2016

Modeling Older Driver Behavior on Freeway Merging Ramps

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by

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A thesis submitted to the School of Civil Engineering

in partial fulfillment of the requirements for the degree of

Master of Science in Civil Engineering

UNIVERSITY OF NORTH FLORIDA

COLLEGE OF COMPUTING, ENGINEERING, AND CONSTRUCTION

August, 2016

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ACKNOWLEDGMENTS

Foremost, I would like to express my gratitude to my supervisor, Dr. Thobias Sando, whose expertise, understanding, and patience, added considerably to my accomplishments throughout my graduate program. Also I would like to thank all the people who supported and contributed in some way to my thesis work.

This project was supported by United States Department of Transportation grant DTRT13-G-UTC42, and administered by the Center for Accessibility and Safety for an Aging Population (ASAP) at the Florida State University (FSU), Florida A&M University (FAMU), and University of North Florida (UNF). We also thank the Florida Department of Transportation for providing the roadway data. The opinions, results, and findings expressed in this manuscript are those of the authors and do not necessarily represent the views of the United States Department of Transportation, The Florida Department of Transportation, The Center for Accessibility and Safety for an Aging Population, the Florida State University, the Florida A&M University, or the University of North Florida.

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LIST OF ACRONYMS

DOT	Department of Transportation
DMC	Dynamic Merge Control
DUI	Driving Under the Influence
FHWA	Federal Highway Administration
HCM	Highway Capacity Manual
HGV	Heavy Goods Vehicle
LOS	Level of Service
MOE	Measure of Effectiveness
NHTSA	National Highway Traffic Safety Administration

ABSTRACT

Merging from on-ramps to mainline traffic is one of the most challenging driving maneuvers on freeways. The challenges are further heightened for older drivers, as they are known to have longer perception-reaction times, larger acceptance gaps, and slower acceleration rates. In this research, VISSIM, a microscopic traffic simulation software, was used to evaluate the influence of the aging drivers on the operations of a typical diamond interchange. First, drivers were recorded on video cameras as they negotiated joining the mainline traffic from an on-ramp acceleration lane at two sites along I-75 in Southwest Florida. Several measures of effectiveness were collected including speeds, gaps, and location of entry to the mainline lanes. This information was used as either model input or for verification purposes. Two VISSIM models were developed for each site – one for the existing conditions and verification, and another for a sensitivity analysis, varying the percentage of older drivers and Level of Service (from A to E), to determine their influence on ramp operational characteristics. According to the results, there was a significant difference in driving behavior between older, middle-aged, and younger drivers, based on the measures of effectiveness analyzed in this study. Additionally, as the level of service and percentage of older adult motorists increased, longer queues were observed with slower speeds on the acceleration lanes and the right-most travel lane of the mainline traffic.

Keywords: merging, ramps, older adult drivers, young drivers, middle age drivers

CHAPTER 1 INTRODUCTION

Background

The United States is considered an aging society. According to a National Highway Traffic Safety Administration (NHTSA) report of July 2012, senior adults (65 and older) constituted 13% of the U.S. resident population (40.4 million people) in 2010 (NHTSA, 2012). The aging population, 65 and older, also referred to as 65+, is anticipated to increase to 83.7 million by 2050, which is double the estimated aging population of 43.1 million in 2012, and approximately a 135% increase from the year 2000 (Ortman, Velkoff, & Hogan, 2014). This is partly due to the baby-boom generation that will be over 85 years of age by 2050 (Orthman & Velkoff, 2014). Baby boomers are referred to people born during the demographic post-World War II era, approximately 18 years within the period of 1946 and 1964. A number of baby boomers turned age 65 in 2011, and this number will significantly increase in the near future (Ortman, Velkoff, & Hogan, 2014). The Bureau of Labor Statistics in the U.S. indicates that the labor force participation rates for the senior population working beyond retirement age increased by 19.7% in 2014, which means more aging drivers on the highways (Holder & Clark, 2008). Based on the data reported in 2013 by the Federal Highway Administration (FHWA), the number of licensed older adult drivers age 70 and older is increasing and accounts for 11% of the driving age population in the U.S (IIHS, 2016). Therefore, there is a need to focus more on aging-driver behavior, especially in stressful and challenging areas such as merging ramps, to avoid conflicts or crashes and to better manage traffic on freeways.

Florida is among the states with a significant number of the older residents, mainly due to migration, and the population of this age group is increasing faster compared to other states. The population in Florida has increased by about 17.3% from the year 2000 with the elder population comprising about 18.2% of the state population (U.S. Census Bureau, 2014). From 2010 to 2030, a population growth of 5 million is expected in Florida, with approximately 55.2% of the gain expected to be older adults. Additionally, Florida has the second largest population of older adult licensed drivers, which is almost 20% of all drivers. Consequently, the number of aging drivers and corresponding crash rates increase each year on Florida roadways. Florida traffic crash statistics reported an increase of 11.3 % in crashes involving aging drivers from 2008 to 2012 (The Florida Legislature Office of Economic Demograhic Research, 2014). Unfortunately, fatality crashes among aging drivers are also on the rise. Just in 2008 alone, 447 aging drivers, nearly 15 % of all vehicle-related fatalities, were killed in crashes. An increase of more than 2 % (from 18.3% to 20.6%) was observed in fatal crashes within a two-year period from 2007 to 2009 (Florida Department of Highway Safety and Motor Vehicles, 2013).

As age increases, driver performance involving motor skills deteriorates significantly, one example being additional time needed to respond to a stimulus (Klavora & Heslegrave, 2002). In this case, older drivers try to compensate by driving almost 20 percent slower than the posted speed limit (Staplin & Lyles, 1991). Freeways can pose challenges to aging drivers due to the higher posted speed limit compared to arterial and collector roads. Merging into the mainline traffic is one of the most challenging driving maneuvers on the freeways, and freeway traffic is often heavy and moving fast. The pressure of adjusting speeds to match the mainline traffic speeds, especially on shorter acceleration lanes, is challenging for drivers, especially if an eager driver in a following vehicle wants to merge quickly and is following too close. Therefore, the rate of conflicts among vehicles is higher at freeway on-ramps, mostly due to the existing differential speeds between the mainline and merging traffic. Moreover, these challenges are further heightened for older drivers who have longer perception-reaction times, larger acceptance gaps, and are recognized to have slower acceleration rates due to cognitive, behavioral, and health limitations (Bélanger, Gagnon, & Yamin , 2010). The merging behavior of aging drivers at freeway on-ramps needs to be evaluated to quantify the differences between the aging driver population and younger drivers to better understand how the operational characteristics of such facilities are affected by the interaction of different driver age groups.

Study Objectives

With motivation from Florida's aging population and their special needs for transportation, and central to meeting the challenges of aging drivers, this study has three distinctive objectives:

- To quantify the effects of age on merging behavior at freeway on-ramps. In particular, the study examines three merging behaviors: approaching speed, merging location, and gap acceptance.
- To conduct sensitivity analysis by creating several scenarios, varying traffic inputs in terms of total volume and the percentage of aging drivers in the system.
- To develop charts that exhibit the relationship between the percentage of aging drivers and traffic performance measures such as speed and merging location.

Potential Study Benefits

The findings of this research will potentially benefit various transportation stakeholders. For example, measures of performance such as merging speeds and gap acceptance can inform transportation agencies on appropriate values to use either as default or thresholds when creating transportation network simulation models for areas with a predominantly aging population. Furthermore, agencies responsible for planning and design of transportation facilities can use the findings of this study in improving design considerations with the focus on older drivers, especially at locations known to have relatively higher aging populations.

With the advent of connected and autonomous vehicle technology, the understanding of critical gaps for different age groups can help with strategies for lane change assist, among other initiatives, to improve safety and mobility at merging ramps.

Traffic simulation software developers would also benefit from incorporating speed differentials and gap acceptance results into their models. For example, values for younger, middle-aged, and older adult drivers can be used as default values for aggressive, moderate, and the least aggressive drivers.

No similar study was found in the literature that combines a field observational study with traffic simulation to examine merging behavior at freeway on-ramps. With the combination of both field data and simulation makes this study have a significant contribution to the research community.

Organization of the Manuscript

The next chapter, Chapter 2, reviews previous studies on the design and operation of merging sections while incorporating older adult driver difficulties, and studies on traffic simulation tools. Chapter 3 gives information on the existing geometric and traffic characteristics of the selected field sites, and the data collection and reduction techniques used in the study. Chapter 4 follows with the analysis of merging vehicles for the field data collected, taking into account the merging location, the age of the drivers, gaps offered, and the approach speed of vehicles from the ramp to the merging section. Critical gaps were determined, and the gap choice was analyzed. In Chapter 5, a sensitivity study was conducted using VISSIM modeling software based on varying traffic volumes and percentages of older adult drivers. This manuscript ends in Chapter 6 where the findings, limitation and proposed recommendation from this study are elaborated.

CHAPTER 2 LITERATURE REVIEW

This chapter is a synthesis of published literature on various topics encompassed in the study. The previous research and other published literature are summarized in specific topics, including the design aspects of the merging ramps, aging driver limitations, merging control strategies, and traffic simulation issues.

Design Aspects of Freeway On-Ramps and Merging Maneuvers

Taper and parallel ramp designs are the two general forms used in freeways design. The parallel design is commonly adopted by most State Departments of Transportation (DOTs) for the entrance ramp (on-ramp), while the taper design is preferred for the exit ramps, or off-ramps (AASHTO, 2004).

When a taper design is used for an entrance ramp, the design standard used is either a 50:1 or 70:1 taper rate when joining the upstream traffic between the outer edge of acceleration lane and the edge of the through-lane traffic stream (AASHTO, 2004). Taper design requires enough distance for drivers to adjust their speeds and be within a 5 mph range of the freeway mainline traffic before merging to avoid a disturbance to mainline traffic and prevent conflicts (Darren, et al., 2012). Furthermore, a taper connection is endorsed for areas with low traffic volume (such as rural areas) and higher speed roadways with posted speed limits above 65 mph (FHWA, 2012; Heavey, 2014).

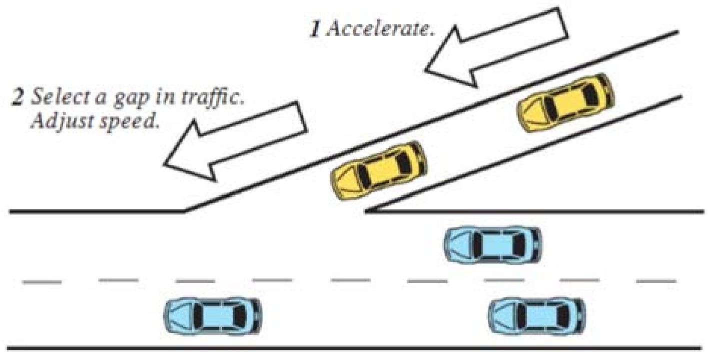


Figure 2.1. A typical freeway merging area with taper connection.

Parallel entrance design is typically preferred by State DOTs and recommended by FHWA to be used at newly built interchanges, mainly due to the safety and operational benefits. (FHWA, 2012). The use of parallel design is recommended for areas with moderate to heavy traffic conditions occurring during peak traffic hours, or in areas that have a potential for frequent traffic delays. Parallel design is also preferred where there are large volumes of trucks entering the highway (greater than 10%), and the mainline traffic speed is 60 mph or less (FHWA, 2012; Heavey, 2014).

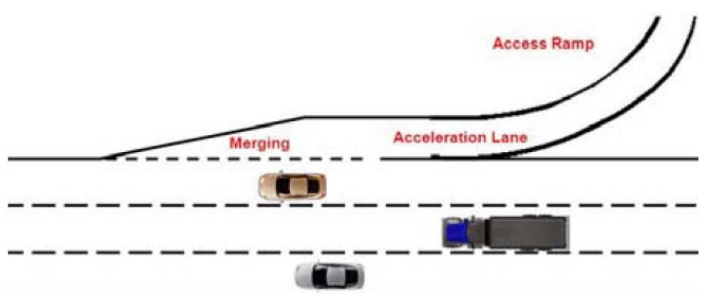


Figure 2.2. A typical freeway merging area with parallel connection.

To attain a smooth flow of traffic, the length of an acceleration lane has to be sufficient to allow adequate gaps for merging maneuvers and to minimize spillback (queuing back to the side street) for the ramp vehicles (AASHTO, 2004). An acceleration lane of adequate length helps ramp drivers with both the gap acceptance decision and the speed change process before joining the mainline traffic. Longer acceleration lanes are generally safer than shorter lanes as reported by Cirillo (1970) and cited by Schurr and Townsend (2010). A study by Dung (2014) reported similar findings. The acceleration lane length also influences the rate of gap acceptance and the selection of the gap sizes for ramp vehicles. The longer the acceleration lane the lower the chance of selecting shorter gaps, and more time is available for merging drivers to choose comfortable gaps (Kita, 1993; Ozbay, Yang, Bartin, & Mudigonda, 2008).

Gap Acceptance for Freeway Merging Ramps

A few studies have reported the implication that gap acceptance behavior has on lane changing maneuvers, which is a significant element of microscopic simulation, i.e., simulation that considers vehicle by vehicle data. In traffic simulation models, different vehicle types are assigned varying characteristics. For example, Jones et.al (2004) suggested that vehicles driven by different age groups could be assigned different values for the minimum deceleration rate, minimum acceptable gap, and the minimum distance required by vehicles to change lanes. These values are used as thresholds in the decision for lane change. For freeway merging sections, for example, Bham (2009) reported a mean value ranging from 1.47 to 1.82 seconds for lead gaps (ahead of mainline traffic), and for lag gaps (behind mainline traffic), the value ranged from 1.58 to 1.82 seconds.

In microscopic simulation, accurate modeling of lane changing behavior is necessary to achieve better results. It has been shown that the lane-changing maneuver may result in unrealistic delays, and the duration of the lane changing process differs among vehicles, such as passenger cars versus trucks (Coifman, Mishalan, Wang, & Krishnamurthy, 2006). Generally, the lane change period ranges from 1.0 to 3.3 seconds and usually follows a specific distribution (Toledo & Zohar, 2007; Thiemann, Treiber, & Kesting, 2008).

Older Adult Driving Behavior and their Difficulties

Over the years, numerous studies have been completed regarding older driver difficulties with specific maneuvers on roadways. There is ample literature reported by various agencies that advocate for the health and safety of older drivers, and show how functions such as eyesight, physical strength, hearing, finely tuned coordination, and the ability to adapt to abrupt changes in body position, decline at an older age. (National Institute on Aging, 2015; DaCoTa, 2013). Various studies such as Ulleberg & Sagberg (2003) have documented a correlation with crash risk and age-related visual and cognitive function impairment. According to the study, older drivers need longer reaction times compared to their younger counterparts. This need is mostly what makes older drivers drive slower. Consistent with the above observations, findings from a study done by Myunghoon, Byung-jung, & Wu (2014) indicated that older drivers are more likely to commit cognitive and decision-making errors due to the decline of physical and cognitive skills, whereas younger drivers are more likely to commit violation errors, such as excessive speeding, driving under the influence (DUI), and driving action errors.

The difficulty faced by senior drivers in accepting gaps, judging the approximate time for collision, and slower response rates compared to younger drivers is well documented (Staplin &

Lyles, 1991; Chandraratna & Stamatiadis, 2003). The study conducted in Kentucky by Chandraratna & Stamatiadis (2003) concluded that older adult drivers have difficulties with high-speed lane changes (in the merging area) and left turn movements against oncoming traffic at intersections. Similarly, Huey (1995) reported that the decision sight distance increased with age at intersections and recommended that this factor should be considered in the design process.

Merging Control Strategies

Different management control strategies have been developed since the 1960s to reduce traffic conflicts and congestion at ramp sections (Pearson , Black, & Wanat, 2001). For instance, in a strategy known as ramp metering, traffic signals are installed along the ramp. Ramp vehicles receive a green light indication only when sufficient gaps exist in the mainline traffic stream. Among the benefits associated with this strategy are reduction of accidents, improved freeway flow and increased capacity, travel time reduction, and the increase in travel speed on the freeway. Different ramp metering control algorithms have been developed since their inception, and various simulation models are used to determine the most optimal algorithm based on prevailing conditions. Most of these control algorithms are based on fixed time, local control, and system-wide control (Zhang, Kim, Nie, & Jin, 2001).

Apart from conventional ramp metering, in Germany and the Netherlands, another unique strategy is used to assist merging traffic. In this approach, the right-most lane of the mainline is closed upstream to provide merging vehicles with a free-flow condition onto the mainline. This approach is referred to as Dynamic Merge Control (DMC). The strategy prioritizes high-volume roadways (ramps) by dropping a lane on the lower-volume roadway (in this case, the mainline) and is best applied when the merging areas involve a lane drop (Jiang, Bared, Maness, & Hale, 2015). According to findings by Jiang et al. (2015), this strategy is beneficial in regard to average vehicle delay and speed. Similarly, the benefits are more significant when the minor road (mainline) capacity is higher. Additionally, it reduces the problem of capacity loss generated by lane changing in the merging areas.

Traffic Simulation Models

Traffic simulation, also known as simulation of transportation systems, involves mathematical modeling of transportation networks by creating models using various computer software for predicting, planning, and designing operational aspects of transportation systems. Generally, traffic simulation models can be classified as microscopic, macroscopic, and mesoscopic models (Boxill & Yu, 2000; Haleem, 2007). Traffic simulation models has been used since the 1950s when computers were first developed (Pursula, 1999).

For detailed analyses of freeway ramp operations, microscopic simulation models are used, where characteristics of individual vehicles and their interaction with other traffic streams can be evaluated. On the other hand, macroscopic simulation models analyze traffic streams in less detail, and are restricted to the description of vehicle dynamics in terms of density and average velocity on freeways (Helbing, Hennecke, Shvetsov, & Treiber, 2002; Mathew, 2014). Mesoscopic simulation models analyze with a much higher details than macro-simulation but lower than the micro-simulation (Mathew, 2014). Many computer programs have been used in freeway ramp operation studies, such as VISSIM, CORSIM, PARAMICS, SimTraffic, Aimsun, and SUMO. Some researchers have also used driving simulators to study the cooperative merging behavior of drivers to determine ways of reducing the “bottleneck” effect that exists at the merging point (Majid, Masao, & Avishai, 2004).

