of the work zone which enhances overall driving conditions upstream of work zone (Tavoola et al., 2004). Therefore, the early and late merging systems have the potential of improving the merging maneuvers in Florida’s work zones especially for trucks. These systems can also reduce hostile driving by reducing random merging (at random locations) to definite merging.

As a typical study based on traffic crash databases, some limitations may exist since some variables (or information) may not be available in these crash databases. For instance, the Florida Crash Records Database did not provide information about the work zone duration and the work zone design or configuration. These variables may be confounded or may interact with other variables in our models. Such data can be obtained and analyzed using driving simulation studies or field data collection.

CHAPTER 4 SIMPLIFIED DYNAMIC LANE MERGING SYSTEM (SDLMS)

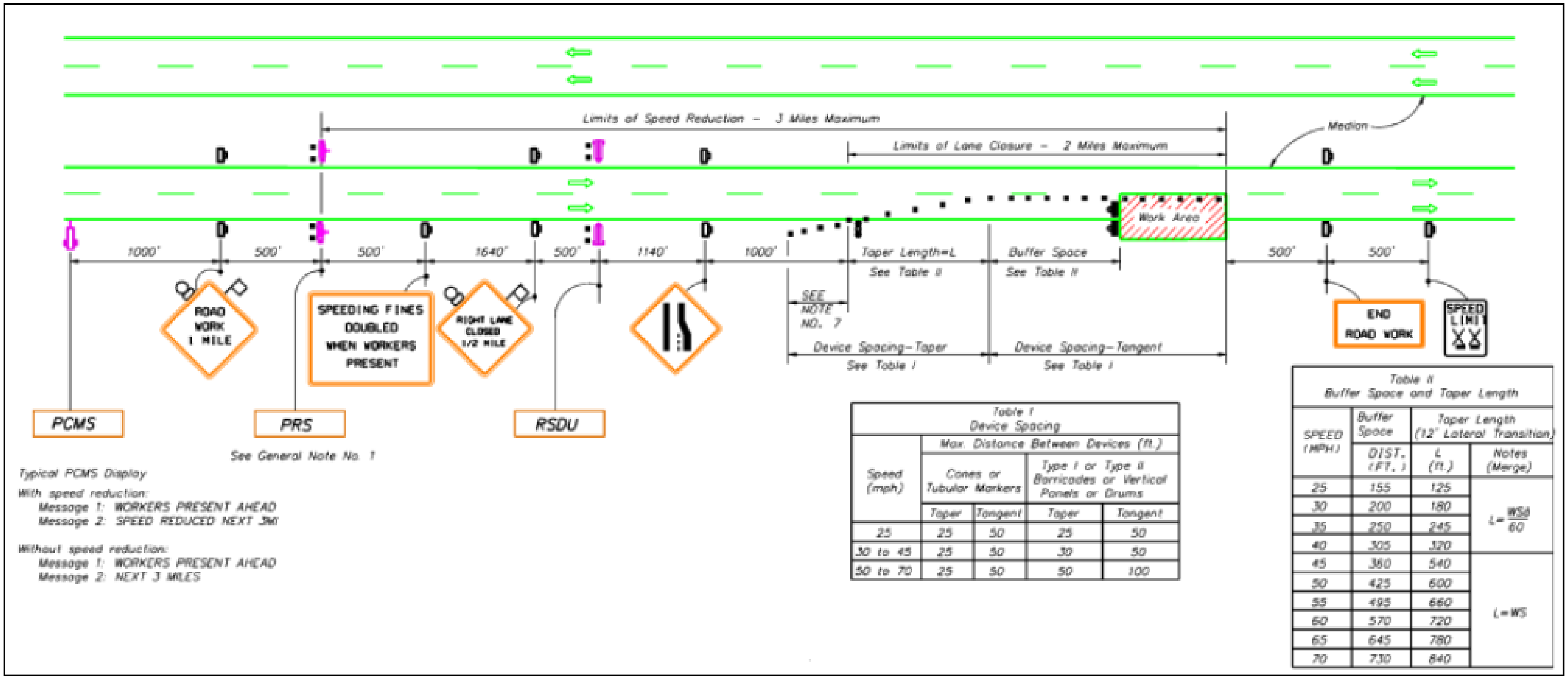
This Chapter entails details on the design of the modified Maintenance of Traffic (MOT) plan that includes the dynamic lane merging component. Two modified MOTs were designed; the Early Simplified Dynamic Lane Merging and the Late Simplified Dynamic Early Merging. This chapter also includes details about the ITS system components, deployment, and communication between the entities involved in deploying the system.

4.1 Current Florida MOT Plans

Currently the Florida Department of Transportation deploys an MOT plan known as the Motorist Awareness System (MAS). According to the Florida Plans Preparation Manual (FPPM), the MAS aims at increasing the motorist awareness of the presence of active work and at providing emphasis on reduced speed limits in the active work area. The Florida manual states that the MAS shall be used on multilane facilities where the posted speed limit is 55mph or greater and where work activity requires a lane closure for more than five days only when workers are present. The MAS, as shown in Figure 4.1, consists of Portable Regulatory Signs (PRS) highlighting the regulatory speed for the work zone and a Radar Speed Display Unit (RSDU) displaying the motorist’s work zone speed. The MAS also comprises a PCMS, a lane drop warning sign, a speeding fines doubled warning sign, in addition to road work ahead warning signs.

4.2 Modified MOT Plans

The modified MAS plans consist of the addition of an ITS-based lane management system to the conventional MAS. Two modified MAS plans (early SDLMS and late SDLMS) are suggested. The first modified MAS plan is a simplified dynamic early merge system and the second modified MAS plan is a simplified dynamic late merge system. Therefore the conventional MAS plans are supplemented with one PCMS and a non-intrusive RTMS trailer as shown in Figure 4.2. The modified MAS plan is referred to in this paper as SDLMS. The additional PCMS and sensor trailer are placed at the same location in both modified MAS plans. The messages displayed by the PCMS will differ as elaborated on in the next section. The modified MOT plans were signed and sealed by a Florida licensed consultant.



Typical PCMS Display

With speed reduction:
Message 1: WORKERS PRESENT AHEAD
Message 2: SPEED REDUCED NEXT 3MI

Without speed reduction:
Message 1: WORKERS PRESENT AHEAD
Message 2: NEXT 3 MILES

Table I
Device Spacing

Speed (mph)	Max. Distance Between Devices (ft.)			
	Cones or Tubular Markers		Type I or Type II Barricades or Vertical Panels or Drums	
	Taper	Tangent	Taper	Tangent
25	25	50	25	50
30 to 45	25	50	30	50
50 to 70	25	50	50	100

Table II
Buffer Space and Taper Length

SPEED (MPH)	Buffer Space	Taper Length (12' Lateral Transition)	
	DIST. (FT.)	L (ft.)	Notes (Merge)
25	155	125	$L = \frac{WS^2}{60}$
30	200	180	
35	250	245	
40	305	320	
45	360	540	L=WS
50	425	600	
55	495	660	
60	570	720	
65	645	780	
70	730	840	

Figure 4.1: Motorist Awareness System in Florida (Index 670 FDOT-Standards)

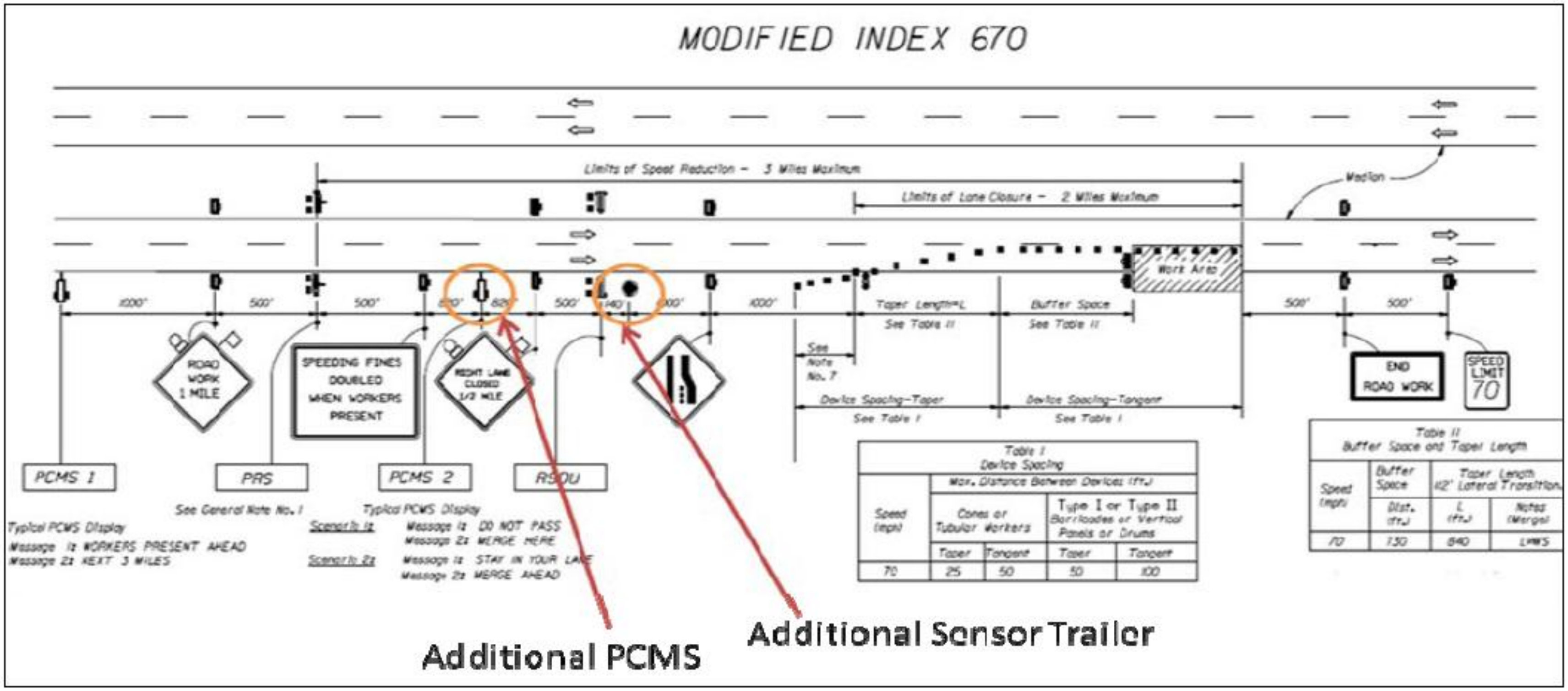


Table I
Device Spacing

Speed (mph)	Max. Distance Between Devices (ft.)			
	Cones or Tubular Markers		Type I or Type II Barricades or Vertical Panels or Drums	
	Taper	Tangent	Taper	Tangent
70	25	50	50	100

Table II
Buffer Space and Taper Length

Speed (mph)	Buffer Space	Taper Length (12' Lateral Transition)	
	Dist. (ft.)	L (ft.)	Notes (Merge)
70	130	840	LWS

Figure 4.2: Modified Motorist Awareness System (SDLMS)

4.4. SDLMS System's Specifications

4.4.1 SDLMS System Components

The SDLMS consisted of one set of the following equipment. The equipment was relocated as needed upon relocation of the work zone.

- ***Traffic detection station*** wirelessly linked to central computer base station. Traffic detection station was mounted to the sensor trailer which was equipped with solar panel, deep cycle batteries and associated circuitry. The RTMS sensors capture speed, volume, and occupancy.
- ***One central computer base station*** environmentally hardened and equipped with appropriate software and dedicated wireless communications to "link" with the traffic sensor station and PCMS. The computer base station was housed in a standard weather proof traffic-signal control cabinet, or other appropriate means, with provision for installation of the central communication antenna. One base station may be used for multiple directions of travel.
- ***Wireless communication links*** consisting of a road-side remote station, duly equipped with radio modems (for transmitting and receiving licensed UHF radio frequencies), micro- processors and antennae.
- ***PCMS*** remotely controlled via a central computer base station or central system controller.

The detection zones were located on the highway, distanced suitably to both gather traffic data and to cover the entire length of the desired stretch of the highway..

4.4.2 SDLMS Features

The SDLMS features are as following:

- The software provided is modular with open architecture providing for future integration with other similar traffic monitoring systems and allowing detailed real-time monitoring of the status including communications-link operational status, current delay predicted for the roadway and current messages displayed on the PCMS. The software also provides options for various types of traffic data to meet the real-time speed control system needs.
- The SDLMS utilized DOT compliant PCMS to convey real-time traffic condition information to motorists.
- The SDLMS can operate continuously (24 hours, 7 days a week) for the duration of the project.
- Critical system operator control functions were password protected.
- The SDLMS is capable of acquiring traffic data and selecting motorist information messages automatically without operator intervention after system initialization.
- SDLMS is an independent standalone unit with provision(s) for future integration with other traffic control / maintenance systems.

- The SDLMS traffic sensor’s accuracy is not degraded by inclement weather of degraded visibility conditions including precipitation, fog, darkness, excessive dust, and road debris.
- All traffic data acquired by the DLMS are archived in a log file with time and date stamps.

4.4.3 SDLMS Traffic Data Acquisition

The SDLMS operation is based on real-time speed data acquired from the traffic detection zones with each data sample ‘Time Stamped’ to indicate currency of the message displayed. Software provided with the SDLMS system allows the operator to have options of various categories of traffic information to suit the needs of the speed control system as follows.

4.4.4 SDLMS Motorist Information Messages

The SDLMS message information characteristics are as following:

- Records of all motorist information messages displayed by the SDLMS are recorded in log files with time and date stamps.
- The SDLMS is capable of displaying default messages when traffic conditions, system algorithms, and user parameters do not dictate that an advisory message should be displayed.
- The SDLMS is capable of displaying separate, independent default messages, as well as separate, independent advisory messages on each PCMS.

- The SDLMS' default and advisory messages are capable of being automatically selected based on traffic conditions at a single traffic sensor point or at multiple traffic sensor points in combination.
- Default and advisory message content shall be programmable from the central base station.
- The SDLMS is capable of adjusting the thresholds for advisory message selection on an individual traffic sensor station basis from the central computer base station.
- For later use, the SDLMS is capable of storing messages created by an authorized user in overriding any default or automatic advisory message.

4.4.5 SDLMS Communications

The SDLMS communications characteristics are as following

- The SDLMS's communications system incorporates an error detection / correction mechanism to ensure the integrity of all traffic conditions data, motorist information messages.
- Any required configuration of the SDLMS's communications system is performed automatically during system initialization.
- Communications between central computer base station and any individual PCMS or traffic sensor station is independent through the full range of deployed locations and not rely upon communications with any other system.

4.4.6 SDLMS' Other Requirements

The SDLMS' other requirements are as following

- Remote sign operation via central computer base station using wireless licensed UHF radio frequencies in the range of 464 MHZ to 470 MHZ and provision(s) to install antenna
- National Transportation Communications for ITS Protocol (NTCIP) version 2 conformant and proprietary communications protocol, if any, shall be provided to the DLMS provider in proper format.
- Licenses / permissions to legally operate a wireless system must be owned by the DLMS system provider, where required.
- The central computer base station shall be housed at a suitable location, to facilitate wireless communications, and in a suitable enclosure with AC power, internet access or a minimum of a reliable, dedicated telephone line.

4.4.7 Remote Traffic Microwave Sensor

RTMS are radar-based, non-intrusive, advanced sensors for the detection and measurement of traffic on roadways. They are known to be easy to install, remove, and maintain without traffic disruption. As shown in Figure 4.3, the RTMS are pole-mounted on the side of the road. They can collect the per-lane presence, volume, vehicle classification, occupancy, and speed in up to 8 user-defined detection zones.

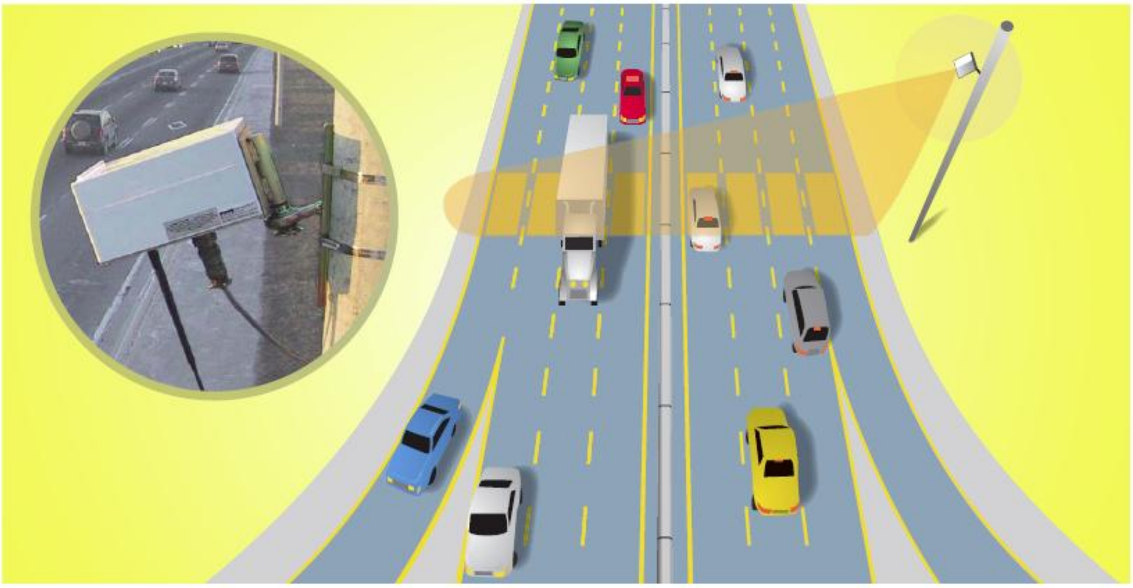


Figure 4.3: Remote Traffic Microwave Sensors

4.5 SDLMS field setup

4.5.1 SDLMS Preparation

The SDLMS preparation is shown in Figures 4.4, 4.5, and 4.6 . The University of Central Florida (UCF) team setup the SDLMS at the site and the details are as following:

- UCF took the sensor trailer to the first site Feb 7, 2008.
- The new chip received from VERMAC was installed in the VERMAC PCMS.
- The communication system including antennas and processing unit was installed
- The RTMSs were mounted on the PCMS and the sensor trailer

4.5.2. SDLMS Testing

The UCF team tested the SDLMS at the site and the details are as following:

- The communication between the sensor trailer and the PCMS was tested (Feb 16, 2008)
- The RTMS was tested including the proper leveling of the sensor and the calibration.
- The UCF team was trained on the calibration of the RTMS.
- The UCF team was trained on the daily setup of the SDLM system including the proper leveling of the sensor trailer and the instantaneous testing of the communication system.
- UCF was also trained on extracting the data from the RTMS.

It should be noted that the communication system on the additional PCMS relies on the proper power supply from the latter. The communication between the PCMS and the sensor trailer may fail if the batteries of the PCMS are not properly charged. It should also be noted that on average it takes about one hour to level the sensor trailer and to calibrate the RTMS upon every relocation.

Figure 4.4: PCMS Chip Modification

Figure 4.5: Antenna Installation and Sensor Trailer Setup

Figure 4.6: SDLMS Controller

4.5.3 RTMS Calibration

The RTMS was calibrated on a daily basis upon reinstallation. The sensor trailer was leveled in a way that the pole on which the RTMS is mounted is perpendicular to the road. The first step in the calibration consisted of creating the capturing or sensing zones as shown in Figure 4.7. In our case, for the first site with two-to-one lane closure, there were two lanes therefore two sensing zones were created. Sequentially, the calibration of the speeds is implemented. It should be noted that the calibration time of the RTMS takes about 30 to 45 minutes to be completed. After completing the calibration process the system was set to operate.

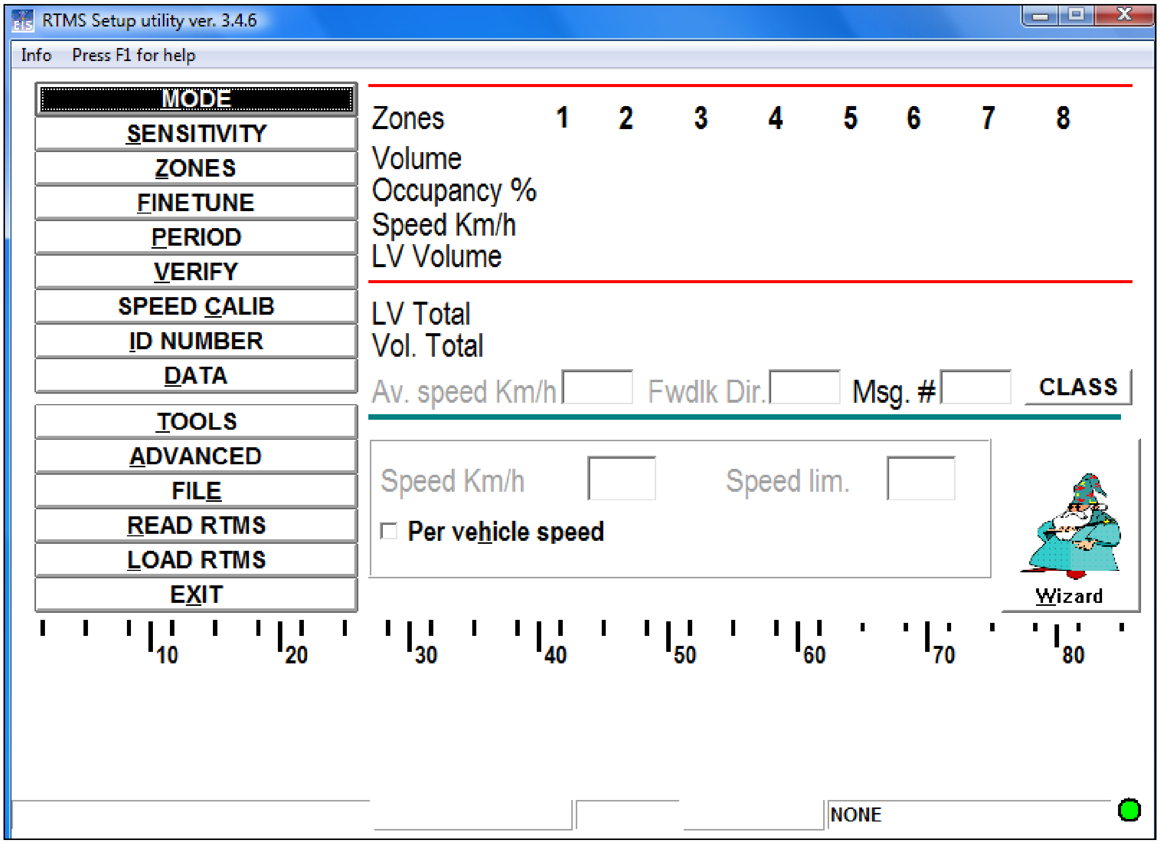


Figure 4.7: RTMS Calibration

4.5.4 System Check-up

The SDLMS provided by IRD, Inc. contains an application that allows us to check on the performance of the system. The system contains an “Adaptir” map (shown in Figure 4.8) that displays the location of the sensor trailer and the PCMS on the map and shows a green light for the correct wireless communication between the sensor (RTMS) and the PCMS. In case there is a miscommunication (wireless defect) the Adaptir map displays a red light and display an error message.

```
19:09:16 New Msg Type Initializing
19:09:16 New Msg Param: 0
19:09:16 New Message Pending: YES
19:09:16 VMS Type: No VMS
19:09:16 Sensor Type: RTMS
19:09:16 Milepost:  0.5
19:09:16 Idest: 005
19:09:17 System Start
19:09:17 VMS00 Battery Voltage 12.0; N/A AC Source
19:09:17 VMS01 Battery Voltage 12.0; N/A AC Source
19:09:17 Starting System Scans 120 secs
19:09:17 Initializing Sensors
```

Figure 4.8: Adaptir Map

4.6 SDLMS Operation

The SDLMS operation is based on real-time speed data acquired from the traffic detection zones with each data sample (time-stamped over 2 minutes) to indicate currency of the message displayed. The RTMS collects the average speed of the vehicles passing through the detection zones over 2-minute time intervals. The SDLMS operates under two modes; the passive mode (not activated) and the active mode (activated). Under the passive mode the additional PCMS was set to display a flashing “CAUTION/CAUTION” message for both the early and late SDLMS. Under the active mode, the PCMS displays “DO NOT PASS” followed by “MERGE HERE” alternately for the early SDLMS and “STAY IN YOUR LANE’ followed by “MERGE AHEAD” alternately for the late SDLMS (as shown in Table 4.1). The early and late SDLMS were activated once the average speed over any 2-minute time interval drops below 50mph. The SDLMS was deactivated (passive mode) once the average speed over the next time stamp goes over 55 mph. It should also be noted that the minimum activation time of the PCMS was set for 5 minutes.

4.7 Additional PCMS Messages

When the RTMS' average collected speed over two minutes drops below 50 mph, the PCMS displays "DO NOT PASS" followed by "MERGE HERE" in the early merge setup and "STAY IN YOUR LANE" followed by "MERGE AHEAD" in the late merge setup. When the average speed goes above 55 mph the PCMS will display a blinking "CAUTION/CAUTION" message.

Table 4.1: SDLMS' Active and Passive Messages

Early Merge

Activated			NOT Activated	
DO	MERGE			
NOT	HERE		CAUTION	CAUTION
PASS				

Late Merge

Activated			NOT Activated	
STAY	MERGE			
IN YOUR	AHEAD		CAUTION	CAUTION
LANE				

4.8 Project communication

The UCF research team communicated with multiple parties to conduct this project (see Figure 4.9):

1. IRD Inc. provided the SDLMS system components
2. Smart Technologies provided the communication system and system training.
3. Highway technologies provided the PCMS through FDOT.
4. VERMAC provided the updated PCMS chip to match the system’s protocol.
5. A Florida licensed professional engineer (consultant) signed and sealed the modified MOT plans.
6. FDOT project manager from the central office along with FDOT district 5 and UCF team selected sites for data collection.
7. UCF team, local operation office, and road rangers were constantly in touch during the data collection.

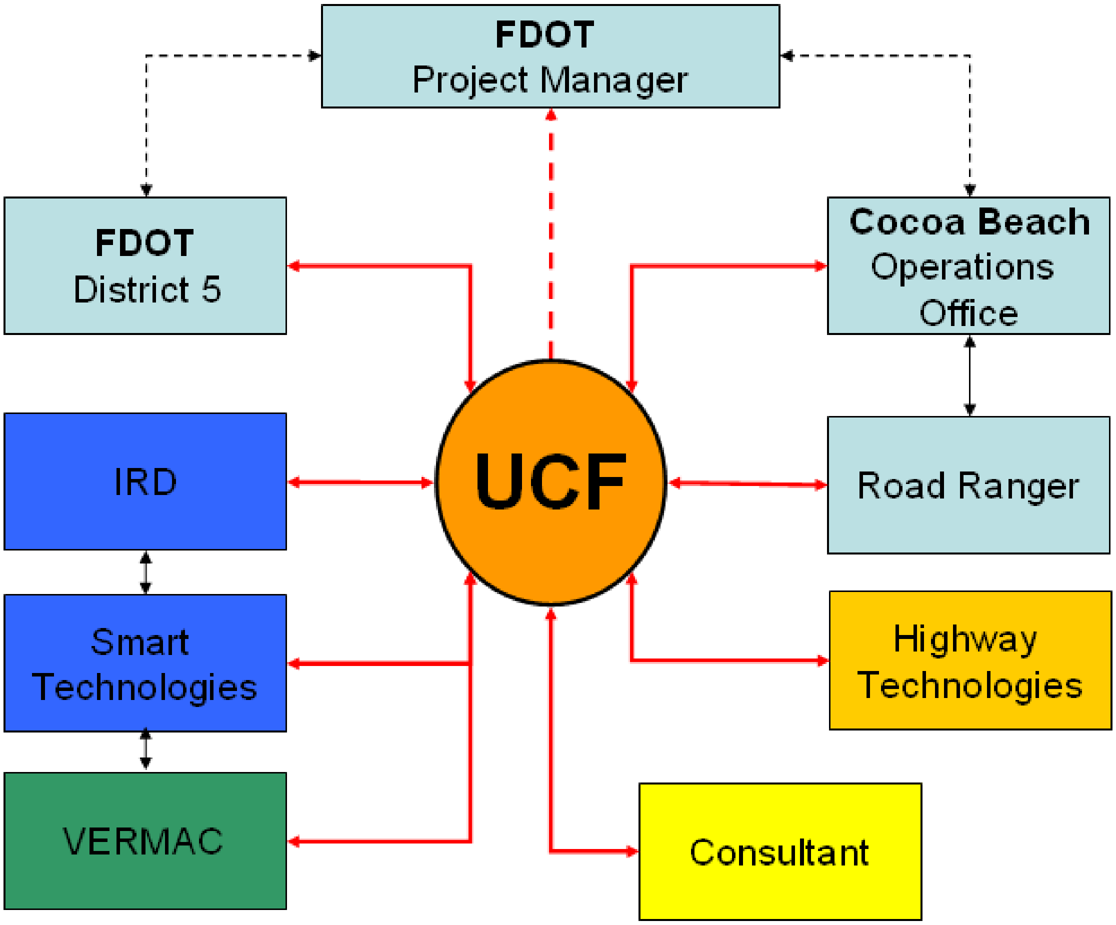


Figure 4.9: Project Communication Flowchart

CHAPTER 5 APPLICATION OF THE SDLMS ON A TWO-TO-ONE LANE CLOSURE

5.1 Data Collection

Data was collected on the first selected site to secure volume and travel time data for the evaluation of the tested system. It should be noted that data collection was not intrusive to the freeway therefore not creating distraction or disruption to traffic.

5.1.1 Site Location

The selected site was located on Interstate-95 in Malabar, Florida as shown in Figure 5.1. I- 95 is two-lane per direction limited access rural freeway with 70 mph speed limit (reduced to 60 mph during work). The work zone consisted of a resurfacing and milling job on the south bound of I-95 on a 13 mile stretch. A two to one lane closure configuration was adopted and the work zone moved on a daily basis covering a length of approximately three miles per day. Data was collected on homogenous basic freeway segment of I-95 with no on/off ramps.

