Phase II: Operational and Safety-Based Analyses of Varied Toll Lane Configurations

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List of Abbreviations

ACS	American Community Survey
ASCE	American Society of Civil Engineering
ANOVA	Analysis of variance
AADT	Annual average daily traffic
CG	Comparison group
CS	Cross-sectional
DR	Deceleration rate
DRD	Deceleration rate difference
DRAC	Deceleration rate to avoid crash
DOT	Department of Transportation
DSD	Desired speed distribution
DeltaS	Difference in vehicle speeds
ETC	Electronic toll collection
EB	Empirical Bayes
ETLs	Express toll lanes
FHWA	Federal Highway Administration
ft	Feet
FDOT	Florida Department of Transportation
GPLs	General-purpose lanes
HOT	High-occupancy toll
HOV	High-occupancy vehicle
ISAT	Interactive Scenario Authoring Tools
LOS	Level of service
MLs	Managed lanes
MaxD	Maximum deceleration
MaxDeltaS	Maximum difference in vehicle speeds
MaxS	Maximum speed
ML	managed lane

MPH	Mile per hour
NADS	National Advanced Driving Simulator
NCHRP	National Cooperative Highway Research Program
PET	Post-encroachment time
PDO	Property damage only
RITIS	Regional Integrated Transportation Information System
SEC	Second
SSAM	Surrogate Safety Assessment Model
TMT	Tile Mosaic Tool
TIT	Time-integrated time to collision
TTC	Time to collision
VSL	Variable speed limit
VEH	Vehicle
VPH	Vehicles per hour

Abstract

On expressways, managed lanes (MLs) have been introduced as an effective dynamic traffic management strategy. This research consists of two parts: a microsimulation study and driving simulator experiments for appropriate designs for the MLs.

The objective of the microsimulation research was to determine optimal access zone density and weaving length. In the simulation, the lane choice replicated drivers' choice behavior at dynamic tolls based on modeling components and algorithms generated in VISSIM. The network was well calibrated and validated by comparing the operational measurements for simulated and field data. Subsequently, forty-two scenarios were built and tested in VISSIM to specify the optimal accessibility level and to decide on the sufficient weaving distance. The findings indicate that there was a significantly lower conflict risk in MLs than in general-purpose lanes (GPLs). Compared to GPLs, the conflict frequency per vehicle in MLs was less by 48% and 11% in the peak and off-peak traffic conditions, respectively. A Tobit model and a log-linear models were developed for investigating the factors and scenarios that affect traffic conflict frequency. The results of the conflict frequency analysis suggest that one access zone is the optimal accessibility density in the 9-mile segment. Moreover, the results revealed that a length of 1,000 feet per lane change is the optimal length for the weaving segments near access zones. A series of linear regression models was developed to explore the effects of access zone design on the operational performance of the network. The modeling results confirm that one access zone is the optimal level, with a higher speed, a lower delay, and a higher time efficiency than other cases. As the accessibility level increases, the operational performance declines. From the revenue perspective, the case of two access zones creates the largest revenue in the studied network. The traffic operation analysis also revealed that the level of service was the same for the base case with no access zones and the case with one access zone when the weaving distance was higher than 1,000 feet per lane change.

The driving simulator experiment aimed to evaluate the impact of different weaving lengths and variable speed limit (VSL) strategy on drivers' speed control and lane-changing maneuvers. It was found that long weaving lengths (i.e., 1,000 feet and 1,400 feet per lane) resulted in a reduction of average speed and that a weaving length of 1,400 feet per lane had significantly higher speed standard deviation when compared with the other two weaving lengths. In addition, the VSL strategy can reduce the average speed and speed variation. As for the lane-changing behavior, drivers can have the safest performance with 1,000-foot weaving length, in terms of time to collision and number of conflicts. Finally, fewer conflicts could be found in the scenarios with VSL strategy. Another research effort was conducted to compare driving behaviors considering drivers' gender and age. The experiment results showed that young drivers were prone to drive more aggressively, which resulted in higher speed standard deviation. Also, it was revealed that males have more conflicts when changing lanes than females. It is expected that the results from this study can help engineers and practitioners employ appropriate weaving length, access zone density, and traffic control strategy to enhance traffic operation and safety for MLs.

1 Introduction

On expressways, managed lanes (MLs) have emerged as an effective dynamic traffic management strategy. They play an important role in improving traffic mobility, efficiency, and safety, in addition to generating revenue for transportation agencies. Previous research has indicated that the installation of MLs has improved the traffic operation and safety of expressways. However, most studies explored safety and operational impacts for the whole segment without considering accessibility levels and weaving distance. In this study, the effects of accessibility levels and weaving on the safety and operation on MLs are investigated. The studied accessibility level varies from one to three access zones along the network. The weaving distance was defined as the distance per lane change to enter the access zone from the on-ramps or to exit the access zone to the off-ramps.

This research consists of two parts: a microsimulation study and driving simulator experiments. In the microsimulation study, the research team collected extensive data from microsimulation scenarios that included a 9-mile network of an ML segment on Interstate 95 in South Florida. VISSIM microsimulation was used for developing the network due to its feature of simulating dynamic priced MLs. In the simulation, the lane choice replicated drivers' choice behavior at dynamic tolls based on modeling components and algorithms generated in VISSIM. The network was well calibrated and validated by comparing the operational measurements for simulated and field data. Subsequently, forty-two scenarios were built and tested in VISSIM to specify the optimal accessibility level and to decide the sufficient weaving distance. Six measures of effectiveness were determined to evaluate the safety and efficiency of different scenarios. For the safety measurements, conflict frequency and conflict rate of the weaving segments were used. For the operational measures of effectiveness, the level of service (LOS), travel speed, time efficiency, and average delay were used. Moreover, the revenue was estimated to compare the monetary benefits of various strategies. The findings indicate that there was a significantly lower conflict risk in MLs than in general-purpose lanes (GPLs). Compared to GPLs, conflict frequency per vehicle in MLs was reduced by 48% and 11% in the peak and off-peak traffic conditions, respectively. A conflict prediction model was developed for investigating the factors and scenarios that affect traffic conflict frequency. The result of the conflict frequency analysis suggests that one access zone is the optimal accessibility density. Hence, it can be concluded that the average distance between access zones should be no less than 4.5 miles. Moreover, the results revealed that a length of 1,000 feet is the optimal length for the weaving segments near access zones. A series of linear regression models was developed to explore the effects of access zone design on the operational performance of the network. The modeling results confirm that one access zone is the optimal level, with a higher speed, a lower delay, and a higher time efficiency than the other cases. As accessibility level increases, the operational performance declines. From the revenue perspective, the case of two access zones creates the largest revenue in the studied network. The traffic operation analysis also revealed that LOS was the same for the base case with no access zones and the case with one access zones when the weaving distance was higher than 1,000 feet per lane change.

Meanwhile, the driving simulator experiment study aimed to evaluate the effects of different weaving lengths and variable speed limit (VSL) strategy on traffic safety when drivers enter and exit toll managed lanes (MLs). Twelve driving simulator scenarios were developed considering three different weaving lengths (600 feet, 1,000 feet, and 1,400 feet), with/without VSL strategy, and peak/off-peak traffic flow. Fifty-four participants were recruited in this experiment. Drivers' speed control and lane-changing maneuvers were investigated. Repeated measures analysis of variance (ANOVA) was employed to analyze the operational and safety effects of different factors. It was found that long weaving lengths (i.e., 1,000 feet and 1,400 feet per lane) resulted in the reduction of average speed and that the weaving length of 1,400 feet per lane had a significantly higher speed standard deviation than the other two weaving lengths. As for the lane-changing behavior, drivers showed the safest performance with a 1,000-foot weaving length, in terms of time to collision (TTC) and the number of conflicts. In addition, the VSL strategy reduced the average speed and speed variation and resulted in fewer conflicts. Subsequently, a comparative analysis was conducted for driving behaviors considering drivers' gender and age. The results showed that young drivers were prone to drive more aggressively, which resulted in higher speed standard deviation. Also, it was revealed that male drivers have more conflicts when changing lanes. It is expected that the results from this study can help engineers and practitioners employ appropriate weaving length and traffic control strategies to enhance traffic safety for drivers when they enter and exit MLs.

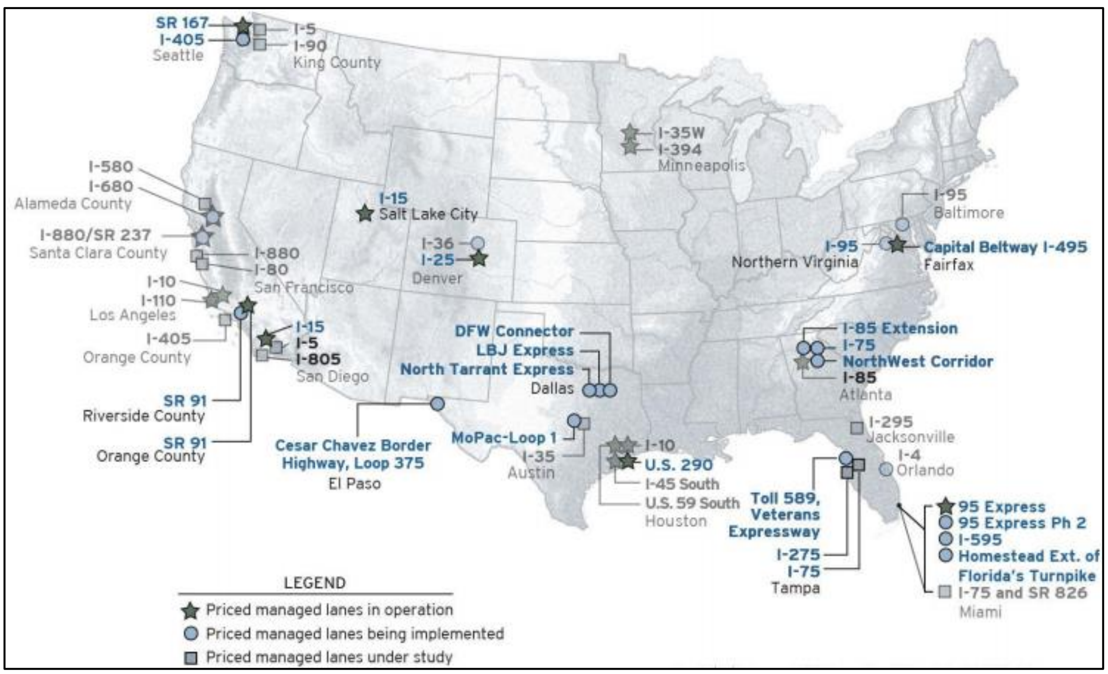
This report consists of four chapters: Chapter 1 is the introduction. Chapters 2 and 3 describe the research efforts for the microsimulation research and driving simulator experiment research, respectively, and both chapters include their own introduction, literature review, experiment design, result, and conclusion sections. Lastly, Chapter 4 summarizes and concludes the report.

2 Microsimulation Approach

2.1 Introduction

Managed lanes are designated lanes where the flow of traffic is managed by limiting vehicle eligibility, restricting facility access, or employing variable-price tolls (1). They have emerged as an effective dynamic traffic management strategy. In recent years, several major cities in the United States have introduced ML systems such as expressway toll lanes (ETLs), high-occupancy toll (HOT) lanes, or high-occupancy vehicle (HOV) lanes.

Managed lanes are a vital option for managing time and congestion through tolling and providing drivers with more choices. In 2013, the American Society of Civil Engineers (ASCE) estimated that the cost of congestion for wasting fuel and time was $101 billion annually and the average time spent for American drivers in traffic is about 38 hours annually. In U.S. states, tens of MLs are being implemented or under development, as shown in Figure 2.1. By 2020, MLs are projected to grow in the U.S. by 6,000 lane-miles because they are an appropriate option to deal with high congestion and high crash frequency with a viable cost effectiveness for promoting economic development. Toll revenue can support half of the repayment of the $1 billion asset of the facility (2).



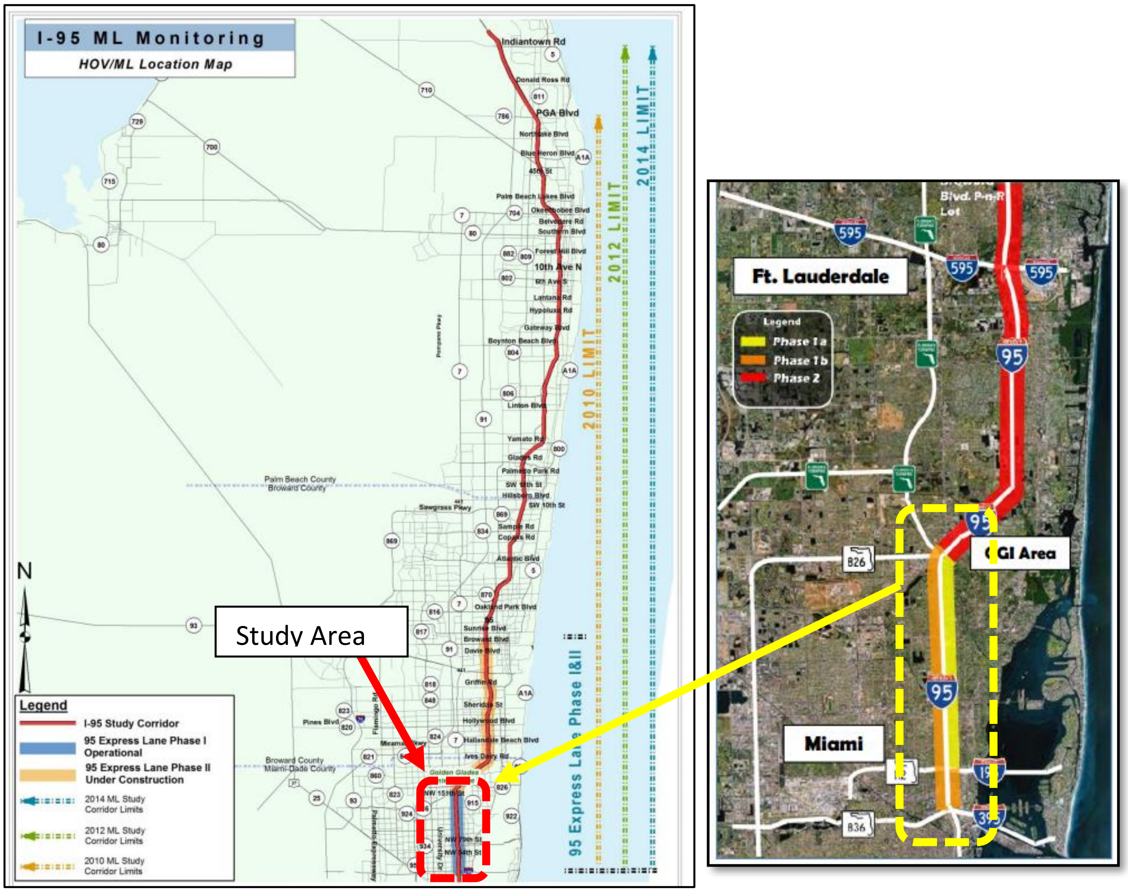
Source: HNTB, 2013 *(2)*

Figure 2.1 - Priced managed lanes in the U.S.