Since the objective of this research was to determine the merging behavior for younger, middle-aged, and older adult drivers on freeway ramps, the merging behavior was related to vehicular interaction. Therefore, a microscopic simulation model was the suitable choice for analyses conducted in this research. The type of modeling such as vehicle-to-vehicle communication included in this research, which delineates the position and velocity of each vehicle in the model, also helps in the choice of which traffic simulation tool to use. VISSIM, CORSIM, and PARAMICS are stochastic software package and considered by many to be suitable for freeway modeling. For this research, VISSIM software was used for micro-simulation modeling due to its versatility and customization ability.

Findings from Literature Review

The aforementioned difficulties experienced by older adult drivers presented above provide evidence that a need for continued research on driving behavior exhibited by older drivers still exists, especially with the expected increase in aging populations in locations such as Florida. To date, driving behavior studies on freeway merging ramps have not focused on age and gender. Hence a need exists to evaluate age implications, specifically older drivers, on the merging behavior for freeway ramps.

CHAPTER 3 DATA COLLECTION

Methodology

A successful field data collection exercise requires prior planning, organization, and timely execution, to obtain desired outcomes. The data collection methodology was devised based on literature review findings and engineering knowledge. Figure 3.1 illustrates the data collection procedure that was employed in this research. The following sections within this chapter provide a summary of each of the steps depicted in Figure 3.1.

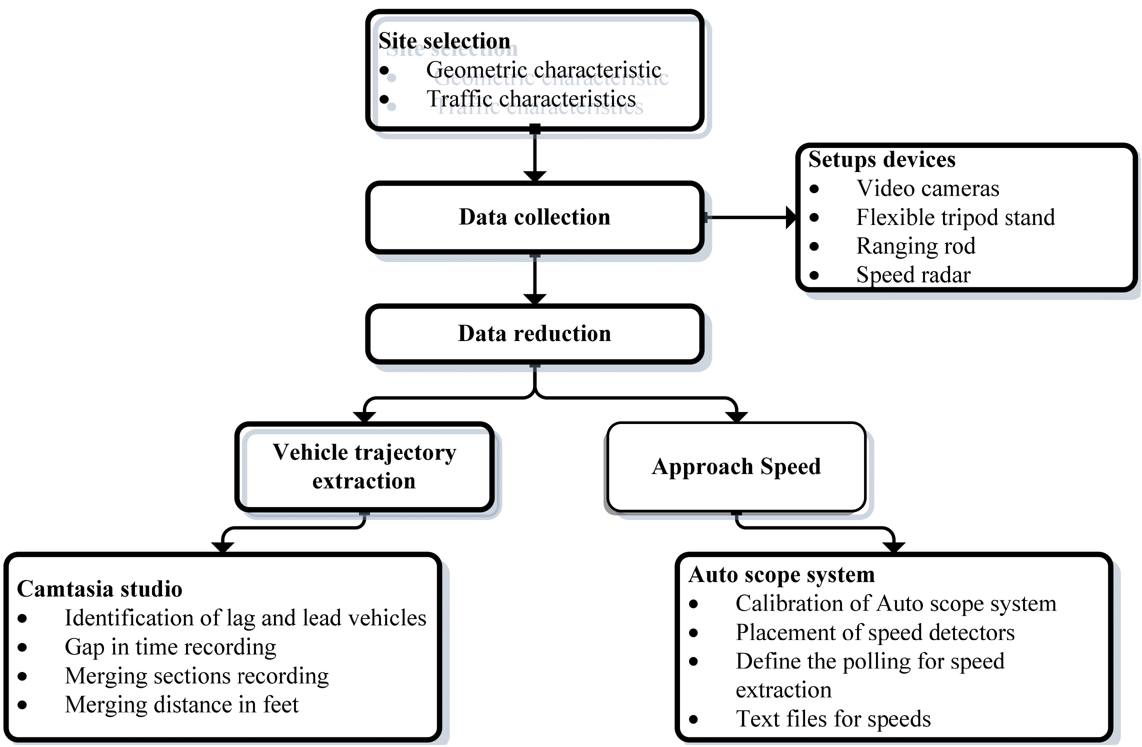


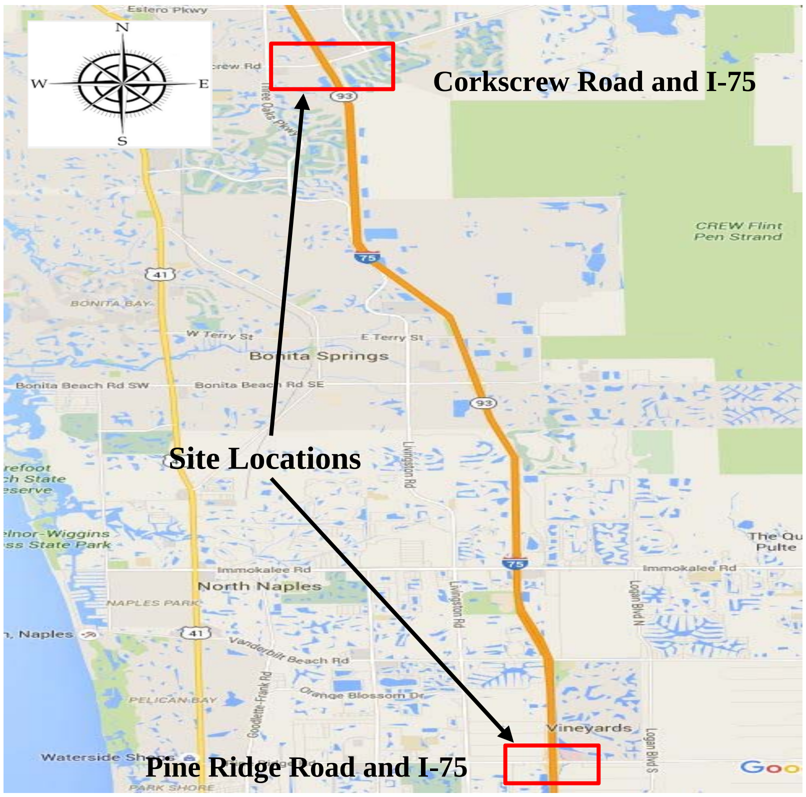
Figure 3.1. Conceptual data collection flowchart.

Site Selection

This study has a special focus on aging drivers. Hence, appropriate study site locations had to contain a significant percentage of aging drivers (65+). Furthermore, not all freeways on-ramps have a merging lane, also known as an acceleration lane. For some ramps, the ramp lane is continuous, adding to the number of mainline through-lanes. In order to investigate driver merging behavior, a study site had to consist of an acceleration lane that ends, keeping the number of mainline lanes the same. In light of the above two requirements, i.e., presence of significant proportion of aging drivers, and presence of a merging lane, none of the local sites in North Florida qualified. Therefore, the Fort Myers area in Lee County was considered due to the high aging population. The 2010 census indicates that in Lee County, Florida, the average age is 45.6 years, higher than the average age of the entire State (40.3 years), thus reflecting a high presence of aging drivers on surroundings highways (U.S. Census Bureau, 2014). Additionally, older people are reported to make up 23.5% of the population in Lee County, which is again higher when compared with the statewide population of 17.4% of age 65 or older. In order to qualify for data collection, the ideal study site had to have:

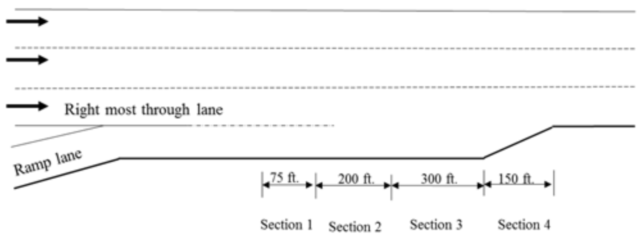
1. Older living communities or attractions such as golf clubs, pharmacies, hospitals etc., near the site;
2. Parallel connection entrance with a lane drop for the merging section;
3. Utility poles or trees adjacent for setting up the video cameras to collect data;

Two sites in Lee County, Fort Myers, Florida, were selected based on the recommendations received late February 2016 from Florida Department of Transportation (FDOT) traffic staff in Lee County. Figure 3.2 shows a location map of the two selected sites. Both sites are 6-lane divided highway sections along I-75, where the mainline has three lanes, each lane 12 feet in width, in both directions, and one lane for the entrance ramp of the same width. The acceleration lane for the entrance ramp at Corkscrew Road has a length of 1000 feet, shorter in length than that of the Pine Ridge Road on-ramp (approximately 1500 feet). Figure 3.3 shows a schematic diagram of each study site.

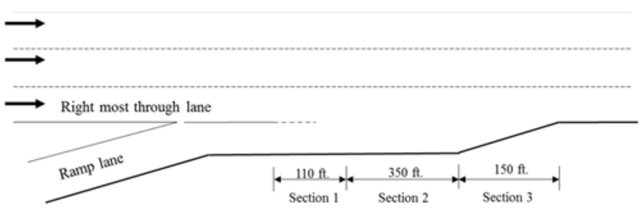


Source: Google Earth, 2016 (not to scale)

Figure 3.2. Site Location Map – I-75 On-ramps at Corkscrew and Pine Ridge Roads.



(a) I-75 at Corkscrew Road



(b) I-75 at Pine Ridge Road

Figure 3.3. On-ramp geometrics at Corkscrew and Pine Ridge Roads.

Study Variables

Various geometric, traffic, and driver characteristics were collected. Geometric variables included the length of acceleration lane for each site, and were measured using Google Earth software prior to the initial data collection site visit (Google Earth, 2016). While collecting data on-site, the acceleration lanes were divided into sections of different lengths to be discussed later in this Chapter. Two key traffic variables collected in this study were the merging speeds and gaps, as well as, whether gaps were accepted or rejected. In addition, traffic volumes, both for the mainline and merging ramps were collected. Due to the focus on aging drivers, it was important to identify merging driver age groups. Hence, age was a key study variable. The following sections provide a thorough discussion of how each study variable was collected.

Data Collection Duration

Each site location was in the vicinity of a number of older adult attractions' venues, including several churches and golf courses. Due to the presence of such features, both the mainline and ramps contained a significant number of aging drivers. As it has been reported in previous studies, including Bruff and Evans (1999), older adult road users tend to be on the road during daytime off-peak hours, typically from 9:00 a.m. to 3:00 p.m., thereby avoiding morning and evening rush-hour traffic. Hence, as shown on Table 3.1 for this study, the data collection exercise was scheduled during off-peak hours from 11 a.m. to 4 p.m., and occurred on consecutive weekdays during the months of March and May 2016, for a total of four days.

Table 3.1

Video Survey Periods and Traffic Conditions for The Merging Ramps and Mainline

Entrance Ramps	Survey Date	Survey Time	Driver composition (%)			Ramp Flow rate (Veh/h-lane)	Mainline Flow rate (Veh/h-3lane)
			Older	Middle age	Young		
I-75 on-ramp at Corkscrew Road	3/18/2016	11:00-12:00	27	22	51	1032	2884
	5/19/2016	12:30 - 1:30	45	32	24	556	2052
		3:30 - 4:30	36	47	17	740	2576
I-75 on-ramp at Pine Ridge Road	3/17/2016	11:00-12:00	40	20	40	990	2884
	5/20/2016	12:00 - 1:00	29	11	59	648	1768
		3:00 - 4:00	23	41	35	876	2646

Data Collection Method

Video recorders were used to capture merging maneuvers. The field setup included three video camera stations, as shown in Figure 3.4. The cameras were strategically located to capture merging events throughout the ramp. For the variables needed for this study, especially speed and gaps, a good vantage point was pivotal. For that reason, the cameras were mounted on utility poles or sign posts, as shown in Figure 3.5, and at least 15 feet above existing grade to allow for clear vision of vehicles. This height was sufficient to provide a vantage point for both lateral and longitudinal movements. Cameras also had to be set at an appropriate angle to see the spacing between vehicles, even during instances of tailgating, discussed further in the data reduction section.

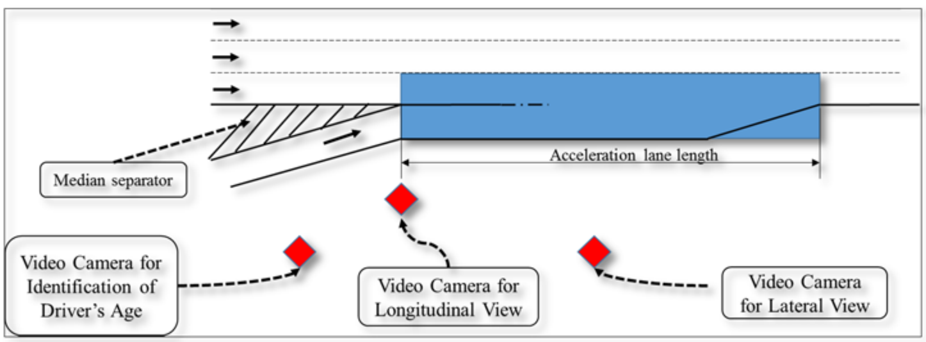


Figure 3.4. Typical configuration of freeway video recording stations.

Figure 3.5. Video camera setup.

In order to determine the merging location for each vehicle, the acceleration lane was divided into four sections, as shown in Figure 3.6. This was accomplished by using ranging rods with attached pink flags. Additionally, flexible delineators found on site were used as fiducial marks. The space between delineators was determined and marked using a tape measure. Ranging rods were placed on the side of the acceleration lane where no delineators were present. A typical data collection site layout with spacing between each fiducial mark along the acceleration lane is shown in Figure 3.6.

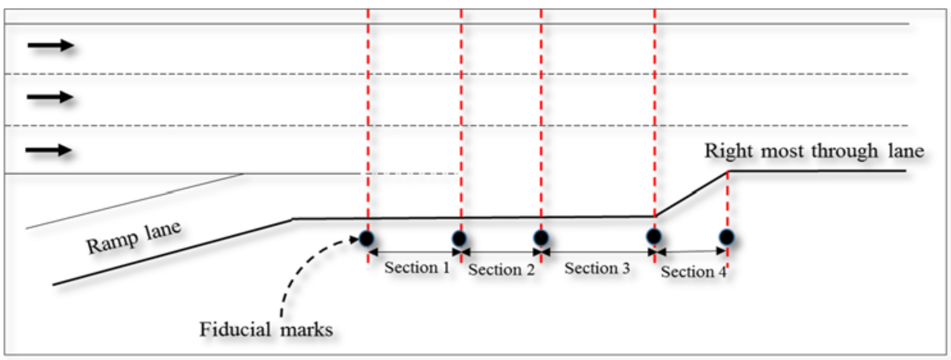


Figure 3.6. Typical merging section showing the fiducial marks.

Variables Collected On-site

One traffic variable, speed, and one driver characteristics, age, were collected on-site. The following two sections provide a short explanation on how data for these three variables were collected.

Driver Age

As shown in Figure 3.4, Camera 1, the first camera at the beginning of the acceleration lane, was used for identifying age. Three research assistants assisted in the data collection process. As vehicles passed Camera 1, a trained research assistant identified the approximate age, speaking loudly so as to be recorded by the camera. Camera 2 was used for capturing the longitudinal movement of vehicles, and Camera 3 was used to capture the lateral movement of vehicles along the acceleration lane.

Driver age was categorized into three age groups: younger (age 25 or less), middle-age (age 25 to 65), and older adult (age 65 and older). The age of the driver was estimated on-site using common physical features, such as grey hair, facial appearance, or driving close to the steering wheel, to distinguish between older and younger drivers. A similar observation method was used by John Lu & Pernia (2000), who concluded that there is no significant difference in the survey results and observation results on the process of identifying driver age.

Merging Speed

The speed of each merging vehicle was collected at the merging location. It is important to note that although speeds used for analyses were collected by video cameras and later processed in the laboratory, merging speeds were also collected on-site using a speed radar detector. This was done for calibration purposes, to be discussed in later sections.

Data Reduction Process

Vehicle Trajectory Extraction

Trajectory extraction of the merging vehicles involved two primary steps. The first and the most challenging step was tracking the vehicle along the acceleration lane to the point where it leaves the acceleration lane to merge into the mainline traffic stream, as shown on Figure 3.7. This step required the identification of the leading and lagging vehicles traveling in the right-most lane of the mainline, and the gaps between the vehicles.

The second step of the process was to determine and record when the ramp vehicle merged onto the freeway, including the time and distance from the beginning of acceleration lane, as shown in Figures 3.7 and 3.8. The fiducial marks along the edge of pavement specified the beginning point of the acceleration lane and the point at which a vehicle was considered fully merged onto the mainline. The total number of merging vehicles observed in the video recordings is listed in Table 3.1.

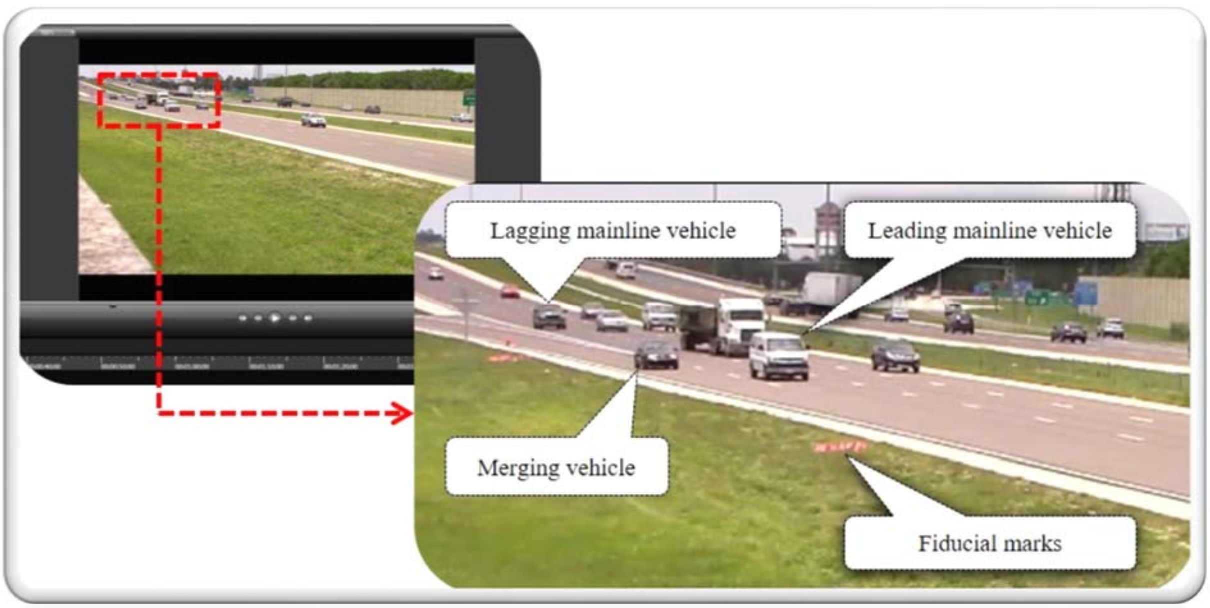


Figure 3.7. Vehicle trajectory extraction from video data using Camtasia Studio.