5.2.1 Data Collection Methodology

Four Digital Camcorders were set in the field labeled C-1, C-2, C-3, and C-4 as shown in Figure 5.2. To synchronize the camcorders spatially (i.e. upon daily relocation), C-1 was always located behind the first PCMS, C-2 was always located behind the radar speed display unit (RSDU), C-3 was always located behind the arrow panel, and C-4 was always located at the end of the lane closure. All four camcorders were started at the same time to synchronize the temporal events and flow of vehicles. Data was collected on the same site for the MAS, early SDLMS, and late SDLMS for two days each. From C-1, C-2, C-3, and C-4, per-lane vehicle counts including vehicle classification were extracted in five minutes intervals in the laboratory. The zone between C-1 and C-2 is identified as zone 1 and the zone between C-2 and C-3 is identified as zone 2. The difference between the vehicle counts (including vehicle classification) in the closed lane between C-1 and C-2 is the number of lane changes made in zone 1. The remaining vehicle counts (including vehicle classification) remaining in the closed lane at C-2 is the number of lane changes in zone2.

The RTMS was temporally synchronized with C-1, C-2, C-3, and C-4 and the PCMS activation time (recorded by the RTMS) was extracted and concatenated temporally to the vehicle count data. From C-1 the demand volume for the work zone was determined. From C-4 the throughput of the work zone was determined. Under the standard MAS configuration, data was collected on February 11th and 12th 2008, under the early SDLMS data was collected on March 17th and 18th 2008, and under the late SDLMS data was collected on March 27th and 28th, 2008. There were several difficulties engaged in the data collection process. In fact, for short term moving work zones, there exist inherent logistic and operational difficulties. For instance, the work, hence data collection was cancelled and/or interrupted unexpectedly multiple times due to adverse weather conditions.

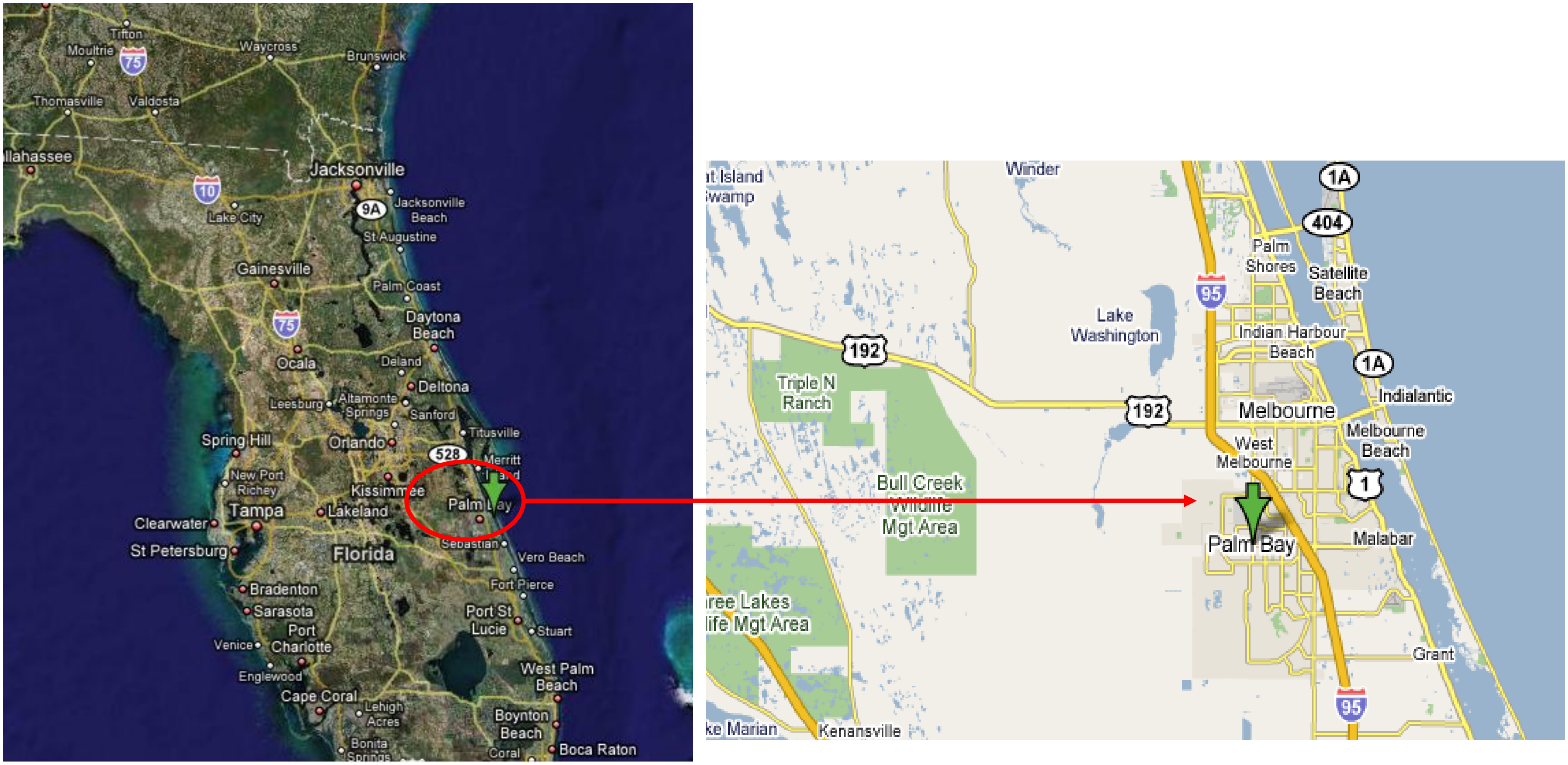


Figure 5.1: Data Collection Site, Malabar, Florida

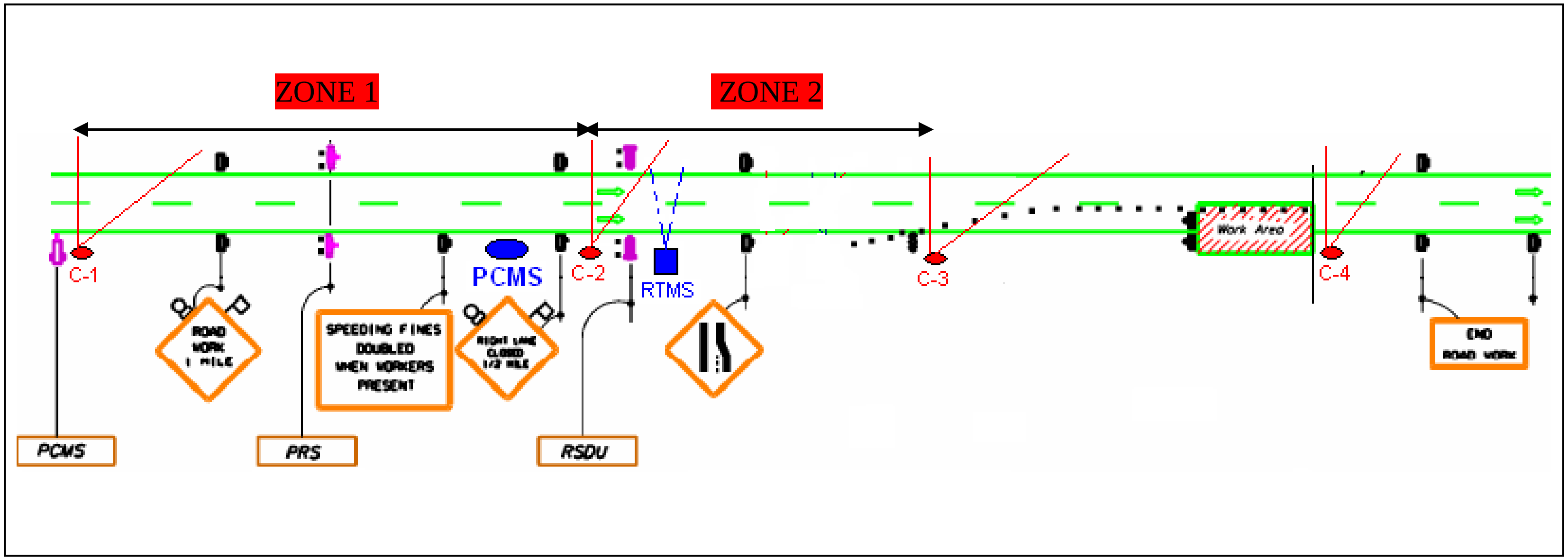


Figure 5.2: Camcorders Location

Work was also unexpectedly cancelled on several occasions without prior notice due to contractor-related logistic issues. Moreover, the freeway shoulders were narrower at some locations which made the installation of the SDLMS equipment almost impossible. It is recommended that a good communication/planning protocol be established between the researcher team and the work zone crew (construction manager) for future data collection on short term moving work zones.

5.2 DATA Analyses

Roadway capacity in which a work zone is located is lower than the normal operating conditions. The impact of the early and late SDLMS on the work zone capacity is studied by comparing the capacity of the work zone under the MAS traffic (control) to the capacity of the work zone under the early SDLMS (test1) and late SDLMS (test2). It should be noted that different researchers, as mentioned by Heaslip et al. (2007) , have different definitions of work zone capacity. “Some researchers (Dudek and Richards, 1981; Kermode and Myra, 1970; Maze et al., 2000) measured the mean queue discharge flow rate as work zone capacity when the upstream of work zones was in sustained congested traffic flow, while other researchers (Dixon et al., 1999; Jiang, 1999) defined the work zone capacity as the traffic flow at the onset of congested traffic conditions” (Ping and Zhu, 2006).

In this study, the work zone capacity under the three different scenarios is determined as the queue discharge flow rate or throughput volume under queuing/congested conditions.

The onset of congestion is detected visually by C-2 shown in Figure 5.2. Since only two days of data collection under each MOT type were available for this site, and to control for the demand volume, the ratio of throughput at the onset of congestion over the demand volume is taken as the operational measure of effectiveness (MOE).

5.2.1 Statistical Summary

Table 5.1 summarizes the data extracted from C-1, C-2, C-3, and C-4. As shown by Table 5.1, the mean and maximum throughputs at the onset of congestion of the early SDLMS are the highest among the three MOT treatments. The mean and maximum capacities of the conventional MAS system are 881 veh/hr and 1092 veh/hr, respectively. The mean and maximum capacities of the early SDLMS are 970 veh/hr and 1272 veh/hr, correspondingly. The mean and maximum capacities of the late SDLMS are 896 veh/hr and 1093 veh/hr in that order. The mean ratio of throughput over the demand volume is also the highest for the early SDLMS taking a value of 0.84 followed by the late SDLMS taking a value of 0.79 the MAS system taking a value of 0.79. This indicates that normalizing for the demand volumes, the early SDLMS resulted in the highest throughput. Also from Table 5.1, the mean number and mean percentage of lane changes in zone 1 for cars and trucks are the highest for the early SDLMS and the lowest for the late SDLMS. These average numbers of lane changes are taken for all times including when the additional PCMS is not activated for the early and late SDLMS. The mean number and percentage of passenger cars changing lanes in zone 1 for the early SDLMS are 293pc/hr and 67.5% respectively (92Trk/hr, 76.9% for trucks). The mean and percentage of passenger cars changing lanes in zone 1 for the late SDLMS are 274 pc/hr and 51.9% respectively (33 Trk/hr, 74.1% for trucks). The mean and percentage of passenger cars changing lanes in zone 1 for the conventional MAS are 143 pc/hr and 66.3% in that order (57Trk/hr, 79.6% for trucks). These results indicate that some drivers are complying with the messages displayed by the additional PCMS in the early and late SDLMS.

Table 5.1: Data Summary Statistics

MOT Type	Variable	Unit	Mean	Std. Dev.	Min	Max
Conventional MAS	Throughput*	Veh/hr	881	120	624	1092
	Ratio	N/A	0.79	0.23	0.55	1
	Car lane changes in zone 1	PC/hr	143	118	84	324
	TRK lane changes in zone 1	TRK/hr	57	46	84	120
	Car lane changes in zone 2	PC/hr	51	53	48	168
	TRK lane changes in zone 2	TRK/hr	16.8	30	12	132
	% TRK	N/A	15.1	6	2.4	25.8
	% Car lane changes in zone 1	N/A	66.3	24.7	12.5	95.7
	% TRK lane changes in zone 1	N/A	79.6	19.2	38.9	100
Early SDLMS	Throughput*	Veh/hr	970	135	696	1272
	Ratio	N/A	0.84	0.24	0.66	1
	Car lane changes in zone 1	PC/hr	293	102	96	516
	TRK lane changes in zone 1	TRK/hr	92	81	24	312
	Car lane changes in zone 2	PC/hr	108	62	21	312
	TRK lane changes in zone 2	TRK/hr	23	26	24	96
	% TRK	N/A	5.5	13.6	13.6	35.7
	% Car lane changes in zone 1	N/A	67.5	7.1	7.1	100
	% TRK lane changes in zone 1	N/A	76.9	10.2	0	100
Late SDLMS	Throughput*	Veh/hr	896	111	696	1092
	Ratio	N/A	0.81	0.19	0.69	1
	Car lane changes in zone 1	PC/hr	274	95	60	516
	TRK lane changes in zone 1	TRK/hr	33	24	24	312
	Car lane changes in zone 2	PC/hr	100	51	12	312
	TRK lane changes in zone 2	TRK/hr	12	13	24	96
	% TRK	N/A	24.6	5.4	23.6	35.7
	% Car lane changes in zone 1	N/A	51.9	15.7	7.1	100
	% TRK lane changes in zone 1	N/A	74.1	23	0	100

During the early and late SDLMS, the additional PCMS may not be activated when the average detected speed does not fall below the preset threshold speed (50 mph).

Therefore, a comparison between the throughputs of the early and late SDLMS with the conventional MAS only when the additional PCMS is activated, hence displaying the lane merging advisory messages is conducted. Therefore, a new variable (labeled ACT) is derived to reflect this issue. This variable (ACT) consists of four levels; early and late SDLMS not activated, early SDLMS activated, late SDLMS activated, and conventional MAS.

5.2.2 Correlation Analysis

To examine the correlation between a categorical variable with more than two levels and a continuous variable, one can compute a Friedman's test with no assumption on the homogeneity of variances. Table 5.2 shows that the ratio (throughput over demand) is significantly correlated with MOT type (*p*-value=0.01). Table 5.2 below also shows that the percent trucks in the traffic, percent car changing lane in zone 1, and percent trucks changing lane in zone 1 are not correlated with MOT type. Although it seems intuitive for a correlation between MOT type and percentage of lane changing in zone 1 for passenger cars and trucks to exist, the results show no significant correlation. This may be due the compliance rate variance. For example during early merge instructions the compliance rate may be low therefore drivers may still merge late. On the other hand, during late merge compliance rate may be low therefore drivers may still merge early. The result of the compliance rate variance in early and late merge may have caused no significant difference in the correlation between MOT type and percentage lane changing in zone 1 for passenger cars and trucks.

Table 5.2: Correlation between MOT type and continuous variables

Variable	Unit	**Pr > F**
Throughput/Demand	N / A	0.01
% Car Lane Changes in Zone 1	N / A	0.2371
% Truck Lane Changes in Zone 1	N / A	0.844
% TRK In Traffic	N / A	0.622

A correlation analysis between truck percentages in the traffic composition, percent truck lane changing in zone 1, percent car lane changing in zone 1 is conducted. For these continuous variables Spearman's rank-order correlation and Pearson's correlation are used. Pearson's correlation is a parametric measure of association which measures both the strength and the direction of the linear relationship. Spearman's correlation is nonparametric measure of association based on the ranks of the data values. From Tables 5.3 and 5.4 one can assume no correlation between all continuous variables in question since all *p*-values are greater than 0.05.

Table 5.3: Pearson's Correlations

		% PC changing lane In zone 1	% TRK changing lane In zone 1	%TRK
% PC changing lane In zone 1	Pearson Correlation	1	-.012	.049
	Sig. (2-tailed)		.907	.620
	N	105	105	105
% TRK changing lane In zone 1	Pearson Correlation	-.012	1	-.030
	Sig. (2-tailed)	.907		.761
	N	105	105	105
%TRK	Pearson Correlation	.049	-.030	1
	Sig. (2-tailed)	.620	.761	
	N	105	105	105

Table 5.4: Spearman's Correlations

Spearman's rho		% PC changing lane In zone 1	% TRK changing lane In zone 1	%TRK
% PC changing lane In zone 1	Correlation Coefficient	1	0.048	0.026
	Sig. (2-tailed)	.	0.623	0.788
	N	105	105	105
% TRK changing lane In zone 1	Correlation Coefficient	0.048	1	-0.12
	Sig. (2-tailed)	0.623	.	0.221
	N	105	105	105
%TRK	Correlation Coefficient	0.026	-0.12	1
	Sig. (2-tailed)	0.788	0.221	.
	N	105	105	105

5.2.3 Ratio of throughput over demand volume analysis

A multiple linear regression model is estimated to explore the effect of the MOT plan type, truck percentage in the traffic composition, percentage of trucks changing lane in zone 1, and percentage of passenger cars changing lanes in zone 1 on the ratio of throughput over demand volume. As mentioned earlier a new variable is ACT reflecting whether the PCMS is activated is added to replace MOT type. The ACT variable has four categories. Early SDLMS activated, late SDLMS activated, SDLMS deactivated, and MAS. Table 5.5 shows the results of the regression model. From Table 5.5, the ACT shows significant effect on the work zone throughput over demand volume ratio. In particular, the early SDLMS treatment affects positively (parameter estimate= 0.103, *P*-value=0.004) and significantly the throughput of the work zone compared to the conventional MAS maintenance of traffic plan. The other variables included in the model

Table 5.5: Multiple Linear Regression Results

ANOVA and Parameter Estimates

Parameter	Categories	Estimate	Standard Error	t Value	Pr > |t|
Intercept	N/A	0.784	0.062	12.710	<.0001
% PC lane changing in zone1		-0.095	0.055	-1.720	0.089
% TRKlane changing in zone1		0.020	0.040	0.490	0.628
%TRK		-0.020	0.193	-0.100	0.917
ACT	Late SDLMS	0.045	0.049	0.920	0.359
	Early SDLMS	0.103	0.035	2.930	0.004
	NOT ACTIVATED	0.043	0.034	1.260	0.209
	CONVENTIONAL MAS	0.000	.	.	.

Overall ANOVA

Source	Sum of Squares	Mean Square	F Value	Pr > F
Model	29.561	2.760	2.45	0.0291
Error	109.684	1.126		
Corrected Total	139.245			

R-square=0.2123

do not have a statistical significant effect on the work zone throughput at 0.05 significance level. The R-square of the model was 0.2123 indicating that the 21.23% of the variance in the ratio (throughput over demand volume) can be explained by the explanatory variables in the model.

5.2.4 Travel Time

Camcorders C-1 and C-4 were used to observe the travel time through the work zone. Past literature (Oppenlander, 1976; Quiroga and Bullock, 1998) documented methods to determine the minimum required sample size for travel time runs to achieve reliable and accurate results. The following Equation is used to determine the number of runs required (May, 1990):

$$n = \left\{ \frac{\hat{\sigma} x * Z}{\varepsilon} \right\}^2$$

Where,

N = Estimated sample size for number of runs at the desired precision and level of confidence

σ = Preliminary estimate of the population standard deviation for average travel speed among the sample runs

Z = Two-tailed value of the standardized normal deviate associated with the desired level of confidence (at a 95% confidence interval, Z=1.96)

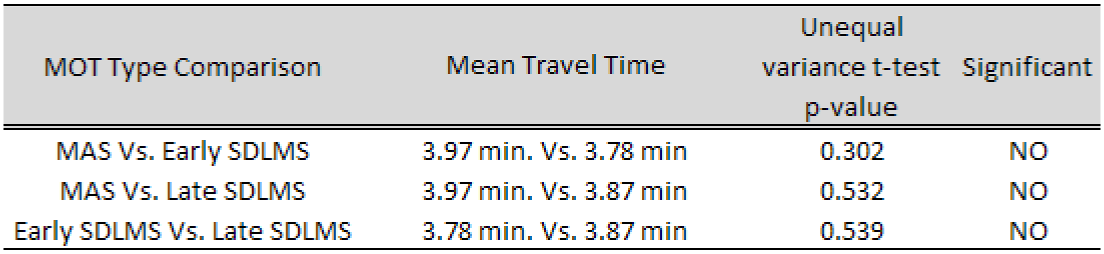
ε = Acceptable Error (±3 mph)

According to Oppenlander (1976), the allowable errors range between ±1 mph to ±3 mph for ‘before and after’ entailing operational improvement of roadways. In this study the allowable error is assumed to be ±3 mph. During the MAS only (before period) 45 travel time runs were determined. The resulting mean and standard deviation for the average travel speed through the work zone were determined to be 37.5 mph and 8.74 mph respectively. The resulting minimum required sample size of travel time runs is determined by the above Equation to be:

$$n = \left\{ \frac{8.74 * 1.96}{3} \right\}^2 = 33$$

The actual number of travel time runs for the MAS, early SDLMS, and late SDLMS exceeded the minimum required number of runs (n_{MAS}=63; n_{early}=67; n_{late}= 69). A Levene's test was conducted to test the homogeneity of travel time variances for the MAS, early SDMLS, and late SDLMS. Levene's test indicated the variances significantly different (*P*-value= 0.024). Therefore, the unequal variance t-test was performed to determine whether there exists a significant difference in the travel times between the three treatments. The average travel time for the MAS, early and late SDMLS are 3.97minutes, 3.87 minutes, and 3.78 min respectively and the resulting *p*-values are 0.302 (comparing early SDLMS to MAS), 0.532 (comparing late SDLMS to MAS), and 0.539 (comparing early and late SDMLS) indicating no statistical significant difference between the travel times of MAS, early and late SDLMS. Table 5.3 summarizes the travel time comparison between the three treatments.

Table 5.3: Travel Time Comparison



MOT Type Comparison	Mean Travel Time	Unequal variance t-test p-value	Significant
MAS Vs. Early SDLMS	3.97 min. Vs. 3.78 min	0.302	NO
MAS Vs. Late SDLMS	3.97 min. Vs. 3.87 min	0.532	NO
Early SDLMS Vs. Late SDLMS	3.78 min. Vs. 3.87 min	0.539	NO

5.3 Conclusions

The throughput over demand volume of the work zone under the control and test MOT plans was used as a measure of effectiveness to explore the impact of the early and late

SDLMS on work zones. Results showed that the early SDLMS enhances work zone mean throughput over demand volume significantly. The late form of SDLMS increased the mean throughput over demand volume, however this increase was not statistically significant.

The travel time through the work zone under the control and test MOT plans were examined. The average travel time for the MAS, early and late SDMLS are 3.97 minutes, 3.87 minutes, and 3.78 min respectively and did not result in statistically significant difference. This indicates that the simplified dynamic early and late merge did not affect the travel time through the work zone. It should be noted that the travel time under each MOT type was taken as the average travel time under all demand volumes and trucks percentages in the traffic composition which may be related to the statistical insignificance among the differences in the travel time means. A disaggregation of the travel time data for different volume levels and trucks percentage level was not possible due to the limitation in the data sample size.

The number and percentage of lane changes in zone 1 were the highest for the early SDLMS and the lowest for the late SDLMS. This indicates that drivers are complying with the messages displayed by the additional PCMS. It was noted during data collection, for the early SDLMS, that drivers usually comply with the messages displayed by the PCMS. However, it was also observed that when a vehicle uses the closed lane to pass vehicles in the queue and merge into the open lane ahead of them, a platoon of vehicles follows this vehicle which defeats the purpose of the early SDLMS.

This first site was used as a pilot for testing the SDLMS system. It should be noted that the sample size of data points was not large enough to conduct thorough analyses of travel times and throughput under different demand volumes and truck percentages. It should also be noted that the delivery of the SLMDS was delayed by the vendor which disabled us from using the RTMS to collect speed data that was intended for use as a safety surrogate measure. Since data sample size was a limited the scope of the results a simulated two-to-one lane closure configuration work zone is coded in VISSIM. The simulation model is calibrated and validated with the available data (See Chapter 7).

CHAPTER 6 APPLICATION OF THE SDLMS ON A THREE-TO-TWO LANE CLOSURE

6.1 Site Location

The selected site was located on Interstate-95 in Palm Beach, Florida as shown in Figure 6.1. At that location I- 95 consisted of three -lane per direction urban freeway with 60 mph speed limit (reduced to 50 mph during work). The work zone consisted of a resurfacing and milling job on the south bound of I-95 on an 8 mile stretch. A three-to-two lane closure configuration was adopted and the work zone moved on a daily basis covering a length of approximately three miles per day. Data was collected on homogenous basic freeway segment of I-95.

6.2 DATA Collection

Four digital camcorders were set in the field labeled C-1, C-2, C-3, and C-4 as shown in Figure 6.2. To synchronize the camcorders spatially (i.e. upon daily relocation), C-1 was always located behind the first PCMS, C-2 was always located behind the additional PCMS, C-3 was always located by the beginning of the lane closure, and C-4 was always located at the end of the lane closure. All four camcorders were started at the same time to synchronize the temporal events and flow of vehicles. Data was collected on the same site for the MAS, early SDLMS, and late SDLMS for two days each. From C-1, C-2, C-3, and C-4, per-lane vehicle counts including vehicle classification were extracted in 5 minutes intervals in the laboratory. The zone between C-1 and C-2 is identified as zone 1 and the zone between C-2 and C-3 is identified as zone 2. The difference between the vehicle counts (including vehicle classification) in the closed lane between C-1 and C-2 is the number of lane changes made in zone 1. The remaining vehicle counts (including vehicle classification) remaining in the closed lane at C-2 is the number of lane changes in zone 2.

Since the TRMS was available during the MAS, early SDLMS, and late SDLMS, speed data is extracted and used as a safety surrogate measure. Recall, from the previous site this was not possible due to delay in the system's delivery. The RTMS was temporally synchronized with C-1, C-2, C-3, and C-4 and the PCMS activation time (recorded by the RTMS) was extracted and concatenated temporally to the vehicle count data. From C-1 the demand volume for the work zone was determined. From C-4 the throughput of the work zone was determined.