In order to efficiently and safely operate the ML systems, it is necessary to determine the optimal access control level. If the access control is strictly restricted, some vehicles on heavily congested GPLs cannot enter the MLs even if they are willing to pay tolls. Also, vehicles currently traveling on the MLs are not able to exit when they want. On the other hand, if there is no access control, vehicles on GPLs can enter the MLs all the time, but the LOS and traffic safety on MLs are not guaranteed. Thus, a tradeoff between the accessibility, efficiency, and safety is inevitable to some extent.

Once the optimal access control level of the MLs is determined, the next step is to decide the configuration and location of the access. Two major parameters need to be considered: first, the distance from an upstream MLs exit to the next downstream off-ramp; second, the minimum distance from an upstream on-ramp to the next downstream MLs entry. VISSIM was used since it simulates lane choice based on dynamic tolling. A logit model is in VISSIM to decide the possibility of choosing MLs based on tolls and time savings. Therefore, the primary research objectives of this project can be summarized as follows: using microscopic simulation to determine an optimal accessibility level to maximize system-wide efficiency and determining sufficient length and location of access zones near on- or off-ramps.

The simulated area consists of nine miles of MLs located in the northbound direction of the I-95 corridor in South Florida. The locations of the existing MLs and the study area are shown in Figure 2.2 *(3, 4)*.



Source: Cambridge Systematics, 2014; FDOT, 2012 (3, 4)

Figure 2.2 - Location of the existing MLs in I-95

The research team worked on building a microsimulation network for evaluating the optimal control level for the MLs. First, the field ML network's geometry and traffic were well replicated in the VISSIM microsimulation. Afterward, the calibration and validation of the VISSIM simulation network were followed. Subsequently, the experimental design was conducted, including various scenarios, which were based on different access levels, access configurations, and traffic conditions. The safety performance of different scenarios was analyzed with the Surrogate Safety Assessment Model (SSAM). Two types of safety measurements were used: the conflict frequency and the conflict rate. The operational measurements included LOS, average speed, average delay, and time saved by using MLs. Furthermore, the revenue generated by the MLs was also computed.

The flow chart of the simulation process is shown in Figure 2.3. This chapter is composed of six sub-chapters. The first two sub-chapters are the introduction (2.1) and the literature review (2.2) of the research. Sub-chapter 2.3 is the experimental design. Sub-chapter 2.4 presents the microsimulation process for the studied network, which mainly includes network building, calibration, and validation. Sub-chapter 2.5 shows the principal findings of this project based on evaluating the safety and operation of different ML designs. Lastly, sub-chapter 2.6 gives a summary and conclusion of the results in addition to discussing the implication of the findings to future research.

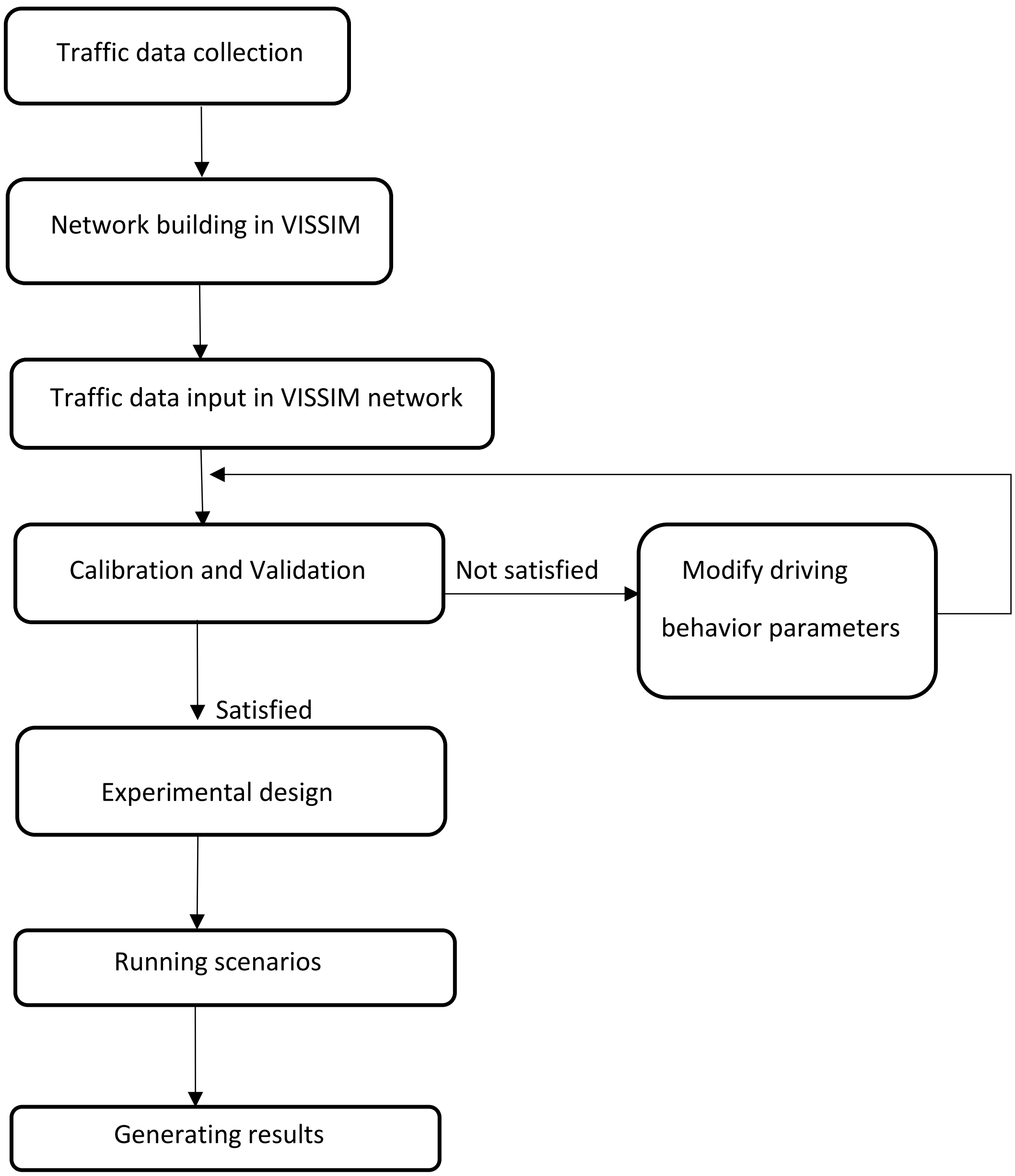


Figure 2.3 - Simulation process flow chart

2.2 Literature Review

2.2.1 Managed Lanes

The primary purpose of the MLs is to manage and expedite the flow in a segment through access control (i.e., entrances and exits), vehicle eligibility (i.e., vehicle type and vehicle occupancy), or pricing (i.e., tolls and dynamic tolls) strategies *(5)*. As presented by the Federal Highway Administration (FHWA) *(6)*, MLs are a valuable option for transportation agencies to manage traffic congestion. In addition, it is a better solution than expanding freeways in terms of construction cost, right-of-way constraints, and environmental impacts. The use of priced ML systems has risen dramatically in the U.S. in recent years due to improving improved time reliability, time savings, mobility, congestion management, and revenue generation *(7)*. The toll revenue is used to fund the facility through the dynamic tolls that vary based on time savings and traffic conditions. As the traffic increases in the MLs (i.e., peak hours), the toll price increases to maintain the operating speed at the MLs *(8)*.

As discussed by Cho et al. *(9)*, the presence of priced MLs proved to reduce traffic congestion and utilize the transportation infrastructure more efficiently. They studied the willingness to pay for travel time savings and found that travel time savings are not the only thing that influences use of dynamic priced MLs. They found that the time value of using the priced lanes is $73/hour in the morning period and $116/hour in the afternoon session on I-394 in Minnesota. Meanwhile, the economic benefit of the tolling lanes was $5 million between 2006 and 2008 *(9)*.

The latest ML guidelines report from the National Cooperative Highway Research Program (NCHRP) pointed out that MLs provide better operational and safety performance than GPLs. The crashes in MLs are mainly due to access zones, congestion, and sight distance. One of the countermeasures suggested by the NCHRP is to appropriately locate the access zones and traffic control devices. The NCHRP report also concluded that the most frequent crash types in the MLs facility are rear-end crashes, because of congestion, as well as sideswipe crashes due to lane changing within access zones *(10)*. Limited research has been conducted on the evaluation of safety and operation benefits when improving the geometric design of the GPL segments close to the access zones. The limitation of the geometric data availability and the small sample size are the main reasons behind limited studies in the MLs *(10)*.

One of the studies that focused on the effect of geometric design on the safety of MLs was conducted by Jang et al. *(11)*. In the study, 153 miles of MLs (13 Southern California segments) and three years' crash data (2005 to 2007) were used. The authors found that there was a relationship between the safety performance of the MLs and the cross-section design, including lane width, shoulder width, and buffer width. Additionally, segments with wide shoulder width were more likely to have fewer crashes. They recommended adding a buffer to all segments and reallocating shoulder width to the buffer *(11)*. The Florida Department of Transportation (FDOT) conducted a study to estimate the expected crash frequency for the MLs of urban freeway segments *(4)*. The results of the study showed that fatal and injury crashes decreased when an appropriate buffer type and width (2-3 feet) are considered. The widening of the left shoulder width also was associated with lower crash frequency *(4)*.

A recent study conducted by Abuzwidah and Abdel-Aty *(8)* analyzed crash data for 156 segments on I-95 for 9 years (2005 to 2013) using three methods, which included a before-after with comparison group (CG) and the empirical Bayes (EB) methods for evaluating the Crash Modification Factors (CMFs) for severe crashes data only. Also, a cross-sectional method (CS) was used for total and property damage only (PDO) crashes. Compared to GPLs, the total crashes in the MLs decreased by 20% and the severe crashes (fatal and injury) reduced by 30%. Traffic operational measurements (i.e., travel speed, volume, LOS) were also used in previous studies for comparing the traffic operation performance between GPLs and MLs. Previous studies concluded that the LOS in the MLs is better than the LOS in the GPLs. Vehicles traveled at a higher speed in the MLs than the GPLs. Meanwhile, the volume in the MLs increases during morning and afternoon peak hour conditions *(8)*.

2.2.2 Access Zones

Access zones are some of the most dangerous locations on GPL segments. Crashes frequently occur near the entrances or exits of MLs. Two types of crashes are common: sideswipe and rear-end crashes. Sideswipe crashes happen due to lane-changing maneuvers upstream from the ML entrances or exits. Meanwhile, rear-end crashes occur because of vehicles that brake before MLs to avoid crashing with other vehicles *(10)*.

There are multiple approaches for providing access to MLs: continuous access, restricted at-grade access, and grade-separated access. Recently, there has been an interest in continuous access, where vehicles could use the priced MLs at any point. Experiences from the design of access zones for MLs have resulted in several recommendations *(14)*. First, the geometric criteria for access zones should be the same as those used for freeway ramps, including locally recognized entrance and exit standards. Second, the location of ingress/egress facilities is influenced by some factors. For example, direct access ramps to/from local streets should be made with candidate streets that currently do not have freeway access to distribute demand better and prevent overloading existing intersections. For at-grade access to the adjacent freeway lanes, designated outlets should be strategically positioned to minimize erratic weaving to reach nearby freeway exits. Third, the location of ingress/egress points should be associated with street access away from intersections that are operating at or near traffic capacity. Fourth, vehicles entering the MLs facility should be required to make a maneuver to get into the lane. Fifth, the ramps to MLs should provide adequate space for possible metering and storage. Sixth, proper advance signing should be provided, and pavement markings should emphasize the mainline. Seventh, safety lighting should be applied for all ingress/egress locations using the same warrants applied for urban freeway entrance and exit ramps. Provision for entrance ramp metering and enforcement should be considered.

Access zones crashes are fundamentally affected by access zone type, traffic condition, and the weaving segment length upstream or downstream of the facility. For access zone type, no significant difference was found between the limited-access HOV lanes and the continuous-access HOV lanes. Traffic crashes on the ML facilities are mainly concentrated near the access zones. Meanwhile, the high crash frequency is associated with small access length and close access points to the on- or -off-ramps *(15, 16)*.

2.2.3 Managed Lanes Simulation

Recently, simulation studies for ML facilities have been increasing in order to analyze driving behavior, as well as the safety and operation impacts in a driving simulator or a microsimulation. The main purpose of the simulation is to test countermeasures and changes to a freeway before implementation *(17)*. The main problem in ML simulation is the calibration of the network to the real conditions. Another issue is the intensive required data that are used for coding the network *(18)*. Recently, microsimulation data have been integrated with the SSAM to compute surrogate safety measures for vehicle interactions. SSAM is software developed by Siemens and sponsored by the FHWA. The primary objective of SSAM is to evaluate the safety performance of current roadway designs or a new strategy before implementation *(19)*. In this study, SSAM was adopted to determine the conflict frequency from the microsimulation data, which is highly correlated with the crash rate in the field *(20)*.

2.2.4 Summary

In general, the literature supports the notion that MLs are an important countermeasure for improving the safety and the traffic operation of expressways. Nevertheless, little is known about the interrelationship between the ML design and the efficiency of the network. Previous studies show that access zones are risky locations in the ML segment. Hence, there is a need for studying the safety and operational impacts of access zones on the facility. The research team used a micro-traffic simulation, as it is a valid approach for studying the safety and operation effectiveness of the access zone design.

2.3 Experimental Design

2.3.1 Accessibility level

Three accessibility levels were tested in this study including one, two, and three access zones (Figure 2.4). The base condition is the current situation of the network, which does not have any access zones along the study area. The first case of the experimental design has one entrance and one exit in the middle of the network. This case is divided into two types: an egress upstream of an ingress (Case 1A), and an ingress upstream of an egress (Case 1B). Case 2 involves adjusting the network to have two ingresses and two egresses. Case 3 has three ingresses and three egresses.