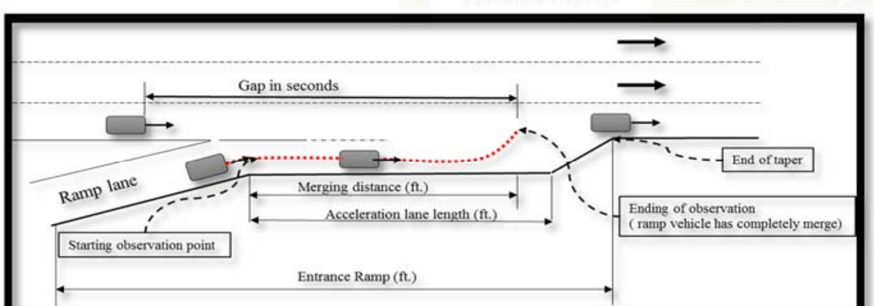


Figure 3.8. Typical merging maneuver data extraction process.

The data reduced from the video recording included freeway lead, lag, and the ramp lead vehicle at each time step. Rewinding of the videos using Camtasia Studio 6 was necessary in the tracking of the vehicles along the merging area. For determination of gap time, a reasonable distance was assumed for the ramp vehicle to detect a vehicle in the right-most lane of the mainline and decide whether to accept or reject the merge gap. A previous study by Levin (1970) used 300 feet for the detection of an opposing vehicle. Another study on merging sections by Chen Kou (1997) used 300 to 400 feet for detection of vehicles in the adjacent right-most traffic lane of a freeway merging section. Accordingly, for the current research, a distance of 300 feet was used for non-elderly ramp vehicles, and 400 feet was used for elderly ramp vehicles to detect the presence of vehicle on the adjacent right-most lane before the ramp vehicles accept or reject the gap and merge to the mainline traffic.

Approach Speed

The average speed of a vehicle along the ramp and the acceleration lane was determined using the Auto scope system. Auto scope is a sophisticated traffic monitoring system that uses video or video with radar to produce highly accurate traffic measurements (Autoscope, 2016). This system detects, calculates, and collects a variety of user-specified traffic data and consists of software and hardware that work together to monitor and report on traffic conditions for intersections, freeways, highways, and tunnels. Using the Auto scope system enabled the accurate collection of travel speeds at both study sites.

For accurate speeds from the Auto scope system, calibration of the software was required. A site image was imported from the video recording to the Auto scope to assist in the calibration process (see Figure 3.9). Other inputs such as lane width, known measured distances identified using fiducial marks, and sample speeds collected from the field study using speed radar, were used as input in the calibration process. After calibration, process detectors, such as speed detectors, count detectors, and detector stations, were placed and defined on the calibrated images to determine the speeds and the number of vehicles in each video recording. Polls were created and specified to collect data at one second interval, and the video recording was run in the Auto scope, as shown in Figure 3.9. Finally, an output text file with the speeds of each vehicle was generated. The collected data was added to a data reduction form in Excel, similar to Figure 3.10, for further processing.

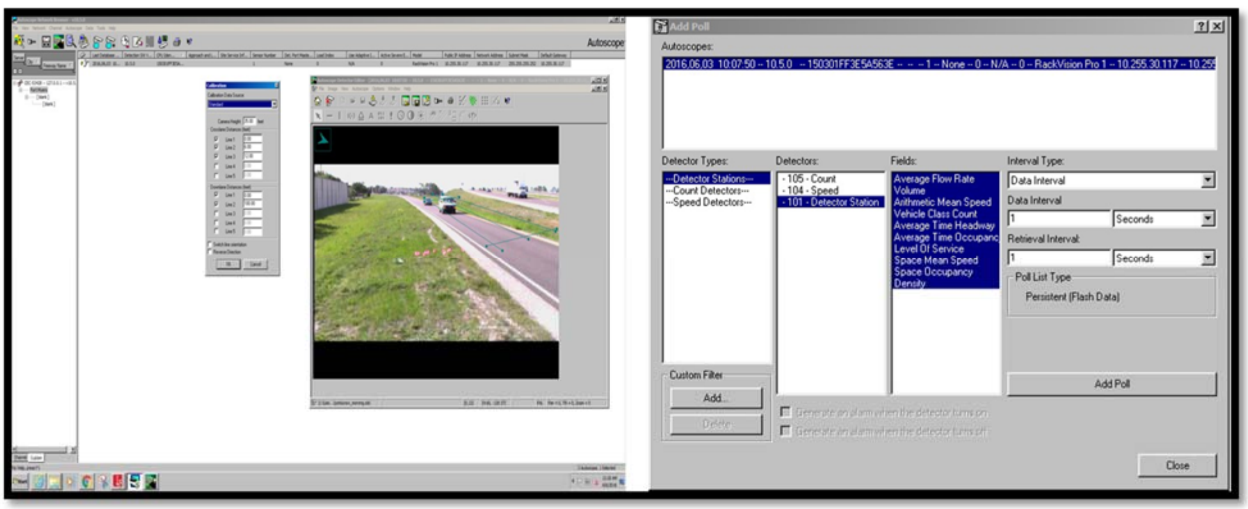


Figure 3.9. Average speed extraction process using Auto scope systems.

Vehicle #	Speed (mph)	Age	Approximately merging distance (ft.)	Gap (sec.)	Merging section on acceleration lane	Response (Accept / Reject)

Figure 3.10. Data reduction form.

CHAPTER 4 ANALYSIS OF FIELD DATA

This chapter discusses the data analysis of an observational study that was conducted at two ramps in Fort Myers, as indicated in Chapter 3. The analysis considers three variables - merging position, approach speed for ramp vehicles, and merging gaps. Additionally, from merging gap data, critical gaps were determined for different age groups. Figure 4.1 illustrates the conceptual data analysis framework.

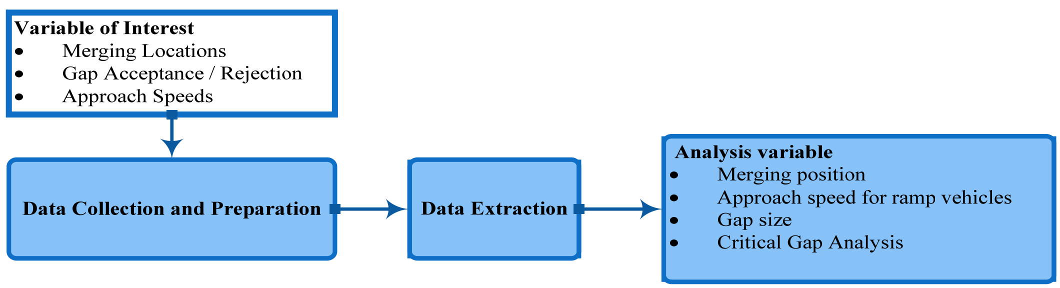


Figure 4.1. Conceptual data analysis framework.

Merging Position Analysis

The position on the acceleration lane at which a majority of drivers start a merging maneuver may offer insight about several traffic and driver characteristics. For example, in heavy traffic conditions, it is expected that most drivers might be forced to merge at the end of the acceleration lane due to a lack of adequate gaps that allow for a safe lane change. For moderate traffic, however, with relatively more adequate gaps available, most aggressive drivers may intuitively merge at the beginning of the acceleration lane. The majority of drivers who need much larger gaps to change lanes would be expected to merge at the end of the acceleration lane.

A comprehensive field study of two sites along I-75, with different acceleration lane lengths, was conducted to investigate the influence of driver age on the merging position of on-ramp vehicles. The location of merging vehicles along the acceleration lane was determined for each driver type identified on site. In this study, the initiation of merging was considered to be when the front left wheel of the ramp vehicle crossed the pavement marking line that separates the acceleration lane and the right-most mainline through lane. At that point, the merging location was noted and categorized as one of four merging sections described in the previous chapter (see Figure 3.6).

Descriptive Statistical Analysis of Merging Position

Figures 4.2(a) and 4.2(b) show the percentage of merges classified by driver type and merging location for the on-ramps at Corkscrew Road and Pine Ridge Road, respectively. As shown in Figure 4.2(a), the percentage of young driver merges at the I-75/Corkscrew Road entrance ramp decreases from Section 1 to Section 4, suggesting that the majority of young drivers prefer to merge near the beginning of the acceleration lane rather than near the end. This observation may be attributed to the tendency of young drivers to be more aggressive. On the other hand, the percentage of older driver merges increased towards the end of the acceleration lane, suggesting that older drivers are more defensive or uneasy about entering the freeway.

While collecting data on site, older drivers were observed to wait until a comfortable gap became available before attempting to merge onto the mainline. Interestingly, the results produced a bell shape curve for middle-age drivers, indicating a more normal distribution, with a central tendency to merge in the middle of the acceleration lane.

Slightly similar results are seen in Figure 4.2(b) for the I-75/Pine Ridge Road on-ramp.

For young drivers, the merge pattern is similar to that observed at the Corkscrew Road site. However, more merges occurred towards the middle of the acceleration lane for both middle-age and older adult drivers, compared to the Corkscrew Road ramp. The difference in patterns between the two sites can be explained by the different acceleration lanes' lengths. The site at Pine Ridge Road has a longer acceleration lane (1500 feet), while the acceleration lane at the Corkscrew Road site is 1,000 feet in length.

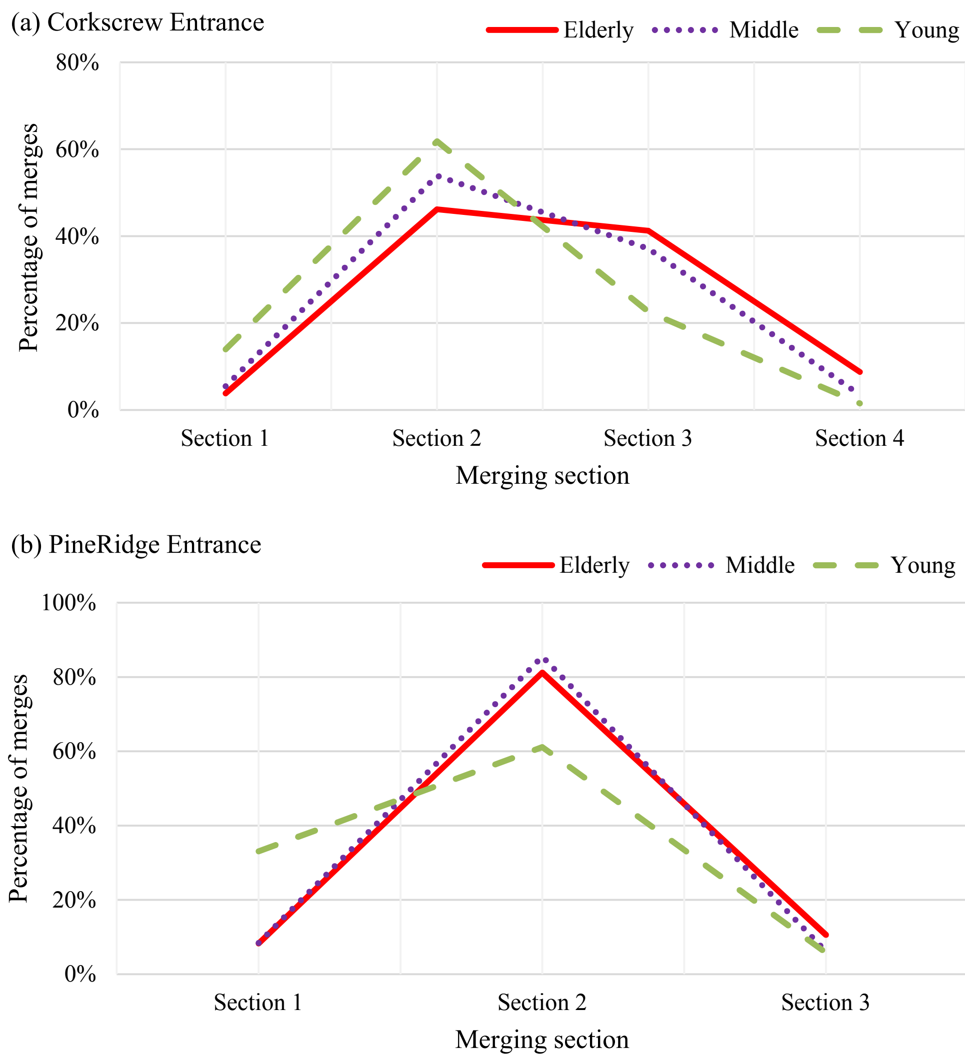


Figure 4.2. Merging location of ramp drivers in acceleration lane.

The results for older drivers suggest that longer acceleration lanes provide more time to scan for a comfortable gap before reaching the end of the acceleration lane.

Inferential Statistical Analysis of Merging Position

To examine the relationship between vehicle merging positions and the prevailing traffic flow conditions, the Friedman Test was utilized. The Friedman test was adopted because it is a non-parametric test and can be used to detect the difference in treatments across multiple test groups within the data listed in Tables 4.1 to 4.6. Unlike the Analysis of Variance (ANOVA) test, the Friedman test requires no assumption on the distribution of the data (Hollander & Wolfe, 1999). Friedman test is an extension of Wilcoxon test, where both are non-parametric tests. It was adopted since it allows the analysis of more than two conditions similar to what we have in this study. Wilcoxon test is only applicable for assessing two conditions (Green & Salkind, 2008), hence it was not suitable for this study.

To compute the Friedman *S*-statistic, ranking is required. The test requires the number of experimental treatments, *k*, to be greater than or equal to two ($k \geq 2$). The observations are arranged in blocks, *b*, as shown in Figure 4.3 (Hollander & Wolfe, 1999).

	Treatment			
Block	**1**	**2**	**...**	***k***
1	X_{11}	X_{12}	...	X_{1k}
2	X_{21}	X_{22}	...	X_{2k}
3	X_{31}	X_{32}	...	X_{3k}
...	...	...	...	...
b	X_{b1}	X_{b2}	...	X_{bk}

Figure 4.3. Block-Treatment example matrix.

The analysis ranks each row together and then considers the values of the ranks by column, where $R\ (X_{ij})$ is the value assigned to X_{ij} within block *i*. Average ranks are used in case of ties. The ranks are summed to obtain the rank value, R_j, as shown in (equation 1).

$$R_j = \sum_{i=1}^{n} r_{ij} \ \ and \ \ R_{.J} = \frac{R_J}{n} \qquad \text{(Eq. 1)}$$

The Friedman *S*-statistics is determined by (equation 2) as follows:

$$S = \frac{12n}{k(k+1)} \sum_{j=1}^{k} (R_{.j} - \frac{k+1}{2})\ 2 = \left[\frac{12}{nk(k+1)} \sum_{j=1}^{k} R_j^2\right] - 3n(k+1) \qquad \text{(Eq. 2)}$$

The following hypothesis:

H_0: The treatments have identical effects

H_1: Not all treatments have identical effects

At α level of significance:

Reject H_0 if $\geq \chi^2_{k-1,\alpha}$; otherwise we do not reject.

Where $\chi^2_{k-1,\alpha}$ is the upper α percentile point of a Chi-square with k-1 degrees of freedom.

In investigating how the driver age categories affect the number of merges in each section (merging positions), data tables constructed for each site were used, as summarized in Tables 4.1 and 4.3 for Pine Ridge and Corkscrew ramps, respectively. The null and alternative hypothesis were formulated as follows:

H_0: Driver merging position is independent of driver age.

H_1: There is correlation between driver age and choice of merging section.

Table 4.1

Number of Merges and Corresponding Driver Age: Pine Ridge Entrance

Driver Type	Number of Merges		
	Section 1	Section 2	Section 3
Older adult Drivers	14	138	18
Middle-Age Drivers	18	183	13
Young Drivers	46	85	8

The output from data shown in Table 4.1 were analyzed using the Friedman test, and results are shown in Table 4.2 in Minitab notation (Minitab, 2013):

Table 4.2

Minitab Results for Number of Merges and Corresponding Driver Age: Pine Ridge Entrance

Section	N	Estimated Median	Sum of Ranks
1	3	17.00	5.0
2	3	138.00	9.0
3	3	15.00	4.0
Grand median = 56.67			
$\chi^2_r = 4.67$ DF = 2 *p*-value = 0.097			

From statistical tables, at a 95% confidence level with two degrees of freedom, the critical Chi-Square value is 5.99. Since the test statistic value, $\chi^2_r = 4.67$, seen in Table 4.2, is smaller than the critical Chi-square, there is no enough evidence to reject the null hypothesis. It should be noted, however, that if the test is conducted at 90% confidence level, the null hypothesis would be rejected as the observed *p*-value of 0.097 is significant at the 90% confidence level. Therefore, this leads to a conclusion that there is a correlation between choice of merging section and driver age.

The Friedman test results of the data listed in Table 4.3, for the Corkscrew Road site, are shown in Table 4.4 in Minitab notation (Minitab, 2013):

Table 4.3

Number of Merges and Corresponding Driver Age: Corkscrew Entrance

Driver Type	Number of Merges			
	Section 1	Section 2	Section 3	Section 4
Older adult Drivers	14	169	151	32
Middle-Age Drivers	14	138	95	9
Young Drivers	55	243	89	6

Table 4.4

Minitab Results for Number of Merges and Corresponding Driver Age: Corkscrew Entrance

Section	N	Estimated Median	Sum of Ranks
1	3	29.38	5.0
2	3	170.88	12.0
3	3	118.13	9.0
4	3	30.13	4.0
Grand median = 87.13			
$\chi^2_r = 8.20$ DF = 3 *p*-value = 0.042			

A similar analysis, as that done for the Pine Ridge entrance, was conducted for the on-ramp at the Corkscrew site (see Table 4.3) and yielded a similar conclusion at a 95% confidence level. The critical Chi-Square of 7.18 was obtained from the statistical tables at 95% (degrees of freedom = 3). The observed *p*-value of 0.0421 and χ^2_r= 8.2 shown in Table 4.4, led to the rejection of the null hypothesis and concluded that the data suggest a correlation between merge location and driver age.

Approach Speed

Approach ramp speed, the speed at which the ramp vehicle entered the acceleration lane, was measured just before the beginning of Section 1 using the Auto scope system for each ramp vehicle. It was hypothesized that the higher the ramp approach speed, the more likely the driver would commence merging at the beginning of the acceleration lane, and the slower the ramp approach speed, the higher the probability merging would occur at the end of the acceleration lane.