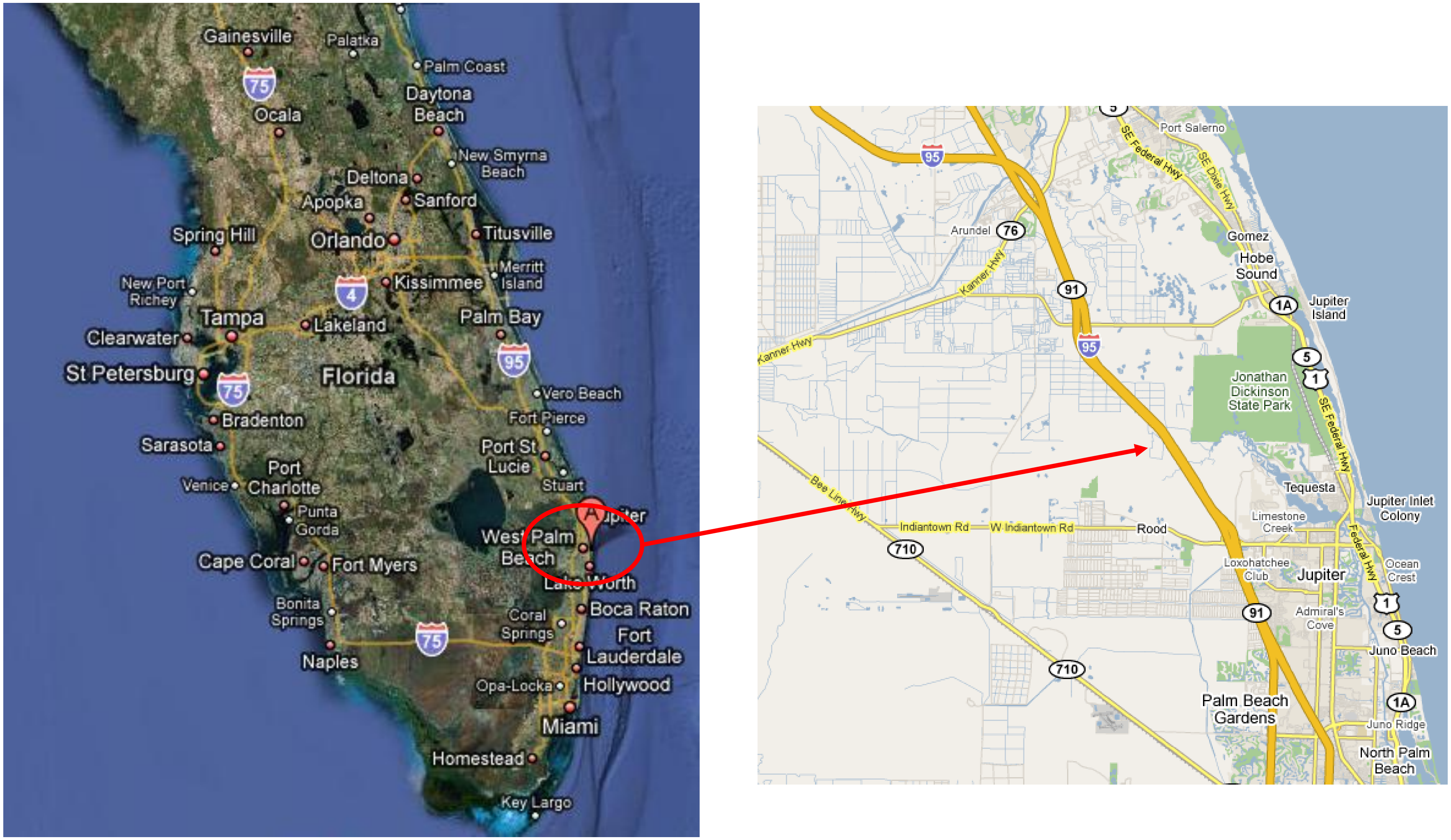


Figure 6.1: Data Collection Site, Palm Beach, Florida

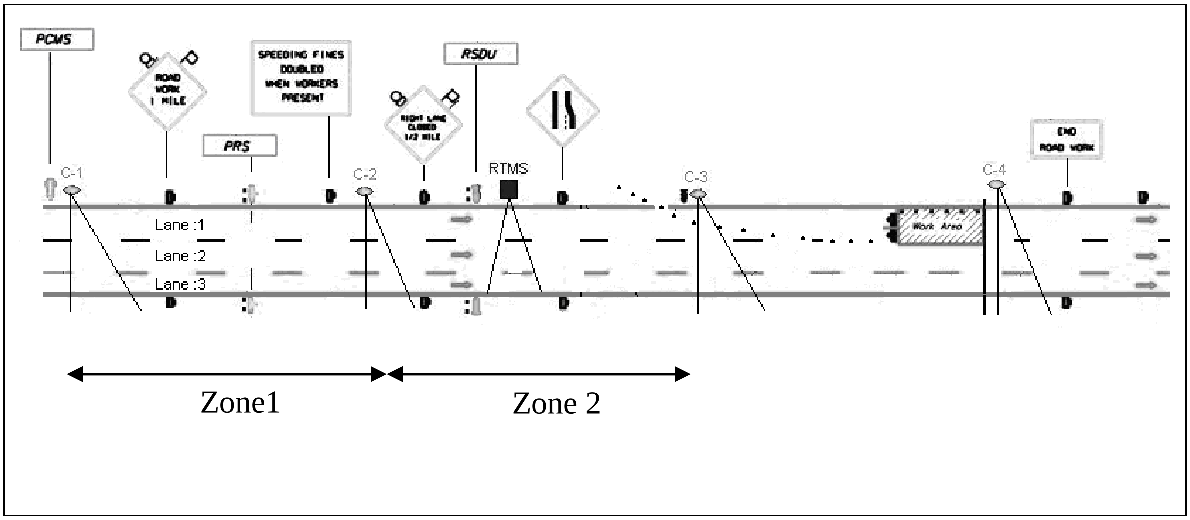


Figure 6.2: Cameras Location

6.2 Safety MOE

The speed fluctuation at the location of the RTMS is taken as the safety measure of effectiveness (MOE). The speed fluctuation is the difference in average speed over two-minute consecutive time intervals. If the speed fluctuation is high one can conclude that the risk of accident is higher. Figure 6.3 shows the distribution of the speed fluctuations under the MAS, early SDLMS, and late SDLMS treatments in that order. Lane 1 is the closed lane, lane 2 is the middle lane and lane 3 is the outer lane. A negative speed fluctuation means a speed drop between two consecutive time intervals and a positive speed fluctuation means a speed increase between two consecutive time intervals. Examining Figure 6.3, one can conclude that work zone under the MAS regime undergoes the highest speed fluctuations. The range of speed fluctuation for the closed lane (lane 1) under the MAS MOT plans varies between -48mph to 47mph, compared to a range of -9 mph to 7 mph for the dynamic early merge and a range of -5 mph to 3 mph to the dynamic late merge. The range of speed fluctuation for the middle lane (lane 2) varies from -12.5 mph to 17.5 mph for the MAS MOT plans compared to -8 mph to 5 mph for the dynamic early merge and -6 mph to 5 mph for the dynamic late merge. As for the outer lane (lane 3) the speed fluctuation varies from -66 mph to 68 mph for the MAS system compared to a range of -13 mph to 10 mph for the dynamic early merge and a range of -5 mph to 7 mph for the dynamic late merge (See Figure 6.3). Figure 6.4 compares the speed fluctuations for lanes 1, 2, and 3 under different demand volumes for the three different MOT types. Looking at MAS, the speed fluctuation for lane 1 (closed lane) and lane 3 (the outer lane) are the highest for demand volumes below 1,500 veh/hr. Figure 6.4 shows that the speed fluctuation for lane 2 is fairly stable under different demand volumes. Looking at early and late charts from Figure 6.4, one can conclude that the speed fluctuation is fairly similar under all demand volumes.

The next step was to examine the speed fluctuations in each lane under different demand volumes. To complete this task, the demand volumes were split into 5 categories. The first demand volume labeled v1 varies between 1 and 500 veh/hr. The second demand volume labeled v2 varies between 501 and 1000 veh/hr. The third demand volume v3 category varies between 1001 and 1500 veh/hr. The fourth demand volume v4 category ranges from 1501 and 2000 veh/hr and the fifth demand v5 is greater than 2001 veh/hr.

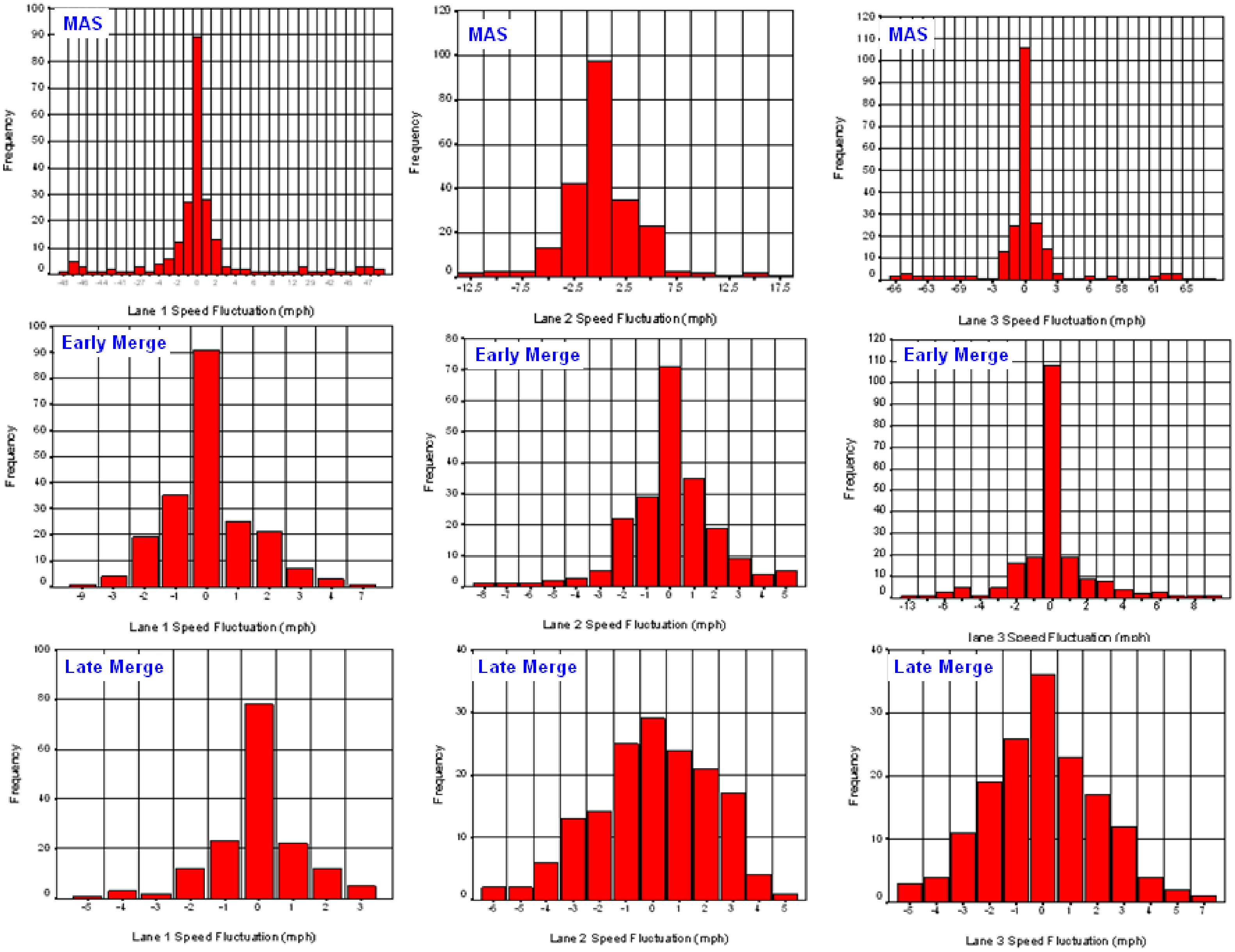


Figure 6.3: Speed Fluctuation per lane

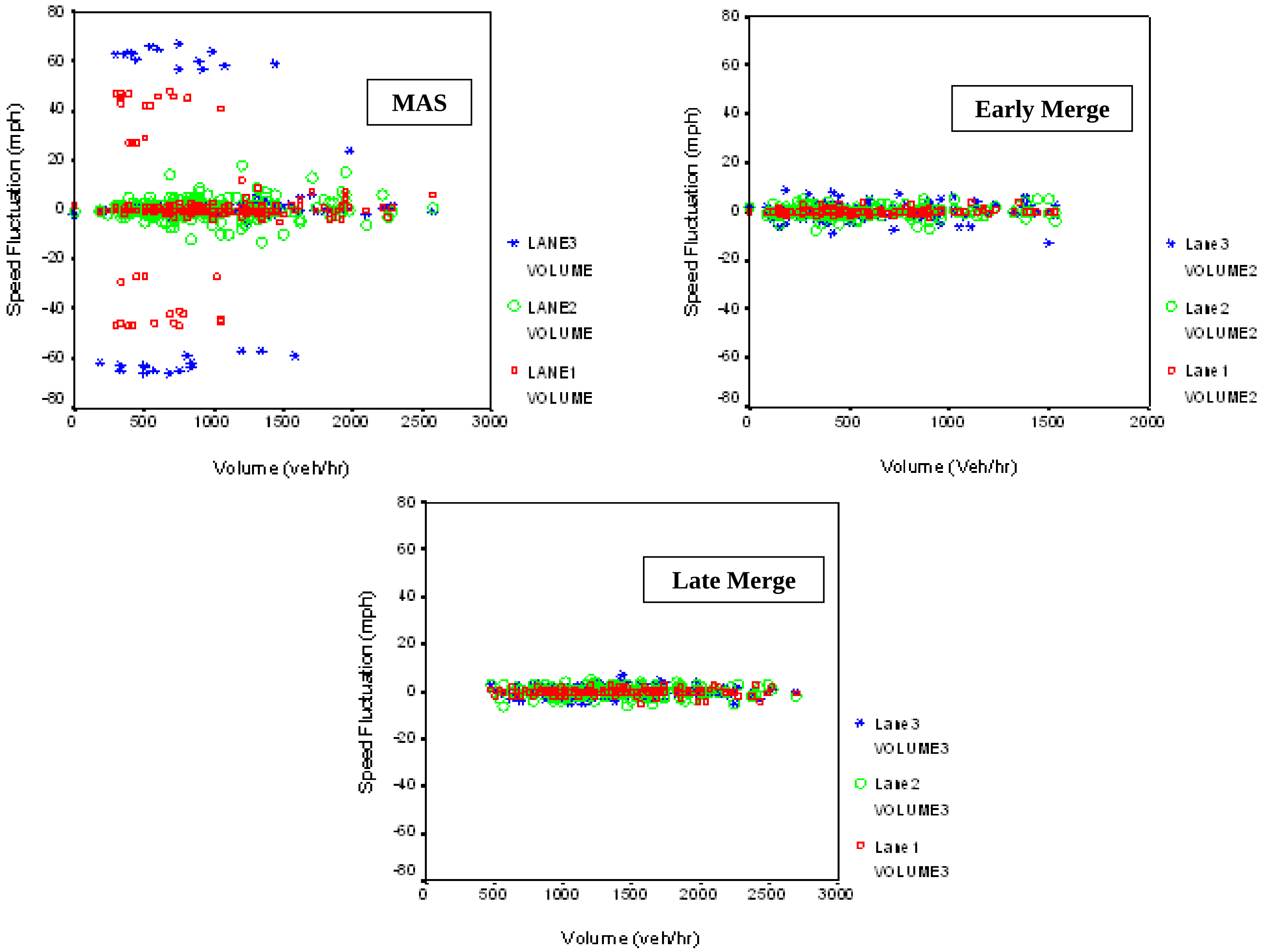


Figure 6.4: Speed Fluctuation for MAS/Early/Late Under Different Volumes

Table 6.1 displays the means of speed fluctuations under different volumes. Levene's tests revealed inhomogeneous variances of the speed fluctuations distributions. Therefore, unequal variance t-test comparing fluctuation means in each lane under different volumes were conducted and results are shown in Table 2. In table 6.2 the statistically significant differences between the speed fluctuation means (p-value<0.05) are highlighted in grey. As shown by Table 6.1, the mean speed fluctuation in lane 1 (closed lane) was the highest under the MAS system for all demand volumes. The *p*-values of the differences in those means are statistically significant (highlighted in grey in Table 6.2). This means that the dynamic late merge and the dynamic early merge have lower speed fluctuations in the closed lane under all demand volumes compared to the MAS system. Comparing the dynamic early merge and the dynamic late merge mean speed fluctuations in the closed lane, Table 6.1 shows that the mean speed fluctuation for the early merge are lower than those of the late merge under demand all demand volumes.

Looking at the speed fluctuations in the middle lane (lane 2), Table 6.1 shows that the mean speed fluctuations are the highest for the MAS system compared to dynamic early merge and dynamic late merge under all demand volumes. However, Table 6.2 shows that the mean speed fluctuations under the MAS are significantly higher than the mean speed fluctuations under the dynamic late merge only for volumes for volumes greater than 1500 veh/hr (and marginally at volumes between 1000 and 1500 veh/hr). Table 6.2 also shows that the mean speed fluctuations under the MAS are significantly higher than the mean speed fluctuations under the dynamic early merge system for volumes ranging between 500 and 1500 veh/hr. Comparing the mean speed fluctuations under the dynamic early merge and the dynamic late merge Table 6.1 shows that the mean speed fluctuations are lower for the dynamic early merge. However, there is no significant difference between the speed fluctuations in the middle lane (Lane 2).

Looking at the speed fluctuations in lane 3 (outer lane), Table 6.2 shows that the mean speed fluctuations are the highest under the MAS system compared to the dynamic early merge and the dynamic late merge under all volumes. However, Table 6.1 and 6.2 show that the mean speed fluctuations for the MAS system is significantly higher than the mean speed fluctuation for dynamic early and dynamic late merge for volumes under 1000 veh/hr. Moreover, Table 6.1 and 6.2 show a marginal significance indicating that the mean speed fluctuation for the late merge is lower than the mean speed fluctuation for the MAS system for volumes ranging 1000veh/hr to 2000 veh/hr.

Table 6.1 Mean Speed Fluctuations

	Mean Speed Fluctuation (mph)		
	Late Merge	**Early Merge**	**MAS**
Lane1 v1	1.50	0.32	16.94
Lane1 v2	0.74	0.63	5.75
Lane1 v3	0.72	0.62	4.78
Lane1 v4	0.98	0.00	2.63
Lane1 v5	1.56	N/A	2.20
Lane2 v1	1.50	1.24	1.22
Lane2 v2	1.69	1.63	2.39
Lane2 v3	1.72	1.43	3.49
Lane2 v4	1.95	2.50	4.32
Lane2 v5	1.50	N/A	3.40
Lane3 v1	2.00	1.56	16.17
Lane3 v2	1.51	1.93	9.27
Lane3 v3	1.79	2.52	5.88
Lane3 v4	1.52	4.75	5.32
Lane3 v5	1.38	N/A	1.40

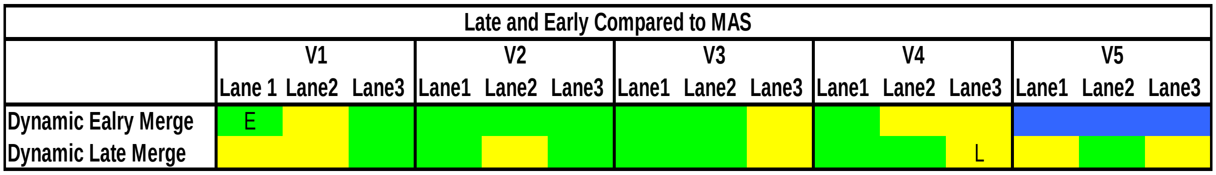
Table 6.2 Unequal variance t-test p-values

	P-value		
	Late Merge Vs. MAS	**Early Merge Vs. MAS**	**Early Merge Vs. Late Merge**
lane1 v1	0.2188	0.0000	0.0153
lane1 v2	0.0277	0.0052	0.5597
lane1 v3	0.0054	0.0850	0.6927
lane1 v4	0.0005	0.0384	0.0733
lane1 v5	0.4236	N/A	N/A
lane2 v1	0.3267	0.9766	0.3257
lane2 v2	0.0715	0.0260	0.9221
lane2 v3	0.0006	0.0157	0.4330
lane2 v4	0.0012	0.4247	0.1242
lane2 v5	0.0286	N/A	N/A
lane3 v1	0.0099	0.0000	0.9162
lane3 v2	0.0275	0.0111	0.0572
lane3 v3	0.0633	0.3343	0.1705
lane3 v4	0.0831	0.9388	0.0202
lane3 v5	0.9849	N/A	N/A

Comparing the mean speed fluctuations between the dynamic early and dynamic late merge, Table 6.1 shows that the means speed fluctuations are lower for the dynamic late merge under volumes higher than 500 veh/hr. However, Table 6.2 shows that the mean speed fluctuation for the dynamic late merge is significantly lower than the mean speed fluctuation for the dynamic early merge for demand volumes ranging between 1500veh/hr and 2000veh/hr.

Table 6.3 summarizes the safety MOE for each lane under different MOT plans. The colors compare the dynamic early and late merge to the MAS. The green color means that the dynamic early or late merge is better than the MAS. The yellow color means that the difference is not significant, and the blue color means that difference is unknown (small sample size). To compare dynamic early and late merge we used the letters E and L. As shown by the Table 6.3, the early SDLMS was better than the late SDLMS for Lane1 V1 and the late SDLMS was better than the early SDLMS for Lane3 V4.

Table 6.3: Comparison of Early SDLMS, Late SDLMS and MAS for Safety



Late and Early Compared to MAS															
	V1			V2			V3			V4			V5		
	Lane 1	Lane2	Lane3	Lane1	Lane2	Lane3	Lane1	Lane2	Lane3	Lane1	Lane2	Lane3	Lane1	Lane2	Lane3
Dynamic Ealry Merge	🟩 E	🟨	🟩	🟩	🟩	🟩	🟩	🟩	🟨	🟩	🟨	🟨	🟦	🟦	🟦
Dynamic Late Merge	🟨	🟨	🟩	🟩	🟨	🟩	🟩	🟩	🟨	🟩	🟩	🟨 L	🟨	🟩	🟨

6.3 Operational MOE

The ratio of throughput over demand volume is taken as an operational measure of effectiveness to test the impact of the early and late SDLMS on the work zone. The ratio of throughput over demand volume of the work zone under the MAS (control) to the capacity of the work zone under the early SDLMS (test1) and late SDLMS (test2) were compared. The onset of congestion is determined by C-3 shown in Figure 6.2.

Table 6.4 summarizes the variables taken into account to analyze the operational aspects of the work zone under three different regimes (MAS, early and late SDLMS). The maximum throughput for the work zone under the MAS system is 2,730 veh/hr. The maximum throughput under the dynamic early merge is 1890 veh/hr, and the maximum throughput under the late merge is 2940 veh/hr. The mean throughputs were 1064.87veh/hr, 763.96 veh/hr, and 1152.81 veh/hr for the MAS, early SDLMS, and late SDLMS respectively. It should noted here that the demand volumes for the MAS and late

SDLMS were higher than the demand volumes for the under the early SDLMS (See Table 6.4). The differences in demand volumes resulted in the difference in the mean and maximum throughputs. To overcome this issue in the analyses the demand volume was categorized into 5 categories as will be elaborated on later in this chapter. Looking at Table 6.4, one can notice that the mean percent car lane changing in zone one is the highest for the dynamic early merge and the lowest for the dynamic late merge. Also looking at the percent truck lane changing in zone 1, the highest mean percent lane changes is for the dynamic early merge and the lowest is for the dynamic late merge. This means that some drivers are obeying the message displayed by the dynamic message boards.

Table 6.4: Descriptive Statistics

MOT Type	Variable	Unit	Mean	Std. Dev.	Min	Max
Conventional MAS	Demand Volume	Veh/hr	911.92	467.4	120	2580
	Throughput	Veh/hr	1064.87	488.58	270	2730
	% TRK	N/A	11.3	10.78	0	50
	% Car lane changes in zone 1	N/A	52.08	28.39	0	100
	% TRK lane changes in zone 1	N/A	60.68	41.59	0	100
Early SDLMS	Demand Volume	Veh/hr	713.17	406.63	120	1530
	Throughput	Veh/hr	763.96	377.49	230	1890
	% TRK	N/A	17.84	19.09	0	74
	% Car lane changes in zone 1	N/A	59.55	30.98	0	100
	% TRK lane changes in zone 1	N/A	66.34	35.43	0	100
Late SDLMS	Demand Volume	Veh/hr	1209.06	577.11	180	3120
	Throughput	Veh/hr	1152.81	596.11	60	2940
	% TRK	N/A	13.84	11.29	0	54
	% Car lane changes in zone 1	N/A	46.35	34.24	0	100
	% TRK lane changes in zone 1	N/A	38.21	37.38	0	100

6.3.1 Correlation Analysis

A correlation analysis between truck percentage, percent truck lane changing in zone1, percent car lane changing in zone 1, and MOT type (MAS, early SDLMS, and late SDLMS) is conducted. For the continuous variables including truck percentage, truck percentage lane changing in zone 1, and passenger car lane changing in zone one, Spearman's rank-order correlation and Pearson's correlation are used. Pearson's correlation is a parametric measure of association which measures both the strength and the direction of the linear relationship. Spearman correlation is nonparametric measure of association based on the ranks of the data values. From Tables 6.5 and 6.6 one can assume no correlation between all continuous variables in question since all correlation coefficient are below 0.2. The correlations are statistically significant with very low coefficient therefore, they are ignored.

Table 6.5: Spearman's Correlation

			TRK %	TRK % Lane Changing in Zone 1	PC % Lane Changing in Zone1
Spearman's rho	TRK %	Correlation Coefficient	1.000	-.188**	-.191**
		Sig. (2-tailed)	.	.000	.000
		N	517	517	517
	TRK % Lane Changing in Zone 1	Correlation Coefficient	-.188**	1.000	.138**
		Sig. (2-tailed)	.000	.	.002
		N	517	517	517
	PC % Lane Change in Zone1	Correlation Coefficient	-.191**	.138**	1.000
		Sig. (2-tailed)	.000	.002	.
		N	517	517	517

**. Correlation is significant at the 0.01 level (2-tailed).

Table 6.6: Pearson's Correlation

		TRK %	TRK % Lane Changing in Zone 1	PC % Lane Changing in Zone1
TRK %	Pearson Correlation	1	-.100*	-.166**
	Sig. (2-tailed)		.023	.000
	N	517	517	517
TRK % Lane Changing in Zone 1	Pearson Correlation	-.100*	1	.146**
	Sig. (2-tailed)	.023		.001
	N	517	517	517
PC % Lane Changing in Zone1	Pearson Correlation	-.166**	.146**	1
	Sig. (2-tailed)	.000	.001	
	N	517	517	517

*. Correlation is significant at the 0.05 level (2-tailed).

To examine the correlation between a categorical variable with more than two levels and a continuous variable, one can compute Friedman's test. Table 6.7 shows that the resulting p-values of all three Friedman's tests are greater than 0.05 indicating no significant correlation between MOT type and the percentage of trucks, the percentage of trucks changing lanes in zone 1, and the percentage of passenger car changing lanes in zone 1.

Table 6.7: Friedman's tests for correlation between MOT type and continuous variables

Test#	Variable	Vs. MOT type	Pr>F
Test 1	TRk%	Vs. MOT type	0.63
Test 2	TRK % lane changing in zone 1	Vs. MOT type	0.11
Test 3	PC % lane changing in zone 1	Vs. MOT type	0.35

Although it seems intuitive for a correlation between MOT type and percentage of lane changing in zone 1 to exist, the results show no significant correlation. This may be due the compliance rate variance. For example during early merge instructions the compliance rate may be low therefore drivers may still merge late. On the other hand, during late merge compliance rate may be low therefore drivers may still merge early. The result of the compliance rate variance in early and late merge may have resulted in no significant difference in the correlation between MOT type and percentage lane changing in zone 1 for passenger cars and trucks. This same trend was encountered in the analysis of the two-to-one work zone lane closure data analyses in Chapter 5.

6.3.2 Evaluating the ratio of throughputs over demand volumes under different volume levels

As mentioned earlier the distributions of the demand volumes were different under all three MOT types. Therefore, comparing mean throughputs without controlling for demand volumes is erroneous. Moreover, comparing the ratios of throughputs over demand volumes without controlling for demand volumes is also incorrect. For instance, demand volumes for the early SDLMS (mean=713.17 veh/hr) were lower than the demand volumes of the late SDLMS (mean=1209.06veh/hr) and if we compare the mean ratios of the early SDLMS (763.96/713.17=1.07) to the mean ratios of the late SDLMS (1152.81/1209.06=0.95) regardless of the demand volume, results would be erroneous. To resolve this issue, demand volumes were split into five categories. Demand volume V1 ranges between 1-500 veh/hr, demand volume V2 ranges between 501-1000veh/hr, demand volume V3 ranges between 1001-1500veh/hr, demand volume V4 ranges between 1501-2000 veh/hr, and demand volume V5 >2000 veh/hr. After splitting demand volumes into five categories the ratios of throughputs over demand volumes were determined.

Five linear regression models (one for each demand volume level) were estimated to determine the effect of the truck percentages in the traffic composition, percent trucks lane changing in zone1, percent cars lane changing in zone 1, and MOT type on the throughput over demand volume of the work zone. Table 6.9 summarizes the parameter estimates and their significance on the ration of throughputs over demand volume under each demand volume level. Looking at the first estimated model in Table 6.9 where the demand volume ranges between 1 and 500 veh/hr., it was found the dynamic late merge displays a significant (*p*-value=0.006) negative effect (parameter estimate= -0.234) on the compared to the MAS system. This indicates that under this range of demand volume (1-500 veh/hr), the MAS resulted in higher throughputs compared to the dynamic late merge system.

Looking at the second estimated model for demand volumes ranging between 501veh/hr and 1000 veh/hr, results showed that the percentage trucks changing lanes in zone one has a significant positive effect on the ratios (parameter estimate = 0.141; *p*-value=0.0001). This indicates that the higher the percentage of trucks changing lane in zone 1 the higher the ratio (i.e. the throughput of the work zone). The same model shows that the dynamic early merge resulted in significantly higher throughputs (parameter estimate = 0.133; *p*-value=0.014) compared to the MAS system.

For demand volume ranging between 1001veh/hr and 1500 veh/hr, the third estimated main effect model showed that the percentage trucks changing lanes in zone 1 (parameter estimate = 0.104; *p*-value=0.018) and the percentage passenger car changing lanes in zone 1 (parameter estimate = 0.141; *p*-value=0.038) have significant positive effect on throughputs. This means that when the truck and passenger cars lane changing in zone 1 increased, the ratio of throughputs over demand volume increased. The same model shows that the dynamic early merge resulted in significantly higher ratios (throughputs over demand volume) compared to the MAS system (parameter estimate = 0.029; *p*-value=0.059). The truck percentage in the traffic composition displays a marginal significance with the ration. In facts, the models shows that the lower the truck percentage in the traffic composition, the higher the ratios of throughputs over demand volume (parameter estimate = -.288; *p*-value=0.09).

For demand volume ranging between 1501veh/hr and 2000 veh/hr, the fourth estimated main effect model (see Table 6.8) showed that the percentage passenger car changing lanes in zone 1 (parameter estimate = 0.166; *p*-value=0.044) have significant positive effect on throughputs. This means that when the passenger cars lane changing in zone 1 increased, the throughput increased. The same model shows that the dynamic early merge (parameter estimate = 0.156; *p*-value=0.002) as well as the dynamic late merge (parameter estimate = 0.204; *p*-value=0.031) resulted in significantly higher ratios compared to the MAS system. This means the dynamic early merge and dynamic late merge resulted in higher throughputs compared to the MAS system under demand volumes ranging between 1501 and 2000 veh/hr.