The preliminary results of the first condition showed no significant difference between the cases (Cases 1A and 1B). However, Case 1A showed fewer conflicts than Case1B in most of the studied scenarios. Therefore, Cases 2 and 3 were only tested with the egress upstream of the ingress. The accessibility level cases are shown in Figure 2.4. The purple arrows represent the directions of vehicles that use the ingress from on-ramps, while the red arrows represent the directions of vehicles from the egress to the off-ramps. L_1 and L_2 are the lengths of the weaving segments near access zones.

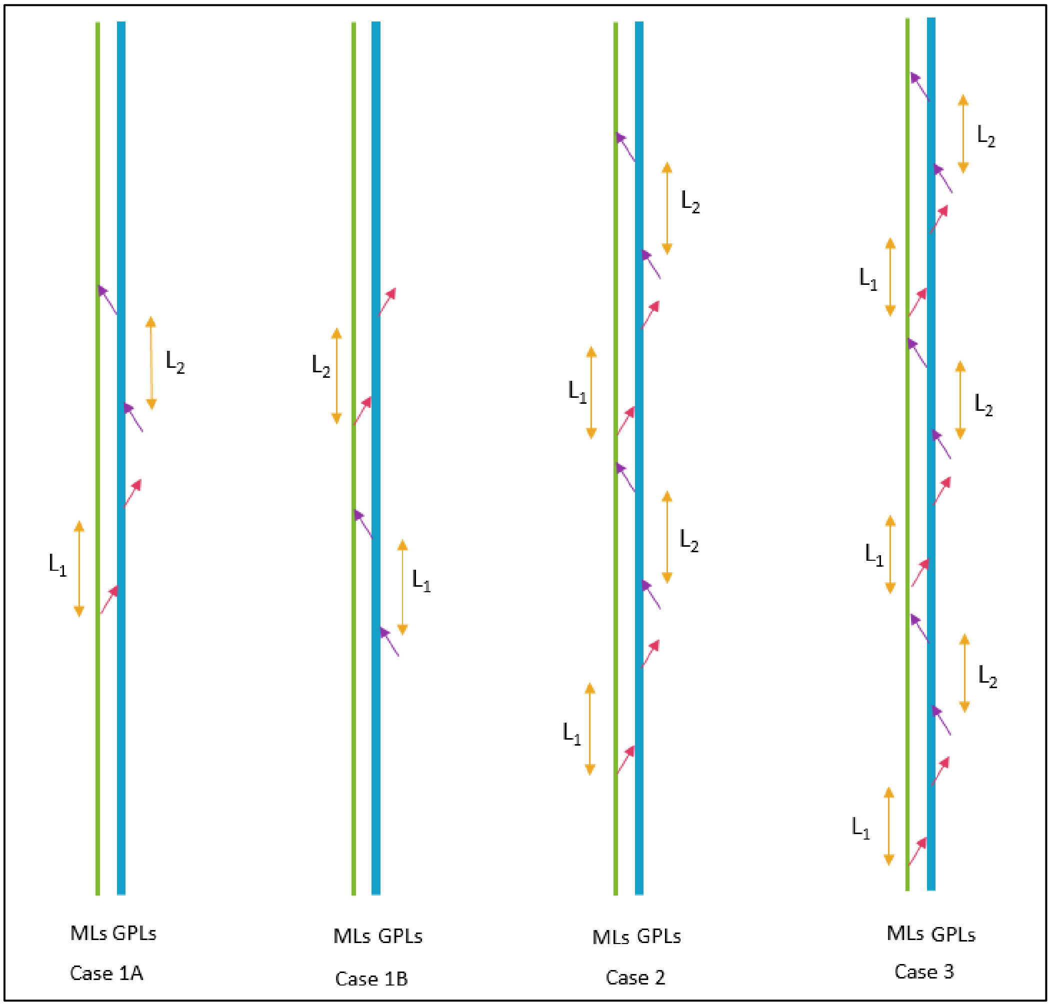


Figure 2.4 – Accessibility level cases

2.3.2 *Weaving Segments*

The access zones usually form weaving segments since on-ramp vehicles want to enter the MLs through ingress and off-ramp vehicles want to exit MLs through egress. These on- and off-ramp vehicles will weave with the mainline traffic on GPLs. Hence, the study of the access zones focuses on the design of the weaving segments. Two types of weaving segments were studied in the VISSIM network: (1) the ingress weaving segment, which is from the on-ramp to the ingress, and (2) the egress weaving part, which is from the egress to the off-ramp. Figure 2.5 shows the weaving segments where L_1 is the ingress weaving segment length and L_2 is the length of the egress weaving segment.

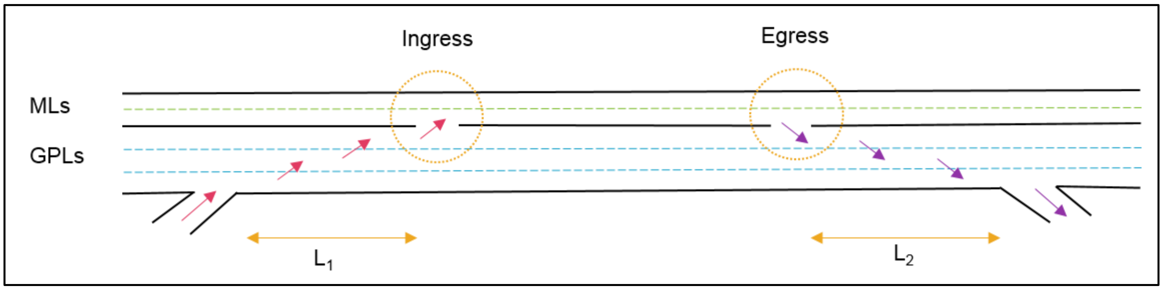
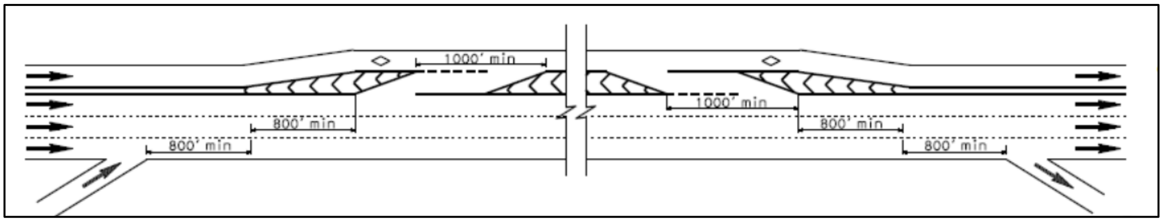


Figure 2.5 - Weaving segments

Previous studies explored the efficient weaving distance. One of these studies was conducted by the California Department of Transportation *(15)*, which suggested a minimum distance of 800 feet per lane change between the on- or off-ramps and the access zones, as shown in Figure 2.6.



Source: California DOT report, 2001 *(15)*

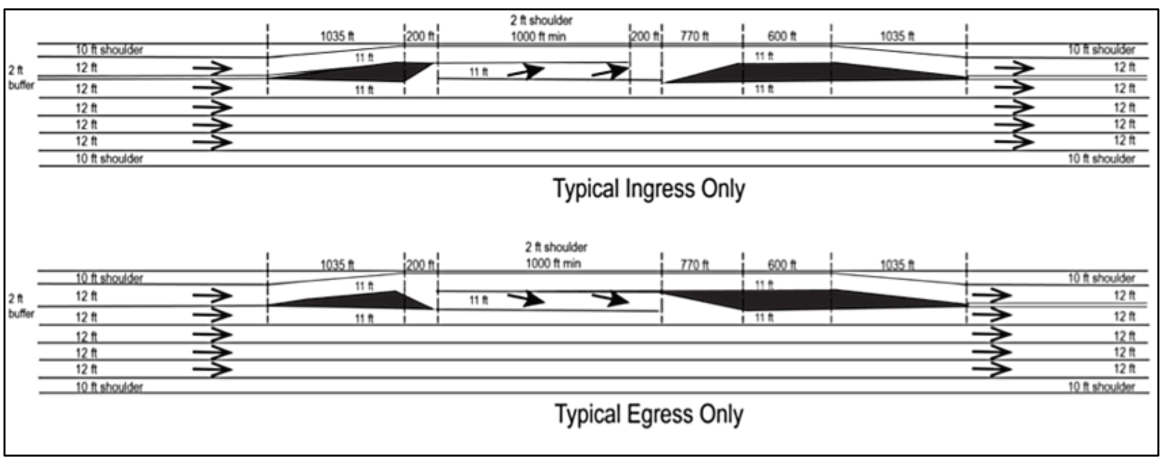
Figure 2.6 – Minimum weaving distance for access zones (min=minimum)

Another study conducted by the Washington Department of Transportation *(21)* proposed the minimum distance between the access zones and the on- or off-ramps to be 500 feet per lane change. Meanwhile, the study recommended that the desired distance is 1,000 feet per lane change, which is double the minimum distance. Also, another study, conducted by Venglar et al. *(22)*, offered that the range of the weaving distance varies between 500 and 1,000 feet. They provided various cases of the weaving distance as shown in Table 2.1. Meanwhile, they concluded that the minimum distance between the ingress and the egress of the MLs was 2,500 feet. Additionally, the NCHRP guidelines for implementing MLs suggested that the spacing between access zones should be between 3 and 5 miles *(10)*. The ingress and egress design of this study followed the recommendation of the FHWA *(23)*. The detailed designs for the ingress and egress are shown in Figure 2.7.

Table 2.1 Weaving distances for MLs

Design Year Volume Level	Allow up to 10 mph Mainline Speed Reduction for Managed Lane Weaving?	Intermediate Ramp (between Freeway entrance/exit and MLs entrance/exit)?	Recommended Minimum Weaving Distance Per Lane (feet)
Medium (LOS C or D)	Yes	No	500
		Yes	600
	No	No	700
		Yes	750
High (LOS E or F)	Yes	No	600
		Yes	650
	No	No	900
		Yes	950

Source: Venglar et al., 2002 *(22)*



Source: FHWA, 2011 *(23)*

Figure 2.7 - Ingress and egress details for different cases

2.3.3 List of Scenarios

This study focuses on studying the design of weaving segments. Three accessibility cases were tested including one, two, and three access zones (Figure 2.4). In each case, five different weaving distances were applied to determine the optimal distance of the access zones. Meanwhile, the traffic volume condition has two conditions: peak and non-peak. Hence, 42 scenarios were tested in VISSIM as shown in Table 2.2. For each scenario, ten random runs with different random seeds were applied.

Table 2.2 - List of scenarios

Cases		Traffic condition	Lane-change length between the access zones and the on- or off-ramps (feet) *					Number of scenarios
Case 0 (Base condition)		Peak	No access zones					1
		Off-peak	No access zones					1
Case 1	Egress then ingress	Peak	600	800	1,000	1,400	2,000	5
		Off peak	600	800	1,000	1,400	2,000	5
	Ingress then egress	Peak	600	800	1,000	1,400	2,000	5
		Off peak	600	800	1,000	1,400	2,000	5
Case 2	Egress then ingress	Peak	600	800	1,000	1,400	2,000	5
		Off peak	600	800	1,000	1,400	2,000	5
Case 3	Egress then ingress	Peak	600	800	1,000	1,400	2,000	5
		Off peak	600	800	1,000	1,400	2,000	5
Total number of scenarios								42

* All distances are per lane change (number of lanes minus one).

2.3.4 Performance Measurements

Three types of measures of effectiveness (MOE) are used to evaluate the performance of the MLs and the GPLs: including safety measurements, operational measurements, and revenue. The detailed information is shown below:

1. Safety measurements
   - Conflict frequency
   - Conflict rate
2. Traffic measurements
   - LOS
   - Travel speed for both MLs and GPLs
   - Average delay for both MLs and GPLs

- Time efficiency (time saved by using the MLs), determined by the difference between the travel times on the MLs and the GPLs

3. Revenue, which can be computed from the dynamic toll pricing calculation.

2.4 Building Microsimulation Network

2.4.1 Study Area

The network was built in the VISSIM software based on the real-world geometric characteristics. The segment that was utilized in the VISSIM included 9 miles of GPLs and MLs on I-95 in Miami, Florida. Three types of lanes were built in the VISSIM network: GPLs, MLs, and ramps. Parts of the VISSIM network are shown in Figures 1.8 and 1.9 with the background Bing map.

The two principal components of the network are links and connectors. Links reflect roadway segments, and connectors are utilized to connect two links. In the VISSIM network, links are shown in blue and connectors are demonstrated by purple, as shown in the right pictures of Figures 2.8 and 2.9. The geometric properties of each link were adjusted to be consistent with the real network. These properties included link length, number of lanes, and lane width. Moreover, link behavior type was set to be “Freeway” since the studied segment was on an interstate. In addition, right or left lane-change behavior can be modified for each link to either permit it or prevent it. Link properties are shown in Figure 2.10.