Descriptive Statistical Analysis of Approach Speed

Figures 4.4(a) and 4.4(b) show the percentage of merges in each defined section of the acceleration lane at the two sites with increasing speed. Figure 4.4(a) , for the Pine Ridge entrance ramp, illustrates that as speed increases from (30 < speed < 45) to (55 < speed < 65) the percentage of merging vehicles on Section 1 increases, which agrees with the hypothesis mentioned before. However, a different trend is seen for Sections 2 and 3, as the percentage of merges slightly decrease with the increase in speed from (30 < speed < 45) to (55< speed < 65). This result may be due to fact that most drivers tend to approach the end of the acceleration lane when a vehicle is present in the adjacent through-lane, resulting in the need to reduce their speed so as to meet a suitable gap and merge.

For the Corkscrew entrance, shown in Figure 4.4 (b), results for Section 1 are similar to observations at the Pine Ridge entrance (Figure 4.4 (a)). However, for Sections 2 and 3, a decrease in the percentage of merges is observed as speed increases from (30 < speed < 45) to (45< speed < 55). Once the speed reached (55< speed < 65), the percentage of merges increases slightly. A decrease in the percentage of merges is seen in Section 4, when the vehicle speed changes from low to high speed. This suggests that only a few vehicles traveling at higher speeds tend to merge at the end of the acceleration lane.

Inferential Statistical Analysis of Approach Speed

The Friedman test was performed again to examine the relationship between the approach speed and percentage of merging vehicles in each section on the acceleration lane. The number of drivers who merged at different sections of the acceleration lane are summarized in Tables 4.5 and 4.7, for Pine Ridge and Corkscrew ramps, respectively. The following null and alternative hypothesis were formulated as follows:

H_0: The choice merging section is independent of the approach speed.

H_1: There is correlation between approach speed and choice of merging section.

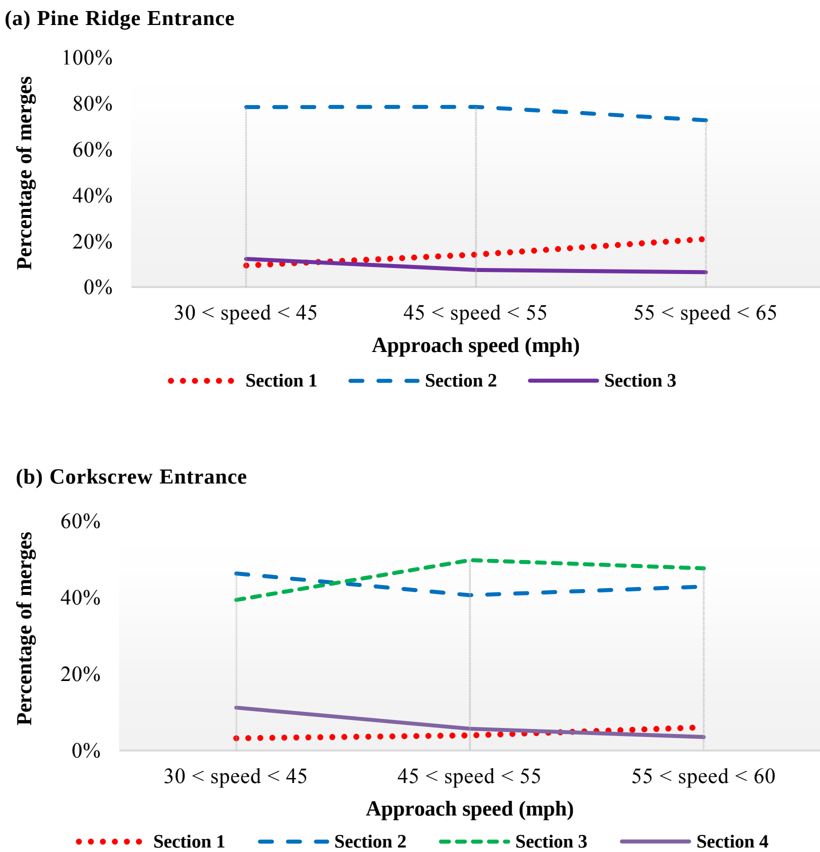


Figure 4.4. Relationship between merge vehicles and approach speed.

Table 4.5

Test for Independence of Merge Position vs. Approach Speed: Pine Ridge Entrance

Approach Speed	Number of Merges		
	Section 1	Section 2	Section 3
30 < speed < 45	17	142	22
50 < speed < 55	25	139	13
55 < speed < 65	36	125	11

The Friedman test results of the data in Table 4.5 are presented on Table 4.6 in Minitab notation (Minitab, 2013).

Table 4.6

Minitab Results for Independence of Merge Position vs. Approach Speed: Pine Ridge Entrance

Section	N	Est Median	Sum of Ranks
1	3	25.00	5.0
2	3	137.00	9.0
3	3	15.00	4.0
Grand median = 59.00			
$\chi^2_r = 4.67$ DF = 2 p-value = 0.097			

From statistical tables, for a 95% confidence level and the degrees of freedom of 2, the critical Chi-Square value is 5.99, which is greater than the observed Chi-Square of 4.67. Since the p-value of 0.097 obtained from the analysis is greater than $\alpha = 0.05$, we do not reject the null hypothesis and conclude that there appears to be no correlation between approach speed and the choice of merging section. Alternatively, at a 90% confidence interval, there appears to be a correlation between the approach speed and the number of merges in each section. A similar analysis was conducted for the Corkscrew Entrance (Table 4.7) and yielded a similar conclusion, with a slightly lower p-value of 0.0602. However, if the test was conducted at 90% confidence level, the null hypothesis would be rejected and for both sites, the conclusion will be that it appears to be a correlation between the approach speed and the number of merges in each section.

Table 4.7

Test for Relationship between the Merge Position vs. Approach Speed: Corkscrew Entrance

Approach Speed	Number of Merges			
	Section 1	Section 2	Section 3	Section 4
30 < Speed < 45	6	87	74	21
45 < Speed < 55	9	93	114	13
55 < Speed < 65	14	99	110	8

The Friedman test results of the data in Table 4.7 are presented on Table 4.8 in Minitab notation (Minitab, 2013).

Table 4.8

Minitab Results for Independence of Merge Position vs. Approach Speed: Corkscrew Entrance

Section	N	Estimated Median	Sum of Ranks
1	3	10.13	4.0
2	3	94.38	10.0
3	3	108.13	11.0
4	3	11.88	5.0
Grand median = 56.13			
$\chi^2_r = 7.40$ DF = 3 p-value = 0.060			

Accepted Gaps / Gap Analysis

Gaps were measured from Section 1 to the point where the ramp vehicle merged. Once a right-most lane vehicle was identified to be at Section 1, near the ramp vehicle, the time difference from that point to where the ramp vehicle merged was defined as gap (see Figure 3.8).

Descriptive Statistical Analysis of Gap size

It was hypothesized that the larger the gap size, the greater the acceptance rate would be for vehicles to merge from the first two sections (Section 1 and Section 2). This implies that as the gap size increases, ramp vehicles are more likely to merge earlier from the acceleration lane compared when the gap size is small. Figure 4.5(a) and 4.5(b), for the Corkscrew and Pine Ridge ramps, respectively, support this expectation, and indicate that most of the shorter gap sizes are accepted by drivers when they are at the end of acceleration lane.

According to Figures 4.5(a) and 4.5 (b), gap sizes greater than 6 seconds are more often accepted by vehicles merging in Sections 1 and 2, yielding a large percentage of merges. Merges decrease in Sections 3 and 4. Nonetheless, the percentage of merges with smaller gaps increases from Section 1 to the end of acceleration lane for gaps under 1 second, as well as those ranging from 1 to 3 seconds.

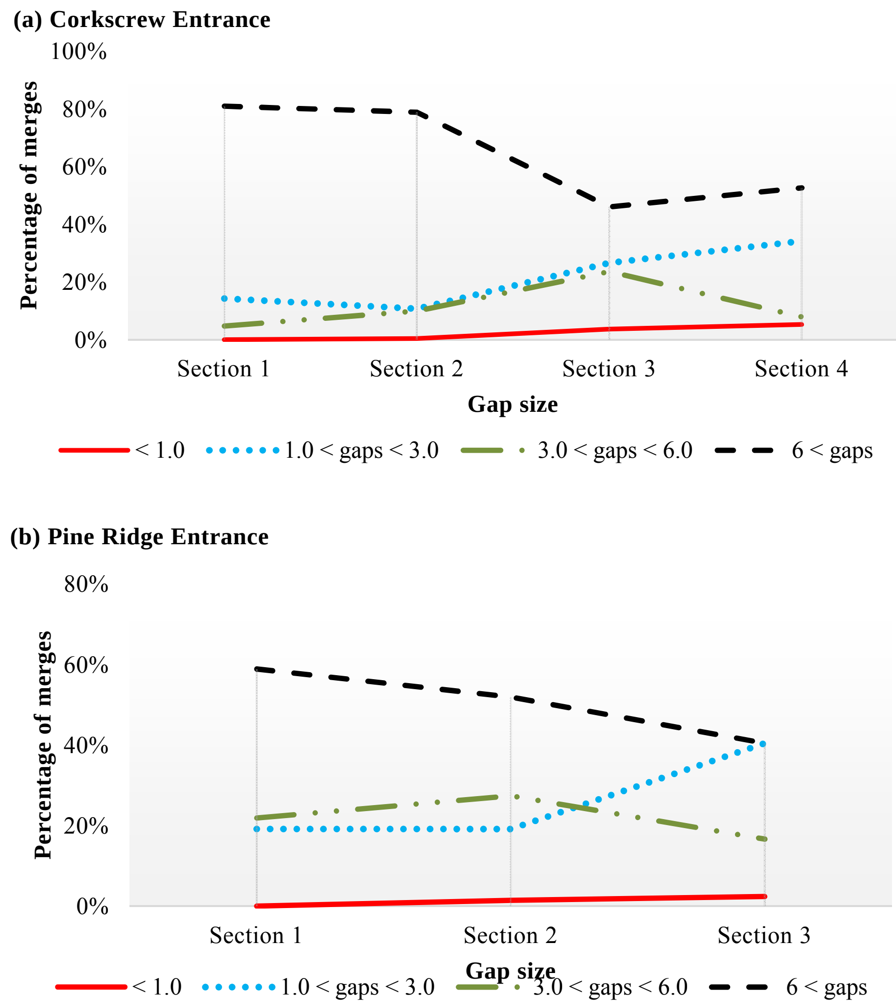


Figure 4.5. Merge percentage vs. gap size offered by freeway traffic.

Inferential Statistical Analysis of Gap Size

A Friedman statistical test was used to determine whether there was a significant difference of the gap size offered at merging sections along the acceleration lane. The number of drivers who merged at different sections on the acceleration lane is summarized in Tables 4.9

and 4.11, for Pine Ridge and Corkscrew ramps, respectively. The following null and alternative hypothesis were formulated as follows:

H_0: Gap size is independent of the choice of merging section.

H_1: There is correlation between gap size and choice of merging section.

Table 4.9

Test for Independence for Gap Size vs. Merging Position: Pine Ridge Road and I-75

Gaps (sec)	Number of Merges		
	Section 1	Section 2	Section 3
gaps < 1.0	0	6	1
1.0 < gaps < 3.0	14	79	17
3.0 < gaps < 6.0	16	113	7
6 < gaps	43	214	17

The Friedman test results of the data listed in Table 4.9 are presented in Table 4.10 in Minitab notation (Minitab, 2013).

From the statistical table, the critical value is 5.99, which is slightly lesser than the computed Chi-square value, χ^2_r=6, shown in Table 4.10. This result falls in the rejection region, and therefore, we reject the null hypothesis and conclude that gap size is dependent to the merging sections.

Table 4.10

Minitab Results for Independence for Gap Size vs. Merging Position: Pine Ridge Road and I-75

Section	N	Estimated Median	Sum of Ranks
1	4	15.00	6.0
2	4	95.67	12.0
3	4	11.33	6.0
Grand median = 40.67			
χ^2_r = 6.00 DF = 2 p-value = 0.050			

A similar analysis was conducted for the Corkscrew Entrance (Table 4.11) and yielded the same conclusion, with a smaller *p*-value of 0.00194. Again, we reject the null hypothesis and conclude that there appears to be a correlation between gap size and merging section, which indicates that drivers rely on the size of the gap offered in each section of the acceleration lane to make the decision of whether to merge or not. The Friedman test results of the data shown in Table 4.11 are presented in Table 4.12 in Minitab notation (Minitab, 2013).

Table 4.11

Test for Independence of Gap Size vs. Merging Position: Corkscrew Entrance

Gaps (sec)	Number of Merges			
	Section 1	Section 2	Section 3	Section 4
gaps < 1.0	0	1	6	2
1.0 < gaps < 3.0	3	27	44	13
3.0 < gaps < 6.0	1	25	39	3
6 < gaps	17	198	76	20

Table 4.12

Minitab Results Test for Independence of Gap Size vs. Merging Position: Corkscrew Entrance

Section	N	Est Median	Sum of Ranks
1	4	2.81	4.0
2	4	27.44	12.0
3	4	39.94	15.0
4	4	7.06	9.0
Grand median = 19.31			
χ^2_r = 9.90 DF = 3 *p*-value = 0.019			

Critical Gap Analysis

The availability of gaps in the mainline traffic stream is influenced by traffic flow, which may affect the gap acceptance behavior of the merging vehicles. If the traffic volume on the mainline is light, most of the available gaps for the ramp vehicles will be relatively large and will be easily accepted. However, if the traffic volume is heavy, the available gaps will be relatively small, resulting in difficulties for the ramp vehicles to accept the gaps and merge. Therefore, when evaluating the gap acceptance behavior of ramp vehicles, it is important to understand the critical gaps.

According to Raft definition (Guo & Wang, 2014), the critical gap is the number at which the accepted gap times that at the critical gap, are smaller is equal to the number of rejected gap times that are longer. This means that the probability of acceptance or rejection is equal. A critical gap can also be used as a threshold to whether a ramp vehicle can enter the mainline stream. It has been used in the analyses of intersections where the critical gap is the accepted gap when the headway of the major roadway traffic is large, and the rejected gap when the headway is smaller, for vehicles entering the intersection from the minor roadway (Guo & Wang, 2014).

A critical gap is a random variable due to the inconsistency and non-homogeneous nature that exists between drivers, and is difficult to measure directly. Different methods have been used to estimate the critical gap, such as the regression method, Siegloch's method, Ashworth's method, maximum likelihood method, Raff method, and others. Typically, the estimation of the critical gap relies on accepted and rejected gaps (Guo & Wang, 2014).

For this study, merging vehicles were categorized according to driver characteristics based on age group. For each driver, the rate of acceptance and rejection was evaluated. This was done due to the observed mixture of drivers with different merging behaviors mentioned in previous chapters. Since the vehicles are in motion along the acceleration lane, only two gap scenarios were considered in the analysis process. The first scenario is where a gap is offered to the vehicle when entering the acceleration lane and the driver rejects it. The second scenario is where a gap is offered to the vehicle when entering the acceleration lane, and the driver accepts it and proceeds to merge with the mainline traffic. Because this method included all of the rejected gaps, it was easier to remove bias.

In this study, critical gaps were determined graphically, as indicated in Figures 4.6 through 4.11. Two cumulative frequency curves were generated for each age group at each study site, and show the number of accepted gaps shorter than time *t* compared to the number of rejected gaps longer than time *t*, where *t* in each curve is the critical gap in seconds.

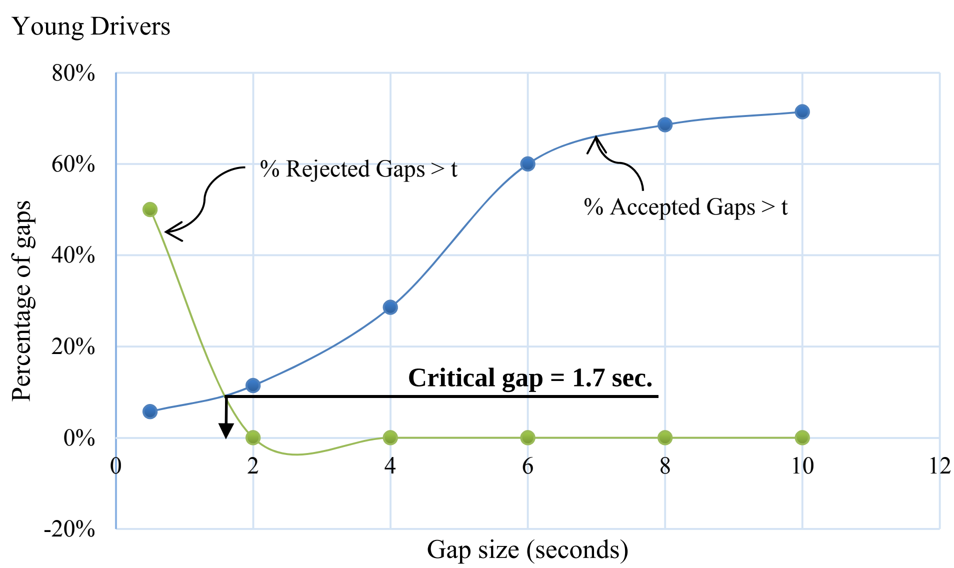


Figure 4.6. Acceptance and rejection curves for young drivers: Pine Ridge Entrance.

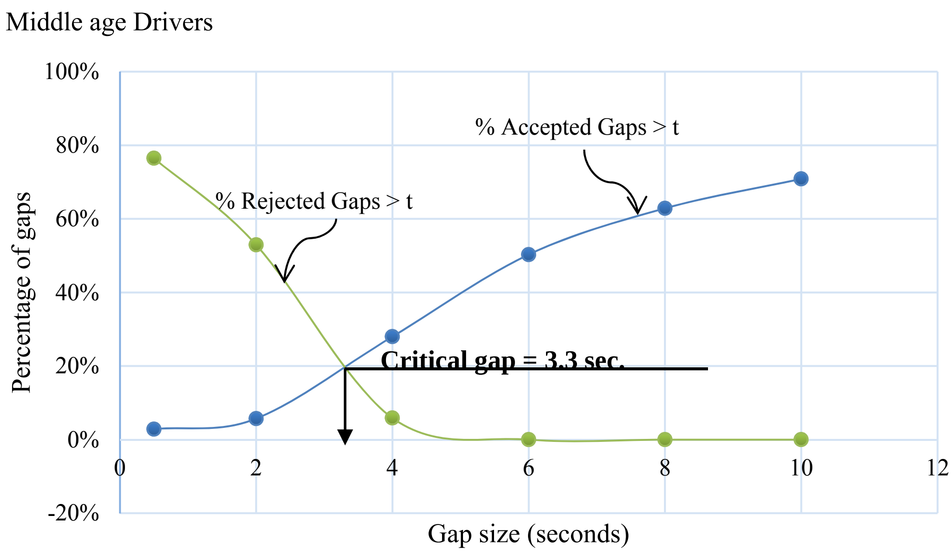


Figure 4.7. Acceptance and rejection curves for middle-age drivers: Pine Ridge Entrance.