Table 6.8 Parameter Estimates Under Different demand volumes (dependent variable ratio of throughput over demand volume

0-500 veh/hr

ANOVA AND PARAMETER ESTIMATES			
Parameter	Estimate	Standard Error	Pr > |t|
Intercept	0.880	0.083	<.0001
% Trucks	-0.088	0.149	0.558
% PC Lane Changing in Zone 1	-0.008	0.089	0.925
%TRK Lane Changing in Zone 1	0.032	0.070	0.653
Dynamic Late Merge	-0.234	0.083	0.006
Dynamic Early Merge	0.058	0.055	0.291
MAS	0	.	.
R-Square=0.19			

501-1000 veh/hr

ANOVA AND PARAMETER ESTIMATES			
Parameter	Estimate	Standard Error	Pr > |t|
Intercept	0.627	0.056	<.0001
% Trucks	-0.222	0.161	0.170
% PC Lane Changing in Zone 1	0.071	0.064	0.269
%TRK Lane Changing in Zone 1	0.187	0.048	0.0001
Dynamic Late Merge	0.082	0.050	0.102
Dynamic Early Merge	0.133	0.054	0.014
MAS	0	.	.
R-Square=0.22			

1001-1500 veh/hr

ANOVA AND PARAMETER ESTIMATES			
Parameter	Estimate	Standard Error	Pr > |t|
Intercept	0.652	0.054	<.0001
% Trucks	-0.288	0.168	0.090
% PC Lane Changing in Zone 1	0.141	0.067	0.038
%TRK Lane Changing in Zone 1	0.104	0.044	0.018
Dynamic Late Merge	0.099	0.042	0.187
Dynamic Early Merge	0.029	0.053	0.059
MAS	0.000	.	.
Ta R-Square=0.203			

1501-2000 veh/hr

ANOVA AND PARAMETER ESTIMATES			
Parameter	Estimate	Standard Error	Pr > |t|
Intercept	0.523	0.074	<.0001
% Trucks	0.004	0.292	0.988
% PC Lane Changing in Zone 1	0.166	0.081	0.044
%TRK Lane Changing in Zone 1	0.122	0.072	0.097
Dynamic Late Merge	0.204	0.063	0.002
Dynamic Early Merge	0.156	0.152	0.031
MAS	0.000	.	.
R-Square=0.19			

>2000 veh/hr

ANOVA AND PARAMETER ESTIMATES			
Parameter	Estimate	Standard Error	Pr > |t|
Intercept	0.760	0.176	0.001
% Trucks	-3.068	1.020	0.010
% PC Lane Changing in Zone 1	0.043	0.152	0.782
%TRK Lane Changing in Zone 1	0.569	0.315	0.094
Dynamic Late Merge	0.203	0.176	0.271
Dynamic Early Merge	0.000	.	.
MAS	0.000	.	.
R-Square=0.23			

For volumes greater than 2000 veh/hr. , the fifth estimated model shows that the dynamic late merge has no significant effect on the throughput over demand volume compared to the MAS system. The data sample size for the dynamic early merge was not large enough therefore; the parameter could not be estimated. This model shows that when the percentage trucks in the traffic increases, the ratio of throughputs over demand volume decreases significantly (parameter estimate = 3.068; *p*-value=0.01).

Table 6.9 summarizes the results from the regression analyses. The red color means lower ratio than MAS, the color yellow means higher but not significant, the color green means higher and significant, the blue color means unknown. To compare dynamic early and late merge we used the letters E and L. As shown by the Table 6.10, the late SDLMS was better than the early SDLMS for V4.

Table 6.9: Comparison of Early SDLMS, Late SDLMS and MAS

Late and Early Compared to MAS					
	V1	**V2**	**V3**	**V4**	**V5**
Dynamic Ealry Merge	🟨	🟩	🟩	🟩	🟦
Dynamic Late Merge	🟥	🟨	🟨	🟩 L	🟨

6.4 Conclusions

The temporal speed fluctuation at the location of the RTMS of the work zone under the control (MAS) and test MOT plans (early and late SDLMS) were compared. The mean speed fluctuation in the closed lane was the highest under the MAS system for all demand volumes. The dynamic late merge and the dynamic early merge have lower speed fluctuations in the closed lane under all demand volumes compared to the MAS system. Comparing the dynamic early merge and the dynamic late merge mean speed fluctuations in the closed lane, results showed that the mean speed fluctuation for the early merge are lower than those of the late merge under demand all demand volumes. However, the difference in the mean speed fluctuation is only statistically significant under demand volume ranging between 1 and 500 veh/hr. Results showed that the speed fluctuations in the middle lane are the highest for the MAS system compared to dynamic early merge and dynamic late merge under all demand volumes. However, results showed that the mean speed fluctuations under the MAS are significantly higher than the mean speed fluctuations under the dynamic late merge only for volumes for volumes greater than 1500 veh/hr (and marginally at volumes between 1001 and 1500 veh/hr). The mean speed fluctuations under the MAS are significantly higher than the mean speed fluctuations under the dynamic early merge system for volumes ranging between 501 and 1500 veh/hr. Comparing the mean speed fluctuations under the dynamic early merge and the dynamic late merge, it was found that the mean speed fluctuations are lower for the dynamic early merge. However, there was no significant difference between the speed fluctuations in the middle lane.

Looking at the speed fluctuations in the shoulder lane, the mean speed fluctuations are the highest under the MAS system compared to the dynamic early merge and the dynamic late merge under all volumes. The mean speed fluctuations for the MAS system is significantly higher than the mean speed fluctuation for dynamic early and dynamic late merge for volumes under 1000 veh/hr. Moreover, there exist a marginal significance indicating that the mean speed fluctuation for the late merge is lower than the mean speed fluctuation for the MAS system for volumes ranging 1001veh/hr to 2000 veh/hr. Comparing the mean speed fluctuations between the dynamic early and dynamic late merge, it was noted that the means speed fluctuations are lower for the dynamic late merge under volumes higher than 500 veh/hr. However, it was shown that the mean speed fluctuation for the dynamic late merge is significantly lower than the mean speed fluctuation for the dynamic early merge for demand volumes ranging between 1500 veh/hr and 2000 veh/hr.

The ratio of the throughput over demand volume was taken as the operational MOE. Results showed that the Dynamic early merge performs significantly better than the regular MAS under demand volume ranging between 500 veh/hr and 2000 veh/hr. Results also showed that the dynamic late merge perform better than the MAS under volumes ranging between 1500 veh/hr and 2000 veh/hr and significantly poorer than the MAS under low volumes. Therefore, the late SDLMS is not recommended for implementation under low volumes. Results also showed that the late SDLMS performs better than the early SDLMS under higher volume (ranging between 1501 veh/hr to 2000 veh/hr).

Combining safety and operational measures discussed above, some recommendations can be drawn regarding the implementation of the early SDLMS and late SDLMS:

- For volumes ranging between 0 and 500 veh/hr, it was found that the dynamic lane merge performs better than the dynamic late merge and MAS. The dynamic late merge shows the poorest performance under this range of volume.

- For volumes ranging between 501 veh/hr and 1000 veh/hr the dynamic early merge exhibits the best performance compared to the dynamic late merge and the MAS system.

- For volumes ranging between 1001 veh/hr and 1500 veh/hr the dynamic late merge exhibits the highest performance compared to the dynamic early merge and the MAS system.

- For volumes larger than 1501 veh/hr and 2000 veh/hr, dynamic early merging data was not available. However, the dynamic lat emerging showed better performance than the MAS system.

CHAPTER 7 TWO-TO-ONE WORK ZONE LANE CLOSURE SIMULATION IN VISSIM

As mentioned earlier, one of the objectives of this research is to provide guidelines on the implementation of the early and late SDLMS. The data sample size from the first site, consisting of a two-to-one lane closure configuration, was limited to certain traffic demand level and to a certain motorists' adherence level to lane management instructions. Therefore, a VISSIM simulation study is conducted to determine the safety and operational effectiveness of the early SDLMS and late SDLMS under different traffic demand volumes and different drivers' compliance rates to the messages displayed by the systems.

7.1 Available tools to evaluate safety and mobility of drivers at work zones

There exist a wide range of tools to evaluate the safety and mobility of drivers at work zone lane closures. The HCM 2000 presents a methodology for estimating the capacity of work zones. This methodology suggests using a base capacity value and applying adjustment factors for intensity of work activity, effect of heavy vehicles, and presence of ramps in the vicinity of the work area. The proposed base capacity is 1,600 pcphpl which is obtained from Texas work zone studies. HCM 2000 does not provide any approach for estimating queue lengths.

QUEWZ, Queue and User Cost Evaluation of Work Zones (QUEWZ) is a DOS-based tool developed by the Texas Transportation Institute. QUEWZ uses the HCM 2000 equations to calculate the capacity of the work zone. For Calculation of queue length it uses the methodology from HCM 1994 (Krammes et al, 1993).

QuickZone is an analytical tool that can be used for estimating the traffic impacts of work zones. It was developed by Mitretek Systems for the FHWA that allows flexible and fast estimation of work zone traffic impacts. QuickZone is an open-source that enables DOT to customize the latter to be applicable in their specific work zones. For instance, MD-QuickZone is the QuickZone customized for Maryland's work zones (MD-QuickZone). QuickZone requires more time and efforts compared to QUEWZ and compares expected travel demand with proposed capacity by facility on an hourly basis to estimate delay and mainline queue length (Quickzone, 2001).

DELAY Enhanced 1.2 is an application developed by Martin Knopp from FHWA to quickly estimate the traffic impacts of incidents. This model could be applied to short term work zone lane closures. The program has a good graphical user interface, which makes it easier for the user to input the data and visualize the queue length (FHWA).

Microscopic Simulation models such as VISSIM, CORSIM, SimTraffic, etc. can be utilized to assess traffic impacts at work zones. Heaslip et al., (2009) used CORSIM to estimate work zone freeway capacity. Chatterjee et al. (2009) replicated work zones in VISSIM and recommended driving behavior parameter values for use in VISSIM. Beacher et al. (2004), evaluated the late merge work zone traffic control strategy using

VISSIM. Park (2002) developed and studied variable speed limit logic in work zones using VISSIM.

The above listed tools have the capability to assess the capacity of work zones but most do not offer the flexibility of adjusting for the lane management strategy suggested in this study. VISSIM is microscopic stochastic simulation software that enables us of creating specific scenarios via vehicle actuated programming (VAP). A program reflecting our algorithm (of dynamic lane merging) can be written in Visual Basic to communicate with VISSIM in real-time. The next sections of this chapter introduce VISSIM and elaborate on the methodology followed in simulating the dynamic lane merging in VISSIM.

7.2 Simulation in VISSIM

VISSIM is a microscopic, time step and behavior based simulation model. VISSIM is a commercially available traffic simulation package developed by PTV AG, Karlsruhe, Germany, and distributed in the United States by PTV America, Inc. The software can analyze traffic and transit operations under user defined conditions, such as lane configuration, traffic composition, traffic signals, transit stops, etc., thus making it a useful tool for the evaluation of various alternatives based on transportation engineering and planning measures of effectiveness (VISSIM User manual, 2007).

According to the VISSIM User Manual the accuracy of a traffic simulation model is mainly dependent on the quality of the vehicle modeling, e.g. the methodology of moving vehicles through the network (VISSIM User manual, 2007). In contrast to less complex models using constant speeds and deterministic car following logic, VISSIM uses the psycho-physical driver behavior model developed by Wiedemann in 1974. The basic concept of this model is that the driver of a faster moving vehicle starts to decelerate as he reaches his individual perception threshold to a slower moving vehicle. Since this driver cannot exactly determine the speed of that adjacent vehicle, his speed will fall below that vehicle's speed until he starts to slightly accelerate again after reaching another perception threshold. This results in an iterative process of each vehicle's acceleration and deceleration.

VISSIM simulates the traffic flow by moving "driver-vehicle-units" through a network. Every driver has a specific behavior characteristics assigned to their specific vehicle type. As a consequence, the driving behavior corresponds to the technical capabilities of his vehicle. Attributes characterizing each driver-vehicle-unit can be categorized into three categories, they are: technical specifications of the vehicle, behavior of driver-vehicle-unit, and independence of driver-vehicle-units. More specifically each category includes parameters such as:

- Technical specifications of the vehicle
    - Length
    - Maximum speed
    - Potential acceleration
    - Actual position in the network
    - Actual speed and acceleration
- Behavior of driver-vehicle-unit

    - Psycho-physical sensitivity thresholds of the driver (also known as their ability to estimate thresholds and level of aggressiveness)
    - Memory of driver
    - Acceleration based on current speed and driver's desired speed
- Interdependence of driver-vehicle-units
    - Reference to leading and following vehicles on own and adjacent travel lanes
    - Reference to current link and next intersection

Not every technical specification that VISSIM employs are applicable in work zone lane closures operations, therefore to reduce model setup and calibration efforts it is important that key specifications be identified as either those that have an impact, or those that do not have an impact on toll plaza modeling. The modeling elements that have a direct effect on toll plaza operations will receive special attention in both the setup and calibration process, while others may not.

7.3 Development of the VISSIM model

The process of coding VISSIM consists of a systematic series of programming processes that must be addressed to duplicate an actual traffic situation. Development of a successful model was broken down into three categories; physical design of the work zone, vehicle characteristics, and driver behaviors. The methodology and process for developing the first two categories is that model characteristics are to remain fixed for all designs while the driver behavior characteristics are reserved as the parameters used for model calibration.

7.3.1 System Layout

To build roadways in VISSIM a series of links and connectors were used to represent the actual geometry of the work zone. The figure below shows the MOT plans for the 2-to-1 lane closure and the corresponding resulting nodes and roadway segments in VISSIM. It should be noted that the MOT plan used in the field was first scaled to match the dimensions embedded into the VISSIM elements. The roadway is traced on top of the image with links and connectors. Figure 7.3.1 shows 5 links and 4 nodes. The first node of the Figure represents the first work zone PCMS. The second node represents the location of the additional PCMS where merging information is provided to drivers. Node 3 represents the lane closure start (1 lane open). Node 4 represents the lane closure end (two lanes open).

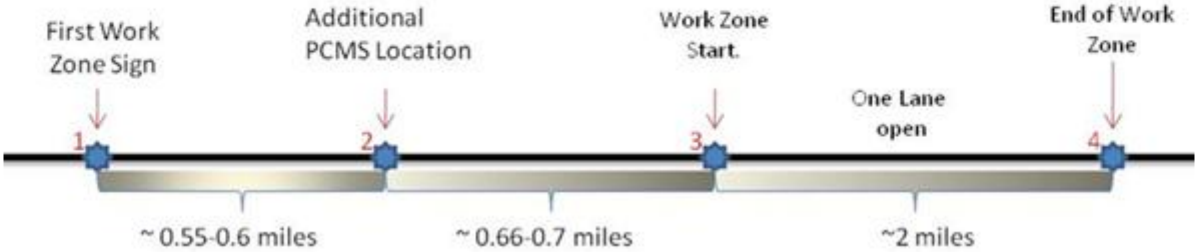


Figure 7.3.1: MOT plan Replication in VISSIM

7.3.2 Dynamic Lane Merging in VISSIM

Recalling the SDLMS algorithm applied in the field, first the RTMS captures the average speed of vehicles over two-minute time intervals and the built-in algorithm checks if the speed threshold is reached. If the speed threshold is reached, the additional PCMS displays the necessary messages. The PCMS keeps displaying the messages until another speed threshold is reached. When the early SDLMS message is displayed, drivers merge to the open lane. When the late SDLMS message is displayed, drivers stay in their lane until the taper. To mimic the early SDLMS in VISSIM, dynamic decision routing were designated. Drivers either follow a decision routing designated to merge early (when speed drops below threshold) or follow a random merging (when speed remains above speed threshold). To imitate the late SDLMS, dynamic decision routing was also designated. Drivers either follow a decision routing designated to stay in their lane until the taper (when speed drops below threshold) or follow a random merging (when speed above speed threshold).

As mentioned above, the routing decision is dynamic since it reacts based on average speeds over two time intervals. Two loop detectors are placed (in VISSIM) at the same location of the RTMS. The loop detectors in VISSIM capture individual vehicles speed. These loop detectors can communicate with signal controllers and can only interact with traffic signals. Since loop detectors cannot directly communicate with the routing decision, Vehicle Actuated Programming (VAP) is used. VAP "is an optional add-on to VISSIM for the simulation of programmable, phase or stage based, traffic actuated signal controls. The control logic is coded in a txt file format and the VAP interpret the control logic commands and creates commands for the VISSIM network. At the same time various detectors variables reflecting the current traffic situation are retrieved from the simulation and processed in the logic" (VISSIM manual). The algorithm shown in Figure 7.3.2 is coded in VAP and the control logic alternate between partial routes 1 and 2 (i.e. MAS or dynamic early merging). The following sections will elaborate on the details of the routing decisions.

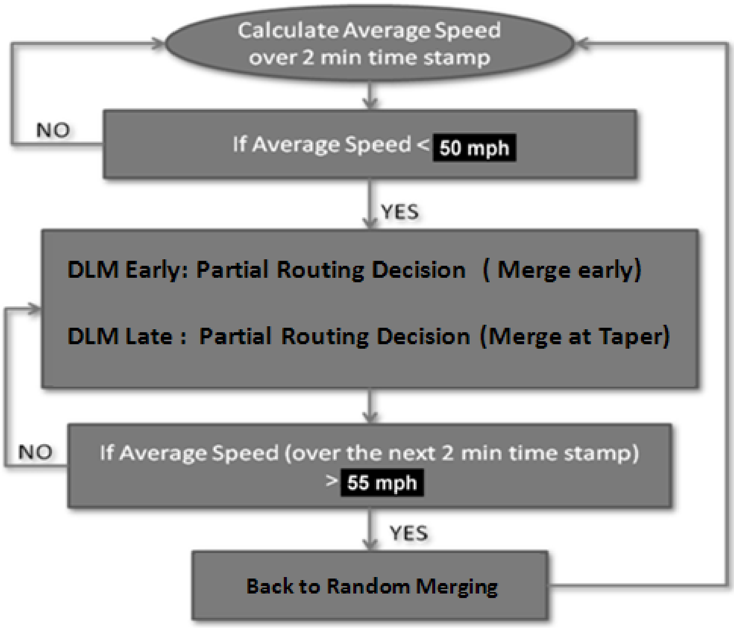


Figure 7.3.2: VAP Logic in VISSIM

7.3.2.1 Static routing decisions

As defined in the VISSIM user manual a static route "routes vehicles from a start point to an end point using a static percentage for each destination". Therefore, static routing decisions are created to ensure that all vehicles entering the work zone (from node 1) exit the work zone (from node 4) (See Figure 7.3.2.1).

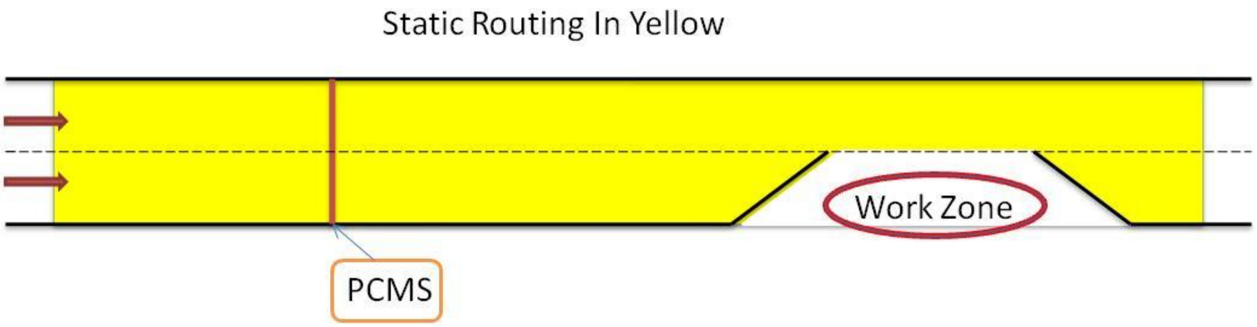
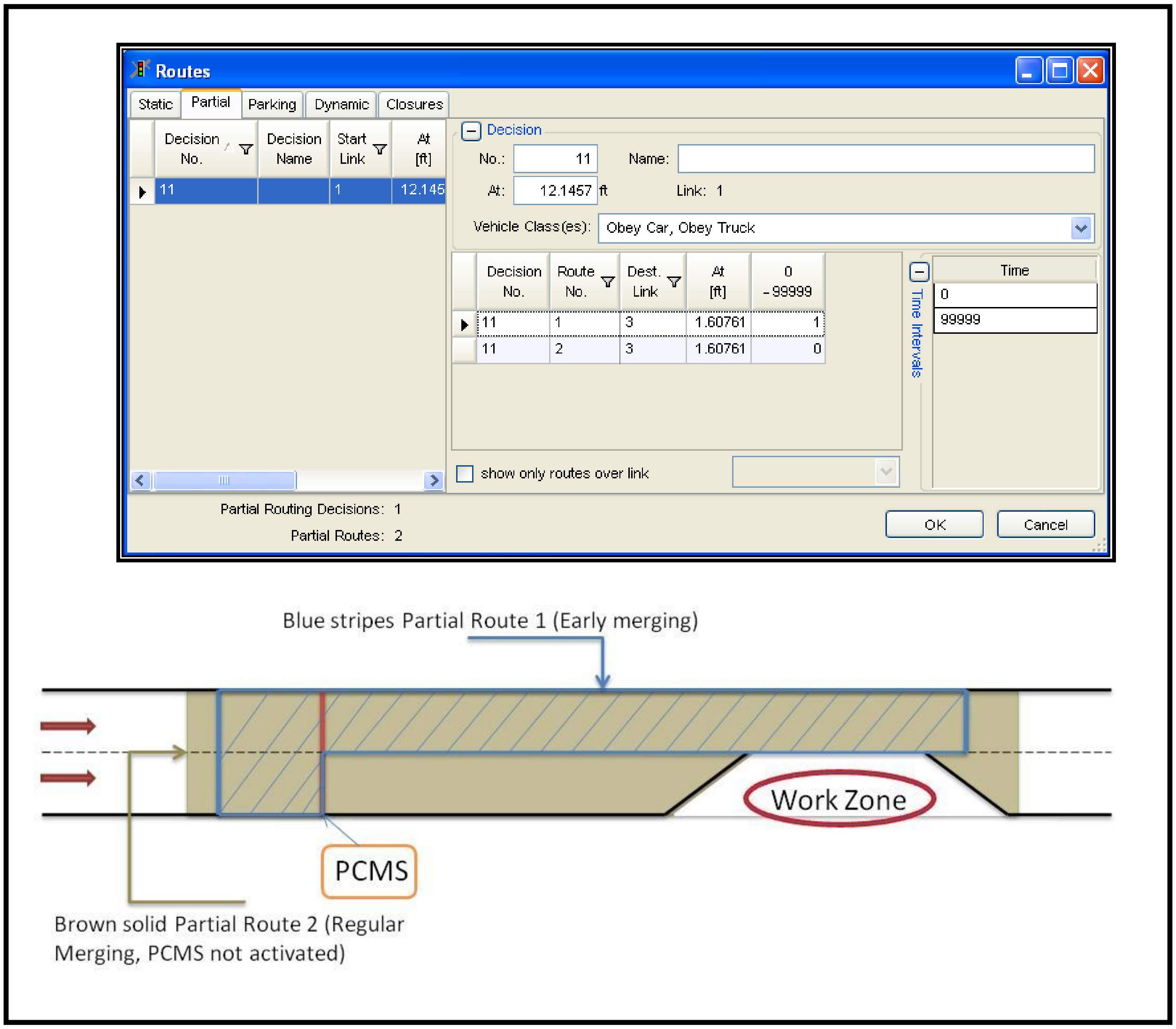


Figure 7.3.2.1: Static Routing Decision

7.3.2.2 Partial routing decisions

The partial route is defined by the VISSIM user manual “defines a section of one or more static routes where vehicles should be redistributed according to the routes and percentages defined by the partial routes. After leaving the partial route, vehicles continue to travel on their original route”. Partial routes are used to create the early and late merge at the work zone. For the dynamic early merge, one partial routing decision with two routes; route 1 and route 2 are created. In routes 1 and 2 fraction of vehicles going on each route can be selected. For instance, as shown by the VISSIM input in Figure 7.3.2.2, route 1 is used by 100% of the vehicles and route 2 by none (0%). Since route 1 means that the vehicles are using the open lane (See illustration in Figure 7.3.2.2), this means that early merging is activated and vehicles are merging at the location of the additional PCMS. Therefore, when the early merge is deactivated the fractions of vehicles on route 1 and 2 change from 1 to 0 and 0 to 1 in that order. The alternation between route 1 and 2 is based on the speed threshold (50mph as selected in the field).



Decision No.	Decision Name	Start Link	At [ft]
11		1	12.145

No.: 11 Name:

At: 12.1457 ft Link: 1

Vehicle Class(es): Obey Car, Obey Truck

Decision No.	Route No.	Dest. Link	At [ft]	0 - 99999
11	1	3	1.60761	1
11	2	3	1.60761	0

Time
0
99999

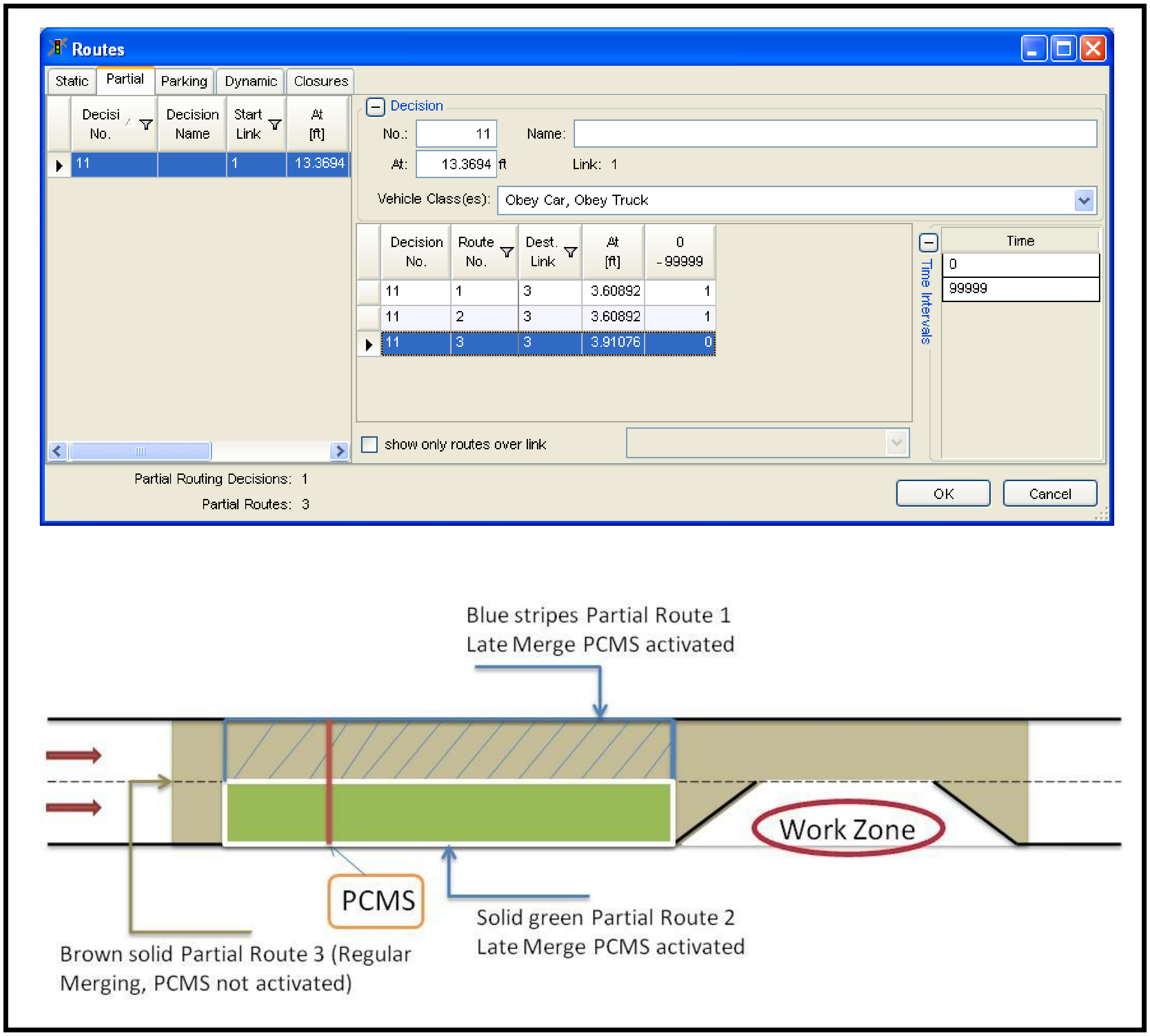
☐ show only routes over link

Partial Routing Decisions: 1

Partial Routes: 2

Figure 7.3.2.2: Early SDLMS Partial Routing Decision

The same concept was followed for the late SDLMS. However, in this case one partial routing decision includes three partial routes. Route 1 is designated for drivers in the open lane, route 2 is designated for drivers in the closed lane and route 3 is designated for all drivers (in both lanes). For instance, as shown by the VISSIM input in Figure 7.3.2.3, route 1 and route 2 have fractions of vehicles of 1 and 1. This means that 100% of the vehicles entering the routing decision in the open lane and the closed lane stay in their lane and follow the routing decision without making lane changes until the taper. Route 3 has a fraction of 0 which means that there are no drivers that are using this route. Figure 7.3.2.3 shows the routes when the late SDLMS is activated. When the late SDLMS is deactivated based on the speed threshold, routes 1 and 2 will have fractions of 0 and 0 and route 3 will have a fraction of 1 (controlled by the VAP). The illustration in Figure 7.3.2.3 shows the three different routes.