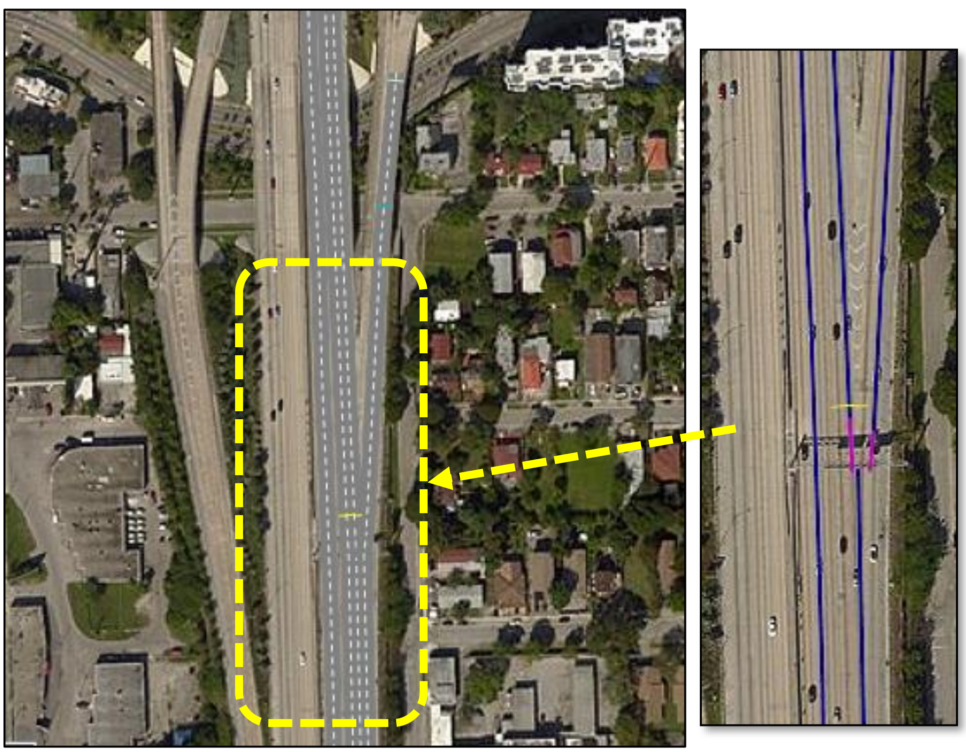


Figure 2.8 - Part of the VISSIM network (off-ramp)

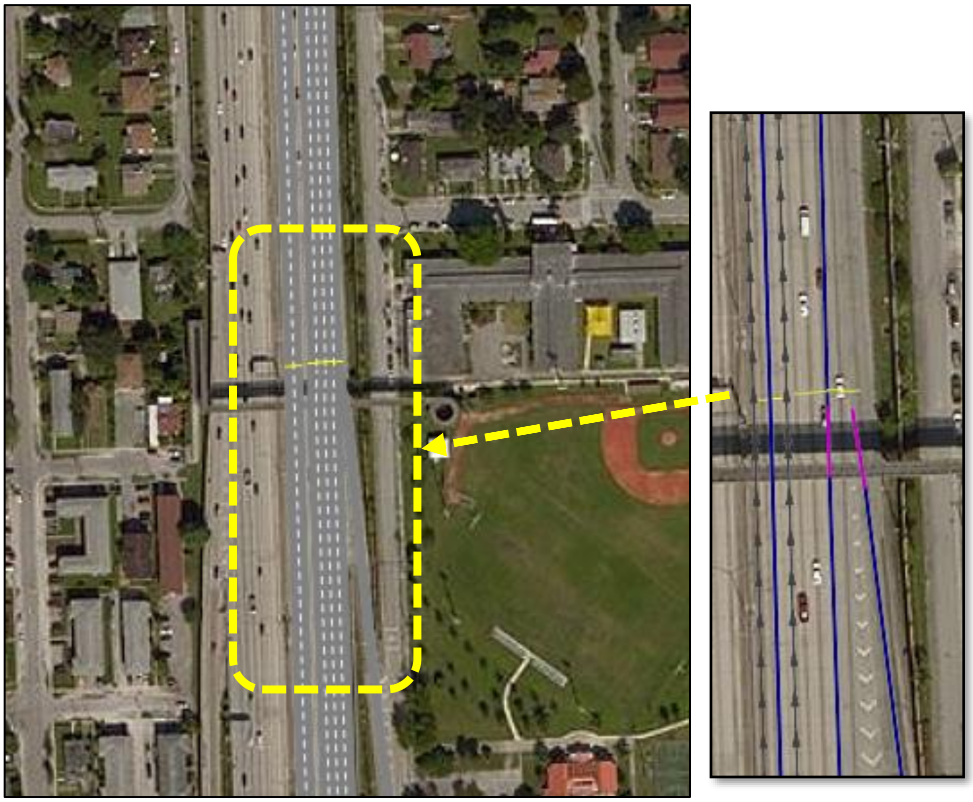


Figure 2.9 - Part of the VISSIM network (on-ramp)

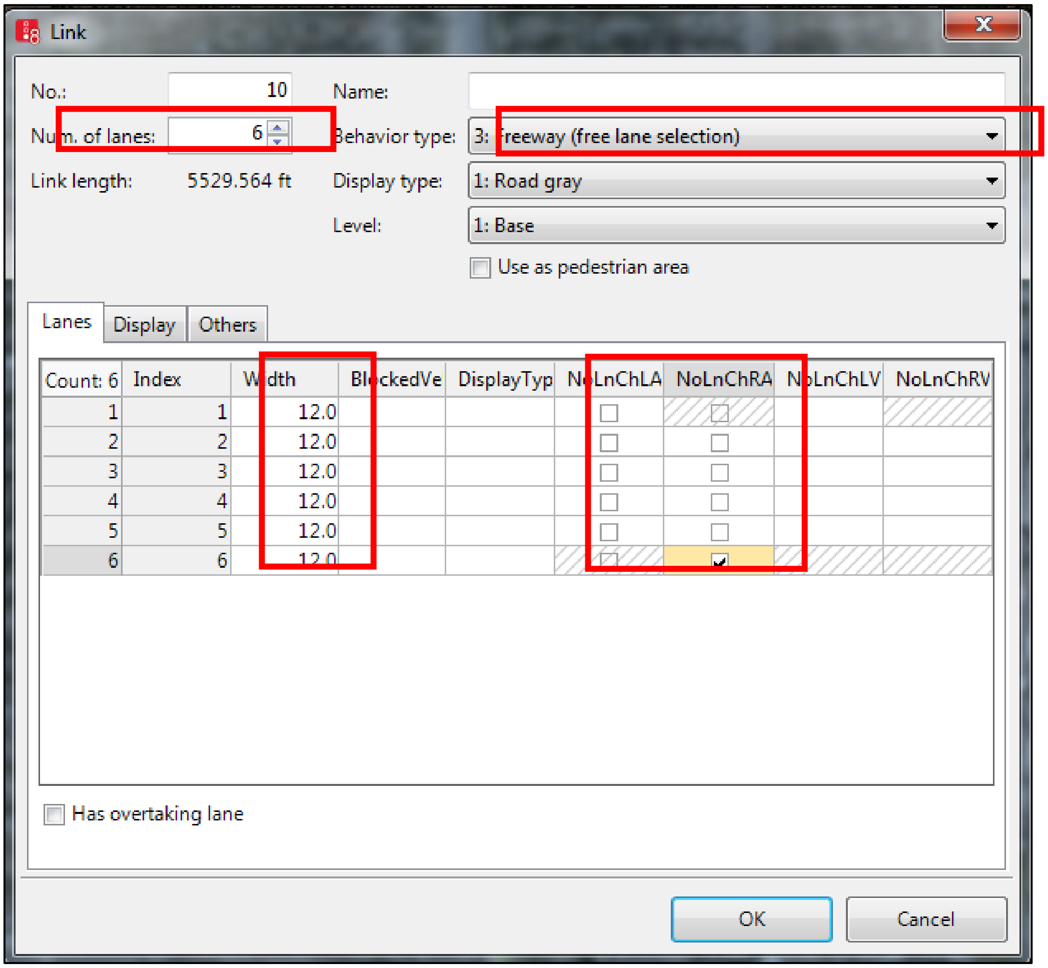


Figure 2.10 - Link properties

2.4.2 Study Period

In this study, peak hour was from 7:30 AM to 8:30 AM. The off-peak period was from 9:30 AM to 10:30 AM. It is worth mentioning that the morning peak period was chosen instead of the afternoon peak period because the morning peak period had the most severe conditions. Compared to the afternoon peak period, the morning peak period had higher volume, as shown in Figure 2.11.

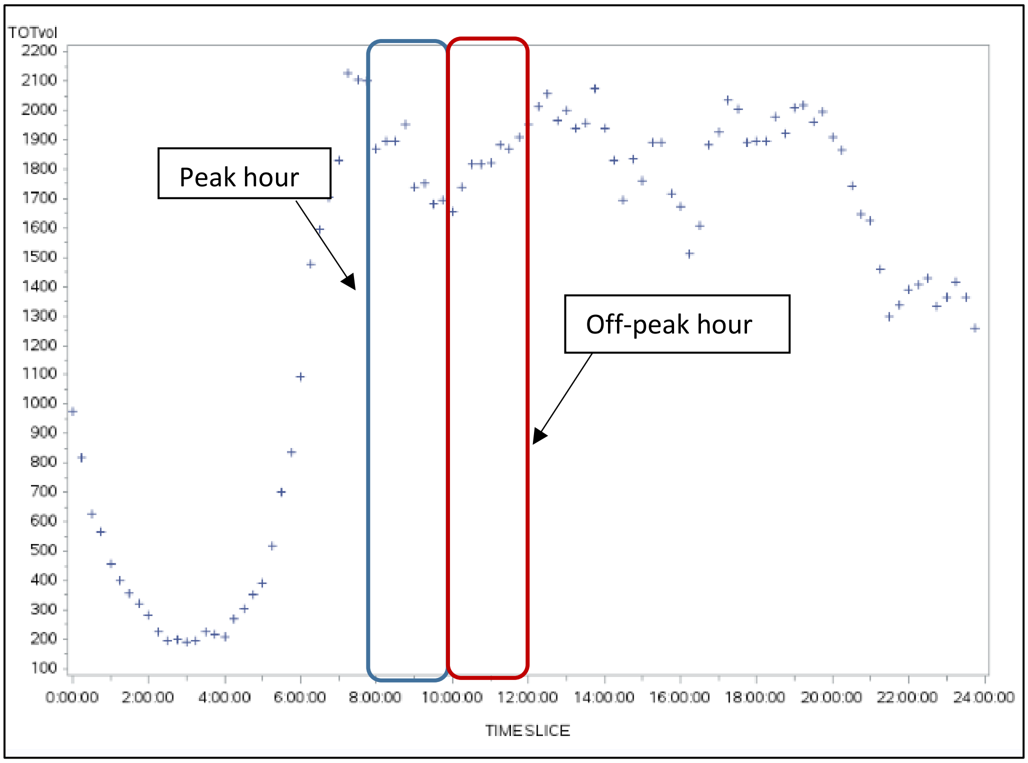


Figure 2.11 - Volume distribution

2.4.3 Network Coding

In order to output traffic information from the VISSIM network, data collection points were added to the network. The locations of the data collection points in VISSIM are the exact locations of the Regional Integrated Transportation Information System (RITIS) detectors on I-95. Figure 2.12 shows part of the coded data collection points for each detector. The code consisted of three parts. The first letter represents whether a lane is GPL (G) or ML (M). The number beside the letter shows the link number. The four numbers that follow represent the detector name in the RITIS data. Then, the number in parentheses is the lane ID. For instance, the lane ID for the rightmost lane is 1.

The information that the VISSIM collected from the data collection points includes the time when the front of a car reaches the point, the time when the rear of the car leaves the point, vehicle type, speed, acceleration, etc.

Data Collection Points

Select layout... <Single List>

Count: 5	No	Name	Pos
1	3	G 2763 (1)	3323.987
2	4	G 2763 (2)	3323.987
3	5	G 2763 (3)	3323.987
4	6	G 2763 (4)	3323.987
5	7	G 2763 (5)	3323.987
6	8	G 2763 (6)	3323.987
7	32	G10 2581 (1)	1139.278
8	33	G10 2581 (2)	1139.600
9	34	G10 2581 (3)	1139.877
10	35	G10 2581 (4)	1140.199
11	36	G12 2876 (1)	1455.914
12	37	G12 2876 (2)	1457.254
13	38	G12 2876 (3)	1456.041
14	39	G12 2876 (4)	1457.381
15	51	G12 2876 (5)	1457.214
16	40	G15 2877 (1)	401.932
17	41	G15 2877 (2)	402.462
18	42	G15 2877 (3)	402.134
19	43	G15 2877 (4)	401.408
20	44	G15 2877 (5)	399.780
21	45	G16 3042 (1)	2226.746
22	46	G16 3042 (2)	2227.757
23	47	G16 3042 (3)	2227.927
24	48	G16 3042 (4)	2226.839
25	9	G2 3088 (1)	1422.000
26	10	G2 3088 (2)	1422.000
27	11	G2 3088 (3)	1422.000
28	13	G3 2826(1)	1982.303
29	14	G3 2826(2)	1981.677
30	15	G3 2826(3)	1984.115
31	16	G3 2826(4)	1981.157
32	17	G3 2826(5)	1982.690
33	22	G6 3136 (1)	1395.915
34	23	G6 3136 (2)	1392.919
35	24	G6 3136 (3)	1391.757
36	25	G6 3136 (4)	1393.101
37	26	G6 3136 (5)	1390.106
38	27	G8 3150 (1)	1896.144
39	28	G8 3150 (2)	1896.262
40	29	G8 3150 (3)	1896.617
41	30	G8 3150 (4)	1897.089
42	31	G8 3150 (5)	1895.363
43	1	M1 3034 (1)	703.328
44	2	M1 3034 (2)	703.328
45	49	M2 2825 (1)	136.729
46	50	M2 2825 (2)	136.500

Figure 2.12 - Data collection points in VISSIM

2.4.4 Traffic Data Input

The traffic data input of the VISSIM network were based on the four Wednesdays in April 2016 to exclude random fluctuations. The locations of RITIS detectors in the study area are shown in Figure 2.13. The RITIS detectors provided detailed traffic information at 20-second intervals for each lane, including average time, mean speed, volume, and lane occupancy. In the RITIS data, if the percentage of the missing data for any detector was higher than 10%, these detectors were excluded from further analyses. The traffic data were aggregated to obtain VISSIM traffic input data at a 15-minute time interval.

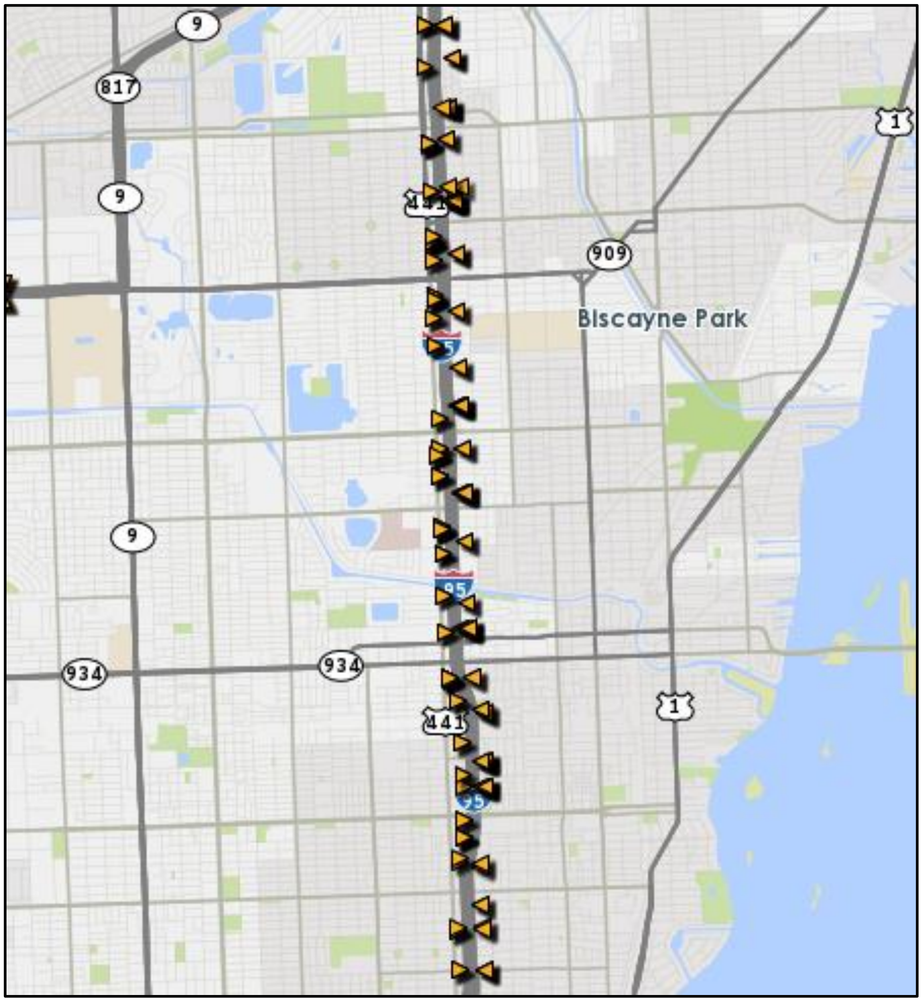


Figure 2.13 - RITIS detectors on the I-95 in Miami-Dade

2.4.5 Vehicle Classes

Three classes of vehicles were utilized in this simulation: passenger cars (PCs), heavy goods vehicles (HGVs), and carpools. According to the FDOT *(24)*, the percentage of HGVs is 5%. Meanwhile, according to the 2015 U.S. Census American Community Surveys (ACS) for Miami-Dade *(25)*, the percentage of carpools is 10%. Considering carpool percentage in this study was important as the policy of the FDOT is that carpools are allowed to use the MLs without paying tolls *(26)*.