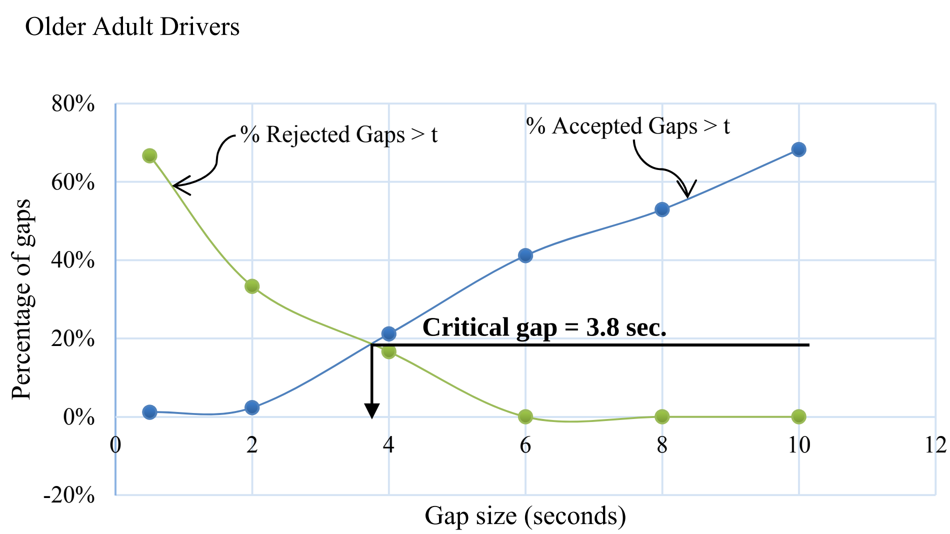


Figure 4.8. Acceptance and rejection curves for older adult drivers: Pine Ridge Entrance.

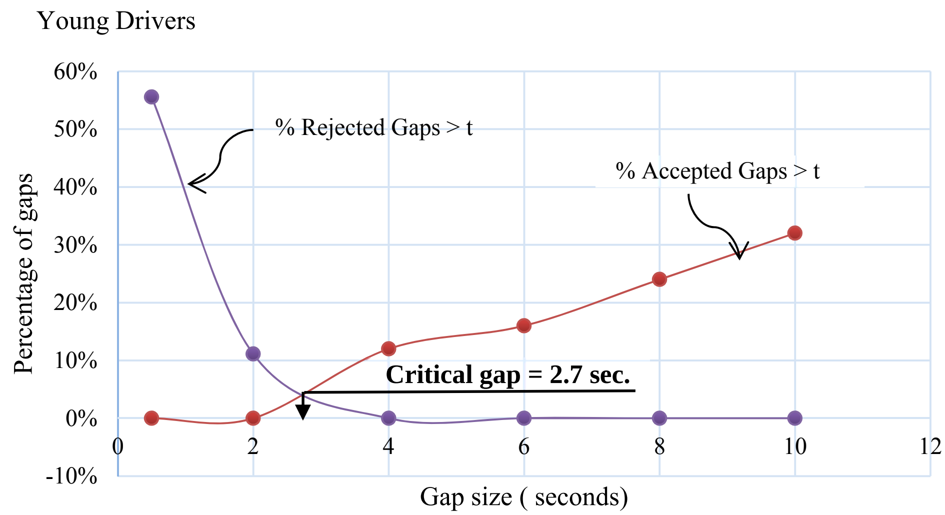


Figure 4.9. Acceptance and rejection curves for young drivers: Corkscrew Entrance.

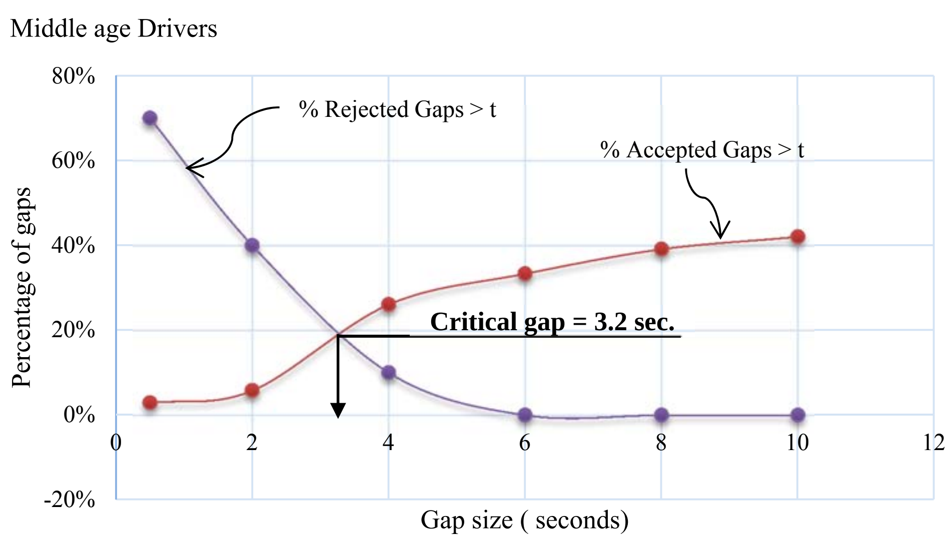


Figure 4.10. Acceptance and rejection curves for middle-age drivers: Corkscrew Entrance.

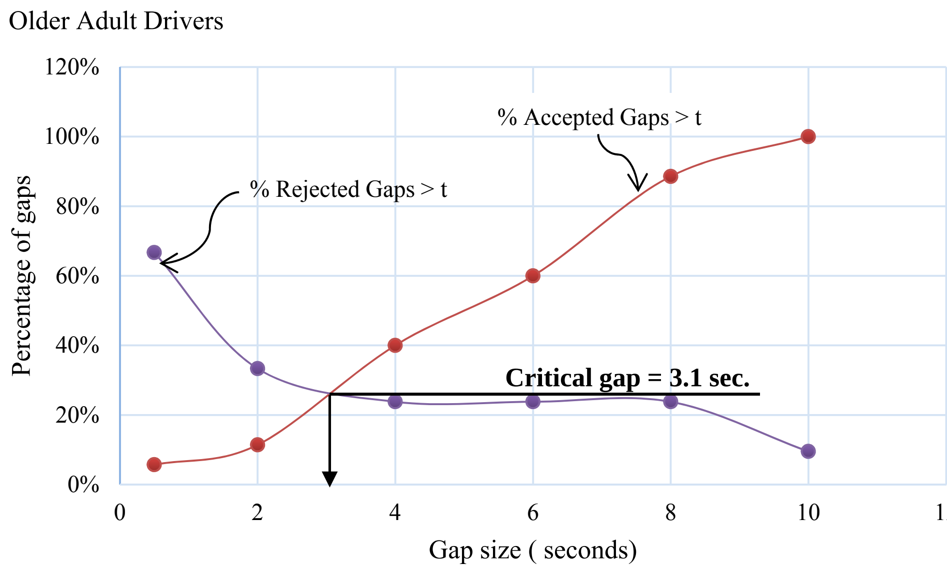


Figure 4.11. Acceptance and rejection curves for older adult drivers: Corkscrew Entrance.

From the cumulative frequency curves shown in Figures 4.6 through 4.11, critical gaps were observed to differ according to driver age group, and increasing with increasing age. The findings reveal that younger drivers typically require a smaller critical gap compared to older drivers.

Since critical gaps act as a threshold, the values obtained on the graphs are more in line with expected critical gap values (Chandraratna & Stamatiadis, 2003). However, many studies have been conducted on left turn maneuvers and none on the merging areas. Younger drivers are often viewed as aggressive drivers, meaning the likelihood of accepting smaller gaps is greater compared to older adult drivers. On the other hand, defensive drivers, such as the older adult population, are prone to accepting larger gaps, as indicated in Figures 4.8 and 4.11. Note that the graphs in Figures 4.6 – 4.10 represent critical gaps at any point along the acceleration lane.

Regression Model of Rejected/Accepted Gaps

A binary regression model was used to quantify the factors influencing a driver's decision to accept or reject a gap in the mainline traffic stream. As mentioned previously, the probability of acceptance or rejection of the critical gap is 0.5, and is one of the key characteristics of gap acceptance behavior of merging drivers. Both logit and probit models can be used, and both produce similar results. However, due to three benefits with using the logit model outlined by Marczak , Daamen, & Buisson (2013), the logit model was deemed the preferred model for this analysis. The advantages of using a logit model are:

- Does not require previous knowledge of the shape of data distribution to use it;
- Its numerical implementation is simpler to use;
- It provides an asymptotically consistent parameter which allows the use of a *t*-test to evaluate the quality of the regression.

From the compiled data set, four different variables were of interest to investigate: age of drivers, approaching speed of ramp vehicles, the gap size offered, merging distance measured from the start of acceleration lane, and the merging section of vehicles on acceleration lane at the moment a gap is accepted.

The appropriate logit model used for the distribution of gap acceptance was a weighted linear regression model with the mathematical form shown in Equation 4.

$$p = \frac{e^t}{1+e^t} \quad and \quad t = \beta_0 + \beta_i X_i \qquad \text{(Eq. 4)}$$

Where, p is the probability of accepting a gap smaller than t, β_0 and β_{i} are regression coefficients to be estimated based on the data collected in the field. The equation can be finally expressed as:

$$p = \frac{1}{1+e^{-(\beta_0 + \beta_i X_i)}} \quad \text{(Eq. 5)}$$

Similar to a previous study by Marczak , Daamen, & Buisson (2013), a logistic regression model was performed with an explanatory variable, $X_i = (X_{age}, X_{speed}, X_{gap}, X_{pos})$, and a dependent variable of Y. From the contributing variables mentioned above, the model can be expressed as:

$$\ln\left(\frac{p}{1-p}\right) = \beta_0 + \beta_{age}X_{age} + \beta_{speed}X_{speed} + \beta_{gap}X_{gap} + \beta_{pos}X_{pos} \quad \text{(Eq. 6)}$$

The results of the logit model for the I-75/Corkscrew Road data are presented in Table 4.13, and in Table 4.14 for I-75/Pine Ridge Road site.

Table 4.13

The Logit Model Results for Corkscrew Entrance

Predictor Variables	Estimated Coefficient	Standard Error of Coefficients	Z-value	Confidence Interval Lower Bound	Confidence Interval Upper Bound	*p*-value
β0	3.69113	1.10193	3.35	1.53135	5.85091	0.001
βsection	-0.90725	0.30043	-3.02	-1.49609	-0.31842	0.003
βgap(sec)	0.30050	0.12510	2.40	0.05530	0.54570	0.016
βage	-0.06024	0.29137	-0.21	-0.63132	0.51083	0.836

Table 4.14

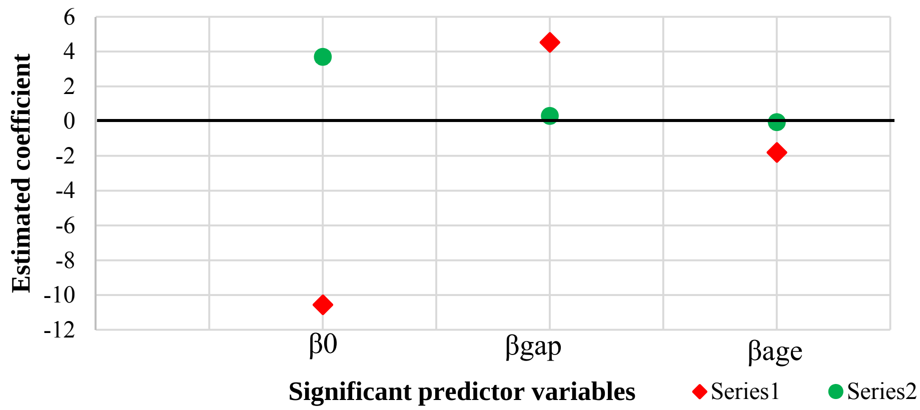
The Logit Model Results for Pine Ridge Entrance

Predictor Variables	Estimated Coefficient	Standard Error of Coefficients	Z-value	Confidence Interval Lower Bound	Confidence Interval Upper Bound	*p*-value
β0	-10.554	5.804	0.800	-21.930	0.823	0.069
βspeed	0.177	0.109	1.630	-0.036	0.390	0.10
βgap (sec)	4.530	1.604	2.820	1.386	7.674	0.005
βage	-1.800	0.908	-1.980	-3.579	-0.021	0.047

To determine if a significant relationship exists between the predictor variables and the response, the *p*-values were considered. From the output presented in Table 4.13, section and gap size yields a *p*-value less than 0.05, indicating that at a 95% confidence level, suggesting a relationship between the significant predictor variables and the response. Variables such as the merging distance and merging section yielded a *p*-value greater than 0.05 indicating that no significant relationship exists with the response, and therefore, were removed from the model. Although the predictor variable related to driver age was not significant, it was deemed important to the model, and so was not removed.

For the Pine Ridge entrance, age and gap size offered to the merging driver were found to have a significant relationship with response, producing a *p*-value less than 0.05 at 95% confidence level, whereas the predictor variable speed can be significant at 90% confidence level as shown on Table 4.14.

Figure 4.12 illustrates the relationship of the significant predictor variables for the two data sets. For both data sets the gap size estimated coefficients are positive, indicating the increase in gap size increases the likelihood of ramp vehicles to accept the gap. Similar results were reported by Marczak , Daamen, & Buisson (2013).



Series 1: Pine Ridge Entrance; Series 2: Corkscrew Entrance
Figure 4.12. Estimated coefficients of the predictor variables.

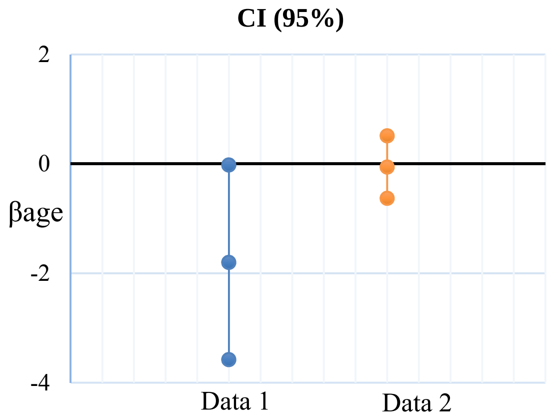
On the other hand, estimated coefficients for the predictor variable age (β_{age}), yielded a negative slope with the response, reflecting that as driver age increases from young to older adult, the probability of accepting gaps decreases.

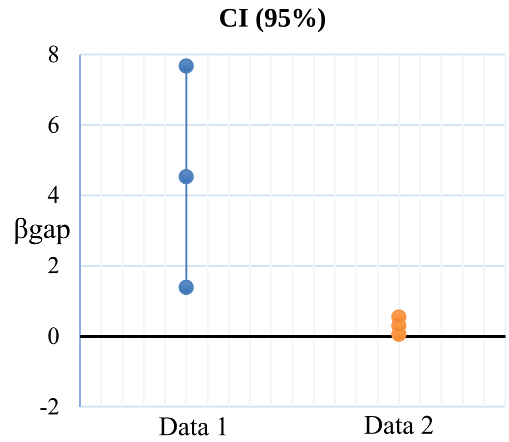
For comparison of the two data sets (Figure 4.13), the confidence interval for the significant variables were considered. The confidence interval for the estimated coefficients was calculated using the following formulas (Equation 7 and 8):

$$CI = \beta_x \pm z_{\frac{\alpha}{2}}(\mathrm{SE}) \qquad \alpha = (1 - 95\%) \;\; and\; x = (\,x_0, x_{distance}, x_{gap}) \qquad \text{(Eq. 7)}$$

$$SE = \frac{\delta}{\sqrt{N}} \qquad \text{(Eq. 8)}$$

Where, *CI* is the confidence interval, *SE* is the standard error, βx is the estimated coefficient, and x are the explanatory variables.





(Data 1: I-75/Pine Ridge Road; Data 2: I-75/Corkscrew Road)

Figure 4.13. Confidence intervals for the estimated coefficients.

As shown in Figure 4.13, the variable β_{age} confidence intervals overlap, meaning that the importance of the variables is not significantly different between the two sites. The confidence intervals for the predictor variable β_{gap} do not overlap, thus showing a significant difference in this variable between the two study sites.

CHAPTER 5 SENSITIVITY ANALYSIS OF MERGING BEHAVIOR

Overview

A widely used complex modeling tool, VISSIM version 8.0, with coding guidelines outlined in the FDOT *Traffic Analysis Handbook* (2014), was used in this study. According to VISSIM manual of 2015, “VISSIM is a microscopic, time step, and behavior-based simulation model developed at the University of Karlsruhe, Germany in 1992, and launched in 1993” (PTV America, 2015).

The quality of the traffic flow model used in VISSIM is essential for obtaining reasonable results (PTV America, 2015). The software uses the psycho-physical perception model developed by Wiedemann (1974). The fundamental theory behind the model is that drivers of faster moving vehicles start decelerating as approaching the individual perception threshold/limit of a slower driver, and begin to accelerate again after approaching another threshold. At this point, the faster moving vehicle will begin to decelerate to a speed below the slower speed vehicle. Different driving behaviors are taken into consideration by VISSIM using the distribution functions of speed and distance behavior.

Figure 5.1 shows the steps used in VISSIM simulation following the FDOT protocol as described in the FDOT *Traffic Analysis Handbook* (2014).