Decision No.	Decision Name	Start Link	At [ft]
11		1	13.3694

No.: 11
Name:
At: 13.3694 ft
Link: 1
Vehicle Class(es): Obey Car, Obey Truck

Decision No.	Route No.	Dest. Link	At [ft]	0 - 99999
11	1	3	3.60892	1
11	2	3	3.60892	1
11	3	3	3.91076	0

Time
0
99999

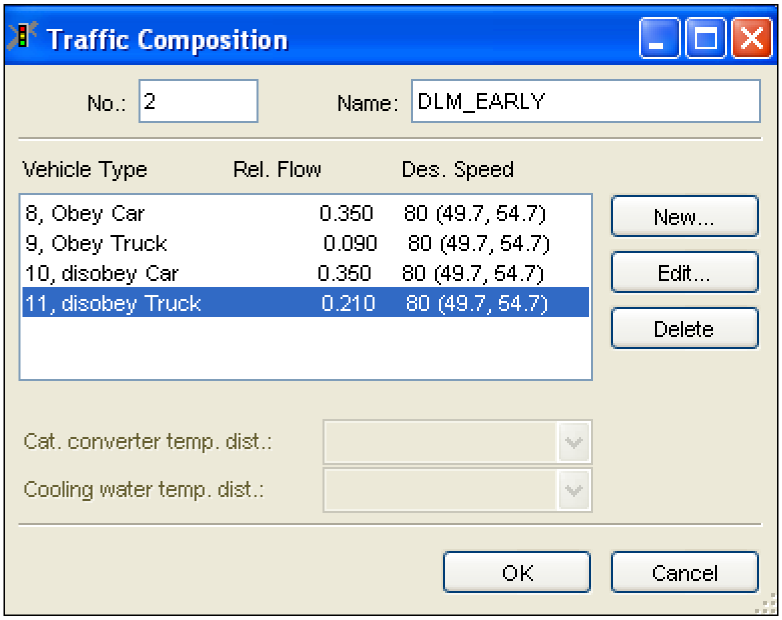
☐ show only routes over link

Partial Routing Decisions: 1
Partial Routes: 3

Figure 7.3.2.3: Late SDLMS Partial Routing Decision

7.3.2.3 Vehicle Classification and Drivers' Compliance

An important factor in the SDLMS is the driver's compliance rate to the messages displayed by the PCMS. Therefore, it is necessary for this simulation model to control for drivers compliance rate. For instance, in the early SDLMS, the average speed collected by the loop detectors may be less than 50mph (PCMS activated) and the routing decision designates route 2 (early merging) then all vehicles follow route 2. In this case compliance rate is ignored. The partial routing decision can control specific vehicle classes. One can create different vehicle classes, some controlled by the partial routing decision and some not controlled by the partial routing decision. Four vehicle classes are created; Obey_car, Obey_TRK, Disobey_Car, Disobey TRK. Obey_car and Obey_TRK vehicle classes represent the vehicles that are controlled by the partial routing decision therefore complying with the PCMS messages. The Disobey_car and Disobey_TRK are not controlled by partial routing decision constituting the non complying vehicles. The traffic composition (entering from node 1in Figure 7.3.1) is set to contain all 4 vehicle classes. The traffic composition was changed manually to reflect different levels of compliance. For instance, Figure 7.3.2.4 shows an example of a traffic composition of 30% trucks and 70% passenger cars. The compliance rate of passenger cars is 50% and the compliance rate for trucks is 30%.



No.: 2

Name: DLM_EARLY

Vehicle Type	Rel. Flow	Des. Speed
8, Obey Car	0.350	80 (49.7, 54.7)
9, Obey Truck	0.090	80 (49.7, 54.7)
10, disobey Car	0.350	80 (49.7, 54.7)
11, disobey Truck	0.210	80 (49.7, 54.7)

Figure 7.3.2.4: Late and Early SDLMS Traffic Composition

7.4 Calibration and validation of the VISSIM model

Calibration and validation of the work zone simulated model are needed to assure that this model generates representative numerical results that replicate traffic operations in the actual work zone in the field. Simulation models calibration is an iterative procedure to fine tune the simulated model's parameters and settings to achieve acceptable numerical results. The validation part is an analytical process that verifies whether the simulated model parameter fine tuning process truly represents actual traffic operations.

7.4.1 Driving behavior

After completing the original work zone model in VISSIM, this model undergoes an initial evaluation to determine its performance level. If the model can simulate the observed conditions within acceptable errors, the model is considered calibrated. Otherwise, driving behavior parameter sets are to be adjusted. The driving behavior presides over the range of parameters and rules of the car following and lane changing models. VISSIM is based on Weidemann's "psycho-physical" car-following model and lane changing model.

The Wiedemann 99 car following model (W-99), has 10 user defined driving behavior parameters. Each parameter is briefly explained below (VISSIM User Manual):

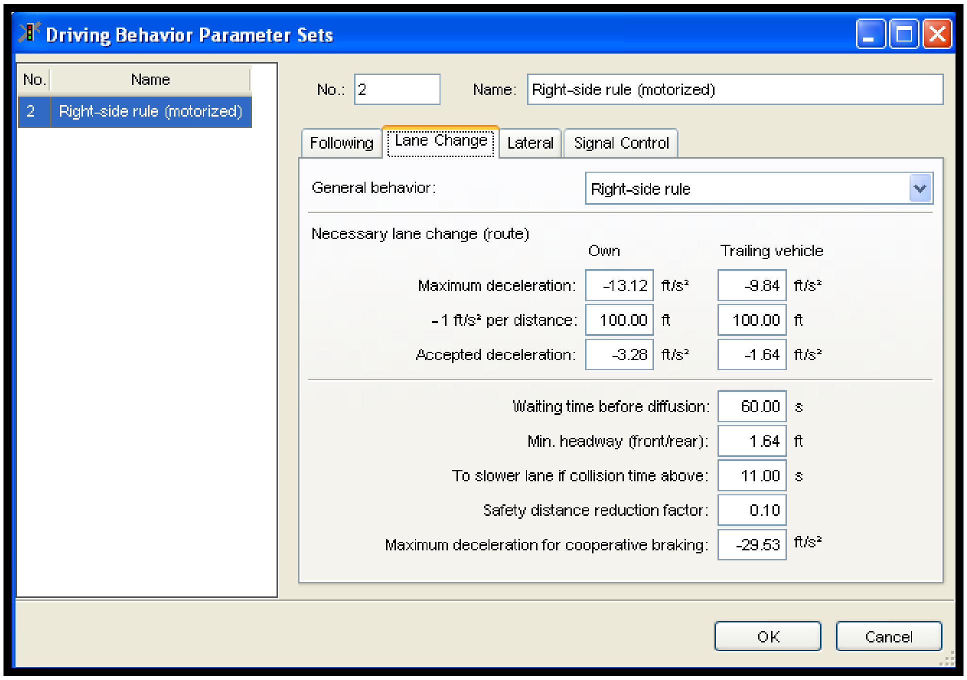
- CC0 (Standstill distance) defines the desired distance between stopped cars. It has no variation.
- CC1 (Headway time) is the time (in s) that a driver wants to keep. The higher the value, the more cautious the driver is. Thus, at a given speed the safety distance dx_safe is computed to: dx_safe = CC0 + CC1 * v. The safety distance is defined in the model as the minimum distance a driver will keep while following another car. In the case of high volumes this distance becomes the value with the strongest influence on capacity.
- CC2 ('Following' variation) restricts the longitudinal oscillation or how much more distance than the desired safety distance a driver allows before he intentionally moves closer to the car in front. If this value is set to e.g. 10 ft, the following process results in distances between dx_safe and dx_safe + 10ft.
- CC3 (Threshold for entering 'Following') controls the start of the deceleration process or when a driver recognizes a preceding slower vehicle. It defines how many seconds before reaching the safety distance the driver starts to decelerate.

- CC4 and CC5 ('Following' thresholds) control the speed differences during the ' Following' state. Smaller values result in a more sensitive reaction of drivers to accelerations or decelerations of the preceding car, i.e. the vehicles are more tightly coupled. CC4 is used for negative and CC5 for positive speed differences.
- CC6 (Speed dependency of oscillation): Influence of distance on speed oscillation while in following process.
- CC7 (Oscillation acceleration): Actual acceleration during the oscillation process.
- CC8 (Standstill acceleration): Desired acceleration when starting from standstill
- CC9 (Acceleration at 50 mph): Desired acceleration at 50mph

The lane changing model in VISSIM is based on the driver response to the perception of the surrounding traffic. The lane changing model is based on the fact the drivers will only change lanes if the available gap is smaller than the critical gap. Lane changing has two categories based on this model: discretionary lane change; and necessary lane change (e.g. work zone lane closure). Readers may refer to Widemann and Reiter (1992) for detailed information about the lane changing model. In case of necessary lane change, the driving behavior parameters contain the maximum acceptable deceleration for the vehicle and the trailing vehicle on the new lane, depending on the distance to the emergency stop position of the next connector of the route (VISSIM manual).

As shown in Figure 7.4.1 below, necessary lane changing in VISSIM is governed by parameters such as accepted and maximum deceleration rates, safety distance reduction factor (SRF) etc. These parameters explain drivers' aggressiveness in accepting/rejecting gaps in the adjacent lane.

- *The waiting time before diffusion* defines the maximum amount of time a vehicle can wait at the emergency stop position waiting for a gap to change lanes in order to stay on its route. When this time is reached the vehicle will be taken out of the network (diffusion).
- *Min. Headway* (front/rear) defines the minimum distance to the vehicle in the front that must be available for a lane change in standstill position.
- *Safety distance reduction factor:* During lane changes the reduction factor is regarded, which takes effect for (1) the safety distance of the trailing vehicle in the new lane for the decision whether to change lane or not (2) the safety distance during a lane change (3) the distance to the leading (slower) lane changing vehicle. During any lane change, the resulting shorter safety distance is calculated as follows: original safety distance x reduction factor. The default factor of 0.6 reduces the safety distance by 40%. After the lane change, the original safety distance is regarded again.



Driving Behavior Parameter Sets

No.	Name
2	Right-side rule (motorized)

No.: 2

Name: Right-side rule (motorized)

General behavior: Right-side rule

Necessary lane change (route)

	Own		Trailing vehicle	
Maximum deceleration:	-13.12	ft/s²	-9.84	ft/s²
-1 ft/s² per distance:	100.00	ft	100.00	ft
Accepted deceleration:	-3.28	ft/s²	-1.64	ft/s²

Waiting time before diffusion:	60.00	s
Min. headway (front/rear):	1.64	ft
To slower lane if collision time above:	11.00	s
Safety distance reduction factor:	0.10	
Maximum deceleration for cooperative braking:	-29.53	ft/s²

Figure7.4.1: Lane Changing Model Driving behavior parameter Set

- *Maximum deceleration for cooperative braking:* defines the maximum deceleration the vehicle would use in case of cooperative braking thus allowing a lane changing vehicle to change into its own lane. Cooperative braking uses (1) up to 50% of the desired deceleration (cf. section 5.1) until the leading vehicle starts changing lane (2) between 50% of the desired deceleration and this user-defined *Maximum deceleration*, since a lane changing leading vehicle will not expect an extremely high deceleration of the trailing vehicle.

7.4.2 VISSIM Calibration Steps

The calibration process in VISSIM was divided into several steps. First, travel time through the work zone was selected as the index of comparison. Second, the required number of simulation runs was determined. Third, an initial evaluation was conducted with the VISSIM's driving behavior's default parameters. If the selected measure of effectiveness is different in simulated and real conditions, the following step would be necessary. Fourth, an examination of the key parameters was conducted and calibration parameters were determined. Multiple runs with different values of the key parameters were run by trial and error until the calibration is completed. Fifth, for the model validation, the work zone throughput (different dataset) was used to verify the homogeneity between the real and simulated environment.

7.4.2.1 Number of Required Simulation Runs

VISSIM is stochastic simulation model; therefore one should determine the required simulation repetitions to prove statistical significance. The random individual vehicle properties are assigned based on the random seed number used for each simulation run. Due to each run's variance, multiple repetitions of the same model with different seed numbers were required to estimate the mean value with a certain level of confidence that the true mean falls within a target interval (Traffic analysis toolbox). Since prior simulation variation data was not available, preliminary simulations were run and the following equation was used to determine the minimum required number of runs.

$$CI_{(1-\alpha)\%} = 2 \times t_{(1-\alpha),N-1} \frac{S}{\sqrt{N}}$$

Where:

CI(1-α)% = (1-α)% confidence interval for the true mean, where alpha equals the probability of the true mean not lying within the confidence interval

t(1-α/2),N-1 = Student's t-statistic for the probability of a two-sided error summing to alpha with N-1 degrees of freedom, where N equals the number of repetitions.

S = Standard deviation of the model results

Table 7.4.2.1: Preliminary Simulation Runs and the Resulting Travel Time

Run	Seed#	Travel Time (sec)
1	1	305.3
2	201	308.7
3	401	305.9
4	601	305.8
5	801	305.4
6	1001	304.9
7	1201	306.3
8	1401	306.1
9	1601	306.4
10	1801	308.9
11	2001	305.4
12	2201	305.9
13	2401	308.4
14	2601	307.7
15	2801	304.3
16	3001	305.8
17	3201	307.7
18	3401	306.6
19	3601	306.2
20	3801	309.6
Average T.T.:		**306.57**

The Table above shows the preliminary simulation runs and the resulting travel time in each run. The following information is now available to determine the required number of runs:

Initial number of runs = 20

Level of confidence = 95%

α = 1-0.95 = 0.05

t(1-α/2),N-1 = t(1-0.05/2),20-1 = 2.093

X' =306.57

S=1.449

To determine the number of runs that satisfies the desired confidence interval, the target interval was calculated to be 5% of the mean value. Therefore, a confidence interval of 5% of average which is 15.33 seconds was the target range to obtain from the above equation.

Table 7.4.2.2: Number of Runs Required

Number of Runs	T-statistics	Confidence Interval
2	12.706	26.04
3	4.303	7.20
4	3.182	4.61
5	2.776	3.60
6	2.571	3.04

From the above Table, it was determined that a minimum of 3 runs (C.I. $_{N=3}$ <15.33) is required with the 95% confidence interval. It was decided that 10 runs (called replications later on) are to be conducted.

7.4.2.2 Initial Evaluation of the Network

In this step of the calibration process, the simulation model is run with VISSIM’s driving behavior default values shown in Tables 7.4.2.2 and 7.4.2.3. In order to determine whether the default driving behavior parameter set provides acceptable travel time values, ten runs with different seed number were executed. Average travel time through the work zone was recorded in VISSIM and compared to the field observed travel time.

Table7.4.2.2: Default Car Following Driving Behavior Set

CC0	Standstill distance	4.92 ft
CC1	Headway Time	1.20 s
CC2	Following Variation	13.12 ft
CC3	Threshold for Entering ‘Following’	-8
CC4	Negative ‘Following Threshold’	-0.35
CC5	Positive ‘Following Threshold’	0.35
CC6	Speed Dependency of Oscillation	11.44
CC7	Oscillation Acceleration	0.82 ft/sec2
CC8	Standstill Acceleration	11.48 ft/sec2
CC9	Acceleration at 50mph	4.92 ft/sec2

Table7.4.2.3: Default Lane Changing Driving Behavior Parameter Set

	Own		**Trailing**
Maximum deceleration	-13.12 ft/s^2		-9.48 ft/s^2
(-1 ft/sec2) per distance	100 ft		100ft
Accepted Deceleration	-3.28 ft/s^2		-1.64 ft/s^2
Waiting Time Before Diffusion		60 s	
Min. Headway (front/rear)		1.64 ft	
To Slower Lane if Collision Time Above		11.00 s	
Safety Reduction Factor		0.1	
Maximum Deceleration for Cooperative Braking		-29 ft/s^2	

Ten simulation runs with different seed numbers were conducted. The Table below shows the average simulated and average field observed travel times. As shown by Table 7.4.2.5, the mean relative percent error is about 4.04% which is lower than the 5% threshold. A t-test was conducted to compare those means and the resulting *p*-value (0.350) demonstrated no significant difference between the simulated and field observed travel times. Although the initial evaluation run shows no need for calibration, a calibration process was conducted to enhance the errors.

7.4.2.3 Driver Behavior Parameter Selection and Calibration

Before tackling the calibration process, a literature review was conducted to evaluate previous freeway and work zone simulation calibration and validation methods. Park et al. (2000) developed a calibration tool for stochastic simulation models (VISSIM and CORSIM) and conducted a case study on a work zone model calibration. In their case study, Park et al. calibrated all parameters of the car following model ignoring the driving behavior parameters of the lane changing model. The optimal calibration parameter set was determined using Genetic Algorithms. At each generation of parameter sets, these parameters were set in VISSIM (and CORSIM) and the resulting MOE was evaluated. This process was repeated until the algorithm converged. Gomes at al. (2004) calibrated a 15 mile freeway section in the VISSIM model. Three driving behavior parameters CC0, CC1, and CC4/CC5 pair were manually selected based on visual interpretation of the results. Lownes and Machemehl (2006) evaluated the Weidmann 99 car following parameters' effect on Freeway simulated capacity in the VISSIM model. Results showed that CC0, CC1, CC2, and lane change distance had a statistically significant impact on the capacity values. Chitturi and Benekohal (2008) state that CC0 and CC1 are the only parameters that impact freeway simulated capacities. Chatterjee et al. (2009) calibrated a simulated freeway work zone in VISSIM. The selected parameters were CC1, CC2 from the car following model. Moreover, Chatterjee at al. were the first to select the Safety distance Reduction Factor (SRF) from the lane changing model as a calibration parameter. SRF as defined earlier reflects the aggressiveness of the drivers when changing lanes.

Previous literature showed that CC0, CC1, CC2, CC4/CC5, and SRF are candidate parameters for the work zone model calibration. Chatterjee et al. (2009) argued that between parameters CC0 and CC1 determining the safety distance d_{xsafe}=CC0+CC1*v (that in turn determines capacity) only CC1 affects the safety distance significantly. According to the same study, it was concluded from visual interpretation that CC4/CC5 pair less than 3 resulted in unstable car following process and values higher than 3.0 did not result in variation of the MOE. After examining previous driving behavior calibration parameter selection, it was decided that CC0 and C4/C5 pair be dropped. The selected parameters for this calibration process are CC1, CC2, and SRF. Table 7.4.2.4 shows the ranges of these parameters.

Table 7.4.2.4: Ranges of Driving Behavior Parameters

Parameters	Minimum	Maximum
CC1	0.9s	1.8s
CC2	10ft	55ft
SRF	0.1	0.55

Different combinations of these parameters were created. CC1 was incremented by 0.1 seconds, CC2 by 5 ft, and SRF by 0.05. CC1, CC2, and SRF resulted in 10 intervals each. Therefore 10^3= 1,000 combinations of these parameters are possible. To minimize the 1,000 possible combinations, a trial and error procedure was followed in this calibration process. For each run in VISSIM, 10 iterations with different seed numbers were completed. The Table 7.4.2.5 summarizes significant runs completed in VISSIM. In run, 2 the headway time (CC1) was increased to 1.5 seconds, the following variation (CC2) was increased to 50ft, and the safety reduction factor (SRF) was increased to 0.5. The resulting simulated mean travel time (253.63 seconds) was significantly larger than the field measured travel time (*P*-value=0.001; error~10%). In this case VISSIM was overestimating travel time. For run 3, CC1 was kept constant, the following variation (CC2) was decreased to 40ft, and the SRF was decreased to 0.45.

Table 7.4.2.5: Dynamic Early Merging Calibration Process

Base Driving Behavior Paramter Set				*Run Number											
Car Following Model Default Parameter Values				Default	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9	Run 10	Run11	Run 12
CC0	Standstill distance	4.92 ft		4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92
CC1	Headway Time	1.20 s		1.2	1.50	1.50	1.50	1.50	1.25	1.10	1.10	1.00	0.50	0.50	0.50
CC2	Following Variation	13.12 ft		13.12	50.00	40.00	35.00	35.00	35.00	20.00	10.00	10.00	10.00	10.00	10.00
CC3	Threshold for Entering 'Following'	-8		-8.00	-8.00	-8.00	-8.00	-8.00	-8.00	-8.00	-8.00	-8.00	-8.00	-8.00	-8.00
CC4	Negative 'Following Threshold'	-0.35		-0.35	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35
CC5	Positive 'Following Threshold'	0.35		0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
CC6	Speed Dependency of Oscillation	11.44		11.44	11.44	11.44	11.44	11.44	11.44	11.44	11.44	11.44	11.44	11.44	11.44
CC7	Oscillation Acceleration	0.82 ft/sec2		0.82	0.82	0.82	0.82	0.82	0.82	0.82	0.82	0.82	0.82	0.82	0.82
CC8	Standstill Acceleration	11.48 ft/sec2		11.48	11.48	11.48	11.48	11.48	11.48	11.48	11.48	11.48	11.48	11.48	11.48
CC9	Acceleration at 50mph	4.92 ft/sec2		4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92
Lane Changing Model Default Parameter Values															
	Own		**Trailing**												
Maximum deceleration	-13.12 ft/s^2		-9.48 ft/s^2	-9.48	-9.48	-9.48	-9.48	-9.48	-9.48	-9.48	-9.48	-9.48	-9.48	-9.48	-9.48
(-1 ft/sec2) per distance	100 ft		100ft	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
Accepted Deceleration	-3.28 ft/s^2		-1.64 ft/s^2	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64
Waiting Time Before Diffusion		60 s		60.00	60.00	60.00	60.00	60.00	60.00	60.00	60.00	60.00	60.00	60.00	60.00
Min. Headway (front/rear)		1.64 ft		1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64
To Slower Lane if Collision Time Above		11.00 s		11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00
Safety Reduction Factor		0.1		0.1	0.50	0.45	0.45	0.35	0.35	0.20	0.20	0.25	0.25	0.50	0.40
Maximum Deceleration for Cooperative Braking		-29 ft/s^2		-29.00	-29.00	-29.00	-29.00	-29.00	-29.00	-29.00	-29.00	-29.00	-29.00	-29.00	-29.00
Travel Time Evaluation															
Average Simulated Travel Time (sec)				240.1	253.63	250.27	244.03	243.11	241.60	239.50	239.26	238.60	236.49	222.10	234.60
Average Observed Travel time (sec)				230.77	230.77	230.77	230.77	230.77	230.77	230.77	230.77	230.77	230.77	230.77	230.77
% Error				4.04%	9.91%	8.45%	5.75%	5.35%	4.69%	3.78%	3.68%	3.39%	2.48%	-3.76%	1.66%
T-TEST				0.350	0.001	0.006	0.157	0.1873	0.27	0.365	0.361	0.38	0.54	0.35	0.53

*10 Iterations with different seed number were computed for each run

The resulting simulated mean travel time (250.27 seconds) was significantly larger than the field measured travel time (*P*-value=0.006; error~8.45%). In run 4, CC1 and SRF were kept constant while CC2 was decreased to 35ft. The resulting simulated mean travel time (244.03 seconds) was not statistically significantly different than the field measured travel time (*P*-value=0.157; error~5.75%). Since the default parameter values provided lower errors, the calibration process was continued. In run 6, CC1 was decreased to 1.25 second, CC2 maintained at 35ft, and the SRF reduced to 0.35. The resulting simulated mean travel time (241.60 seconds) was not statistically significantly different than the field measured travel time (*P*-value=0.27; error~4.69%). Note that the resulting error from this simulation is acceptable (<5%). In run 7, CC1 was further decreased to 1.10 second, CC2 was decreased to 20ft, and the SRF reduced to 0.2. The resulting simulated mean travel time (239.50 seconds) was not statistically significantly different than the field measured travel time (*P*-value=0.365; error~3.78%). It should be noted that the error in run 7 was enhanced compared to the error resulting from the initial evaluation run. In run 8, CC1 and SRF were kept constant while CC2 was decreased to 10ft. The resulting mean simulated travel time (239.26sec) did not vary significantly compared to run 7. In run 9, CC1 was decreased to 1 second, SRF increased to 0.25 while CC2 maintained at 10ft. The resulting mean simulated travel time decreased slightly to 238.60 seconds and the error decreased to 3.39%. In run 10, CC1 was further decreased to 0.5 seconds while SRF and CC2 remained constant. The resulting simulated travel time decreased to 236.49 seconds decreasing the error to 2.48%. In run 11, CC1 and CC2 were kept constant and SRF was increased to 0.50. The simulated travel time decreased significantly to 222.10 seconds making this simulation model underestimate travel time by 3.76%. In run 12, CC1 and CC2 were kept constant and SRF was decreased to 0.4, the resulting mean simulated travel time increased to 234.60 seconds and the error to 1.66%. During this calibration process, it was shown that the error was reduced from 4.04% in the initial evaluation (with the default parameter set for the driving behavior) to 1.66% in run 12.

7.4.2.5 SDLMS and MAS Validation

The validation of the VISSIM work zone model consisted of several parts. First, the early SDLMS is validated using throughput at the onset of congestion as the MOE. A different field dataset is used for that purpose. Second, the late SDLMS was validated with the same driving behavior parameter sets using travel time and throughput at the onset of congestion as MOEs. Third, the MAS was validated with the same driving behavior parameter sets using throughput at the onset of congestion as a MOE.

Table 7.4.2.6 shows the early SDLMS validation runs. From the calibration process, runs 7 through 12 resulted in acceptable p-values (>0.05) and acceptable errors (<5%) that were also improved compared to the initial evaluation run. The driving behavior parameters sets corresponding to these runs are used for the validation process. For each validation run 10 iterations with different seed numbers are completed and the resulting throughputs were collected. Looking at the Table below, Run 12 results in the best error (error=-0.75%, *p*-value =0.47). The next step of the validation process was to validate the model for the late SDLMS.

Table 7.4.2.6: Early SDLMS Validation

Base Driving Behavior Paramter Set				*Run Number				
Car Following Model Default Parameter Values				Run 7	Run 8	Run 9	Run 11	Run 12
CC0	Standstill distance	4.92 ft		4.92	4.92	4.92	4.92	4.92
CC1	Headway Time	1.20 s		1.10	1.10	1.00	0.50	0.50
CC2	Following Variation	13.12 ft		20.00	10.00	10.00	10.00	10.00
CC3	Threshold for Entering 'Following'	-8		-8.00	-8.00	-8.00	-8.00	-8.00
CC4	Negative 'Following Threshold'	-0.35		-0.35	-0.35	-0.35	-0.35	-0.35
CC5	Positive 'Following Threshold'	0.35		0.35	0.35	0.35	0.35	0.35
CC6	Speed Dependency of Oscillation	11.44		11.44	11.44	11.44	11.44	11.44
CC7	Oscillation Acceleration	0.82 ft/sec2		0.82	0.82	0.82	0.82	0.82
CC8	Standstill Acceleration	11.48 ft/sec2		11.48	11.48	11.48	11.48	11.48
CC9	Acceleration at 50mph	4.92 ft/sec2		4.92	4.92	4.92	4.92	4.92
Lane Changing Model Default Parameter Values								
	Own		**Trailing**					
Maximum deceleration	-13.12 ft/s^2		-9.48 ft/s^2	-9.48	-9.48	-9.48	-9.48	-9.48
(-1 ft/sec2) per distance	100 ft		100ft	100.00	100.00	100.00	100.00	100.00
Accepted Deceleration	-3.28 ft/s^2		-1.64 ft/s^2	-1.64	-1.64	-1.64	-1.64	-1.64
Waiting Time Before Diffusion		60 s		60.00	60.00	60.00	60.00	60.00
Min. Headway (front/rear)		1.64 ft		1.64	1.64	1.64	1.64	1.64
To Slower Lane if Collision Time Above		11.00 s		11.00	11.00	11.00	11.00	11.00
Safety Reduction Factor		0.1		0.20	0.20	0.25	0.50	0.40
Maximum Deceleration for Cooperative Braking		-29 ft/s^2		-29.00	-29.00	-29.00	-29.00	-29.00
Throughput Evaluation								
Average Simulated Simulated Throughput (veh/hr)				1207.00	1216.00	1220.00	1260.80	1262.00
Average Observed Throughput (Veh/hr)				1271.60	1271.60	1271.60	1271.60	1271.60
% Error				-5.08%	-4.37%	-4.06%	-0.85%	-0.75%
T-TEST				0.1	0.17	0.18	0.41	0.47

*10 Iterations with different seed number were computed for each run

The driving behavior parameters sets corresponding to runs 7 through 12 from the early SDLMS calibration process are used for the validation process of the late SDLMS. Ten iterations were completed for each run and the resulting throughputs and travel times are recorded (See Tables 7.4.2.7 and 7.4.2.8). Table 7.4.2.7 shows the validation of the late SDLMS using travel time. According to the results, Run 11 (error = 0.18%, *p*-value=0.85) and Run 12 (error = -0.33%, *p*-value=0.83) resulted in the best error percentages. Table 7.3.3.2.8 shows the validation of the late SDLMS using throughputs. According to the results, Run 11 (error = 3.85%, *p*-value=0.16) and Run 12 (error = 3.56%, *p*-value=0.18) resulted in the best error percentages. The next step of the validation process was to validate the work zone simulation model with the MAS system. The driving behavior parameters sets corresponding to runs 7 through 12 from the early and late SDLMS calibration process are used for the validation process of the MAS simulation model. Table 7.4.2.9 shows the calibration of the MAS using throughputs. Runs 9 (error=4.19%, *p*-value=0.31) and run 12 (error=4.54%, *p*-value=0.34) resulted in the best error compared to the other runs.

Looking at the overall calibration and validation process of the early SDLMS, late SDLMS, and MAS, the driving behavior parameters of run 12 were selected since they resulted in the most acceptable errors. The final headway time (CC1) value is 0.5 seconds, the following variation (CC2) value is 10 ft, and the safety reduction factor (SRF) is 0.40.