2.4.6 Vehicle Composition

There are four types of vehicle composition in this study. The first type is vehicles that start from the beginning of the network and might have the choice to use the MLs. The second type is vehicles that start from the on-ramps and might have the choice to use the MLs. The third type is vehicles that start from the on-ramps located downstream of the access zones and cannot enter the MLs. The fourth group is vehicles that start from the beginning of the network and do not have the choice to use the MLs because they exit the network upstream of the access zone.

2.4.6.1 Type 1 (Beginning)

Type 1 refers to vehicles that come from the beginning of the network, which is located upstream of the start of the MLs. This type of vehicle might have a choice between the GPLs and the MLs. There are five groups in this type. The first group is the vehicles that start from the beginning and use GPLs to exit off-ramps without reaching the end of the network. The second group is the vehicles that have a choice between the MLs or GPLs and reach the end of the network. The third group is the vehicles that use the first MLs egress to exit the network using the off-ramps, which are located downstream of the egress. The fourth group is the vehicles that use the second MLs egress and head to the off-ramps downstream of the second egress. The fifth group is the vehicles that use the third MLs egress to the off-ramps. The percentages of vehicles in all groups are shown in Table 2.3. These percentages were calculated and organized based on the field traffic volume (RITIS data), U.S. Census data, and FDOT data.

Table 2.3 - Type 1 vehicle composition

	No Access Zones			One Access Zone			Two Access Zones			Three Access Zones		
	PCs	Carpools	HGVs	PCs	Carpools	HGVs	PCs	Carpools	HGVs	PCs	Carpools	HGVs
Group 1	55%	6%	3%	47%	5%	2%	45%	5%	2%	43%	5%	2%
Group 2	30%	4%	2%	30%	4%	2%	30%	4%	2%	30%	4%	2%
Group 3	-	-	-	8%	1%	1%	6%	0.6%	0.6%	4%	0.4%	0.4%
Group 4	-	-	-	-	-	-	4%	0.4%	0.4%	4%	0.3%	0.3%
Group 5	-	-	-	-	-	-	-	-	-	4%	0.3%	0.3%
Total	85%	10%	5%	85%	10%	5%	85%	10%	5%	85%	10%	5%

2.4.6.2 Type 2 (On-ramps)

Type 2 includes vehicles that come from on-ramps and have the choice between GPLs and MLs. Vehicles enter MLs through the access zones. The percentages of vehicles are based on the traffic volume of vehicles that start from the on-ramps and exit the off-ramps. Vehicles are divided into three groups. The first group consists of the vehicles that start from the on-ramp using the GPLs and exit the network using the off-ramps, and these vehicles do not reach the end of the network. The second group is the vehicles that start from the on-ramps, use the MLs, and exit the network using the off-ramps. The third group includes the vehicles that reach the end of the network and have the choice to use the GPLs or the MLs utilizing the access zones. Table 2.4 shows the percentages of PCs, carpools, and HGVs for each group.

Table 2.4 - Type 2 vehicle composition

On-Ramp ID	First Group			Second Group			Third Group		
	PCs	Carpools	HGVs	PCs	Carpools	HGVs	PCs	Carpools	HGVs
1*	31%	3.6%	1.8%	51%	6%	3%	3%	0.4%	0.2%
2	28%	3.6%	1.8%	54%	6%	3%	3%	0.4%	0.2%
3	23%	2.7%	1.8%	60%	7%	3%	3%	0.3%	0.2%
4	20%	2.7%	1.8%	63%	7%	3%	2%	0.3%	0.2%
5	13%	2.7%	1.8%	71%	7%	3%	1%	0.3%	0.2%
6	10%	1.8%	0.9%	74%	8%	4%	1%	0.2%	0.1%
7#	7%	1.8%	0.9%	77%	8%	4%	1%	0.2%	0.1%

* is the first on-ramp that is downstream from the beginning of the network

is the seventh on-ramp that is downstream from the beginning of the network

2.4.6.3 Type 3 (Other)

In the third case, the vehicles use the GPLs from the on-ramps downstream from the access zones and are unable to access the MLs. In this case, the percentages are 85%, 10%, and 5% for PCs, carpools, and HGVs, respectively. An example of the vehicle composition for the base case is shown in Figure 2.14.

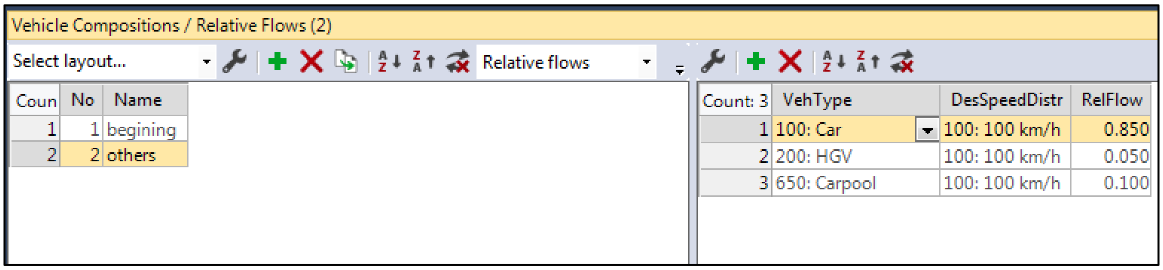


Figure 2.14 - Vehicle composition for the base case in VISSIM

2.4.7 Desired Speed Distribution

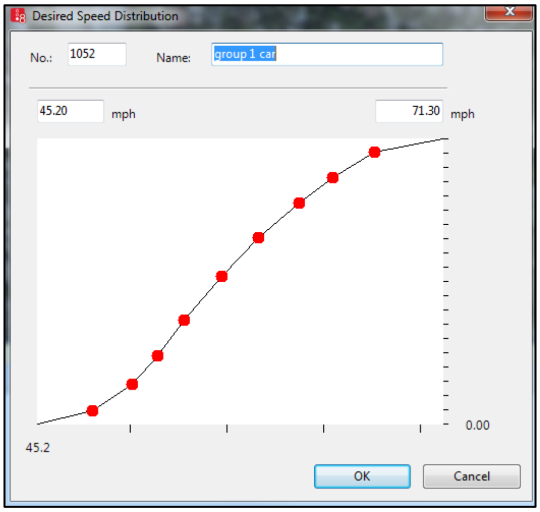
The desired speed distribution (DSD) is the distribution of speed when the vehicles' speed is not affected by other vehicles or network obstacles *(27)*. The DSD has to be input in VISSIM for different types of vehicles (i.e., PCs, carpools, and HGVs). The off-peak speed values were employed for generating the DSD in VISSIM. It is worth mentioning that the off-peak period was chosen because of the low possibility for a vehicle to be constrained by other vehicles. Thus, in the off-peak period, vehicles were more likely to travel at their desired speed.

In the case of PCs or carpools, their speed distributions were the same and were divided into four groups. The groups were determined depending on the speed percentile for the RITIS speed data. First, the speed data was sorted according to the 50th percentile. Subsequently, four groups were defined, and the DSDs in each group had similar 50th percentile speed. Among the four groups, two groups were dedicated to the GPLs and the other two were dedicated to the MLs.

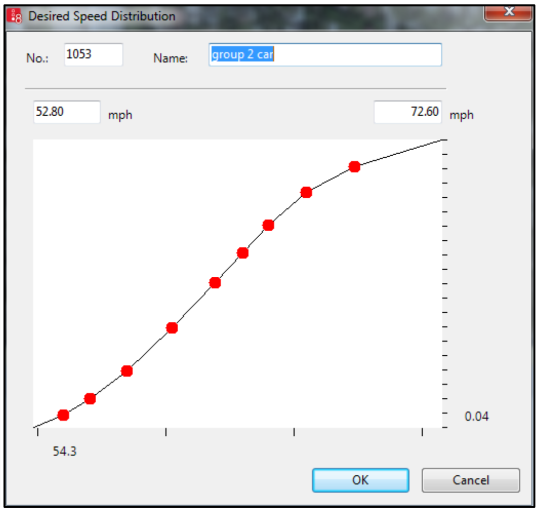
The DSDs of the HGVs were conducted from the speed distributions of PCs and carpools. Johnson and Murray *(28)* concluded that the average speed difference between cars and trucks was 8.1 mile per hour. The HGV percentage is 5%. Suppose x is the speed of PCs or carpools, then the speed for HGV is equal to (x-8.1), the average speed is y, which is provided by RITIS, and

$$\mathrm{Y} = 0.95 \times \mathrm{PC} + 0.05 \times (\mathrm{PC} - 8.1) \tag{1}$$

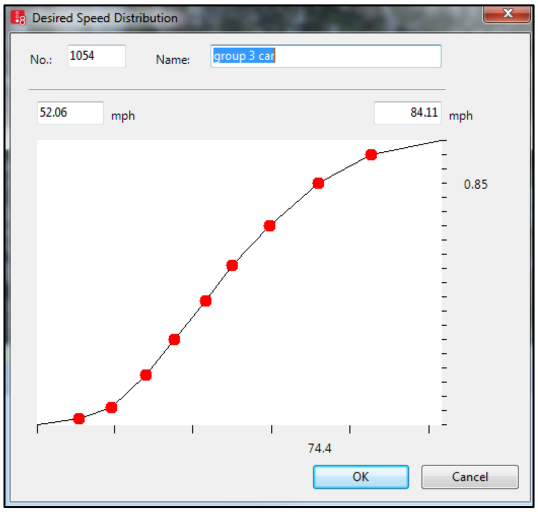
From the equation, the speed of the PC or carpools was about (y+0.5), and the truck speed was about (y-7.6). By shifting the total desired speed distribution by 0.5 mph to the right, PC speed distributions can be gained. Also, by shifting the total DSD for all vehicles by 7.6 mph to the left, HGV speed distributions can be gained. The desired speed distribution for each group for the PCs and the HGVs are represented in Figures 2.15 and 2.16, respectively.



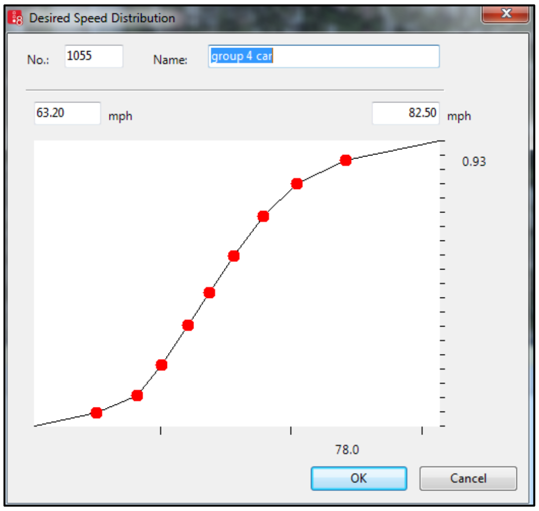
(a) DSD of PC for group 1



(b) DSD of PC for group 2

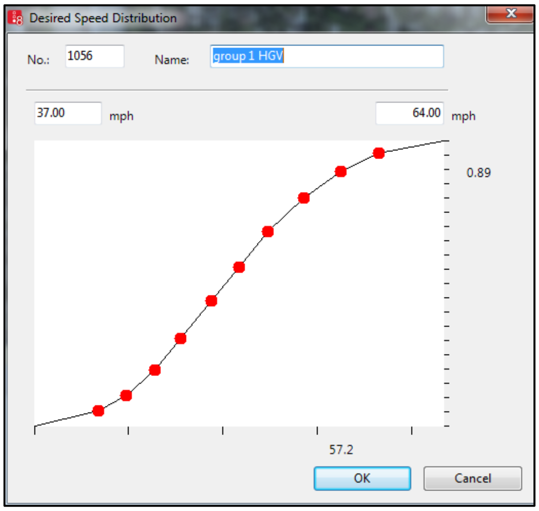


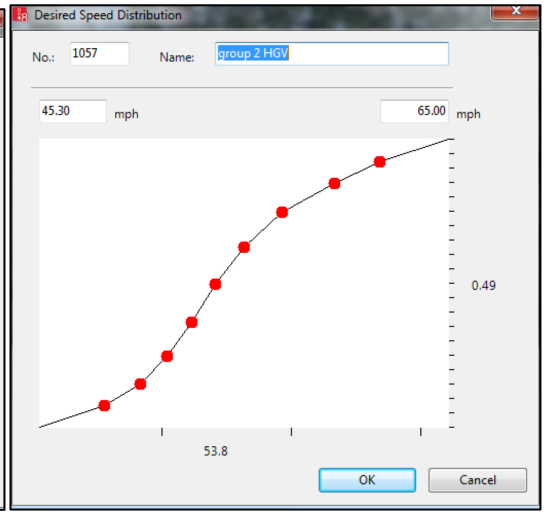
(c) DSD of PC for group 3



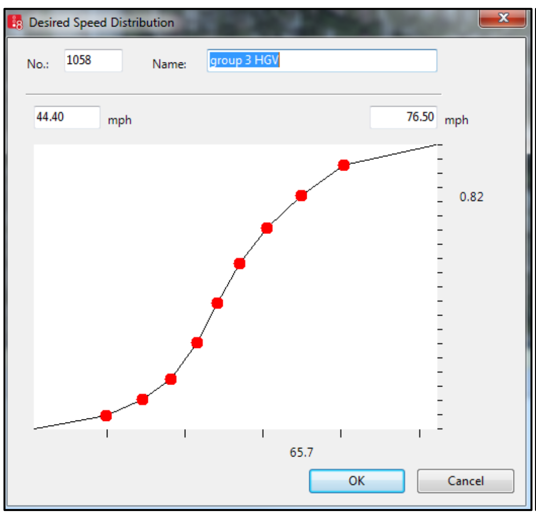
(d) DSD of PC for group 4

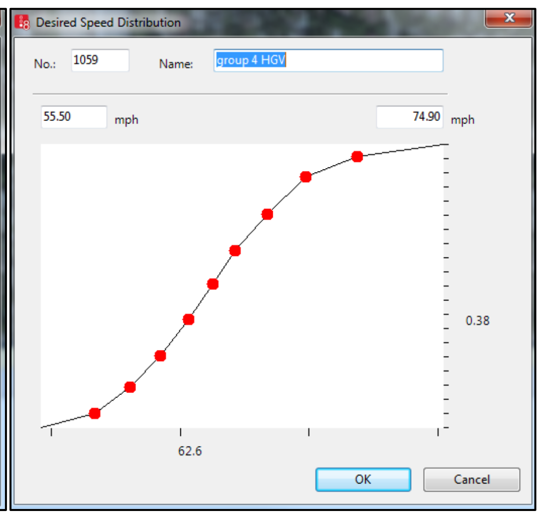
Figure 2.15 - Desired speed distributions for PCs