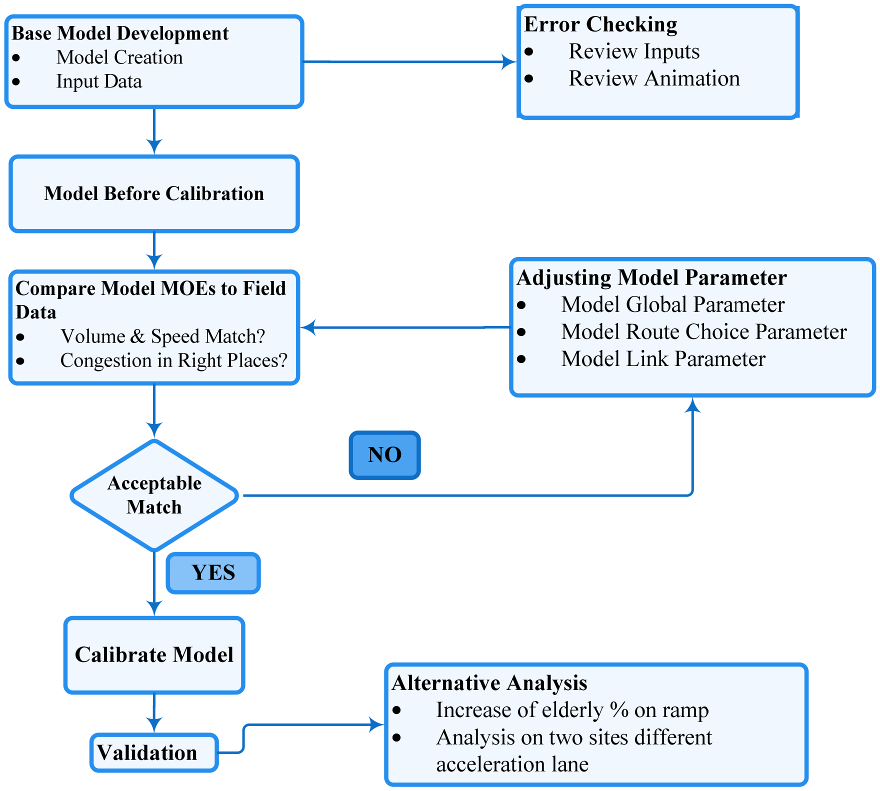


Figure 5.1. Steps used for VISSIM simulation.

Basic Network Coding

Network coding involves links and connectors which represent the uncontrolled input parameters, such as roadway segments, that reflect the traffic movement and curvature of the roadway. The coding of the merging areas in VISSIM is controlled by the vehicle routing process, and the lane change distance – a controllable input parameter. An example of the VISSIM model is shown in Figure 5.2 for the Pine Ridge study site. For proper coding of merging areas, the following principles, outlined by PTV (2015), were used for the model:

- The merging area should include the entire auxiliary lane (acceleration lane). Vehicles in VISSIM will utilize the extra link length when necessary.

- The merging section should be one link including both the links for mainline traffic and the acceleration lane for the merging vehicles.
- Just one connector should be used at the lane drop.

Figure 5.2. VISSIM model of Pine Ridge Entrance.

Base Data for Simulation

Once the merging section was drawn to match the existing geometry at the study site, the vehicle characteristics were defined. One unique element of VISSIM is its ability to stochastically simulate vehicles through the network. According to the VISSIM manual, The stochastic nature of the VISSIM simulation model allows for variation of several parameters, such as desired speed distributions, maximum acceleration and deceleration, dwell time, and vehicle type, where by the Wiedemann car following model represents the variability in driver behavior (PTV America, 2015).

Vehicle Types and Vehicle Composition

Vehicle types with different color codes were created to represent the three different driver groups defined during the data collection. Four vehicle types included: older adult vehicle (green), middle-age vehicle (red), younger driver vehicle (yellow), and lastly a black vehicle representing the mainline traffic. Each of the vehicle types had the same attributes such as vehicles power, length, weight and minimum deceleration (PTV America, 2015). Figure 5.3 shows an example data entry window used for a vehicle type.

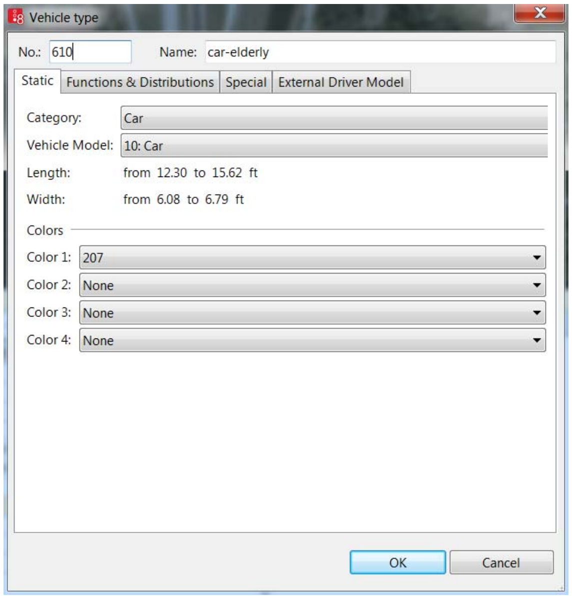


Figure 5.3. Vehicle type classification – VISSIM window.

Vehicle composition includes the proportion of each defined vehicle type relative flow (PTV America, 2015). In this study, two vehicle compositions were created, one for the ramp vehicles, and another for the mainline traffic. The ramp composition consisted of four vehicle types: older adult, middle-aged, younger vehicles, and heavy goods vehicles (HGV). The mainline traffic composition was composed of only two vehicle types, defined as mainline traffic and HGV.

Speed Distribution

Speed control in VISSIM was coded by the use of speed decisions or reduced speed areas on the network links. These speed decisions were based on a speed distribution confined by the 15th and 85th percentiles as the minimum and maximum values, respectively. Desired speed decisions were placed on the merging ramps to change the vehicle speed as they traveled from the arterial roadway to the freeway. Reduced speed areas were not used in the study since there were no such areas where a significant reduction in speed was observed in the field. Spot speeds collected during the off-peak hours (free-flow conditions) were used to determine the speed distribution, and to later code the desired speed decisions. Speed data were converted to speed profiles and functioned as inputs into the VISSIM software. Figure 5.4 shows the cumulative frequency curve for older adult driver speeds before merging into the mainline traffic stream. The observed speed data were used to create the speed distribution shown in the VISSIM graph in Figure 5.4.

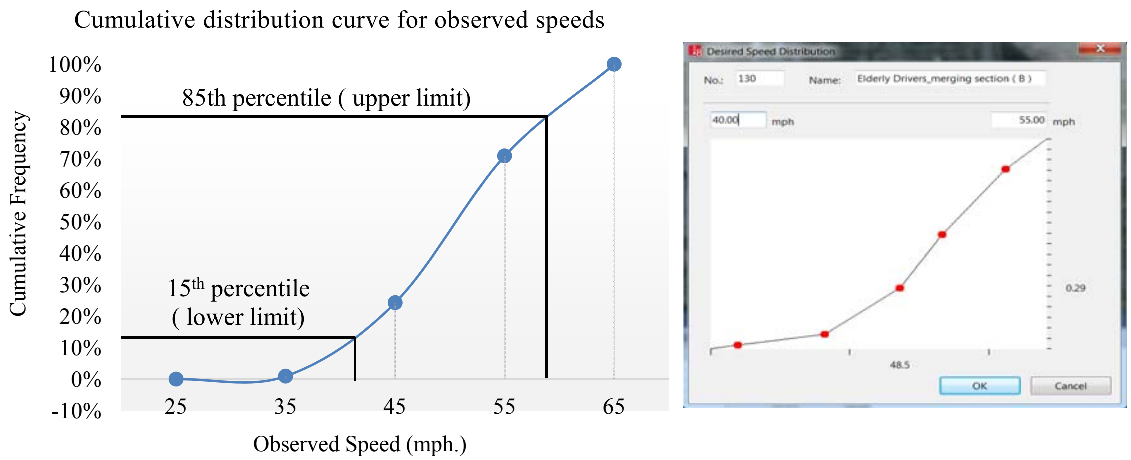


Figure 5.4. Cumulative distribution curve for observed data (left) and VISSIM graph of speed distribution curve (right).

Traffic Volume Input

After entering the vehicle types, and defining the vehicle composition and speed distributions, the actual volumes were entered into the network. From field observations, the hourly volume remained fairly consistent throughout the data collection hours. Therefore, vehicle input was coded in hours, according to the direction of travel, instead of 15 minute increments, the preferred unit of volume. At this stage, the volumes used reflected the off-peak hourly volumes observed in the data extraction process from the video recordings.

Route Decision

To run the model and simulate the existing conditions, static routes were established. The route decision determines the path for a given vehicle, and consists of a sequence of links and connectors. Each route decision contained at least one destination point, placed far upstream on the link to allow for a maximum merging distance for ramp vehicles entering the highway.

Determining the Required Number of Simulation Runs

VISSIM models use random seed numbers to perform simulation runs. The random seed used in each run assigns properties to each individual vehicle. These properties include; the decision on which type of vehicle will enter the simulation, when the vehicle will enter, which lane it will use, the aggressiveness level of the driver, and the interaction between vehicles (i.e., lane change behavior) once the vehicles are in the system (Russo, 2008). Running multiple simulations of similar models with different random seed numbers returns a unique simulation result for each run. This is mainly due to the stochastic function in VISSIM, allowing variation of vehicle arrivals in the network. For meaningful results, an arithmetic mean is calculated by VISSIM for all simulated runs.

Preliminary simulation results were used to estimate the sample standard deviation, and henceforth, used in calculating the number of runs. Using VISSIM, the selected MOEs for calibration included speed and merging location. From the speed results, determined from 10 preliminary simulation runs, the standard deviation was estimated. According to Russo (2008), 90% and 95% confidence levels are preferred in microscopic simulation calibration. For this research, a 95% level of confidence was used.

Without the knowledge of how the results will vary, it is basically impossible to estimate how many repetitions are required to make statistical conclusions without testing. A minimum number of repetitions (runs) were calculated from Equation 6 as follows:

$$n = \left[\frac{s * t_{\propto/2}}{\mu * \varepsilon}\right]^2 \qquad \text{(Eq. 6)}$$

Where: n is the required number of simulation runs,

S is the standard deviation of the system performance measure based on the previously simulation runs,

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

μ is the mean of the system performance measure,

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

Ten simulation runs with different seed numbers for each run, for a total of 10 seed numbers, and the average speed along the merging section for each vehicle types were evaluated. The results are shown in Table 5.1.

Table 5.1

Average Speed Along The Merging Section for Each Vehicle Type

Simulation Run	Seed Number	Average Speed (mph.)			
		Older	Middle Age	Younger	Mainline Traffic
1	42	52.1	57.0	59.5	65.0
3	44	53.1	55.6	58.4	64.9
4	45	54.0	56.2	59.8	65.0
5	46	53.5	57.1	60.0	64.8
6	47	53.0	56.4	59.5	64.9
7	48	52.6	56.8	58.6	65.1
8	49	53.4	56.9	60.0	64.9
9	50	52.6	57.1	59.5	65.0
10	51	53.4	56.0	59.2	64.9
Average		52.9	56.5	59.4	64.9
Standard deviation		0.70	0.60	0.60	0.10
Maximum		54.0	57.1	60.0	65.1
Minimum		51.8	55.5	58.4	64.7

For a confidence level of 95%, a confidence interval α of 0.05 is determined, with degrees of freedom of 9, a standard deviation *S* for average speed on the merging section of 0.14, the statistic table $t_{\alpha/2}$ was found to be 2.262. Using the recommended tolerable error of 10% from the FDOT Traffic Analysis Handbook (2014), the number of simulation runs calculated using Equation (6) was 10. This value agrees with what is reported to be adequate by the Traffic Analysis Handbook and the Protocol for VISSIM Simulation by the Oregon Department of Transportation (Mai, et al., 2011; FDOT Traffic Analysis Handbook, 2014).

Calibration

Calibration process was required to ensure that simulated traffic replicates the field observations. With three driving behaviors used in the study, an examination of the key parameters was conducted based on modeling judgment and the collected data. Multiple runs for each parameter, in each of the three driving behaviors, were completed using the trial and error method until the selected Measure of Effectiveness (MOE) accurately reflected the prevailing field conditions.

Model Calibration

Before changing the default parameters, an initial evaluation of the network was conducted to check if the default values provided acceptable results using the 10 runs previously generated. The average merging speed was the measure of effectiveness used to compare field conditions to VISSIM findings. Since the study deals with three different driver types, it was expected that the default parameter values would not replicate what was observed on site. Thus, the calibration process was necessary for the study. Two steps were performed for this process:

1. First step – system calibration, which involved the evaluation of all input data including, vehicle route choice, traffic compositions, seeding period, vehicle input, defined speed distribution, and geometry characteristics. All were checked for consistency.
2. Second step – operation calibration, which involved the adjustment of model parameters that affect the overall traffic operation of the network. This included driving behavior parameters, such as those affecting the aggressiveness of the driver and location of lane changing. Unlike Gomes (2004) study, driver behavior in this study was assumed to correlate with the vehicle type only, and not the position (Gomes, May, & Horowitz, 2004).

The FDOT Traffic Analysis Handbook (FDOT, 2014) provides a different range of parameters acceptable for the calibration process. Freeway merging sections rely mostly on lane changing and Wiedemann 99 car following parameters. Therefore, the car following parameters were changed to be in line with previous sensitivity studies conducted by other researchers such as Gomes, May, & Horowitz (2004), Habtemichael & Picado Santos (2013), and PTV (2015). The range specified in the FDOT Traffic Analysis Handbook was also considered. Table 5.2 shows the values of the car-following parameters used in this study.

Car Following Parameters

Wiedemann 99, used for freeways, includes 10 parameters, which can be adjusted. Listed below is an explanation of each parameter and the effect when the parameter is changed as defined in the VISSIM manual (2015):

1. CC0 standstill distance – the desired standstill distance between two vehicles Gomes, (2004) observed an influence to the capacity when the value is +/- 2 feet.
2. CC1 headway time – the desired distance a driver wishes to maintain when traveling at a certain speed. According to the VISSIM manual (2015), the higher the value, the more cautious the driver will be. This is the multiple part of the safety distance as define in VISSIM, $d_x = CC0 + CC1 * V$ where d_x, the desired minimum distance a driver will maintain from a preceding vehicle. This distance influences capacity in the model for high volume inputs (congestion), also has an influence on throughput for the mainline and ramp section (Habtemichael & Picado Santos, 2013).
3. CC2 following variation – acts as a threshold of how much additional distance the driver can maintain from a preceding vehicle, or simply put, how much can be added to the desired safety distance. The distance of 13.12 feet is recommended by PTV for stable following behavior (VISSIM manual, 2015). Furthermore, the CC2 following variation is also considered as one measure of aggressiveness for drivers (Gomes, et al., 2004).
4. CC3 threshold for entering following – controls the beginning of the deceleration process before reaching the desired safety distance. In other words, at what second, when a driver recognizes a stimulus such as a slower vehicle, the driver will start decelerating.

5. CC4 & CC5 Positive and Negative following threshold respectively – controls the speed difference during the following process. The lower the value, the more sensitive the driver is to acceleration and deceleration of the vehicle in front (PTV, 2015).
6. CC6 speed dependency of oscillation – the parameter for the influence of distance on speed oscillation when following a vehicle. A value of zero indicates independence of speed oscillation with distance, whereas, a greater value shows a large oscillation of speed with increasing distance.
7. CC7 oscillation of acceleration – the actual acceleration during oscillation, reported by Gomes et al. (2004) to have more influence on driver aggressiveness (Gomes, et al., 2004).
8. CC8 standstill acceleration – the desired acceleration when starting from a stopped position. Parameter limitation relies on the maximum acceleration defined on the acceleration curve.
9. CC9 acceleration at 50 mph – the desired acceleration at 50 mph. This parameter is limited to the maximum acceleration defined on the acceleration curve.

Table 5.2

Calibration Parameter Adjusted

Calibration Parameter	Default Values	Older	Middle Age	Younger	Mainline Traffic
CC0 (ft.)	4.92	4.9	3.0	3.0	4.9
CC1 (sec.)	0.9	3.0	0.5	0.3	0.5
CC2 (ft.)	13.12	19.7	12.0	9.0	13.1
CC4	-0.35	-0.4	-0.4	-0.4	-0.7
CC5	0.35	0.4	0.4	0.4	0.7
CC7 (ft/s^2)	0.82	0.8	2.5	2.5	0.8
CC9 (ft/s^2)	4.92	4.0	8.0	8.0	4.9

Lane Changing Parameters

Lane changing behavior in VISSIM is classified into two categories: necessary lane change and free lane change (PTV America, 2015). The necessary lane change parameter controls the vehicle route in decision making, such as choosing a lane that provides the best interaction with other vehicles, as well as, how aggressively a vehicle can perform the lane changing maneuver (Gomes, May, & Horowitz, 2004; PTV America, 2015). The free lane change parameter deals with the safety distance of the trailing vehicle on the new lane, and depends on the speeds of the vehicles changing lanes that precede it. The parameters were adjusted for lane changing as shown in Table 5.3.

Table 5.3

Lane Changing Parameters

Lane Change Parameters	Default Values	Older drivers	Middle Age drivers	Younger drivers	Mainline Traffic
Maximum deceleration (ft/s2)	-13.12 (own) -9.84 (Trailing)	-10(own) -11 (Trailing)	-32 (own) -10 (Trailing)	-32 (own) -10 (Trailing)	-32 (own) -32 (Trailing)
(-1 ft/s2) per distance (ft.)	200 (own &Trailing)	100 (own) & 80 (Trailing)	50 (own) & 100(Trailing)	50 (own) & 100 (Trailing)	50 (own) & 100(Trailing)
Accepted deceleration (ft/s2)	-3.28 (own & training)	-2.50 (own) & -1.50 (training)	-21.84 (own) & -9.84 (training)	-21.84 (own) & -9.84 (training)	-16.28 (own) & -20.00 (training)
Waiting time before diffusion (sec.)	60	60	60	60	60
Minimum headway (front/rear) (ft.)	1.64	6.0	1.64	1.64	1.64
Safety distance reduction factor	0.6	0.8	0.3	0.1	0.6
Maximum deceleration for cooperation braking (ft/s2)	-9.84	-3.0	-32.0	-32.0	-32.0
Overtake reduced speed area	Unchecked	Unchecked	Unchecked	Unchecked	Unchecked
Advanced merging	Checked	Unchecked	Checked	Checked	Checked
Cooperative lane change	Checked	Checked	Checked	Checked	Checked

For confirmation of the calibration process, classical calibration targets suggested in the FDOT Traffic Analysis Handbook (2014), and the one developed by Wisconsin DOT for freeways, were used. Here, the merging speed and the movement of vehicles in the model were considered. Table 5.4 displays the average speed measured for the Pine Ridge site for each defined vehicle type, regardless of the merging section. In this case, all values comply with the defined range. According to the FDOT Traffic Analysis Handbook (2014), if the calibration item is speed, then it should be within 10 mph (+/-) of the field measured speeds on at least 85% of all network links. This condition was satisfied or indicated in Table 5.4.