Table 7.4.2.7: Late SDLMS Validation Process (Travel Time)

Base Driving Behavior Paramter Set				*Run Number					
Car Following Model Default Parameter Values				Default	Run 7	Run 8	Run 9	Run11	Run 12
CC0	Standstill distance	4.92 ft		4.92	4.92	4.92	4.92	4.92	4.92
CC1	Headway Time	1.20 s		1.2	1.10	1.10	1.00	0.50	0.50
CC2	Following Variation	13.12 ft		13.12	20.00	10.00	10.00	10.00	10.00
CC3	Threshold for Entering 'Following'	-8		-8.00	-8.00	-8.00	-8.00	-8.00	-8.00
CC4	Negative 'Following Threshold'	-0.35		-0.35	-0.35	-0.35	-0.35	-0.35	-0.35
CC5	Positive 'Following Threshold'	0.35		0.35	0.35	0.35	0.35	0.35	0.35
CC6	Speed Dependency of Oscillation	11.44		11.44	11.44	11.44	11.44	11.44	11.44
CC7	Oscillation Acceleration	0.82 ft/sec2		0.82	0.82	0.82	0.82	0.82	0.82
CC8	Standstill Acceleration	11.48 ft/sec2		11.48	11.48	11.48	11.48	11.48	11.48
CC9	Acceleration at 50mph	4.92 ft/sec2		4.92	4.92	4.92	4.92	4.92	4.92
Lane Changing Model Default Parameter Values									
	Own		**Trailing**						
Maximum	-13.12 ft/s^2		-9.48 ft/s^2	-9.48	-9.48	-9.48	-9.48	-9.48	-9.48
(-1 ft/sec2)	100 ft		100ft	100.00	100.00	100.00	100.00	100.00	100.00
Accepted D	-3.28 ft/s^2		-1.64 ft/s^2	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64
Waiting Time Before Diffusion		60 s		60.00	60.00	60.00	60.00	60.00	60.00
Min. Headway (front/rear)		1.64 ft		1.64	1.64	1.64	1.64	1.64	1.64
To Slower Lane if Collision Time Above		11.00 s		11.00	11.00	11.00	11.00	11.00	11.00
Safety Reduction Factor		0.1		0.1	0.20	0.20	0.25	0.50	0.40
Maximum Deceleration for Cooperative Braking		-29 ft/s^2		-29.00	-29.00	-29.00	-29.00	-29.00	-29.00
Travel Time Evaluation									
Average Simulated Travel Time (sec)				249.1	237.81	238.02	238.60	235.80	234.60
Average Observed Travel time (sec)				235.38	235.38	235.38	235.38	235.38	235.38
% Error				5.83%	1.03%	1.12%	1.37%	0.18%	-0.33%
T-TEST				0.060	0.69	0.6	0.46	0.85	0.83

*10 Iterations with different seed number were computed for each run

Table 7.4.2.8: Late SDLMS Validation Process (Throughputs)

Base Driving Behavior Paramter Set			*Run Number				
Car Following Model Default Parameter Values			Run 7	Run 8	Run 9	Run 11	Run 12
CC0	Standstill distance	4.92 ft	4.92	4.92	4.92	4.92	4.92
CC1	Headway Time	1.20 s	1.10	1.10	1.00	0.50	0.50
CC2	Following Variation	13.12 ft	20.00	10.00	10.00	10.00	10.00
CC3	Threshold for Entering 'Following'	-8	-8.00	-8.00	-8.00	-8.00	-8.00
CC4	Negative 'Following Threshold'	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35
CC5	Positive 'Following Threshold'	0.35	0.35	0.35	0.35	0.35	0.35
CC6	Speed Dependency of Oscillation	11.44	11.44	11.44	11.44	11.44	11.44
CC7	Oscillation Acceleration	0.82 ft/sec2	0.82	0.82	0.82	0.82	0.82
CC8	Standstill Acceleration	11.48 ft/sec2	11.48	11.48	11.48	11.48	11.48
CC9	Acceleration at 50mph	4.92 ft/sec2	4.92	4.92	4.92	4.92	4.92
Lane Changing Model Default Parameter Values							
	Own	**Trailing**					
Maximum deceleration	-13.12 ft/s^2	-9.48 ft/s^2	-9.48	-9.48	-9.48	-9.48	-9.48
(-1 ft/sec2) per distance	100 ft	100ft	100.00	100.00	100.00	100.00	100.00
Accepted Deceleration	-3.28 ft/s^2	-1.64 ft/s^2	-1.64	-1.64	-1.64	-1.64	-1.64
Waiting Time Before Diffusion		60 s	60.00	60.00	60.00	60.00	60.00
Min. Headway (front/rear)		1.64 ft	1.64	1.64	1.64	1.64	1.64
To Slower Lane if Collision Time Above		11.00 s	11.00	11.00	11.00	11.00	11.00
Safety Reduction Factor		0.1	0.20	0.20	0.25	0.50	0.40
Maximum Deceleration for Cooperative Braking		-29 ft/s^2	-29.00	-29.00	-29.00	-29.00	-29.00
Throughput Evaluation							
Average Simulated Simulated Throughput (veh/hr)			1099.20	1104.10	1105.30	1103.20	1100.20
Average Observed Throughput (Veh/hr)			1062.33	1062.33	1062.33	1062.33	1062.33
% Error			3.47%	3.93%	4.04%	3.85%	3.56%
T-TEST			0.21	0.15	0.12	0.16	0.18

*10 Iterations with different seed number were computed for each run

Table 7.4.2.9: Validation of MAS

Base Driving Behavior Paramter Set				*Run Number				
Car Following Model Default Parameter Values				Run 7	Run 8	Run 9	Run 11	Run 12
CC0	Standstill distance	4.92 ft		4.92	4.92	4.92	4.92	4.92
CC1	Headway Time	1.20 s		1.10	1.10	1.00	0.50	0.50
CC2	Following Variation	13.12 ft		20.00	10.00	10.00	10.00	10.00
CC3	Threshold for Entering 'Following'	-8		-8.00	-8.00	-8.00	-8.00	-8.00
CC4	Negative 'Following Threshold'	-0.35		-0.35	-0.35	-0.35	-0.35	-0.35
CC5	Positive 'Following Threshold'	0.35		0.35	0.35	0.35	0.35	0.35
CC6	Speed Dependency of Oscillation	11.44		11.44	11.44	11.44	11.44	11.44
CC7	Oscillation Acceleration	0.82 ft/sec2		0.82	0.82	0.82	0.82	0.82
CC8	Standstill Acceleration	11.48 ft/sec2		11.48	11.48	11.48	11.48	11.48
CC9	Acceleration at 50mph	4.92 ft/sec2		4.92	4.92	4.92	4.92	4.92
Lane Changing Model Default Parameter Values								
	Own		**Trailing**					
Maximum deceleration	-13.12 ft/s^2		-9.48 ft/s^2	-9.48	-9.48	-9.48	-9.48	-9.48
(-1 ft/sec2) per distance	100 ft		100ft	100.00	100.00	100.00	100.00	100.00
Accepted Deceleration	-3.28 ft/s^2		-1.64 ft/s^2	-1.64	-1.64	-1.64	-1.64	-1.64
Waiting Time Before Diffusion		60 s		60.00	60.00	60.00	60.00	60.00
Min. Headway (front/rear)		1.64 ft		1.64	1.64	1.64	1.64	1.64
To Slower Lane if Collision Time Above		11.00 s		11.00	11.00	11.00	11.00	11.00
Safety Reduction Factor		0.1		0.20	0.20	0.25	0.50	0.40
Maximum Deceleration for Cooperative Braking		-29 ft/s^2		-29.00	-29.00	-29.00	-29.00	-29.00
Throughput Evaluation								
Average Simulated Simulated Throughput (veh/hr)				1026.60	1014.30	1011.20	1016.81	1014.60
Average Observed Throughput (Veh/hr)				970.50	970.50	970.50	970.50	970.50
% Error				5.78%	4.51%	4.19%	4.77%	4.54%
T-TEST				0.28	0.31	0.33	0.29	0.34

*10 Iterations with different seed number were computed for each run

7.5.1 MOE and data collection

Work zone throughputs and travel times are the selected as operational measures of effectiveness. To collect travel time data collection points are located at the beginning of the work zone (node 1 where the first PCMS in the MOT plans is located) and the end of the lane closure. To collect throughputs, data collection points were selected at the end of the lane closure just before both lanes are open again. For the safety evaluation speed variance is selected as the surrogate measure of effectiveness. To collect speeds, two data collection points are located at the location of the RTMS in the open and closed lane. In all three simulation models (early SDLMS, late SDLMS, and MAS) data collection points were located exactly at the same locations.

As mentioned earlier the objective of the simulation study is to determine the effectiveness of each MOT type (early SDLMS, late SDLMS, and MAS) under different driver's compliance rate, different truck percentage in the traffic composition, and different traffic demand volumes. For that purpose, different levels of each of these variables are considered. Four different level of drivers' compliance rate, C20 (20% of drivers comply to the merging instruction), C40 (40% of drivers comply to the merging instruction), C60 (60% of drivers comply to the merging instruction), C80 (80% of drivers comply to the merging instruction) are created. Three different level of truck percentage in the traffic composition are created, T10 (trucks constitute 10% of the demand volume), T20 (trucks constitute 20% of the demand volume), T30 (trucks constitute 30% of the demand volume). Five different traffic demand volume levels are created. V0500 means the traffic demand volume is 500 veh/hr. V1000 means the traffic demand volume is 1000 veh/hr. V1500 means the traffic demand volume is 1500 veh/hr. V2000 means the traffic demand volume is 2000 veh/hr. V2500 means the traffic demand volume is 2500 veh/hr. Having 4 compliance rate levels, 3 truck percentage level, and 5 traffic demand volume levels resulted in 60 combinations for the early and late SDLMS. For the MAS there is no compliance rate since there is no merging instructions, therefore the MAS has a total of 15 combinations. Sixty combinations (runs) were completed for the early SDLMS and the late SDLMS, and 15 combinations for the MAS resulting in a total of 135 runs (combinations). For each run or combination, 10 iterations with different seed number were executed.

7.5.2 Work Zone Throughputs

Figures 7.5.2-a and 7.5.2-b show the trend of the mean throughputs under different combinations of compliance rate, percent trucks, demand volumes, and MOT type. Examining these two figures one can notice that the mean throughputs seem similar within each combination under demand volume levels of V500, V1000, V1500 for all MOT types. However, the mean throughputs seem dissimilar within each combinations under volume levels V2000 and V2500. Particularly, the early SDLMS seem to have the highest mean throughputs, followed by the MAS, then the late SDLMS. Looking at charts and Tables 7.5.2-a and 7.5.2-b with the percentage of trucks of 10% and different compliance rate, one can see the early SDLMS mean throughputs increase slightly as the compliance rate increases.

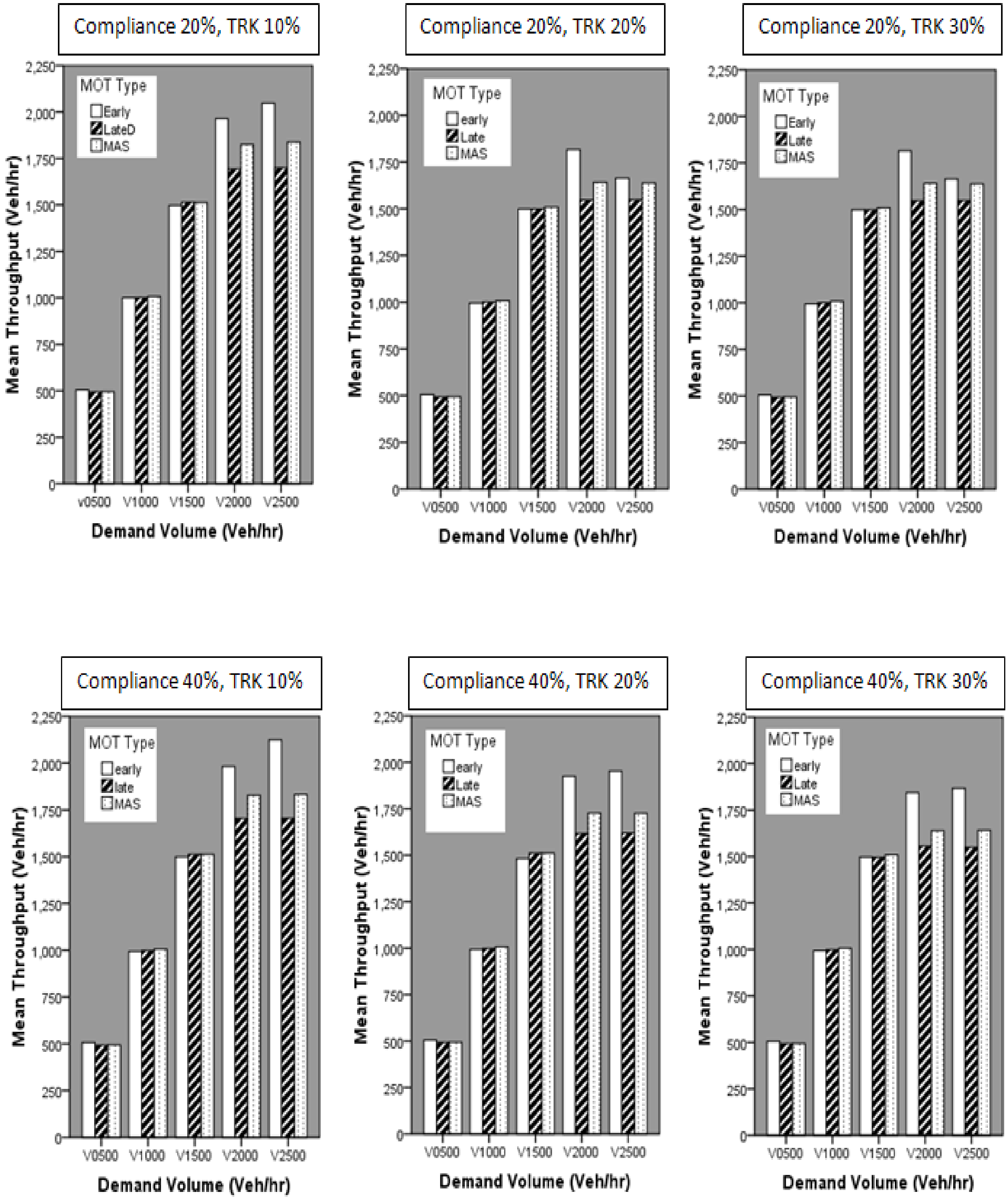


Figure 7.5.2-a: Throughputs under different combinations (C20, C40)

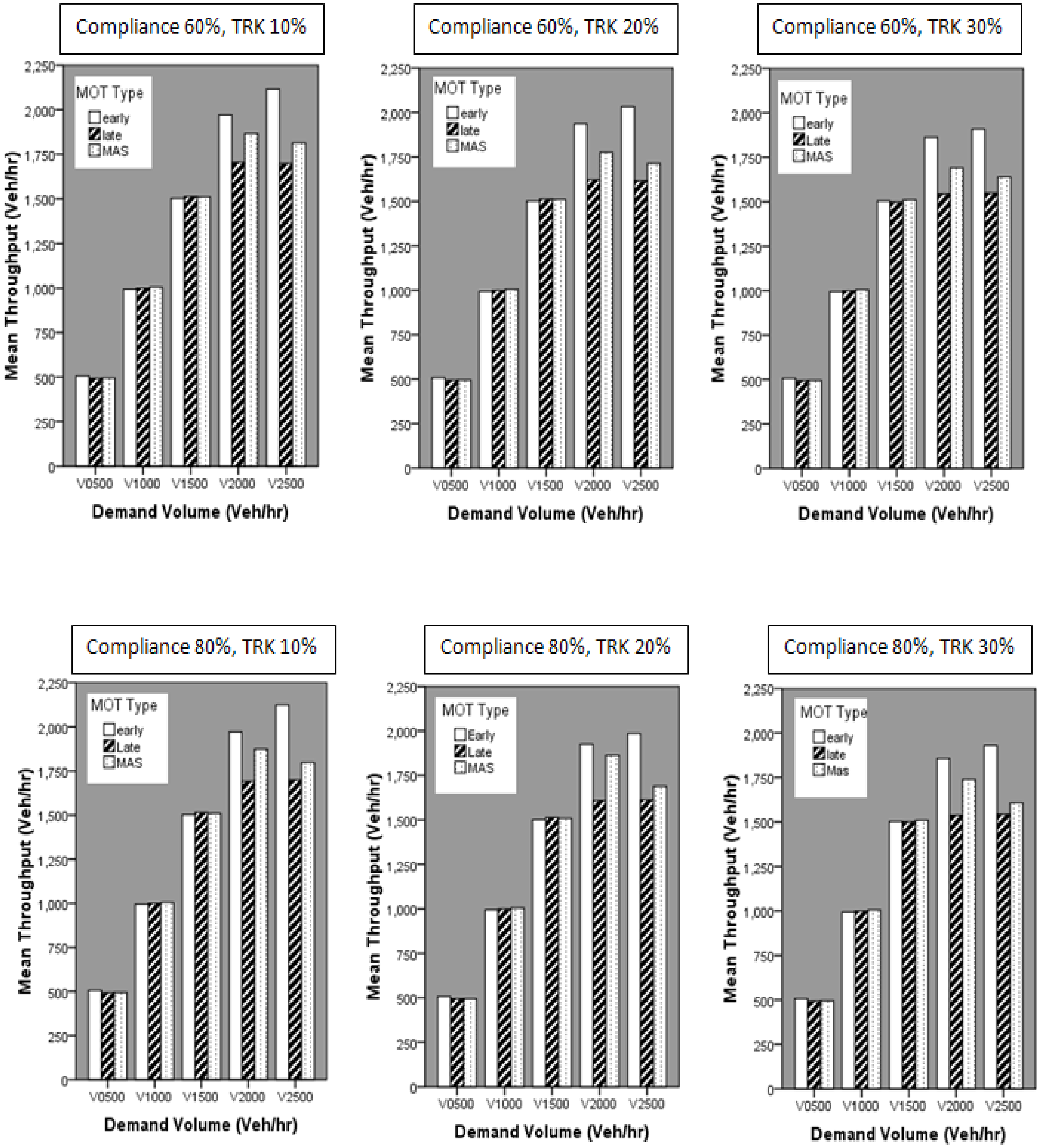


Figure 7.5.2-b: Throughputs under different combinations (C60, C80)

For the exact values of these means the reader may consult Tables 7.5.2-a and 7.5.2-b. However, it is noticed that when the percent trucks increases to 20%, if we compare C20T20, C40T20, C60T20, and C80T20 for the early SDLMS, one can notice that as the compliance rate increases the mean throughout decreases. The same trend is shown under percent trucks 30%. Even though the trends show a decreasing mean throughput for the early SDLMS as the truck percentage and compliance rate increase, these mean throughputs are the highest among all MOT types. Looking at charts with the percentage of trucks of 10% and different compliance rate, one can see the late SDLMS mean throughputs does not vary considerably when the compliance rate increases. For the exact values of these means the reader may consult Tables 7.5.2-a and 7.5.2-b. Moreover, it is noticed that when the percent trucks increases to 20%, if we compare C20T20, C40T20, C60T20, and C80T20 for the late SDLMS, one can notice that as the compliance rate increases the mean throughout does not differ noticeably. The same trend is shown under percent trucks 30%.

The objective of this simulation study is to determine the MOT with the best performance under different combinations. Tables 7.5.2-a and 7.5.2-b provide a summary statistics of the work zone throughputs under each combination. Ideally, we would like to know under each demand volume level, each compliance rate level, and truck percentage level, which MOT type results in the highest throughout. Therefore, for each combination an overall F-test was conducted with a null hypothesis that mean throughputs under all three MOT types are the same. If the null hypothesis is rejected, pair wise Tukey's comparisons are completed to determine the difference between each pair of throughput means. In Tables 7.5.2-a and 7.5.2-b, under last three columns, the shaded areas mean that there was no need for pairwise comparison of means since the F-test null hypothesis was not rejected. In the same column all mean differences superscripted with stars are significant at 0.05 significance level. From these tables one can notice that under demand volumes V500, V1000, V1500, there were no significant differences in the mean throughputs for all compliance rates and trucks percentage in the traffic.

Table 7.5.2-a: Throughputs Comparisons (C20, C40)

Throughput (veh/hr) Analyses													
Compliance % And Truck%	Volume (Vehhr)	Summary Statistics									Overall F-Test P-value (Pr> F)	Tukey's Comparison		
		MAS (μ), (S), Max			EarlyDLM (μ), (S), Max			LateDLM (μ), (s), Max				$\mu_{EarlyDLM} - \mu_{MAS}$ P-value (Pr> F)	$\mu_{MAS} - \mu_{LateDLM}$ P-value (Pr> F)	$\mu_{EarlyDLM} - \mu_{LateDLM}$ P-value (Pr> F)
		Mean (μ)	Std. Dev.	Max	Mean (μ)	Std. Dev.	Max	Mean (μ)	Std. Dev.	Max				
Compliance 20% TRK10%	V0500	494.67	31.23	576.00	505.67	49.16	620.00	493.47	34.56	572.00	0.61			
	V1000	1007.47	45.25	1056.00	1000.80	39.78	1040.00	1001.73	65.54	1116.00	0.93			
	V1500	1512.27	93.77	1656.00	1497.07	70.21	1664.00	1514.48	69.48	1628.00	0.76			
	V2000	1824.53	24.18	1888.00	1965.33	72.99	2076.00	1692.90	35.97	1748.00	<0.001	140.80***	131.63***	272.43***
	V2500	1838.20	20.02	1884.00	2048.20	30.05	2108.00	1698.93	19.65	1740.00	<0.001	210.00***	139.27***	349.27***
Compliance 20% TRK20%	V0500	494.67	31.22	576.00	505.07	29.14	556.00	491.11	39.98	572.00	0.49			
	V1000	1004.57	43.79	1112.00	997.13	41.35	1068.00	999.11	69.45	1112.00	0.88			
	V1500	1510.93	91.48	1660.00	1496.53	70.63	1664.00	1517.11	75.97	1608.00	0.76			
	V2000	1731.47	24.10	1784.00	1945.33	53.01	2028.00	1618.44	15.16	1652.00	<0.001	213.87***	113.02***	326.89***
	V2500	1722.60	28.32	1776.00	1956.20	32.43	2016.00	1619.78	16.14	1648.00	<0.001	233.60***	102.82***	336.42***
Compliance 20% TRK30%	V0500	494.67	31.01	576.00	505.67	29.14	556.00	493.33	40.42	572.00	0.59			
	V1000	1008.00	43.94	1052.00	994.13	40.92	1068.00	1000.27	66.20	1112.00	0.76			
	V1500	1059.33	88.51	1652.00	1499.20	67.19	1664.00	1499.20	71.80	1592.00	0.92			
	V2000	1640.53	26.04	1680.00	1815.73	744.71	1912.00	1546.67	21.09	1572.00	<0.001	175.20***	96.86***	269.06***
	V2500	1637.80	17.28	1668.00	1664.00	23.70	1896.00	1548.20	25.08	1592.00	<0.001	26.20	89.60	115.80***
Compliance 40% TRK10%	V0500	494.13	33.90	580.00	507.20	28.50	546.00	493.06	40.86	572.00	0.47			
	V1000	1006.13	46.05	1056.00	993.67	45.51	1068.00	998.40	63.06	1100.00	0.27			
	V1500	1512.27	94.72	1664.00	1498.93	74.07	1668.00	1512.27	82.87	1624.00	0.53			
	V2000	1828.00	24.19	1880.00	1981.33	93.36	2189.00	1704.27	19.09	1732.00	<0.001	153.33***	123.73***	277.06***
	V2500	1832.40	18.75	1856.00	2124.40	30.62	2180.00	1705.60	17.33	1740.00	<0.001	292.00***	126.80***	418.80***
Compliance 40% TRK20%	V0500	494.13	33.90	580.00	505.60	27.49	556.00	493.33	40.64	572.00	0.56			
	V1000	1006.67	44.73	1052.00	993.67	43.33	1068.00	998.40	63.60	1100.00	0.77			
	V1500	1511.73	92.87	1660.00	1480.50	97.82	1676.00	1512.00	82.72	1608.00	0.55			
	V2000	1726.67	23.88	1764.00	1925.07	45.29	1980.00	1616.80	17.90	1652.00	<0.001	198.40***	109.87***	308.27***
	V2500	1726.00	23.87	1780.00	1952.20	31.88	2004.00	1620.40	21.71	1664.00	<0.001	226.20***	105.60***	331.80***
Compliance 40% TRK30%	V0500	494.13	32.90	580.00	505.00	27.49	556.00	493.33	40.04	572.00	0.55			
	V1000	1006.67	44.73	1052.00	993.87	42.76	1068.00	998.40	63.60	1100.00	0.79			
	V1500	1509.87	88.40	1660.00	1497.33	71.77	1676.00	1494.67	67.34	1572.00	0.84			
	V2000	1638.13	20.83	1680.00	1843.73	35.89	1900.00	1555.47	24.56	1584.00	<0.001	205.60***	82.66***	288.26***
	V2500	1642.00	25.51	1700.00	1866.60	27.17	1932.00	1548.60	21.69	1588.00	<0.001	224.60***	93.40***	318.00***

Table 7.5.2-b: Throughputs Comparisons (C60, C80)

Throughput (veh/hr) Analyses													
Compliance % And Truck%	Volume	Summary Statistics									Overall F-Test P-value (Pr> F)	Tukey's Comparison		
		MAS (μ), (S), Max			EarlyDLM (μ), (S), Max			LateDLM (μ), (s), Max				μEarlyDLM - μMAS P-value (Pr> F)	μMAS - μLateDLM P-value (Pr> F)	μEarlyDLM - μLateDLM P-value (Pr> F)
		Mean (μ)	Std. Dev.	Max	Mean (μ)	Std. Dev.	Max	Mean (μ)	Std. Dev.	Max				
Compliance 60% TRK10%	V0500	494.67	33.59	584.00	506.93	30.56	564.00	493.33	40.67	572.00	0.51			
	V1000	1004.80	44.98	1056.00	994.13	43.99	1064.00	999.47	65.09	1100.00	0.86			
	V1500	1511.73	92.62	1664.00	1501.87	72.58	1668.00	1513.07	85.52	1640.00	0.92			
	V2000	1866.40	26.13	1904.00	1971.20	70.48	2060.00	1705.60	23.75	1744.00	<0.001	104.80***	160.80***	265.60***
	V2500	1814.80	117.91	1924.00	2116.20	57.97	2208.00	1698.80	16.93	1728.00	<0.001	301.40***	116.00***	417.00***
Compliance 60% TRK20%	V0500	494.67	33.59	584.00	507.47	30.01	564.00	493.60	40.56	572.00	0.49			
	V1000	1005.07	45.67	1056.00	994.13	43.92	1064.00	998.93	64.02	1100.00	0.85			
	V1500	1511.20	92.92	1660.00	1501.60	70.21	1660.00	1512.27	84.07	1608.00	0.93			
	V2000	1775.47	30.31	1812.00	1935.47	53.62	2012.00	1621.87	16.55	1648.00	<0.001	160.00***	153.60***	313.60***
	V2500	1713.40	141.13	1828.00	2033.60	110.73	2200.00	1615.40	16.83	1640.00	<0.001	320.20***	98.00***	418.20***
Compliance 60% TRK30%	V0500	494.67	33.73	584.00	506.93	33.73	584.00	493.60	40.53	572.00	0.52			
	V1000	1005.07	45.67	1056.00	994.40	43.65	1060.00	999.20	63.65	1096.00	0.85			
	V1500	1510.93	92.03	1592.00	1504.80	68.79	1668.00	1498.40	73.35	1592.00	0.91			
	V2000	1691.47	33.97	1752.00	1863.20	38.58	1948.00	1543.20	25.08	1588.00	<0.001	171.73***	148.27***	320.00***
	V2500	1641.20	129.07	1780.00	1908.20	102.33	2040.00	1548.20	92.17	1592.00	<0.001	267.00***	93.00***	360.00***
Compliance 80% TRK10%	V0500	494.13	34.74	588.00	506.67	29.53	560.00	493.33	39.73	572.00	0.51			
	V1000	1004.27	39.96	1040.00	994.93	45.64	1068.00	999.47	67.20	1096.00	0.89			
	V1500	1510.40	91.24	1644.00	1501.87	72.17	1656.00	1514.13	85.19	1620.00	0.92			
	V2000	1874.20	102.62	2000.00	1971.73	77.58	2068.00	1691.20	27.00	1720.00	<0.001	97.53***	183.00***	308.60***
	V2500	1797.80	280.19	2016.00	2125.20	87.75	2256.00	1699.60	19.90	1736.00	<0.001	327.40***	96.20***	425.60***
Compliance 80% TRK20%	V0500	494.13	34.74	588.00	506.40	29.66	560.00	493.33	39.73	572.00	0.52			
	V1000	1004.80	38.93	1040.00	994.13	44.15	1064.00	999.20	67.29	1096.00	0.85			
	V1500	1509.87	89.64	1640.00	1501.33	71.60	1656.00	1514.40	86.99	1616.00	0.91			
	V2000	1864.00	58.94	1940.00	1925.60	48.07	2004.00	1608.27	10.95	1628.00	<0.001	61.60***	255.73***	317.33***
	V2500	1689.40	272.50	1960.00	1985.40	88.34	2112.00	1613.40	24.11	1656.00	<0.001	296.00***	76.00	372.00***
Compliance 80% TRK30%	V0500	494.67	35.28	592.00	506.40	29.66	560.00	493.33	39.73	572.00	0.54			
	V1000	1004.80	39.95	1040.00	994.13	44.05	1064.00	999.20	68.25	1096.00	0.89			
	V1500	1509.33	91.29	1648.00	1502.13	74.76	1664.00	1499.47	72.22	1600.00	0.94			
	V2000	1739.43	81.59	1848.00	1856.27	26.29	1928.00	1537.60	17.36	1580.00	<0.001	116.84***	201.83***	318.67***
	V2500	1607.05	257.85	1892.00	1930.10	104.70	2080.00	1543.00	37.72	1600.00	<0.001	323.05***	64.05	387.10***

However, for traffic demand volume levels V2000 and V2500 the early SDLMS results in the significantly highest mean throughputs compared to the late SDLMS under all combinations. Tables 7.5.2-a and 7.5.2-b show the early SDLMS compared to the MAS has significantly higher mean throughputs under all combinations except when the compliance rate is 20% and the truck percentage is 30%. Comparing the MAS to the late SDLMS, the mean throughputs were the highest for the MAS under all combinations. Moreover, the differences were statistically significant except for three instances (combinations) C20T30 and V2500, C80T20, and V2500, C80 T30 and V2500.

7.5.3 Work Zone Travel Times

The second selected operational MOE is the travel time through the work zone. Figures 7.5.3-a and 7.5.3-b show the trends of the mean travel times through the work zones under different combinations of compliance rate, percent trucks, demand volumes, and MOT type. Examining these two figures one can notice that the mean travel times seem similar within each combination under demand volume levels of V500, V1000, and V1500 for all MOT types. However, the mean travel times seem disparate within each combination under volume levels V2000 and V2500. Particularly, the early SDLMS seem to have the lowest mean travel time, followed by the MAS, then the late SDLMS. Looking at charts with the percentage of trucks of 10% and different compliance rate, one can see the early SDLMS mean travel times decrease slightly as the compliance rate increases. Specifically when the compliance rate is 80% and the truck percentage is 10% the mean travel time drops significantly at demand volume level V2500 (599.16 sec) compared to the same combinations under lower compliance rates.