(a) DSD of HGV for group 1 (b) DSD of HGV for group 2





(c) DSD of HGV for group 3 (d) DSD of HGV for group 4

Figure 2.16 - Desired speed distribution for HGVs

2.4.8 Dynamic Toll Pricing

2.4.8.1 Decision Model

The VISSIM software applies a logit model to calculate the probability of a driver deciding to use the MLs. The utility function and the logit model equation is shown as follows:

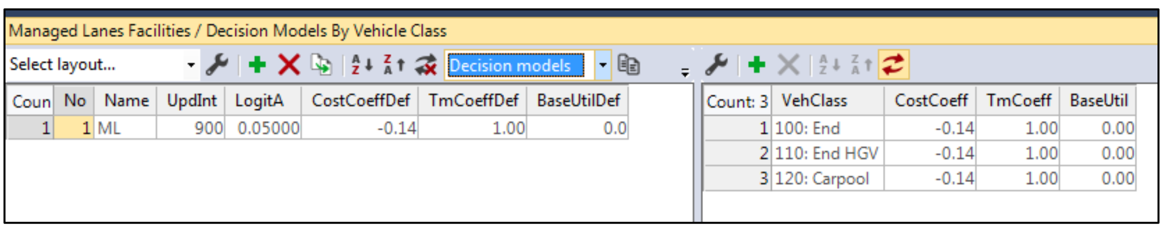
$$\text{Utoll} = \text{Cost coefficient } \times \text{Toll rate } + \text{ Time coefficient } \times \text{ Time gain } + \text{ Base Utility} \quad (2)$$

$$P_{toll} = 1 - \frac{1}{1 + e^{a \times U_{toll}}} \quad (3)$$

The base utility depends on the vehicle class, and zero is the default of the software. The time and cost coefficients were calculated from the value of time (VOT). The ratio of the cost coefficient and the time coefficient was utilized to define the VOT as shown in the following equation.

$$VOT = \frac{\beta_{time}}{\beta_{\cos t}} (\$ / hr) \quad (4)$$

In this study, the VOT was assumed to be \$8.67 per hour based on the result of a multinomial logit model conducted by Jin et al. *(29)*. The time coefficient was assumed to be one minute, and the cost coefficient was -0.14 (\$8.67/60) for all types of vehicles that use the MLs. The negative sign of the cost coefficient means MLs utility increases with the decrease of the tolls. Figure 2.17 shows the decision model parameters in VISSIM.


Managed Lanes Facilities / Decision Models By Vehicle Class

Select layout... Decision models

Coun	No	Name	UpdInt	LogitA	CostCoeffDef	TmCoeffDef	BaseUtilDef
1	1	ML	900	0.05000	-0.14	1.00	0.0

Count: 3	VehClass	CostCoeff	TmCoeff	BaseUtil
1	100: End	-0.14	1.00	0.00
2	110: End HGV	-0.14	1.00	0.00
3	120: Carpool	-0.14	1.00	0.00

Figure 2.17 - Decision model in VISSIM

2.4.8.2 Linear Regression Model

According to the historical field toll prices, the research team found that the toll price was not fixed, but changeable according to the traffic condition. Hence, a linear regression model was utilized to determine the dynamic toll pricing in the MLs depending on the time saved by using the MLs and the average speed in the MLs. The data for model estimation were collected from the field dynamic toll pricings for the same days that were used in collecting the traffic inputs. The data were collected from the FDOT for District 6. The linear regression results and the model performance are shown in Table 2.5. The adjusted R-squared value for the model was 0.86, which indicates that the estimated model can be employed for accurately determining the dynamic toll pricing depending on the speed at the MLs and the time saved by using the MLs.

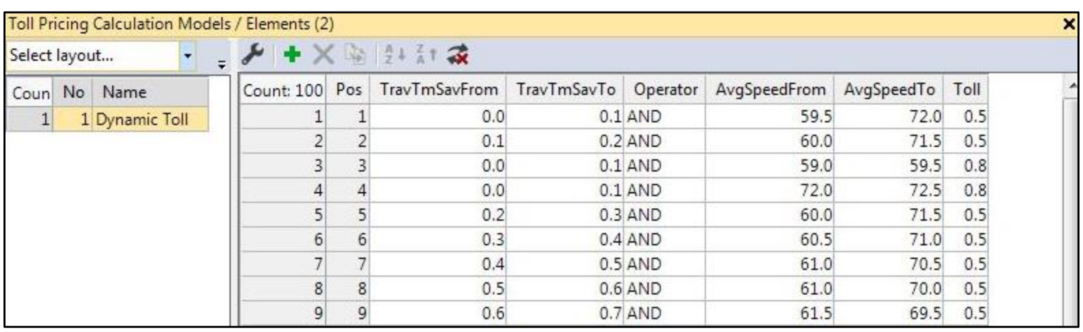
Table 2.5 - Linear regression model results

Variable	Parameter Estimate	Standard Error	t Value	Pr > |t|
Intercept	-0.375	0.148	-2.540	0.012
(Speed at MLs - 65.46*)^2	0.026	0.005	4.990	<.0001
Time saved	1.008	0.031	31.78	<.0001
Interaction term	-0.003	0.001	-2.960	0.004
		Model performance		
Root MSE			1.361	
Dependent Mean			3.193	
Coefficient of Variance			42.613	
R-Square			0.864	

* 65.46 is the average speed of the MLs for the four studied days.

2.4.8.3 Dynamic Toll Pricing

Depending on the linear regression model results, different cases were put into the toll pricing of the MLs in VISSIM. Figure 2.18 and Table 2.6 shows part of the dynamic toll pricing in VISSIM. The cases are mainly affected by two components: first, the time saved by using the MLs, which varied from 0 to 8.5 minutes; and second, the speed in the MLs, which was between 30 mph and 73.5 mph. The dynamic toll prices varied between a minimum value of $0.50 and a maximum value of $10.50.


Toll Pricing Calculation Models / Elements (2)

Select layout...

Coun	No	Name
1	1	Dynamic Toll

Count: 100	Pos	TravTmSavFrom	TravTmSavTo	Operator	AvgSpeedFrom	AvgSpeedTo	Toll
1	1	0.0	0.1	AND	59.5	72.0	0.5
2	2	0.1	0.2	AND	60.0	71.5	0.5
3	3	0.0	0.1	AND	59.0	59.5	0.8
4	4	0.0	0.1	AND	72.0	72.5	0.8
5	5	0.2	0.3	AND	60.0	71.5	0.5
6	6	0.3	0.4	AND	60.5	71.0	0.5
7	7	0.4	0.5	AND	61.0	70.5	0.5
8	8	0.5	0.6	AND	61.0	70.0	0.5
9	9	0.6	0.7	AND	61.5	69.5	0.5

Figure 2.18 - Inputting dynamic toll pricing into VISSIM

Table 2.6 - Part of the dynamic toll pricing

Travel time saved	Managed lane speed	Toll price	Travel time saved	Managed lane speed	Toll price

From	To	From	To	($)	From	To	From	To	($)
0	0.1	59.50	71.99	0.5	0.1	0.2	58.50	58.99	1.00
0.1	0.2	60	71.49	0.5	0.2	0.3	58.50	59.49	1.00
0.2	0.3	60	71.49	0.5	0.3	0.4	59	59.49	1.00
0.3	0.4	60.50	70.99	0.5	0.4	0.5	59	59.99	1.00
0.4	0.5	61	70.49	0.5	0.5	0.6	59.50	59.9	1.00
0.5	0.6	61	69.99	0.5	0.6	0.7	59.50	60.49	1.00
0.6	0.7	61.50	69.49	0.5	0.7	0.8	60	60.99	1.00
0.7	0.8	62	68.99	0.5	0.8	0.9	60.50	61.49	1.00
0.8	0.9	63	68.49	0.5	0.9	1.0	60.50	61.99	1.00
0.9	1.0	63.50	67.49	0.5	1.0	1.5	61	62.49	1.00
0	0.1	59	59.49	0.75	0	0.1	72.50	73.49	1.00
0.1	0.2	59	59.99	0.75	0	0.1	57.50	57.49	1.25
0.2	0.3	59.50	59.99	0.75	0	1.5	57.50	57.99	1.25
0.3	0.4	59.50	60.49	0.75	0.1	0.2	58	58.49	1.25
0.4	0.5	60	60.99	0.75	0.2	0.3	58	58.49	1.25
0.5	0.6	60	60.99	0.75	0.3	0.4	58	58.99	1.25
0.6	0.7	60.50	61.49	0.75	0.4	0.5	58.50	58.99	1.25
0.7	0.8	61	61.99	0.75	0.5	0.6	58.50	59.49	1.25
0.8	0.9	61.50	62.99	0.75	0.6	0.7	59	59.49	1.25
0.9	1.0	62	63.49	0.75	0.7	0.8	59	59.99	1.25
1.0	1.5	62.50	68.99	0.75	0.8	0.9	59.50	60.49	1.25
0.9	1.0	67.50	68.49	0.75	0.9	1.0	59.50	60.49	1.25
0.1	0.2	71.50	72.49	0.75	1.0	1.5	60	60.99	1.25
0	0.1	72	72.49	0.75	1.5	2	62.50	68.99	1.25
0	0.1	58	58.99	1.00	0.5	0.6	56	56.49	1.50

For the access zone cases, the price was decided by the traveled distance on the MLs. For example, for one access zone, the dynamic toll price for the vehicles that come from the on-ramps to the ingress or from the beginning to the egress is half of the dynamic toll price for the vehicles that come from the beginning and reach the end of the network.

2.4.9 Other Parameters

The following section describes the VISSIM parameters and their acceptable ranges employed in the calibration and validation process. These parameters included the emergency stopping distance and the lane-change distance. First, the emergency stopping distance is used to define the last possible location for a vehicle to make a lane change. The emergency stopping distance was assigned to all links of the network. The acceptable range of the emergency stopping distance was defined from 6.5 feet to 23 feet *(30)*. In this study, 20 feet was used for the diverging segments (off-ramps) of the network to prevent network congestion in the gore areas. For all other segments, the default value of the emergency stopping distance was used as the default value, which was equal to 16.5 feet *(27, 30)*. Second, the lane-change distance was assigned in the network based on the distance from the overhead signs to the off-ramps. In the case of merging segments (i.e., on-ramps), the lane-change distance was used as the default value, which was 656 feet. In the case of diverging segments (i.e., off-ramps), the lane-change distance was the distance at which vehicles start to change lanes upstream from the off-ramps. The exact value was decided by the location where the overhead signs are located. Figure 2.19 gives two examples: the left figure shows that the lane-change distance is 0.5 miles, and the right figure shows that the distance is 1 mile.

Source: Google Earth

Figure 2.19 - Lane-change distance on the overhead sign

2.4.10 *Calibration and Validation*

After construction of the VISSIM network, it is important to calibrate and validate it. The comparison between the VISSIM simulated traffic and the field traffic was conducted. If the difference between the two sets of data is significant, the simulation network cannot be utilized to represent the field network. Therefore, only after the successful calibration and validation of the simulation network can it be employed for further applications. Traffic volume data were used for the calibration process of the VISSIM network, and speed data were utilized for the validation process. A total of 180 minutes (from 7:30 AM to 10:30 AM) of VISSIM data were used in the calibration and validation process after excluding 30 minutes of warm-up time and 30 minutes of cool-down time.

In order to calibrate the simulation network and to compare field volume and simulated volume, a method developed by Wisconsin DOT was utilized *(31)*. In this method, the calibration procedure was done by calculating the Geoffrey E. Havers (GEH) value for the traffic volume of the simulated network and the field network. The formula for the GEH value is as follows:

$$GEH = \sqrt{\frac{(E-V)^2}{(E+V)/2}} \tag{5}$$

where E is the traffic volume for the simulated network (vehicle per hour) and V is the traffic volume at the field network (vehicle per hour). If the value of GEH is less than five, it indicates that the difference between VISSIM volume and the field volume in a specific location and that time interval (15 minutes) is acceptable. The VISSIM network is well calibrated when the percentage of the GEHs that are lower than 5 is higher than 85% for all measurement locations and for all time intervals *(32)*.

In the case of network validation, the absolute difference between the speed of the simulated traffic data and the speed of the field traffic data was calculated. The VISSIM network is well validated when the absolute speed difference is lower than 5 mph for 85% of the measurement locations and for all time intervals *(33)*.

In order to confirm the calibration and validation results, ten simulation runs with various random seeds were utilized. Calibration and validation results for each simulation run are shown in Tables 2.7 and 2.8, respectively. For the calibration process, the average GEH was 2.39 and the average percentage of GEHs that were less than 5 was 91.08%. For the validation process, the average absolute speed difference was 1.9 mph, and the average percentage of absolute speed differences that are less than 5 was 95.56%. Consequently, the VISSIM network was satisfactorily calibrated and validated.