Table 5.4

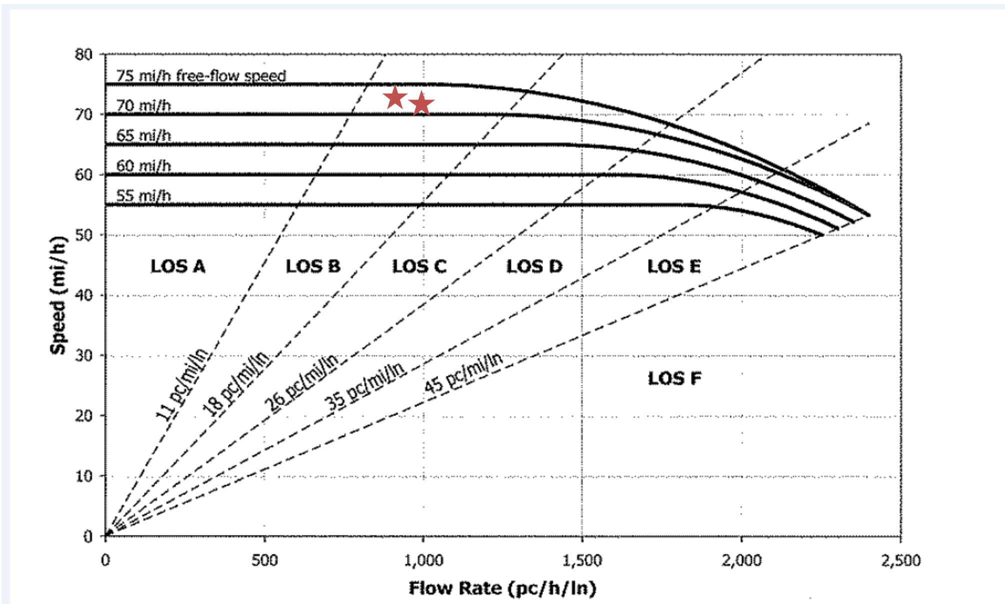
Calibration Target for Speeds

Acceleration lane length (ft)	Driver Type	Measurement Location	Average Field Speed (mph.)	Average Micro-simulation speed
1500	Young	Start of merging section	64	64.2
	Middle Age		62.1	59.5
	Older Adult		61.45	56.7

Simulation Scenarios

Forty simulation scenarios were executed for the Corkscrew entrance, and 50 scenarios, for the Pine Ridge entrance. Each simulation lasted 75 minutes with a seeding time of 15 minutes using different random of seeds. The 15 minutes seeding time was enough to feed the network with traffic and stabilize the flow, as suggested by Mai, et al. (2011). The change in random seed allows the model to simulate stochastic variations of a vehicle in order to produce meaningful results. In this case, an increment of one (1) was selected, and a random seed of 42 was used for simulation number one. The scenarios were created as the level of service (LOS) was changed from A to E.

As defined by the *Highway Capacity Manual* (HCM), the level of service is the qualitative measure that describes the traffic condition in terms of speed, travel time, freedom to maneuver, comfort, traffic interruptions, and safety (Highway Capacity Manual, 2010). A LOS of A, represents a free-flow operation. With a LOS of B, vehicles ability to maneuver is slightly restricted, and this was observed at both sites as indicated by the stars in Figure 5.5. For both sites, the level of service was increased to LOS E to represent full capacity with vehicles very closely spaced within the traffic stream. This was so done to observe the change in merging behaviors for ramp vehicles. As the level of service changed, the percentage of older adult drivers was increased to 50% for the ramp vehicle composition. For simulation, a LOS B was used along the mainline segment just before the merging sections, based on existing traffic conditions. Figure 5.5, shows the selection of the level of service on the mainline segment as a function of flow rate of the mainline and speed.



Source: HCM 2010

Figure 5.5. Level of service chart.

The simulation was also performed with varying the percentage of older drivers from 29% (as observed on site) to 45% for the Pine Ridge entrance. On the other hand, for the Corkscrew entrance, no variation of older adult composition was explored since the percentage of observed older adult drivers was 45% on-site.

Validation

The validation process followed the calibration process, starting with the visualization method. This method involved checking the consistency of lane choice, the presence of bottlenecks, if any, and the merging location of vehicles according to the defined vehicle types. This step was important to ensure realistic outcomes from the model. At this stage the model was observed while running to detect any vehicle overlaps, vehicle disappearance, as well as unrealistic driving maneuvers.

Secondly, a comparison was done between the simulated and observed percentage of merges on the merging section (Figures 5.6 and 5.7) and approach speed for ramp vehicles (Figure 5.8). Data collected during 3 PM to 4 PM at each site location were used for the validation process. Figures 5.6 (a) and 5.7 (a) represent the percentage of merges in each section for older drivers. The simulated and the observed percentage of merges were almost similar, with less than a 2% difference between the two sites. However, for the middle age and young drivers (see Figures 5.6 (b & c) and 5.7 (b & c)), the difference exhibited between the simulated and observed percentage of merges was far greater than 2%. It should be noted that it was difficult to achieve a difference of less than 10% of the calibrated target recommended by the FDOT Traffic Analysis Handbook for all driver types. However, since the study primarily concentrated on the older adult, more focus was placed on ensuring that older adult driver parameters met the required calibration target for the both sites (FDOT Traffic Analysis Handbook, 2014).

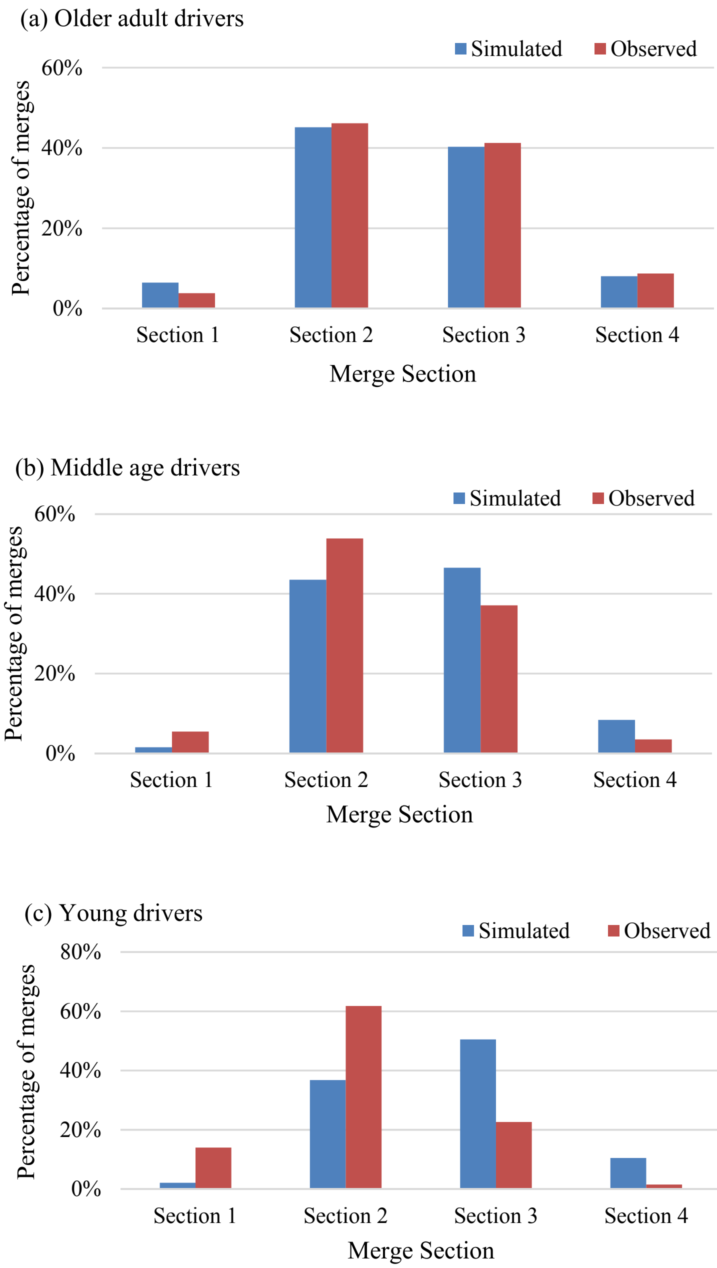


Figure 5.6. Comparison of the percentage of merges for observed and simulated data: Corkscrew entrance.

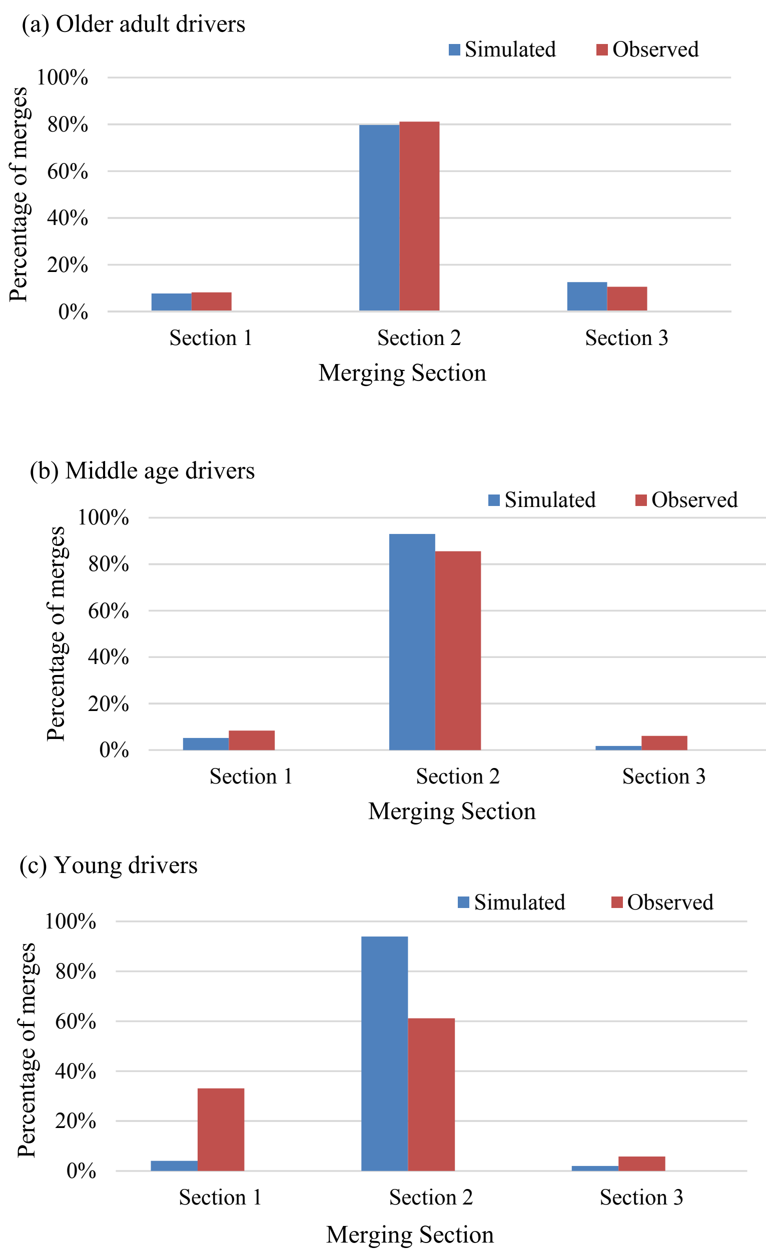
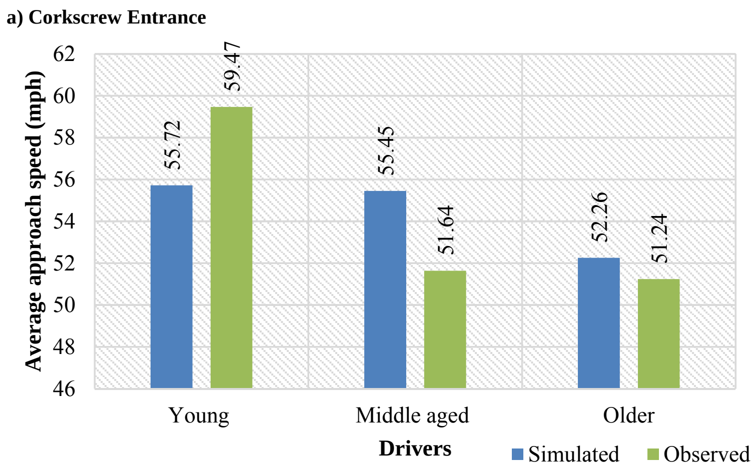


Figure 5.7. Comparison of the percentage of merges for observed and simulated data: Pine Ridge entrance.

Additionally, the comparison of the simulated and observed approach speeds from the ramp shows a smaller difference for older adult drivers for both sites on Figure 5.8 (a) and (b), which was not more than 1 mph. However, for the middle age and young drivers (see Figures 5.8 (a & b), the difference exhibited between the simulated and observed percentage of merges was far greater than 1mph, but still the difference did not exceed the 10 mph target recommended on the FDOT Traffic Analysis Handbook for speeds (FDOT Traffic Analysis Handbook, 2014).



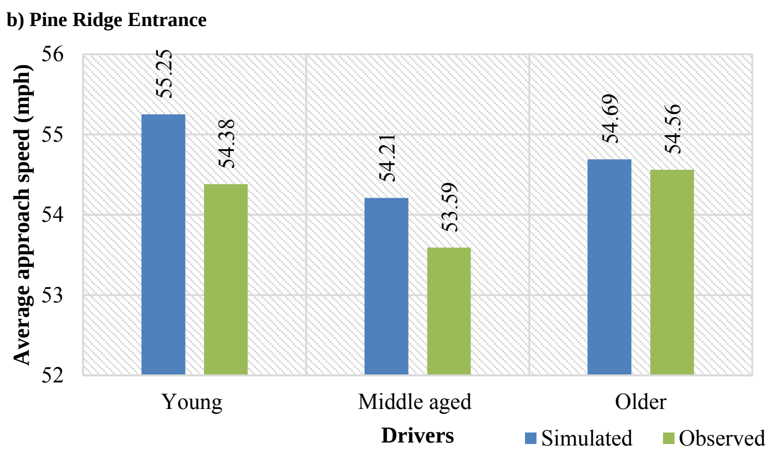


Figure 5.8. Comparison of the observed and simulated data: Pine Ridge entrance.

Results

Average Speed

The average speeds of each vehicle type provided insight into how the traffic merges, and the reaction of vehicles on the right-most mainline lane. As shown in Figure 5.8 and 5.9, merging speeds for the merging traffic decreases as the level of service change from LOS A to LOS E in each section. A huge drop in speed is observed from LOS C to D, which might mainly be due to the queues accumulated on the merging section.

In Figure 5.8, for sections 1 to 3 the older drivers have lower speed as the LOS changes from A to C, however, from LOS C to E the merging speeds are constant for every driver type, indicating bumper to bumper condition. In section 4, all the vehicle types has similar speeds regardless of the level of service, which can be explained by vehicles decelerating at the end of acceleration lane waiting for enough gap to merge.

For Figure 5.9, the older drivers had lower speeds than middle age and young drivers on section 1 as the LOS changed from A to E. For section 2 and 3, the merging speed of older drivers at LOS D and E were much lower compared to the other drivers.

The difference in merging speed for section 2 and 3 as the level of services changed from D to E between the two sites might be due the difference in length of acceleration lane.

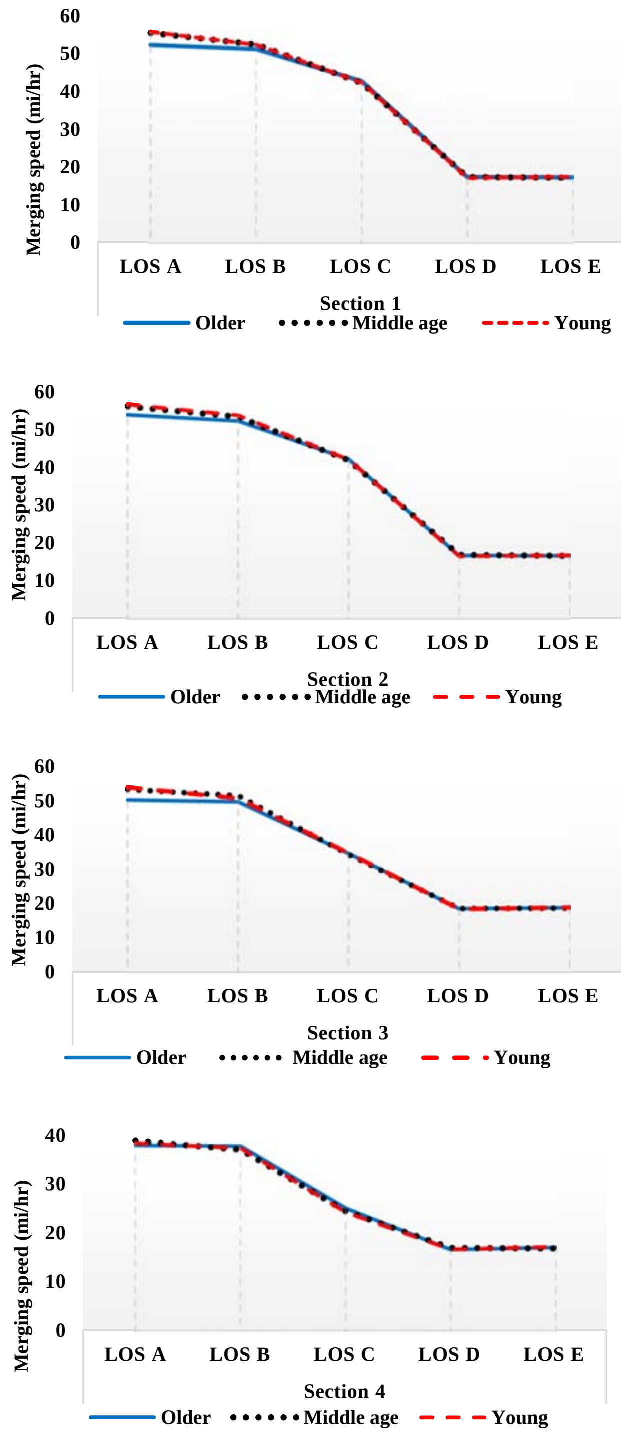


Figure 5.9. Merging speed distribution with change in LOS (Corkscrew Entrance).