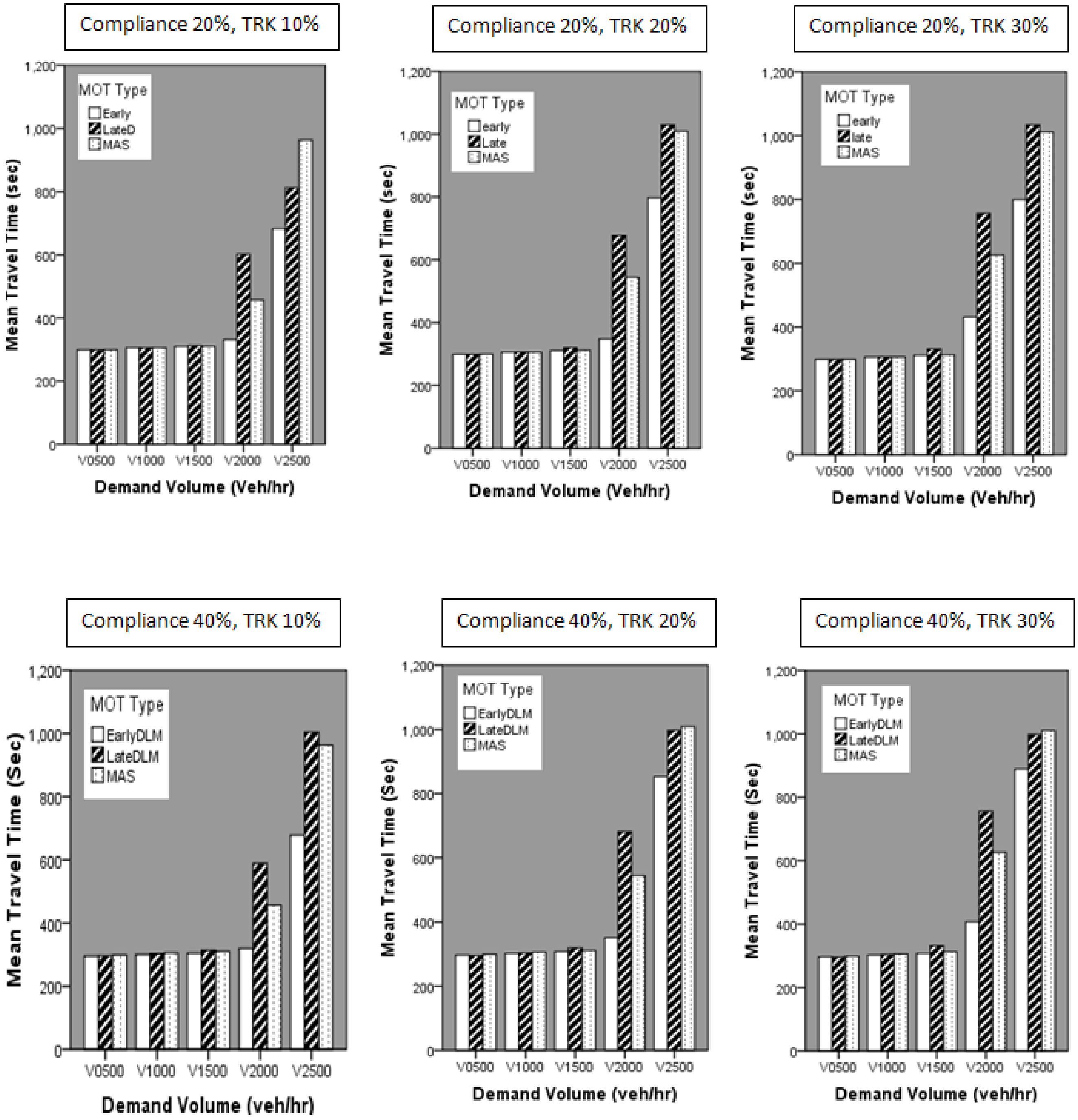
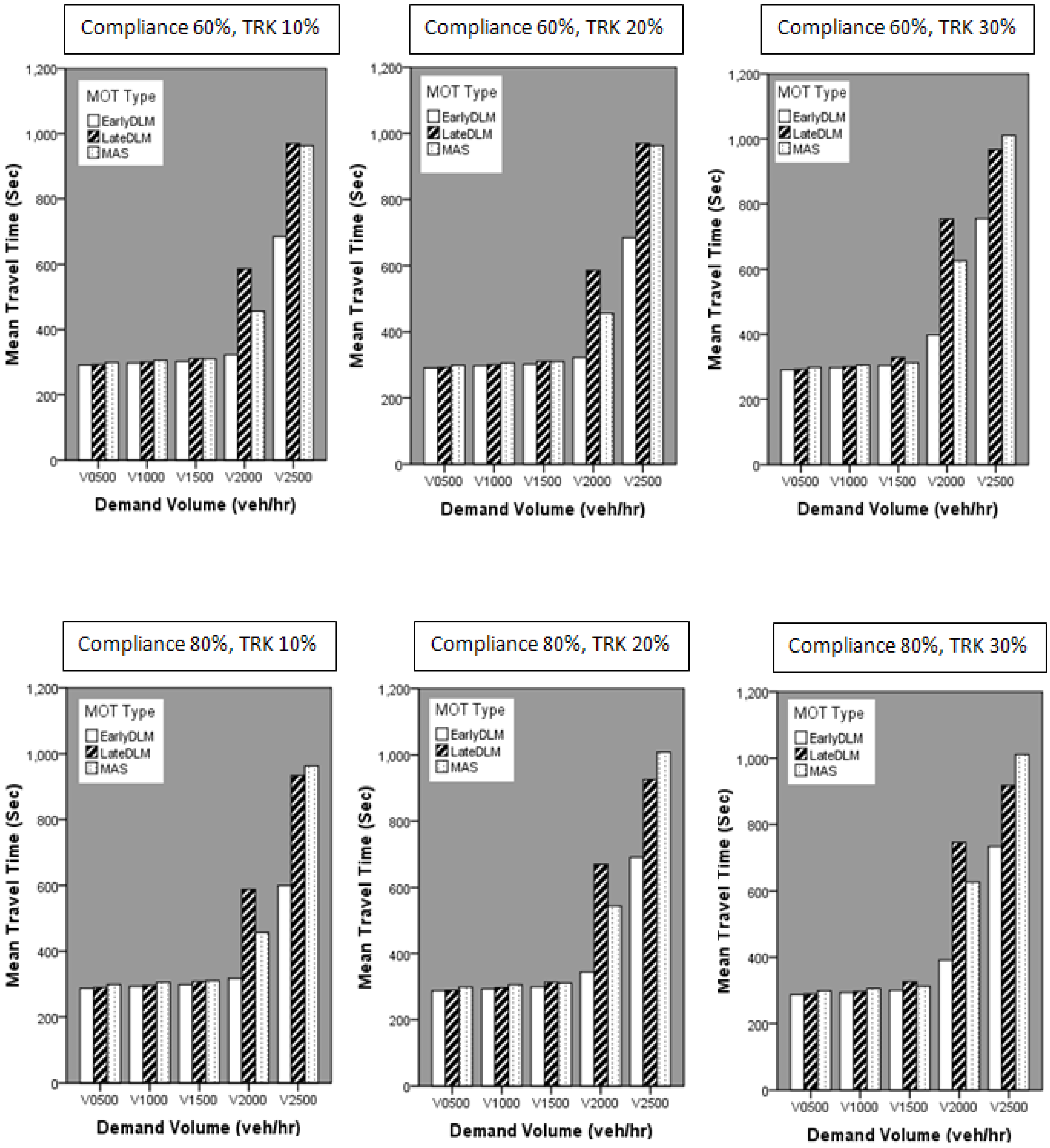


Figure 7.5.3-a: Travel Times under different combinations (C20, C40)


Compliance 60%, TRK 10%

Compliance 60%, TRK 20%

Compliance 60%, TRK 30%

Compliance 80%, TRK 10%

Compliance 80%, TRK 20%

Compliance 80%, TRK 30%

Figure 7.5.3-b: Travel Times under different combinations (C60, C80)

For the exact values of these means the reader may consult Tables 7.5.3-a and 7.5.3-b. It is also noticed that when the percent trucks increases to 20%, if we compare C20T20, C40T20, C60T20, and C80T20 for the early SDLMS, one can notice that as the compliance rate increases the mean travel time decreases slightly to the exception of C40T20 where the mean travel time increases to 852.90sec. The same trend is shown under percent trucks 30% with the same exception of C40T20 where the mean travel time increases to 889.02 sec. For the late SDLMS no clear trend is noticed that when we compare the effect of increasing compliance rate at the same truck percentage levels.

Tables 7.5.3-a and 7.5.3-b provide a summary statistics of the work zone travel times for each combination of compliance rate level, percentage trucks level in the traffic, traffic demand volume level under each MOT type (early SDLMS, late SDLMS, MAS). Since preliminary analyses indicated inhomogeneous variances between travel times for each combination for the early SDLMS, late SDLMS, and MAS, a Levene’s test is conducted for each combination with the null hypothesis that travel times under the early SDLMS, late SDLMS, and MAS are homogenous (See Tables 7.5.3-a and 7.5.3-b). If the null hypothesis is rejected, Friedman’s nonparametric test is conducted for each combination with the null hypothesis that travel times means (early SDLMS, late SDLMS, and MAS) are equal. If the null hypothesis is rejected meaning at least one travel time mean is different than the others, then unequal variance pairewise t-tests are conducted. In Tables 7.5.3-a and 7.5.3-b, under last three columns, the shaded areas mean that there was no need for pair wise comparison of means since the F-test or Friedman’s test null hypothesis was not rejected.

In Tables 7.5.3-a, for compliance rate of 20% and truck percentage 10%, one can see that the early SDLMS has significantly lower travel then the late SDLMS and MAS under demand volume levels V2000 and V2500. However, comparing the late SDLMS to the MAS, MAS has a significantly lower travel time mean under demand volume level V2000 and a significantly higher travel time under demand volume V2500.

For a compliance rate of 20% and truck percentage of 20% and 30%, the early SDLMS resulted in statistically significant lower travel times compared to the late SDLMS under demand volume levels V1500, V2000, and V2500 and statistically significant lower travel times compared MAS under demand volume levels of V2000 and V2500. The MAS has a significantly lower travel compared to the late SDLMS at demand volume levels V1500 and V2000 for 20% and V1500 for 30% trucks.

For compliance rate of 40% and truck percentage of 10%, the early SDLMS resulted in statistically significant lower travel times compared to the late SDLMS and the MAS under demand volume level V1500, V2000, and V2500. The MAS has a significantly lower travel compared to the late SDLMS at demand volume levels V1500 and V2000.

For compliance rate of 40% and truck percentages of 20% and 30%, the late SDLMS has s significantly lower travel times compared to the early SDLMS and MAS under demand volume level V0500. The early SDLMS has a significantly lower travel times compared to the late SDLMS and MAS under demand volume levels of V1000, V1500, V2000, and V2500. The MAS has significantly lower travel times compare to the late SDLMS under demand volume levels of V1000, V1500, V2000 and significantly higher travel times compared to the late SDLMS at demand volume level V2500.

For compliance rates of 60% and 80% and trucks percentage of 10%, 20%, and 30% the early SDLMS resulted in lower travel times compared to the MAS and late SDLMS under all demand volume levels to the exception of C60T30 under level demand volume V0500 where results show that there was no significant difference between the mean travel times for the early SDLMS and late SDLMS. For compliance rates of 60% and 80% and trucks percentage of 10%, 20%, and 30% the late SDLMS resulted in significantly lower travel times compared to the MAS under demand volume level of V0500 and V1000. Under compliance rate of 60% and trucks percentage of 10%, the late SDLMS resulted in significantly lower travel times compared to the MAS only under demand volume level of V2000. For compliance rate of 60% and trucks percentage 20% and 30%, the late SDLMS resulted in significantly higher travel time compared to the MAS under demand volume levels of V1500, and V2000. For compliance rate 80% and truck percentage of 10%, the late SDLMS resulted in significantly higher travel times under demand volume level of V2000 and significantly lower travel times under demand volume level V1500 compared to the MAS. For compliance rate 80% and truck percentage of 20%, the late SDLMS resulted in significantly higher travel times under demand volume level of V2000 and significantly lower travel times under demand volume level V2500 compared to the MAS. For compliance rate 80% and truck percentage of 30%, the late SDLMS resulted in significantly higher travel times under demand volume level of V1500 and V2000, and significantly lower travel time under demand volume level V2500 compared to the MAS.

Table 7.5.3-a: Travel times Comparisons (C20, C40)

Travel Time (Sec) Analyses																
		Summary Statistics												Unequal Variance T-test		
Compliance % And Truck%	Volume (Vehhr)	MAS (μ), (S), Max			EarlyDLM (μ), (S), Max			LateDLM (μ), (s), Max			Levene's Test (Pr>F)	Overall F-Test P-value (Pr> F)	Friedman's Test (Pr>F)	μEarlyDLM Vs. μMAS P-value (Pr> F)	μMAS Vs. μLateDLM P-value (Pr> F)	μEarlyDLM Vs. μLateDLM P-value (Pr> F)
		Mean (μ)	Std. Dev.	Max	Mean (μ)	Std. Dev.	Max	Mean (μ)	Std. Dev.	Max						
Compliance 20% TRK10%	V0500	299.09	1.97	302.70	299.05	2.71	305.60	298.70	2.62	304.40	0.184	0.722				
	V1000	305.53	1.20	307.60	305.53	1.20	309.20	305.54	2.32	308.80	0.184	0.990				
	V1500	310.49	1.45	312.80	309.16	1.62	313.00	312.73	4.54	329.30	0.010		0.840			
	V2000	456.57	81.32	580.50	331.23	13.31	354.40	602.10	118.42	928.70	<0.001		<0.001	<0.001***	<0.001***	<0.001***
	V2500	962.90	138.40	1156.00	682.42	188.94	1126.80	812.47	212.20	1124.00	<0.001		<0.001	<0.001***	0.004***	0.024***
Compliance 20% TRK20%	V0500	299.18	1.97	302.80	298.95	2.32	303.10	298.78	2.87	304.40	0.120	0.809				
	V1000	305.77	1.21	307.80	305.48	1.73	308.00	306.33	1.30	309.00	0.100	0.210				
	V1500	311.41	1.91	315.50	310.75	1.94	315.10	319.77	9.10	341.70	<0.001		<0.001	0.189	<0.001***	<0.001***
	V2000	543.86	113.30	720.50	347.72	21.43	378.60	676.53	162.02	1040.30	<0.001		<0.001	<0.001***	0.002***	<0.001***
	V2500	1008.48	135.00	1182.20	797.90	184.85	1119.50	1029.36	237.87	1357.00	<0.001		<0.001	<0.001***	0.64	<0.001***
Compliance 20% TRK30%	V0500	299.25	1.98	302.80	299.23	2.33	303.10	298.96	2.81	304.40	0.120	0.871				
	V1000	306.03	1.27	308.10	305.84	1.75	309.40	306.65	1.35	309.20	0.130	0.100				
	V1500	312.50	2.55	319.00	311.59	2.13	316.00	330.75	24.30	398.20	0.001		<0.001	0.1615	0.003***	0.002***
	V2000	626.32	147.85	908.30	430.74	102.73	844.00	756.41	211.14	1222.10	0.014		<0.001	<0.001***	0.079	<0.001***
	V2500	1011.33	132.83	1184.70	799.18	235.10	1334.90	1033.33	240.13	1344.80	<0.001		0.001	0.005***	0.622	0.006***
Compliance 40% TRK10%	V0500	299.09	1.97	302.70	295.60	3.42	305.50	295.69	3.19	301.60	0.210	0.620				
	V1000	305.53	1.20	307.60	300.01	1.91	304.10	303.38	1.49	307.20	0.130	0.450				
	V1500	310.49	1.45	312.80	305.05	1.68	308.20	313.67	4.60	326.40	<0.001		<0.001	<0.001***	0.008***	0.008***
	V2000	456.57	81.32	580.50	319.01	7.88	338.10	588.53	107.79	802.10	<0.001		<0.001	<0.001***	<0.001***	<0.001***
	V2500	962.90	138.40	1156.00	677.78	188.06	1151.40	1004.14	158.41	1258.70	0.044		<0.001	<0.001***	0.210	<0.001***
Compliance 40% TRK20%	V0500	299.18	1.97	302.80	296.26	3.04	303.90	295.74	3.20	301.70	<0.001		<0.001	<0.001***	<0.001***	<0.001***
	V1000	305.77	1.21	307.80	302.26	2.26	306.70	303.67	1.48	307.40	<0.001		<0.001	0.006***	<0.001***	<0.001***
	V1500	311.41	1.91	315.50	307.24	2.65	313.30	318.72	10.02	341.00	<0.001		<0.001	<0.001***	0.004***	<0.001***
	V2000	543.86	113.30	720.50	350.60	29.96	408.70	681.35	158.46	994.10	<0.001		<0.001	<0.001***	<0.001***	<0.001***
	V2500	1008.48	135.00	1182.20	852.90	213.98	1274.10	997.47	151.60	1192.30	0.002		<0.001	0.006***	0.712	0.004
Compliance 40% TRK30%	V0500	299.25	1.98	302.80	296.68	2.95	303.90	295.86	3.19	301.70	0.003		<0.001	<0.001***	<0.001***	0.27
	V1000	306.03	1.27	308.10	302.55	2.33	307.40	304.04	1.47	307.60	<0.001		<0.001	<0.001***	<0.001***	0.0049***
	V1500	312.50	2.55	319.00	307.88	2.64	315.80	331.67	21.89	377.60	<0.001		<0.001	<0.001***	<0.001***	<0.001***
	V2000	626.32	147.85	908.30	407.70	68.35	569.30	755.66	185.75	1124.80	0.002		<0.001	<0.001***	0.0017***	<0.001***
	V2500	1011.33	132.83	1184.70	889.02	161.64	1235.60	998.03	159.76	1232.40	<0.001		0.003	0.020***	0.650	0.070

Table 7.5.3-b: Travel times Comparisons (C60, C80)

Travel Time (sec) Analyses																
Compliance % And Truck%	Volume	Summary Statistics									Levene's Test (Pr>F)	Overall F-Test P-value (Pr> F)	Friedman's Test (Pr>F)	Unequal Variance T-test		
		MAS (μ), (S), Max			EarlyDLM (μ), (S), Max			LateDLM (μ), (s), Max						$\mu_{EarlyDLM}$ - μ_{MAS} P-value (Pr> F)	μ_{MAS} - $\mu_{LateDLM}$ P-value (Pr> F)	$\mu_{EarlyDLM}$ - $\mu_{LateDLM}$ P-value (Pr> F)
		Mean (μ)	Std. Dev.	Max	Mean (μ)	Std. Dev.	Max	Mean (μ)	Std. Dev.	Max						
Compliance 60% TRK10%	V0500	299.09	1.97	302.70	291.25	3.48	299.40	292.71	3.07	297.50	0.040		<0.001	<0.001***	<0.001***	0.060***
	V1000	305.53	1.20	307.60	297.34	1.98	300.70	300.38	1.90	305.10	0.092		<0.001	<0.001***	<0.001***	<0.001***
	V1500	310.49	1.45	312.80	302.54	1.87	36.80	311.15	4.34	324.10	0.012		<0.001	<0.001***	0.431	<0.001***
	V2000	456.57	81.32	580.50	322.81	11.46	351.00	586.23	58.70	751.20	<0.001		<0.001	<0.001***	<0.001***	<0.001***
	V2500	962.90	138.40	1156.00	684.58	215.29	1120.00	969.19	106.67	1137.60	0.050		<0.001	<0.001***	0.850	<0.001***
Compliance 60% TRK20%	V0500	299.18	1.97	302.80	290.66	3.53	298.90	292.85	3.09	297.60	0.040		<0.001	<0.001***	<0.001***	0.0075***
	V1000	305.77	1.21	307.80	296.54	2.02	300.50	300.69	1.91	305.50	0.080		<0.001	<0.001***	<0.001***	<0.001***
	V1500	311.41	1.91	315.50	302.61	2.44	308.60	318.94	1.56	312.10	<0.001		<0.001	<0.001***	0.0038***	<0.001***
	V2000	543.86	113.30	720.50	343.92	30.55	412.20	670.88	146.60	948.10	<0.001		<0.001	<0.001***	<0.001***	<0.001***
	V2500	1008.48	135.00	1182.20	709.10	185.33	992.30	964.07	95.62	1108.80	<0.001		<0.001	<0.001***	0.130	<0.001***
Compliance 60% TRK30%	V0500	299.25	1.98	302.80	291.61	3.47	300.10	292.91	3.09	297.80	0.047		<0.001	<0.001***	<0.001***	0.100
	V1000	306.03	1.27	308.10	297.90	1.99	300.90	301.03	1.96	306.00	0.175	<0.001		<0.001***	<0.001***	<0.001***
	V1500	312.50	2.55	319.00	304.22	2.79	310.00	329.54	22.37	372.40	<0.001		<0.001	<0.001***	<0.001***	<0.001***
	V2000	626.32	147.85	908.30	397.68	56.54	518.70	754.25	169.44	1081.80	<0.001		<0.001	<0.001***	0.003***	<0.001***
	V2500	1011.33	132.83	1184.70	755.91	156.66	1241.80	966.95	103.94	1124.00	0.001		0.002	0.002***	0.120	<0.001***
Compliance 80% TRK10%	V0500	299.09	1.97	302.70	287.61	3.12	294.40	289.43	2.92	294.70	0.065		0.020	<0.001***	<0.001***	0.015***
	V1000	305.53	1.20	307.60	293.16	2.09	298.20	296.32	1.70	301.00	0.060		0.013	<0.001***	<0.001***	<0.001***
	V1500	310.49	1.45	312.80	298.72	2.17	303.60	308.07	4.66	322.10	0.017		<0.001	<0.001***	0.012	<0.001***
	V2000	456.57	81.32	580.50	316.65	11.76	347.90	587.89	109.76	770.80	<0.001		<0.001	<0.001***	<0.001***	<0.001***
	V2500	962.90	138.40	1156.00	599.16	149.73	889.10	933.23	76.85	1079.80	<0.001		<0.001	<0.001***	0.21	<0.001***
Compliance 80% TRK20%	V0500	299.18	1.97	302.80	287.76	3.19	295.40	289.55	2.91	294.70	0.060		0.001	<0.001***	<0.001***	0.020***
	V1000	305.77	1.21	307.80	293.33	2.05	297.90	296.66	1.75	301.80	0.120	,0.001		<0.001***	<0.001***	<0.001***
	V1500	311.41	1.91	315.50	299.55	2.21	304.70	313.75	11.24	348.00	0.016		0.012	<0.001***	0.25	<0.001***
	V2000	543.86	113.30	720.50	343.20	26.13	384.70	669.00	145.35	912.50	0.020		<0.001	<0.001***	<0.001***	<0.001***
	V2500	1008.48	135.00	1182.20	690.88	196.04	1195.70	925.02	62.69	1024.50	<0.001		<0.001	<0.001***	0.01***	<0.001***
Compliance 80% TRK30%	V0500	299.25	1.98	302.80	287.96	3.29	295.90	289.63	2.92	294.80	0.047		<0.001	<0.001***	<0.001***	0.030***
	V1000	306.03	1.27	308.10	293.50	1.99	298.70	297.01	1.77	302.10	0.195	<0.001	<0.001	<0.001***	<0.001***	<0.001***
	V1500	312.50	2.55	319.00	300.37	2.52	305.70	326.31	22.90	380.60	<0.001		<0.001	<0.001***	0.0017***	<0.001***
	V2000	626.32	147.85	908.30	391.27	62.23	506.60	746.35	154.69	1008.10	<0.001		<0.001	<0.001***	0.0032***	<0.001***
	V2500	1011.33	132.83	1184.70	734.32	157.70	1275.60	917.14	60.27	999.70	0.027		0.020	<0.001***	0.023***	<0.001***

7.5.4 Speed Variance

Spot Speed studies confirmed that higher speed variances have been correlated with higher crash rates (Garber and Gadiraju, 1989), and higher crash frequency (Taylor et al., 2000). In this section, the speed variance is used as a safety surrogate measure to evaluate the safety of the MOT types (early SDLMS, late SDLMS, and MAS) for each combination. Figure 7.5.4-a and 7.5.4-b show box plot of the speed distributions. For example the first chart of Figure 7.5.4-a shows the speed distributions for a compliance rate of 20% and a truck percentage of 10% under demand volume level V0500, V1000, V15000, V2000, V2500 for all three MOT types (early SDLMS, late SDLMS, and MAS). Box plots have the ability to graphically show obvious differences in variances among groups. For example, looking at the same chart of Figure 7.5.4-a, one can tell that there is a clear difference in speed variances under the MAS, early SDLMS, and late SDLMS, for the demand volume level of V2000. Under different demand volume levels it is not clear whether there is a difference or not.

The objective of this section is to determine for each combination of demand volume level, truck percentage, and compliance rate level, the MOT type that results in the lowest speed variance. Tables 7.5.4-a and 7.5.4-b provide the speed standard deviation values for each combination of compliance rate level, percentage trucks level in the traffic, traffic demand volume level under each MOT type (early SDLMS, late SDLMS, MAS). A Levene’s test is conducted for each combination with the null hypothesis that speed variances under the early SDLMS, late SDLMS, and MAS are homogenous (See Tables 7.5.4-a and 7.5.4-b).

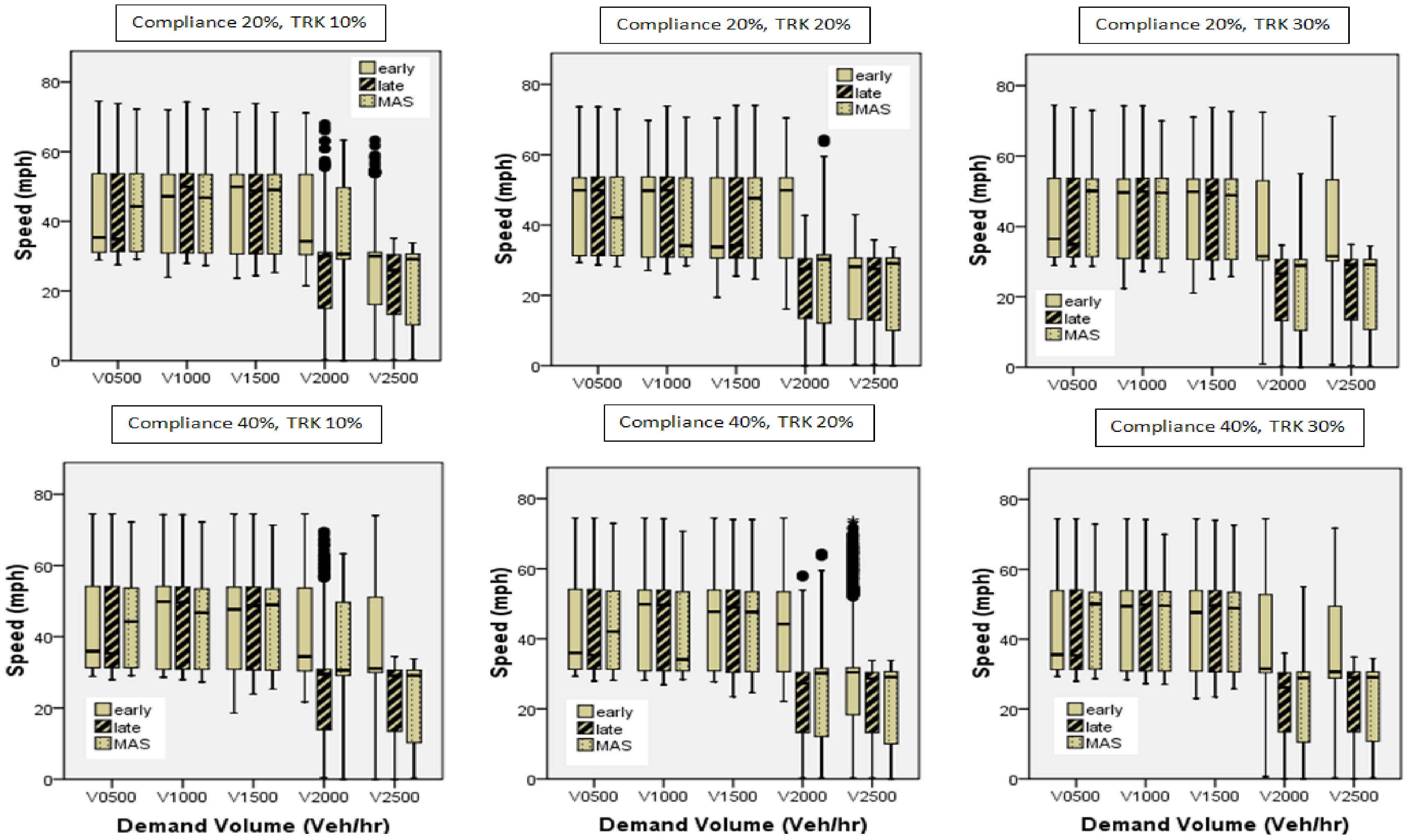


Figure 7.5.4-a: Speed distributions (C20, C40)