Table 2.7 - Calibration results

	Calibration (traffic volume)			
Run number	Good (GEH<5)	All	Percentage of acceptance	Average GEH
1	123	132	93.1%	2.3
2	124	132	93.9%	2.29
3	118	132	89.4%	2.32
4	114	132	86.4%	2.71
5	117	132	88.6%	2.62
6	123	132	93.2%	2.3

7	114	132	86.4%	2.6
8	124	132	93.3%	2.24
9	124	132	93.4%	2.24
10	123	132	93.1%	2.27
Average	120.4	132	91.1%	2.39

Table 2.8 - Validation results

Validation (average speed)				
Run number	Good (absolute speed difference<5)	All	Percentage of acceptance	Average absolute speed difference
1	126	132	95.4%	1.92
2	126	132	95.45%	1.91
3	127	132	96.2%	1.92
4	126	132	95.45%	1.91
5	127	132	96.2%	1.88
6	127	132	96.2%	1.87
7	126	132	95.45%	1.90
8	125	132	94.7%	1.90
9	125	132	94.4%	1.90
10	127	132	96.2%	1.88
Average	126.2	132	95.56%	1.90

2.5 Results for Safety and Operational Measurements

2.5.1 Results for Safety Measurements

2.5.1.1 Surrogate Safety Assessment Model (SSAM)

The main objective of SSAM can be either to develop the safety performance of the current roadway designs or for a new strategy before implementation *(19)*. In this study, SSAM was adopted to determine the conflict frequency, which is highly correlated with the crash frequency in the field *(20)*. The FHWA defines a conflict as "an observable situation in which two or more road users approach each other in time and space to such an extent that there is risk of collision if their movements remain unchanged" *(19)*. Three types of conflicts can be extracted from SSAM: rear-end, lane-change, and crossing conflicts. The classification of conflicts was dependent on the conflict angle diagram as shown in Figure 2.20. Two types of conflicts were used in this report: rear-end and lane-change conflicts. The crossing conflicts were excluded from this study since the percentage of crossing conflicts was less than 1%, and crossing crashes are not likely to happen on freeways.

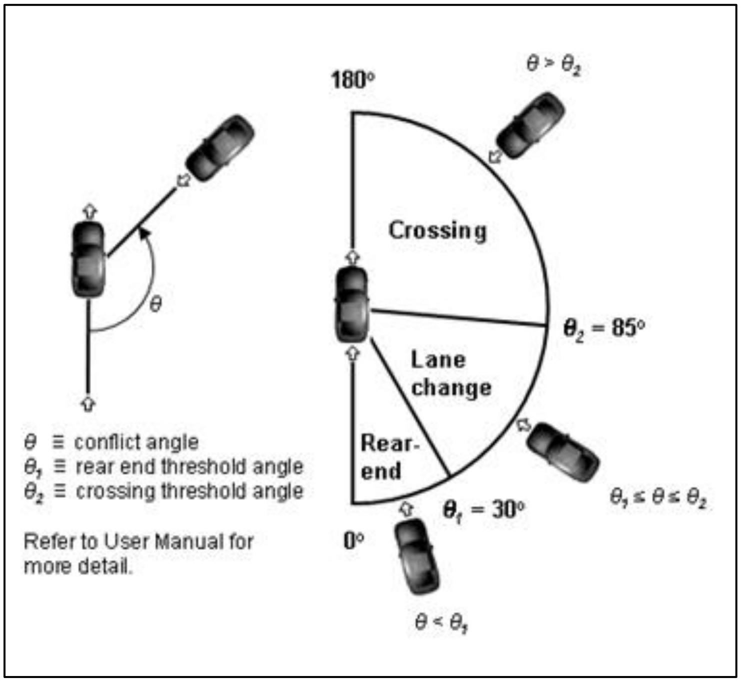


Figure 2.20 - Conflict angle diagram in SSAM

Ten different simulation runs were carried out for each scenario to eliminate the random effects. The vehicle trajectory files from VISSIM were imported in SSAM to obtain detailed information about the conflicts. In each simulation run, there were "virtual" crashes whose TTC was zero. These observations might lead to the inaccuracy of the simulation models *(19)*. Consequently, the cases of TTC=0 (crash) were excluded before implementing statistical analysis calculations.

Eight surrogate measurements were extracted from SSAM to evaluate the safety of the network: TTC, post-encroachment time (PET), maximum speed (MaxS), difference in vehicle speeds (DeltaS) deceleration rate (DR), maximum deceleration (MaxD), maximum difference in vehicle speeds (MaxDeltaS), and conflict angle.

According to FHWA *(19)*, TTC is the minimum time to collision, which is calculated based on the speed and location of vehicles. The threshold of TTC was set to be 1.5 s. When the TTC is less than 1.5 s, a conflict happens. PET is the minimum post-encroachment time, which is defined as the time between two vehicles occupying the same point. The maximum value of PET was determined to be 5.0 s to identify a conflict. MaxS is the maximum speed for either of the two vehicles that participated in the conflict. DeltaS and MaxDeltaS are the difference in speed and the maximum difference in speed, respectively, between the vehicles in the conflict. DR and MaxD are the initial and the maximum deceleration rate for a vehicle to avoid the conflict with the other vehicle.

The descriptive statistics of the surrogate measures are shown in Table 2.9 for both peak and off-peak conditions. An ANOVA test was carried out to compare the surrogate measures in MLs and GPLs. The results showed that TTC (estimate=0.095, P-value=0.0003) and PET (estimate=1.026, P-value<0.0001) were higher in the MLs, which indicated that MLs were safer than GPLs. Meanwhile, the maximum speed of the two vehicles in the conflict was higher in the MLs than the GPLs (estimate=14.669, p-value<0.0001). Compared to GPLs, MLs had lower conflict risk with higher MaxD (estimate=0.892, p-value<0.0001). Another significant result was that MLs had lower conflict angle than GPLs (estimate=-4.288, p-value<0.0001). The result was expected since more lane-change maneuver could be observed in GPLs. Additionally, the results showed no significant difference in DeltaS (estimate=-0.063, p-value=0.8476) between MLs and GPLs.

Table 2.9 - Descriptive statistics of the surrogate safety measures

		MLs				GPLs			
		Mean	SD	Min	Max	Mean	SD	Min	Max
Peak	TTC (sec)	1.07	0.44	0.08	1.50	1.01	0.36	0.10	1.50
	PET (sec)	2.40	1.46	0.10	5.00	1.34	1.18	0.05	4.90
	MaxS (ft/sec)	30.34	3.20	13.86	36.63	15.62	8.86	1.38	35.43
	DeltaS (ft/sec)	8.26	5.14	0.08	24.46	8.30	5.17	0.01	26.80
	MaxD (ft/sec^2)	-6.22	1.02	-7.45	-0.01	-5.29	2.09	-8.00	-0.03
	Conflict angle	3.76	6.29	0.14	43.1	8.62	10.81	0	72.18
Off-peak	TTC (sec)	1.13	0.39	0.10	1.50	1.02	0.39	0.20	1.50
	PET (sec)	2.68	1.42	0.09	5.00	1.42	1.14	0.10	4.90
	MaxS (ft/sec)	31.44	2.84	17.03	36.71	17.49	9.39	1.62	35.30
	DeltaS (ft/sec)	8.21	2.92	0.06	17.91	8.28	2.56	0.79	13.92
	MaxD (ft/sec^2)	-5.92	1.44	-7.25	-0.01	-5.21	2.02	-8.07	-0.05
	Conflict angle	3.49	6.07	0.33	38.85	7.36	9.78	0	71.36

In order to compare the surrogate safety measures between the GPLs and the MLs in the whole segment, a binary logistic regression model was developed. The event has a binary outcome for each i^{th} observation, which is MLs (y_i=1) and GPLs (y_i=0). The probability of the y_i =1 and y_i =0 are pi (y_i =1) and 1-pi (y_i =0), respectively. The model can be formulated as follows:

$$y_i \sim \text{Bernoulli}\ (p_i) \qquad (6)$$

$$ln\left(\frac{P_i}{1-P_i}\right) = \beta_0 + \sum_{i=1}^{n} \beta_k * X_{ki} \qquad (7)$$

where y_i follows a Bernoulli distribution whose probability of success is p_i, β_0 is the intercept, β_k is the regression coefficient, and X_{ki} denotes the explanatory variables for the *k* variable (e.g., TTC, PET, etc.) and the i^{th} observation.

The multicollinearity was tested between variables using the Spearman's rank test. As the correlation value increase, it indicates a higher correlation between variables. When the correlation value is equal to zero, that indicates there is no correlation between variables, and the highest correlation occurs at the value of 1 *(19)*. The results revealed that there is a correlation between most of the surrogate measures; only TTC, MaxS, DR, and lane-change conflict were found to have a low correlation (r<0.3). Hence, these four surrogate measures were used in the logistic regression model. Figures 2.21 and 2.22 show the frequencies of TTC and MaxS for MLs and GPLs.

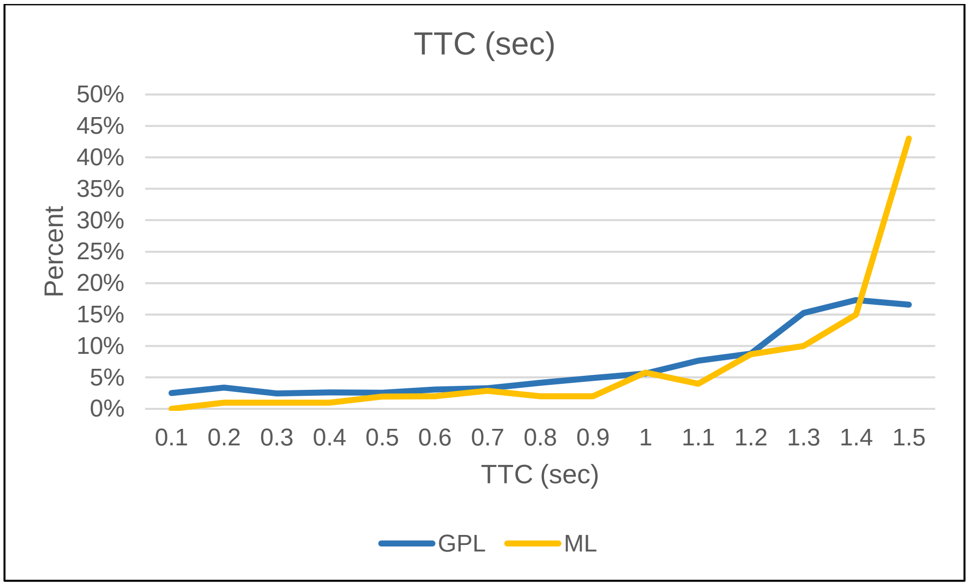


Figure 2.21 - TTC chart for GPLs and MLs

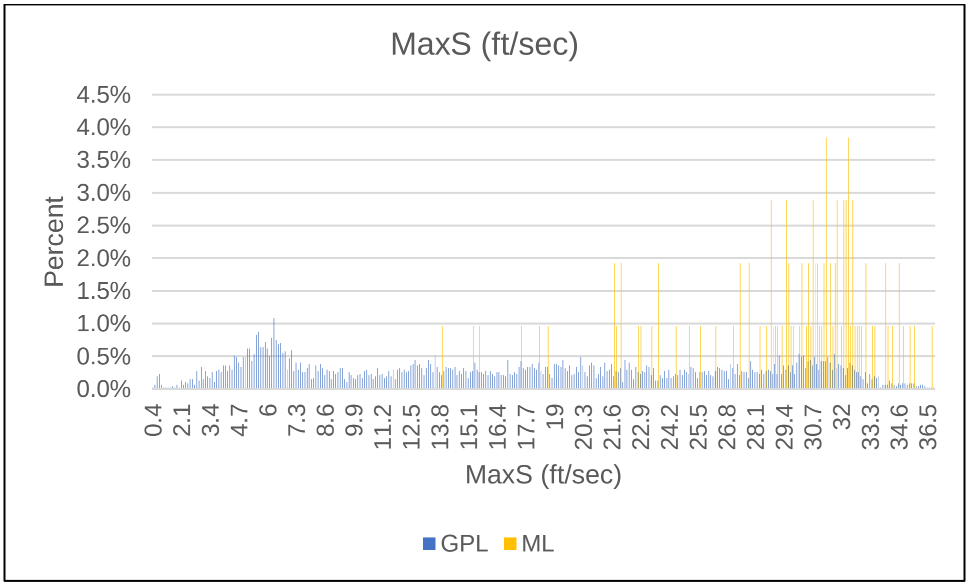


Figure 2.22 - MaxS chart for GPLs and MLs

The results of the model are shown in Table 2.10. Closer inspection of the table reveals that the TTC is higher in the MLs, which indicates that MLs are safer than GPLs. Meanwhile, the maximum speed of the two vehicles in the conflict was significantly higher in MLs. For the case of conflict type, GPLs had a significantly higher number of lane-change conflicts than MLs. Hence, the safety surrogate measures were improved in MLs. The area under the receiver operating characteristic (ROC) value (0.92) indicated that the model provides excellent discrimination between the two binary outcomes (GPLs and MLs).

Table 2.10 - Logistic regression model for MLs

Parameter	Estimate	Standard Error	Wald Chi-Square	Pr > ChiSq
Intercept	-9.703	0.683	201.849	<.0001
TTC	2.209	0.204	117.356	<.0001
MaxS	0.419	0.021	427.196	<.0001
Lane-change conflict	-4.618	0.313	217.259	<.0001

2.5.1.2 Base Case Results

The base condition is the current situation of the network without access zones. In peak-hour conditions, the conflict frequency in GPLs is higher than in MLs by 78.64% (66.67% higher for lane-change conflicts and 85.07% higher for rear-end conflicts). In off-peak-hour conditions, the conflict frequency in GPLs is higher than in MLs by 54.54% (80.00% higher for lane-change conflicts and 33.34% higher for rear-end conflicts), as shown in Figure 2.23. When taking the volume of the GPLs and MLs into account, by dividing the number of conflicts over the total number of vehicles, it was found that the conflict in GPLs is higher than in MLs by 48% and 11% in peak and off-peak traffic conditions, as shown in Figure 2.24. This higher conflict frequency in GPLs than in MLs is because of the lane changing of vehicles near the access zone area in GPLs that can generate both lane-change and rear-end crashes. Also, the conflict frequency per vehicle is higher in peak conditions than off-peak conditions by 68% in GPLs and 45% in MLs.