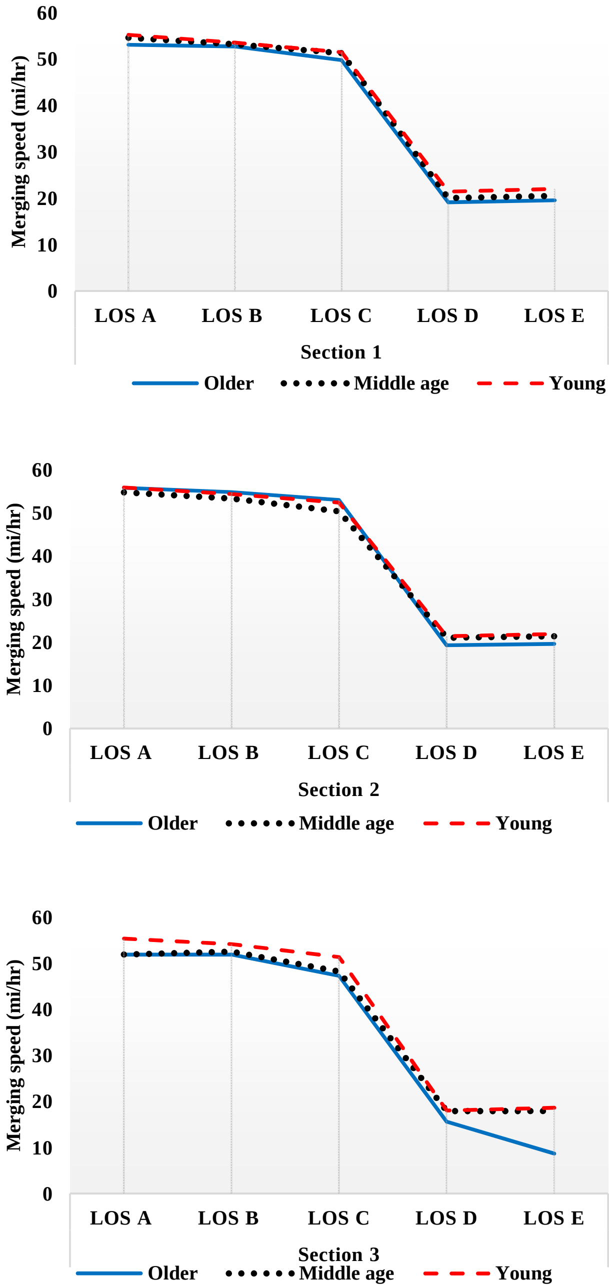
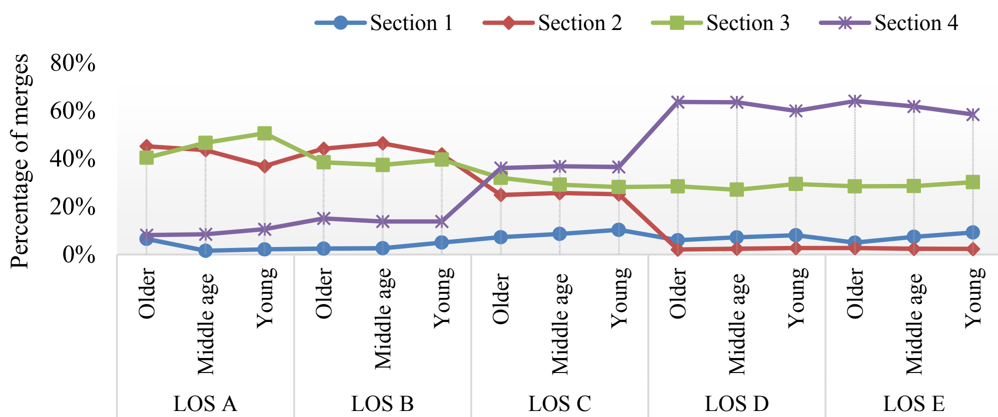


Figure 5.10. Merging speed distribution with change in LOS (Pine Ridge Entrance).

Merging Position

The merging position of the ramp vehicles changed with the level of service, changes from A to E, as shown in Figure 5.9. This was expected. An overall increase in the percentage of merges was seen in sections 3 and 4 for both the Pine Ridge and Corkscrew entrances, respectively. As the level of service changes from A to E, a decrease is observed for sections 2 and 3 at the Corkscrew entrance, and sections 1 and 2 for the Pine Ridge entrance. This implies that, for the Corkscrew entrance, sections 2 and 3 have more interactions between merging vehicles and mainline vehicle in the right-most lane, making the merging maneuver more difficult for vehicles entering the freeway. This can also be said for sections 1 and 2 at the Pine Ridge entrance. Therefore, it can be concluded that vehicle queues begin to form, and vehicle interaction becomes more noticeable with a level of service C.

(a) Corkscrew Entrance



(b) Pine Ridge Entrance

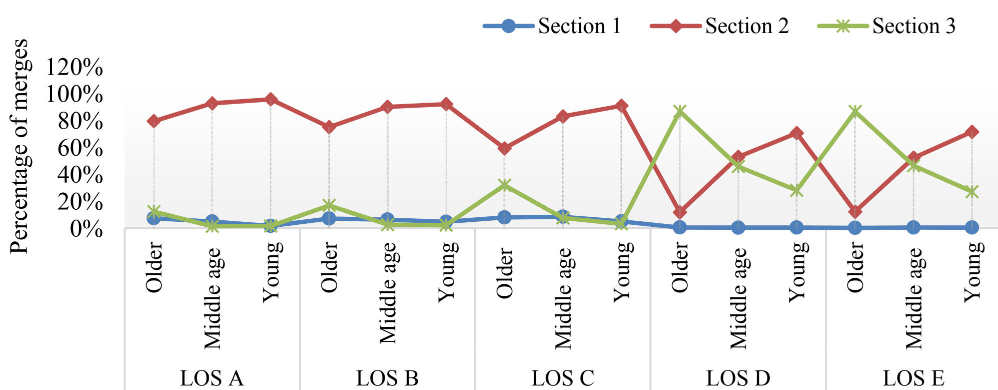


Figure 5.11. Percentage of merges with change in LOS.

A detailed observation of the percentage of merges for each driver type as they travel along the acceleration lane when the level of service changes is shown in Figures 5.10 and 5.11. In Figure 5.10, each section, for each driver type, a similar trend is seen as the level of service changes. The percentage of merges in sections 2 and 3 decrease as the level of service changes from A to E, although remain fairly constant for LOS D and E. A different trend was seen in section 4, where the percentage of merges increase as the level of service changes, which was, expected. Interestingly, although the percentage of merges in section 1 was small compared to the other sections, it appears that for section 1 more merges occurred LOS C conditions.

The percentage of merges trend shown in Figure 5.11 shows a decreasing trend for sections 1 and 2, and an increasing trend for section 3. For middle age and young drivers, the percentage of merges on sections 2 and 3, when LOS is D and E, is slightly similar, whereas for older drivers, a larger percentage of merges was seen on section 3 for these two LOS conditions.

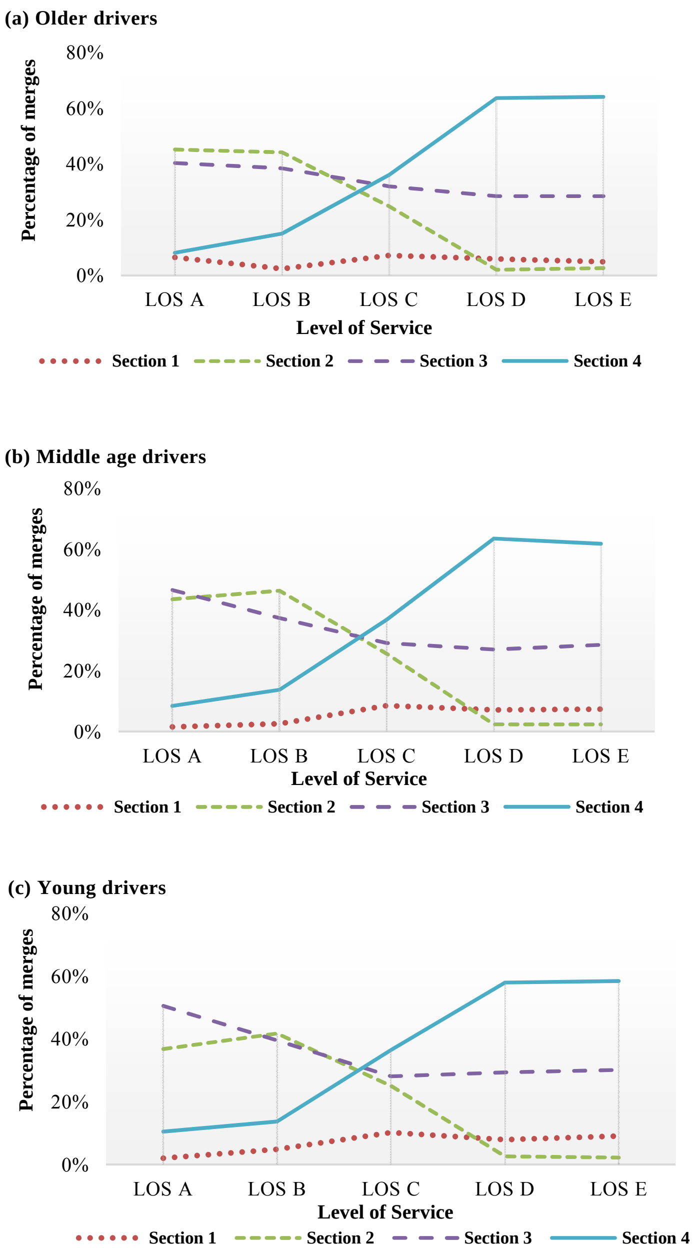


Figure 5.12. Percentage of merges with change in LOS: Corkscrew entrance.

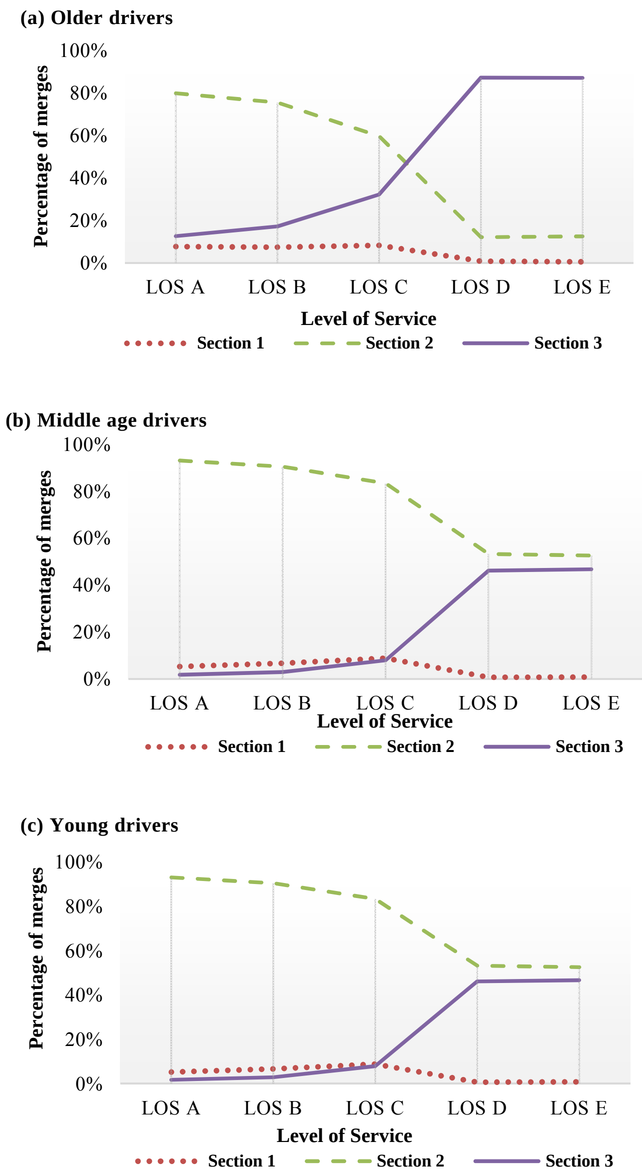


Figure 5.13. Percentage of merges with change in LOS: Pine Ridge entrance.

CHAPTER 6 CONCLUSIONS AND FUTURE WORK

This research focused on the evaluation of merging behavior with several age groups under different geometric conditions. Two sites were selected along I-75 in Lee County, Florida. Lee County was selected as a suitable study area due to its attractiveness as a retirement destination and the relatively higher percentage of older drivers. The length of acceleration lane differed at the two study sites, with the I-75 on-ramp at Pine Ridge Road having a 1500 feet acceleration lane, and the I-75 on-ramp at Corkscrew Road with a 1300 feet acceleration lane.

Merging Behavior

An extensive field study was conducted, which provided data necessary for evaluating of age on merging speeds, gap acceptance and merging location. Findings reveal that the merging position appears to be significantly influenced by driver age at both sites. Older drivers were observed to merge more at the end of acceleration lane, whereas younger drivers tend to merge earlier, near the beginning of the acceleration lane. However, vehicle approach speed for ramp vehicles appeared to have no correlation with the merging position of vehicles along the acceleration lane at either site. The gap size, however, appeared to influence the merging position of vehicles at both the Corkscrew and Pine Ridge entrances. The influence of gap size at both ramps suggests that most drivers are more likely to accept a larger gap regardless of driving age, and that larger gaps appear to encourage more early merges.

Gap Acceptance Analysis

A regression analysis was conducted to determine the effects of driver age, merging position, and approach speed and gap size on gap acceptability. Two models were proposed for each site for the rejection/acceptance of gaps. A Binary Logistic regression analysis was used, expressing the probability of acceptance and rejection of gaps as a function of driver age, gap size, merging position, merging distance and approach speed. Interesting to note, gap size, and merging section were the most influencing factors on I-75/Corkscrew on-ramp, whereas for the I-75/Pine Ridge on-ramp, the gap size, age, and approach speed have influence on the rate of accepting gaps.

Sensitivity Analysis

VISSIM, a popular simulation software, was used in the sensitivity analysis. Two models, one for each site were developed and calibrated to replicate on-site observations with the use of guidelines from the /The results reveal that the merging speed decreases when the level of service changes from a LOS A to a LOS E for each driver type. Younger drivers tend to merge at the beginning of acceleration lane, thus classifying them as aggressive drivers.

Limitations of the Study and Future work

Although positive contributions were found from this study, several limitations exist. These limitations and suggestions for future work are summarized below.

Firstly, this study was limited to the geometric characteristics of the study sites, where the merging ramp is a parallel entrance connection to the mainline traffic stream, ending with a pavement taper section and not extended to next freeway interchange. The length of the acceleration lanes included in the study was 1000 feet and 1500 feet. Additional research on taper ramp connections, acceleration lanes extending to the next ramp, as well as a greater range of acceleration lane lengths is needed to corroborate the current analysis results and add to the general body of knowledge.

Secondly, since this study focused on merging ramp-vehicle behavior with the consideration of driver age difference, the behavior of the leading and lagging vehicle in the right-most mainline traffic lane was not analyzed due to lack of sufficient data. The analysis of driving behavior exhibited by vehicles in right-most mainline lane at the merging sections would be a good research topic.

Thirdly, the identification of driver age group was limited to visual observation in the field research to obtain the age information. This method did not consider surveying each driver since it was hard to track them to place where they can stop. Furthermore, more distant methods such as collecting the license plates and obtaining the age information by a questionnaire can be considered for further research.

Regarding vehicle types, this study was limited to passenger cars, and did not consider heavy vehicles due to the low number of heavy vehicles observed at the two sites. Heavy vehicles may have a significant impact on gap choice, acceleration rates, merging positions along the acceleration lane, and the speed change when travelling the acceleration lane. Future research on this topic should be considered for areas with greater heavy truck volume.

Based on the results from this study, some variables were significant for one site and not the other. Future studies can research and identify more contributing factors influencing different merging behaviors of ramp vehicles. Additionally, a control site can be studied with similar geometrics but a lower percentage of older adult drivers and compare the drivers merging behavior with the results from this study.

GLOSSARY

Acceleration rates – The rate of change of velocity of vehicles. In this research, acceleration rate is the velocity change of merging vehicles as they travel on the acceleration lane.

Approach speeds – The speed of ramp vehicle as it approaches the merging section.

Aggressive drivers – In this research, aggressive drivers are referred as those drivers who express properties of tailgating, flashing headlights, speeding or weaving through traffic and it related to drivers who are under the age of 25 years (young drivers).

Arterial road – Is one of the functional classification of roadways with highest level of service at the greatest speed for the longest uninterrupted distance with some degree of access control.

Comfortable gap – An ideal gap accepted by the ramp vehicles to enter the mainline traffic stream without causing a freeway vehicle to reduce its speed or change lanes.

Critical gap – The minimum gap that most of the ramp vehicle will consider as acceptable.

Defensive drivers - In this research, defensive drivers are referred as those drivers who express properties of driving with low speed and large headway between vehicles, which related to drivers over the age of 65 years (older drivers).

Early merge – In this research, early merges are those merging maneuver performed by ramp vehicles at the beginning of the acceleration lane.

Entrance ramp – Is a small section of the road which allow vehicles to enter from local roads to arterials such as freeway.

Gap – The time that a subject vehicle needs to merge adequately safely between two vehicles. In this research, is considered as the time interval after the arrival of a ramp vehicle at the merging end until the arrival of the first right most lane vehicle at the exact point.

Fiducial marks – Is an object placed in the field of view of an imaging system which appears in the image produced for the use of reference or a measure.

Forced merge – The ramp vehicle effects the merging maneuver into the freeway traffic stream which forces the oncoming freeway vehicle to either change lane or slow down. In this research, it was referred as those maneuvers which occurred at the end of acceleration lane due to the presence of the end of acceleration lane.

Merging – The process by which the vehicles in two separate traffic streams moving in the same direction join to form a single stream. In this research, the two stream include the mainline traffic stream on the freeway and the traffic stream coming from the entrance ramp.

Merging section – In this research, is the area along the acceleration lane where vehicles can join the main traffic stream of the freeway.

Lane change – The movement of vehicles from one traffic lane to the next adjacent traffic lane.

Lag spacing – The distance between ramp vehicle and the reference point subtracted from the distance between the following right most vehicles and the same reference point measured in feet.

Leading spacing - The distance between ramp vehicle and the reference point subtracted from the distance between the leading right most vehicles and the same reference point measured in feet.

Level of service – A measurement used to describe the quality of traffic on roadways, which ranges from free flowing traffic to constant traffic jams.

Ramp –The small section of roadway which allow vehicles to enter of exit the control access highway (freeway).

Right most lane – The lane to which the ramp vehicles will eventually merge.

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