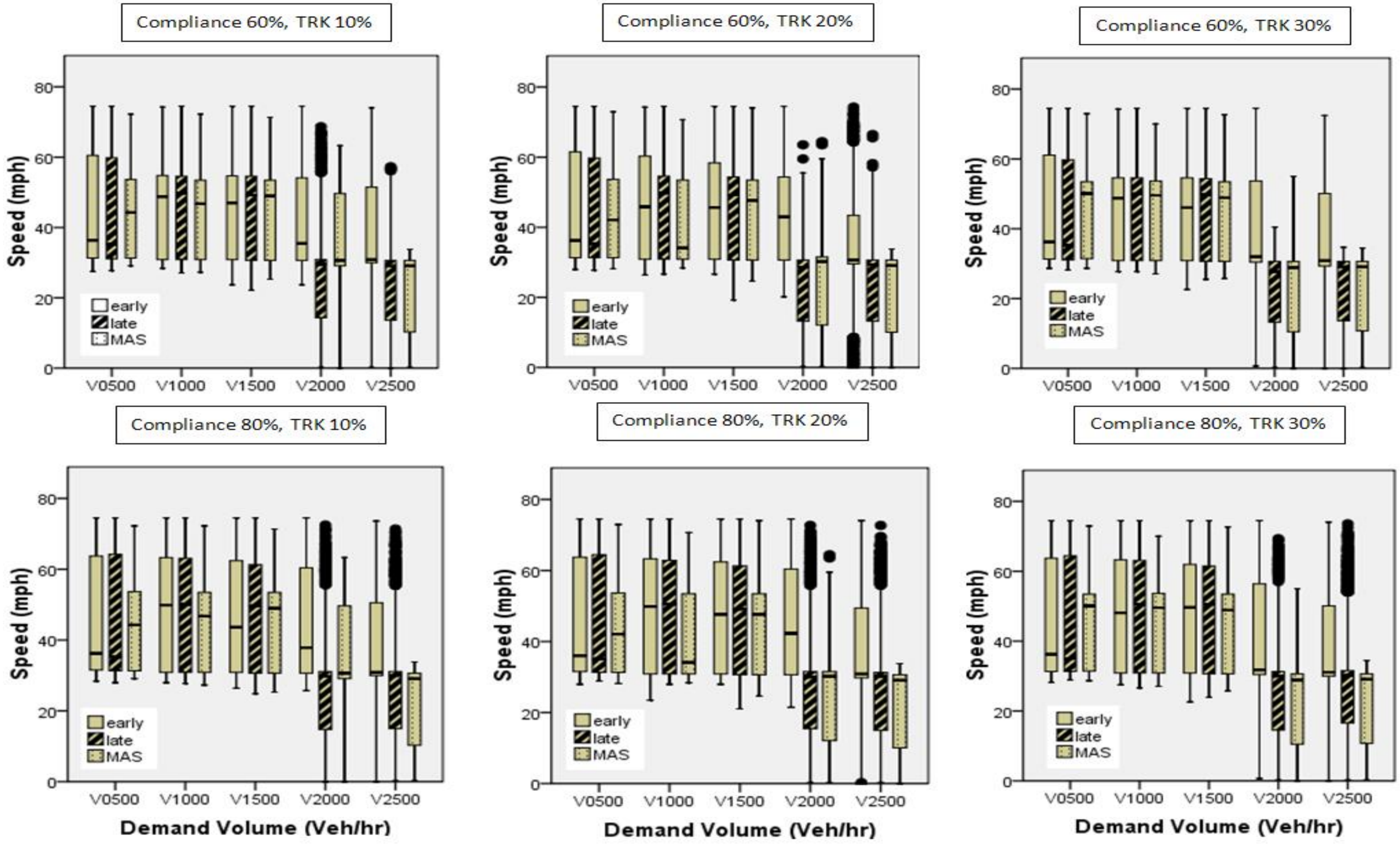
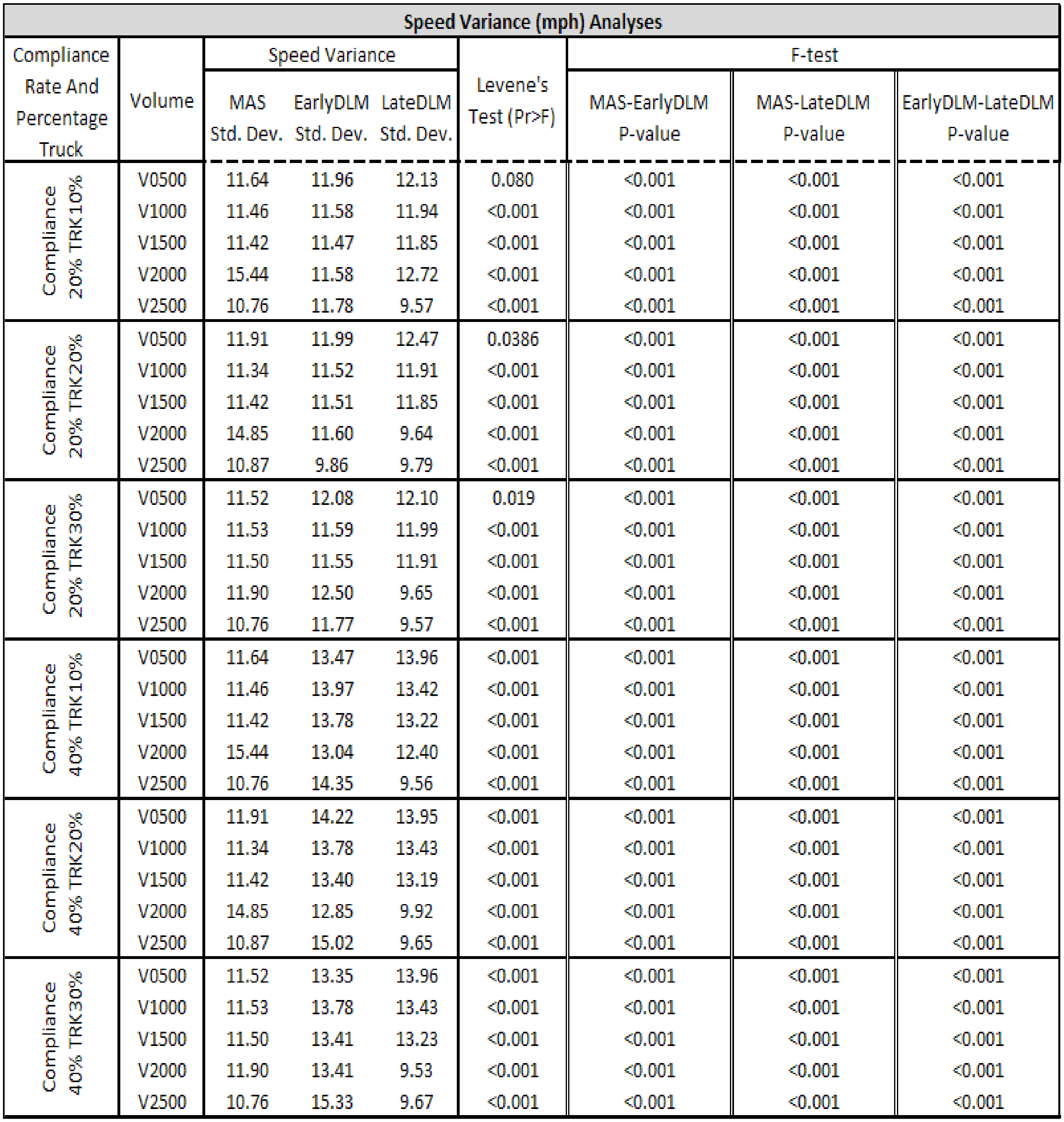


Figure 7.5.4-b: Speed distributions (C60, C80)

If the null hypothesis is rejected meaning at least one of the speed variances is different than the other two, pair wise F-tests are conducted to determine which speed variances are significantly different (early SDLMS Vs. late SDLMS; early SDLMS Vs. MAS; late SDLMS Vs. MAS). From Table 7.5.4-a it is shown that for compliance rates of 20% and trucks percentages of 10%, 20% and 30%, the speed variance under the late SDLMS is significantly higher than the speed variance under the early SDLMS and the MAS and that the speed variance under the early SDLMS is significantly higher than the speed variance under the MAS for demand volume levels of V0500, V1000, and V1500. For compliance rate of 20% and truck percentage of 10% it was found that the speed variance under the early SDLMS is significantly lower than the speed variance of the late SDLMS and the MAS and that the speed variance under the late SDLMS is significantly lower than MAS at demand volume level V2000. For compliance rate of 20% and truck percentage of 10% and demand volume level V2500, it was found the early SDLMS resulted in significantly the highest speed variance compared to the late SDLMS and the MAS and the late SDLMS resulted in the significantly lowest speed variance.

From Table 7.5.4-a, it is shown that for compliance rate of 40% and trucks percentages of 10%, 20%, and 30%, the speed variance under the MAS is significantly lower than the speed variance under the early SLDMS and late SDLMS for demand volume levels of V0500, V1000, and V1500. For a compliance rate of 40% and trucks percentages of 10% and 20%, it was shown the speed variance was significantly the highest for the MAS compared to the late SDLMS and early SDLMS for demand volume level of V2000.

Table 7.5.4-a: Speed Variance Comparisons (C20, C40)



Speed Variance (mph) Analyses								
Compliance Rate And Percentage Truck	Volume	Speed Variance			Levene's Test (Pr>F)	F-test		
		MAS Std. Dev.	EarlyDLM Std. Dev.	LateDLM Std. Dev.		MAS-EarlyDLM P-value	MAS-LateDLM P-value	EarlyDLM-LateDLM P-value
Compliance 20% TRK10%	V0500	11.64	11.96	12.13	0.080	<0.001	<0.001	<0.001
	V1000	11.46	11.58	11.94	<0.001	<0.001	<0.001	<0.001
	V1500	11.42	11.47	11.85	<0.001	<0.001	<0.001	<0.001
	V2000	15.44	11.58	12.72	<0.001	<0.001	<0.001	<0.001
	V2500	10.76	11.78	9.57	<0.001	<0.001	<0.001	<0.001
Compliance 20% TRK20%	V0500	11.91	11.99	12.47	0.0386	<0.001	<0.001	<0.001
	V1000	11.34	11.52	11.91	<0.001	<0.001	<0.001	<0.001
	V1500	11.42	11.51	11.85	<0.001	<0.001	<0.001	<0.001
	V2000	14.85	11.60	9.64	<0.001	<0.001	<0.001	<0.001
	V2500	10.87	9.86	9.79	<0.001	<0.001	<0.001	<0.001
Compliance 20% TRK30%	V0500	11.52	12.08	12.10	0.019	<0.001	<0.001	<0.001
	V1000	11.53	11.59	11.99	<0.001	<0.001	<0.001	<0.001
	V1500	11.50	11.55	11.91	<0.001	<0.001	<0.001	<0.001
	V2000	11.90	12.50	9.65	<0.001	<0.001	<0.001	<0.001
	V2500	10.76	11.77	9.57	<0.001	<0.001	<0.001	<0.001
Compliance 40% TRK10%	V0500	11.64	13.47	13.96	<0.001	<0.001	<0.001	<0.001
	V1000	11.46	13.97	13.42	<0.001	<0.001	<0.001	<0.001
	V1500	11.42	13.78	13.22	<0.001	<0.001	<0.001	<0.001
	V2000	15.44	13.04	12.40	<0.001	<0.001	<0.001	<0.001
	V2500	10.76	14.35	9.56	<0.001	<0.001	<0.001	<0.001
Compliance 40% TRK20%	V0500	11.91	14.22	13.95	<0.001	<0.001	<0.001	<0.001
	V1000	11.34	13.78	13.43	<0.001	<0.001	<0.001	<0.001
	V1500	11.42	13.40	13.19	<0.001	<0.001	<0.001	<0.001
	V2000	14.85	12.85	9.92	<0.001	<0.001	<0.001	<0.001
	V2500	10.87	15.02	9.65	<0.001	<0.001	<0.001	<0.001
Compliance 40% TRK30%	V0500	11.52	13.35	13.96	<0.001	<0.001	<0.001	<0.001
	V1000	11.53	13.78	13.43	<0.001	<0.001	<0.001	<0.001
	V1500	11.50	13.41	13.23	<0.001	<0.001	<0.001	<0.001
	V2000	11.90	13.41	9.53	<0.001	<0.001	<0.001	<0.001
	V2500	10.76	15.33	9.67	<0.001	<0.001	<0.001	<0.001

Table 7.5.4-b: Speed Variance Comparisons (C60, C80)

Speed Variance (mph) Analyses								
Compliance Rate And Percentage Truck	Volume	Speed Variance			Levene's Test (Pr>F)	F-test		
		MAS Std. Dev.	EarlyDLM Std. Dev.	LateDLM Std. Dev.		MAS-EarlyDLM P-value	MAS-LateDLM P-value	EarlyDLM-LateDLM P-value
Compliance 60% TRK10%	V0500	11.64	15.34	15.57	<0.001	<0.001	<0.001	<0.001
	V1000	11.46	15.15	14.85	<0.001	<0.001	<0.001	<0.001
	V1500	11.42	14.95	14.48	<0.001	<0.001	<0.001	<0.001
	V2000	15.44	14.11	11.96	<0.001	<0.001	<0.001	<0.001
	V2500	10.76	14.37	9.97	<0.001	<0.001	<0.001	<0.001
Compliance 60% TRK20%	V0500	11.91	15.81	15.55	<0.001	<0.001	<0.001	<0.001
	V1000	11.34	15.57	14.89	<0.001	<0.001	<0.001	<0.001
	V1500	11.42	15.19	14.56	<0.001	<0.001	<0.001	<0.001
	V2000	14.85	14.29	10.15	<0.001	<0.001	<0.001	<0.001
	V2500	10.87	14.84	10.45	<0.001	<0.001	<0.001	<0.001
Compliance 60% TRK30%	V0500	11.52	15.32	15.55	<0.001	<0.001	<0.001	<0.001
	V1000	11.53	15.14	14.83	<0.001	<0.001	<0.001	<0.001
	V1500	11.50	14.77	14.37	<0.001	<0.001	<0.001	<0.001
	V2000	11.90	14.41	9.69	<0.001	<0.001	<0.001	<0.001
	V2500	10.76	15.27	9.54	<0.001	<0.001	<0.001	<0.001
Compliance 80% TRK10%	V0500	11.64	16.71	16.92	<0.001	<0.001	<0.001	<0.001
	V1000	11.46	16.58	16.51	<0.001	<0.001	<0.001	<0.001
	V1500	11.42	16.23	16.06	<0.001	<0.001	<0.001	<0.001
	V2000	15.44	15.55	13.14	<0.001	<0.001	<0.001	<0.001
	V2500	10.76	15.20	12.99	<0.001	<0.001	<0.001	<0.001
Compliance 80% TRK20%	V0500	11.91	16.72	16.95	<0.001	<0.001	<0.001	<0.001
	V1000	11.34	16.62	16.52	<0.001	<0.001	<0.001	<0.001
	V1500	11.42	16.20	16.03	<0.001	<0.001	<0.001	<0.001
	V2000	14.85	15.52	14.54	<0.001	<0.001	<0.001	<0.001
	V2500	10.87	15.21	14.04	<0.001	<0.001	<0.001	<0.001
Compliance 80% TRK30%	V0500	11.52	16.69	16.98	<0.001	<0.001	<0.001	<0.001
	V1000	11.53	16.58	16.55	<0.001	<0.001	<0.001	<0.001
	V1500	11.50	16.22	16.03	<0.001	<0.001	<0.001	<0.001
	V2000	11.90	16.11	13.94	<0.001	<0.001	<0.001	<0.001
	V2500	10.76	15.35	14.68	<0.001	<0.001	<0.001	<0.001

However, for a demand volume level of V2500 the early SDLMs resulted in the highest speed variance compared to the MAS and late SDLMS. From Table 7.5.4-b, it is shown that for a compliance rate of 60% and trucks percentages of 10%, 20%, and 30% the MAS resulted in the lowest speed variances for demand volume levels V0500, V1000, and V1500. For a compliance rate of 60% and trucks percentages of 10%, 20%, and 30% the late SDLMS resulted in the lowest speed variances at demand volume levels of V2000, and V2500.

From Table 7.5.4-b, at compliance rate of 80% and trucks percentages of 10%, 20%, and 30%, the MAS seems to have the lowest speed variances at demand volume levels V0500, V1000, V1500, and V2500 compared to the early SDLMS and the late SDLMS. At the same compliance rate and trucks percentages of 10% and 20% the late SDLMS resulted in the lowest speed variances under demand volume level of V2000. For the trucks percentage of 30%, compliance rate of 80%, and demand volume level of V2000, the MAS resulted in the lowest speed variance.

7.6 Conclusions

The field study conducted on a two-to-one work zone lane closure configuration was limited to certain traffic demand level and to a certain motorists' adherence level to lane management instructions. Therefore, a simulated work zone model was created in VISSIM, calibrated and validated with the field data. The objective of this simulation study was to provide guidelines on the implementation of the early and late SDLMS on a two-to-one work zone lane closure configuration under different traffic demand volumes and different drivers' compliance rates to the messages displayed by the systems.

Safety and operational evaluation of the three MOT types tested, namely the MAS, the early SDLMS, and the late SDLMS was conducted. Table 6.6 below summarizes the safety and operational effectiveness of the three MOT types. In Table 6.6, the first three columns under operations summarize the throughputs for each combination of compliance rate, truck percentage in the traffic composition, and demand volume level. Only statistically significant results are presented in this table. For each combination the results were numbered 1, 2, and 3. One meaning that it is the best to use, 2 meaning second best to use, and 3 meaning the third best to use. The best MOT types to use, numbered 1, are highlighted in this table. For instance, one may want to know which MOT type is best for a work zone at a demand volume level of 1500 veh/hr, truck percentage of 20%, and compliance rate of 60%. The cells left blank in table 6.6 reflect no significant difference between the combinations. By looking at Table 6.6, one can tell that in terms of throughputs, there is no difference in the three MOT types. In terms of travel times through the work zone the early SDLMS would be the best choice. In terms of safety the MAS is the best choice.

Table 7.6: Summary of Operational and Safety MOEs

Compliance Rate And Percentage Truck	Volume	Operations						Safety		
		Throughputs			Travel Time			Speed Variance		
		MAS	EarlyDLM	LateDLM	MAS	EarlyDLM	LateDLM	MAS	EarlyDLM	LateDLM
C20% TRK10%	V0500							1	2	3
	V1000							1	2	3
	V1500							1	2	3
	V2000	2	1	3	2	1	3	3	1	2
	V2500	2	1	3	3	1	2	2	3	1
C20% TRK20%	V0500							1	2	3
	V1000							1	2	3
	V1500				1	1	2	1	2	3
	V2000	2	1	3	2	1	3	3	2	1
	V2500	2	1	3	2	1	2	3	2	1
C20% TRK30%	V0500							1	2	3
	V1000							1	2	3
	V1500				1	1	2	1	2	3
	V2000	2	1	3	2	1	2	2	3	1
	V2500	2	1	3	2	1	2	2	3	1
C40% TRK10%	V0500							1	2	3
	V1000							1	3	2
	V1500				2	1	3	1	3	2
	V2000	2	1	3	2	1	3	3	2	1
	V2500	2	1	3	2	1	2	2	3	1
C40% TRK20%	V0500				3	2	1	1	3	2
	V1000				3	1	2	1	3	2
	V1500				2	1	3	1	3	2
	V2000	2	1	3	2	1	3	3	2	1
	V2500	2	1	3	2	1	2	2	3	1
C40% TRK30%	V0500				2	1	1	1	2	3
	V1000				3	1	2	1	3	2
	V1500				2	1	3	1	3	2
	V2000	2	1	3	2	1	3	2	3	1
	V2500	2	1	3	3	1	2	2	3	1
C60% TRK10%	V0500				3	2	1	1	2	3
	V1000				3	2	1	1	3	2
	V1500				2	1	3	1	3	2
	V2000	2	1	3	2	1	3	3	2	1
	V2500	2	1	3	2	1	2	2	3	1
C60% TRK20%	V0500				3	1	2	1	3	2
	V1000				3	1	2	1	3	2
	V1500				2	1	3	1	3	2
	V2000	2	1	3	2	1	3	3	2	1
	V2500	2	1	3	2	1	2	3	2	1
C60% TRK30%	V0500				3	1	2	1	2	3
	V1000				3	1	2	1	3	2
	V1500				2	1	3	1	3	2
	V2000	2	1	3	2	1	3	2	3	1
	V2500	2	1	3	2	1	2	2	3	1
C80% TRK10%	V0500				3	1	2	1	2	3
	V1000				3	1	2	1	3	2
	V1500				3	1	2	1	3	2
	V2000	2	1	3	2	1	3	3	2	1
	V2500	2	1	3	2	1	2	1	3	2
C80% TRK20%	V0500				3	1	2	1	2	3
	V1000				3	2	1	1	3	2
	V1500				2	1	2	1	3	2
	V2000	2	1	3	2	1	3	2	3	1
	V2500	2	1	3	3	1	2	1	3	2
C80% TRK30%	V0500				3	1	2	1	2	3
	V1000				3	1	2	1	3	2
	V1500				2	1	3	1	3	2
	V2000	2	1	3	2	1	3	1	3	2
	V2500	2	1	3	3	1	2	1	3	2

<u>Key Code:</u>
Blank: No Significant difference
1: Best (highlighted), 3: Worst

The results obtained by this simulation study show that overall; the early SDLMS outperforms the MAS and late SDLMS to the exceptions of very few cases in terms of operations (i.e. throughputs and travel times). However, in terms of safety the early SDLMS performs poorly compared to the late SDLMS and the MAS. This fact, underlines the compromise between safety and operations of a two-to-one work zone lane closure. From the safety point of view, the late SDLMS performed well under higher volumes (2,000 veh/hr to 2,500 veh/hr) to the exception of higher compliance rates of 80% compared to the early SDLMS and MAS.

CHAPTER 8 CONCLUSIONS AND DISCUSSIONS

After investigating Fatality and Analysis Reporting System (FARS), it was found that Florida’s work zones fatalities are rising significantly compared to other states. Subsequently a Florida freeway work zone crash data analysis was conducted and crash traits were exposed. Results indicated the majority of freeways work zone crashes resulted from merging conflicts leading to rear-end and sideswipe crashes. The Florida Traffic Crash Records Database for years 2002, 2003 and 2004 were employed and statistical models were assembled to draw drivers/vehicles/ environment traits of work zone crashes. Results indicated that for the single-vehicle crashes, trucks are more likely to be involved in a work zone single-vehicle crash compared to trucks and large trucks in non-work zone locations. Straight-level has increased likelihood compared to straight-upgrade /downgrade, curve-level, and curve-upgrade/ downgrade. Lighting condition is also one of the risk factors associated with work zone single-vehicle crashes. In fact, results showed that work areas with poor or no lighting during dark, motor vehicles are more prone for crashes compared to non-work zone locations with poor or no lighting during dark. The weather condition is also associated with single-vehicle work zone crashes. In fact, during rainy weather, drivers are less likely to be involved in work zone crashes compared to the same weather conditions in non-work zone locations.

For the two-vehicle crashes, the second model’s results illustrate that drivers younger than 25 years old and drivers older than 75 years old have the highest risk to be the at-fault driver in a work zone crash. Male drivers have significantly higher risk than female drivers to be the at-fault driver. Results noticeably show that drivers under the influence of narcotics/alcohol are drastically more likely to cause crashes (i.e. at-fault driver) at work zones. Out-of-state drivers are slightly less likely to be the source (i.e. at-fault driver) of a work zone crash compared to local drivers. Road geometry and the lighting condition were significant risk factors for two-vehicle work zone crashes. Freeways straight segments are more susceptible to crashes in work zone areas. Poor lighting or no lighting at all during dark can lead to significantly higher crash hazard on work zones compared to non-work zones. Results also showed that foggy weather causes a significant mount in work zone crash risk compared to non-work zone locations. In addition to that, work zones located in rural areas have higher crashes potential than work zones located in urban areas.

After conducting the Florida work zone crash analyses and consulting with work zone practices in other States of the U.S., it was concluded that ITS lane management systems could be potential countermeasures worthy of implementation and testing on Florida's work zones. For instance, previous studies showed that dynamic early merging can smoothen the merging operation in advance of a lane closure (Tarko, 1998), decrease the rear-end accident rate (Tarko, 1998), and reduce the number of forced merges (Wayne State University, 2001). The dynamic late merging can reduce conflict points (or locations) to one single location at the taper of the work zone which enhances overall driving conditions upstream of work zone (Tavoola et al., 2004). Therefore, the early and late merging systems have the potential of improving the merging maneuvers in Florida's work zones especially for trucks. These systems can also reduce hostile driving that is overrepresented in Florida’s work zones by reducing random merging locations (at random locations) to a definite merging location.

An examination of the current Florida work zone Maintenance of Traffic (MOT) plans, known as the Motorist Awareness System (MAS) was conducted. It was realized that this system is static hence does not react to changing traffic conditions, and does not incorporate a lane management system. Therefore, an ITS-based lane management system, primarily designed to advise drivers on definite merging locations is suggested to supplement the existing Florida MOT plans (i.e. MAS) for short term work zones. Since, previously deployed dynamic lane merging systems comprise several PCMSs and traffic sensors and since the addition of multiple PCMSs to the current MAS plans may encumber the latter and require extensive time for installation on a daily basis, two SDLMS were designed and tested at two sites. The first SDLMS is a simplified dynamic early merging system and the second SDLSM is a simplified dynamic late merging system.

The first work zone configuration was a freeway two-to-one lane closure. The throughput over the demand volume of the work zone was used as a measure of effectiveness to explore the impact of the early and late SDLMS on work zones. Results showed that the early SDLMS enhances work zone mean throughput over demand volume significantly. However, the late form of SDLMS increased the mean throughput over demand volume slightly compared to the MAS, and this increase was not statistically significant. The average travel time for the MAS, early and late SDMLS did not result in statistically significant differences between the mean travel times. It was also noted that the sample size of the data was limited which warranted a simulation study. Results from the first site also indicated that the number and percentage of lane changes in zone 1 were the highest for the early SDLMS and the lowest for the late SDLMS. This indicates that some drivers were complying with the messages displayed by the additional PCMS.

The second work zone configuration was a freeway three-to-two lane closure. The temporal speed fluctuation at the location of the RTMS of the work zone under the control (MAS) and test MOT plans (early and late SDLMS) were compared. The mean speed fluctuation in the closed lane was the highest under the MAS system for all demand volumes. The dynamic late merge and the dynamic early merge have lower speed fluctuations in the closed lane under all demand volumes compared to the MAS system. Comparing the dynamic early merge and the dynamic late merge mean speed fluctuations in the closed lane, results showed that the mean speed fluctuation for the early merge are lower than those of the late merge under all demand volumes. However, the difference in the mean speed fluctuation is only statistically significant under demand volume ranging between 1 and 500 veh/hr. Results showed that the speed fluctuations in the middle lane are the highest for the MAS system compared to dynamic early merge and dynamic late merge under all demand volumes. However, results showed that the mean speed fluctuations under the MAS are significantly higher than the mean speed fluctuations under the dynamic late merge only for volumes greater than 1500 veh/hr (and marginally at volumes between 1001 and 1500 veh/hr). The mean speed fluctuations under the MAS are significantly higher than the mean speed fluctuations under the dynamic early merge system for volumes ranging between 501 and 1500 veh/hr. Comparing the mean speed fluctuations under the dynamic early merge and the dynamic late merge, it was found that the mean speed fluctuations are lower for the dynamic early merge. However, there was no significant difference between the speed fluctuations in the middle lane. Looking at the speed fluctuations in the shoulder lane, the mean speed fluctuations are the highest under the MAS system compared to the dynamic early merge and the dynamic late merge under all volumes. The mean speed fluctuations for the MAS system is significantly higher than the mean speed fluctuation for dynamic early and dynamic late merge for volumes under 1000 veh/hr. Moreover, there exist a marginal significance indicating that the mean speed fluctuation for the late merge is lower than the mean speed fluctuation for the MAS system for volumes ranging 1001 veh/hr to 2000 veh/hr. Comparing the mean speed fluctuations between the dynamic early and dynamic late merge, it was noted that the means speed fluctuations are lower for the dynamic late merge under volumes higher than 500 veh/hr. However, it was shown that the mean speed fluctuation for the dynamic late merge is significantly lower than the mean speed fluctuation for the dynamic early merge for demand volumes ranging between 1501 veh/hr and 2000 veh/hr.

The ratio of the throughput over demand volume was taken as the operational MOE. Results showed that the Dynamic early merge performs significantly better than the regular MAS under demand volume ranging between 500 veh/hr and 2000 veh/hr. Results also showed that the dynamic late merge perform better than the MAS under volumes ranging between 1500 veh/hr and 2000 veh/hr and significantly poorer than the

MAS under low volumes. Therefore, the late SDLMS is not recommended for implementation under low volumes. Results also showed that the late SDLMS performs better than the early SDLMS under higher volume (ranging between 1500 veh/hr to 2000 veh/hr).

The field study conducted on a two-to-one work zone lane closure configuration was limited to certain traffic demand level and to a certain motorists' adherence level to lane management instructions. Therefore, a simulated work zone model was created in VISSIM, calibrated and validated with the field data. The objective of this simulation study was to provide guidelines on the implementation of the early and late SDLMS on a two-to-one work zone lane closure configuration under different traffic demand volumes and different drivers' compliance rates to the messages displayed by the systems.

Safety and operational evaluation of the three MOT types tested, namely the MAS, the early SDLMS, and the late SDLMS was conducted. A table that summarizes the safety and operational effectiveness of the three MOT types was developed. In this table, the first three columns under operations summarize the throughputs for each combination of compliance rate, truck percentage in the traffic composition, and demand volume level. Only statistically significant results are presented in this table. For each combination the results were numbered 1, 2, and 3. One meaning that it is the best to use, 2 meaning second best to use, and 3 meaning the third best to use. The best MOT types to use, numbered 1, are highlighted in this table.

The results obtained by this simulation study show that overall; the early SDLMS outperforms the MAS and late SDLMS to the exceptions of very few cases in terms of operations (i.e. throughputs and travel times). However, in terms of safety the early SDLMS performs poorly compared to the late SDLMS and the MAS. This fact, underlines the trade-off between safety and operations of a two-to-one work zone lane closure. From the safety point of view, the late SDLMS performed well under higher volumes (2,000 veh/hr to 2,500 veh/hr) to the exception of higher compliance rates of 80% compared to the early SDLMS and MAS.

Future research may on simulating the three-to-two work zone lane closure and determining the safety and operational effectiveness of the early SDLMS, late SDLMS and MAS under different traffic demand volume levels, different motorists' adherence level to lane management instructions, and different trucks percentages in the traffic composition.

Future research may also focus on studying the safety of the different MOT types using different safety surrogate measure such as deceleration rates and time to collision at different locations in the work zone.

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