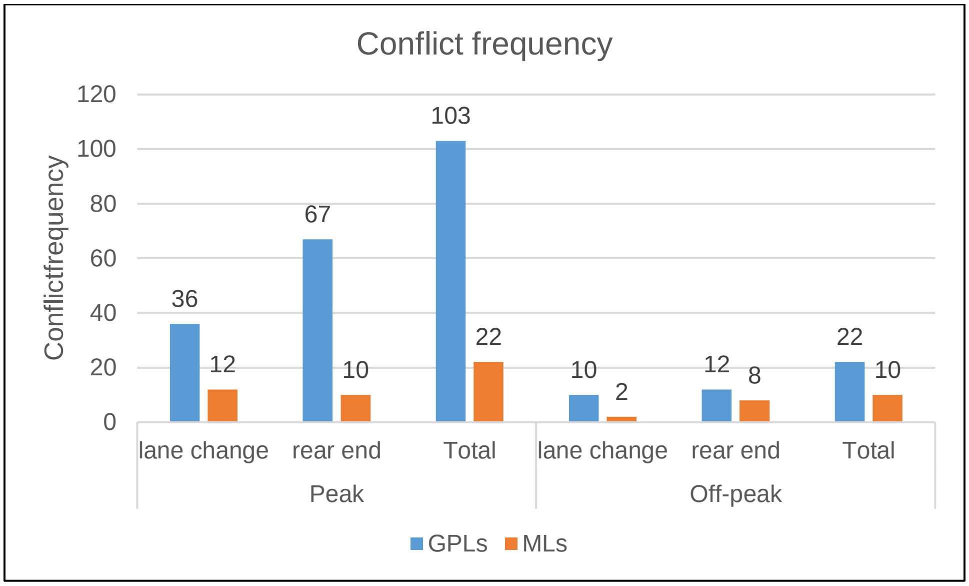


Figure 2.23 - Conflict frequency for each conflict type in different lanes

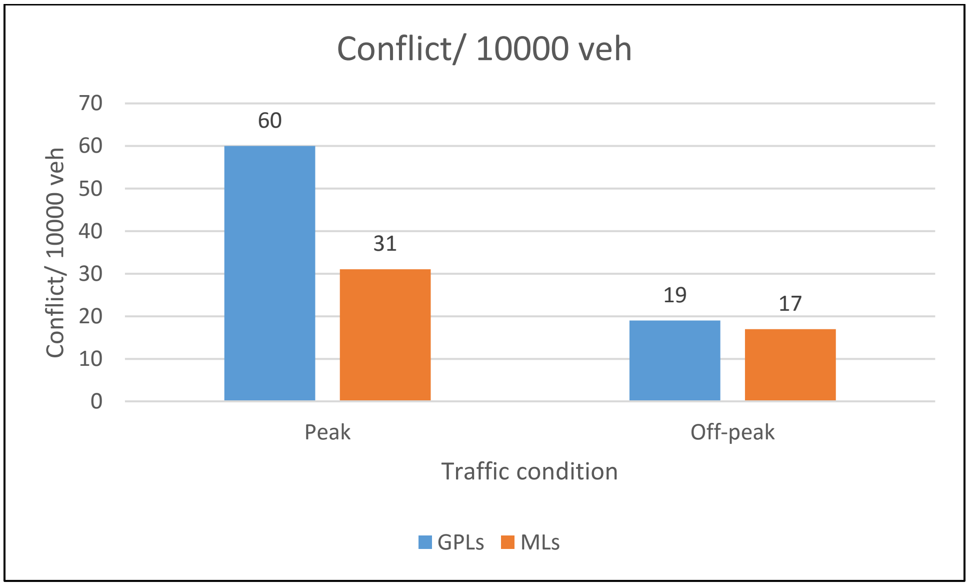


Figure 2.24 - Conflict frequency per vehicle for GPLs and MLs in different conditions

2.5.1.3 Total Conflict Frequency Results

The results from SSAM were used for comparing the safety (conflict frequency and conflict rate) between the various cases. Figures 2.25 and 2.26 show the comparison of the total conflict numbers among the various weaving lengths at different traffic conditions. From the safety point of view, the results depict that the case of one access zone (one ingress and one egress) is the optimal density for the access zone accessibility cases. Meanwhile, lane-change lengths of 800 and 1,000 feet per lane change have the lowest conflict frequency among all possible lengths. It is apparent from the figures that the cases of two and three accessibility levels had higher total conflicts than the cases of one access zone (Cases 1A and 1B). Additionally, Case 1A is safer than Case 1B when the access distance is 1,000 feet or higher. Closer inspection of the two figures summarized that there is a consistent trend between the results in the different traffic conditions.

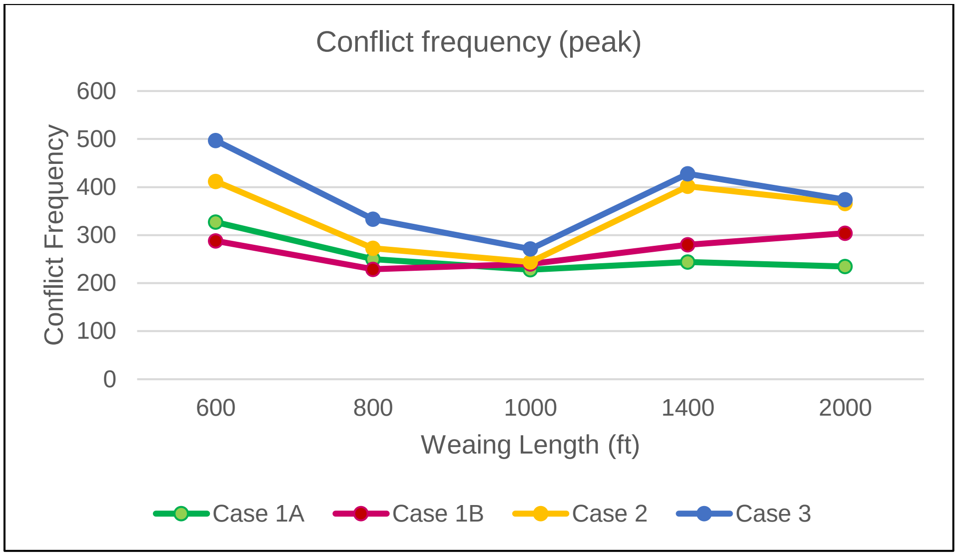


Figure 2.25 - Conflict frequency at peak condition

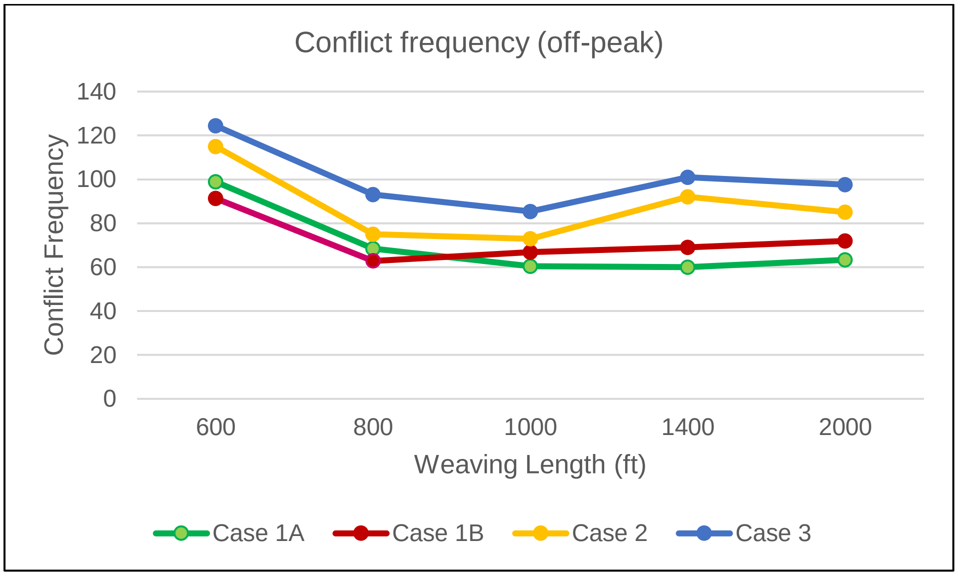


Figure 2.26 - Conflict frequency at off-peak condition

2.5.1.4 Conflict Rate Results

Conflict rate was identified to compare the safety impacts among different scenarios with various accessibility levels and weaving distances. Conflict rate was calculated for weaving segments near access zones and from the total number of conflicts over the weaving segment length.

$$\text{Conflict rate (conflict/100 ft)} = \frac{\text{Conflict Frequency in the Weaving Segment}}{\text{Length of the Weaving Segment}} \tag{8}$$

The conflict rate information can be found in Tables 2.11 and 2.12. In general, the results showed that the conflict rate decreased in the off-peak conditions compared to the peak conditions. That is because of the higher volume and density on GPLs than MLs. It is worth mentioning that, as the weaving length increased, the crash rate declined. That also can be explained by the vehicle having less urgency to make a lane change when the segment length increased. In most of the records, the weaving segment near the ingress has a higher crash rate than the egress areas. The accessibility level also affects the conflict rate. The conflict rate increased with the increase of the accessibility density. For two and three accessibility levels, the last access zone had a lower conflict rate due to the low volume of vehicles that use the MLs in the fourth quarter of the network. Hence, the lane-changing behavior deteriorated.

Table 2.11 - Conflict rate for weaving segments near the egress for different conditions (conflict/100 ft)

		Case 1A	Case 1B	Case2-First Access	Case2-Second Access	Case3-First Access	Case3-Second Access	Case3-Third Access
Peak	600	2.84	2.42	2.98	1.90	2.96	3.13	2.43
	800	2.17	1.54	2.25	1.37	2.48	2.67	2.38
	1,000	1.64	1.66	2.02	1.17	2.02	2.78	2.25
	1,400	1.51	1.80	2.13	2.16	2.09	2.95	2.54
	2,000	1.48	1.54	2.06	1.78	1.49	2.64	2.33
Off-peak	600	0.53	0.42	0.68	0.29	0.44	0.54	0.40
	800	0.37	0.28	0.55	0.17	0.47	0.33	0.46
	1,000	0.31	0.33	0.42	0.12	0.35	0.38	0.30
	1,400	0.25	0.35	0.32	0.54	0.31	0.43	0.56
	2,000	0.28	0.27	0.25	0.41	0.21	0.39	0.44

Table 2.12 - Conflict rate for weaving segments near the ingress for different conditions (conflict/100 ft)

		Case 1A	Case 1B	Case2-First Access	Case2-Second Access	Case3-First Access	Case3-Second Access	Case3-Third Access
Peak	600	2.38	1.74	2.34	1.73	3.87	4.08	1.74
	800	1.63	1.47	1.83	1.39	3.57	3.44	1.13
	1,000	1.35	1.56	1.61	1.25	3.33	3.20	0.92
	1,400	1.47	1.51	2.56	1.22	3.52	3.77	0.81
	2,000	1.00	1.42	1.66	1.14	2.82	3.24	0.63
Off-peak	600	0.39	0.30	0.61	0.27	0.57	0.70	0.54
	800	0.32	0.24	0.40	0.19	0.54	0.56	0.45
	1,000	0.28	0.25	0.29	0.10	0.43	0.47	0.30
	1,400	0.30	0.20	0.39	0.12	0.37	0.46	0.43
	2,000	0.27	0.18	0.32	0.17	0.33	0.49	0.13

Figures 2.27 and 2.28 show the conflict rate for the first accessibility level when the egress upstream from the ingress (Case 1A) at the segments near the access zones in both peak and off-peak conditions. The charts show that the recommended optimal weaving length is 1,000 feet per lane change for both the ingress and egress cases. It can be noted that the peak and off-peak results are consistent with respect to the trend of the conflict rate. Similarly, for Case 1B, the optimal distance was 800 feet, and the conflict rate in the peak condition was higher than the off-peak case. Thus, it can be concluded that in the case with one access zone when the ingress is upstream from the egress (Case 1B), 800 feet per lane change is the optimal weaving distance. in the case with one access zone when the egress is upstream from the ingress (Case 1A), 1,000 feet per lane change is the optimal weaving length from the nearest ramp to the access zone. Additionally, it can be concluded that, in weaving segments, rear-end conflicts occurred more frequently than lane-change conflicts.

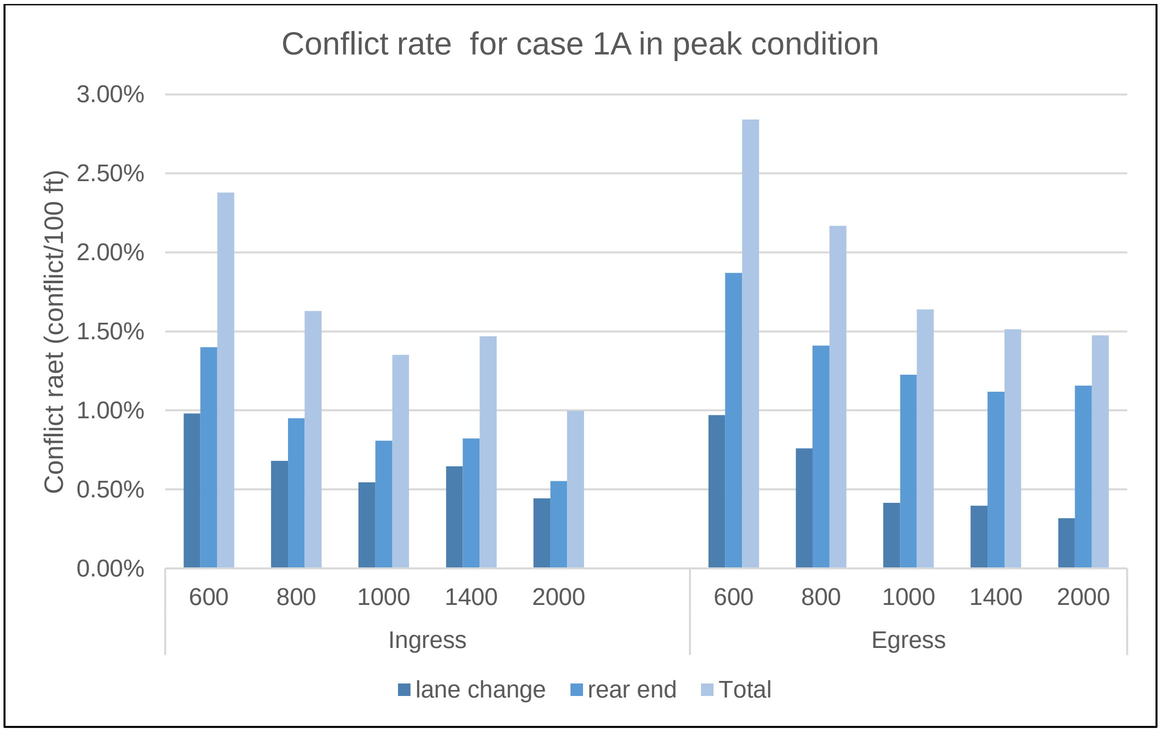


Figure 2.27 - Conflict rate for Case 1A in peak condition

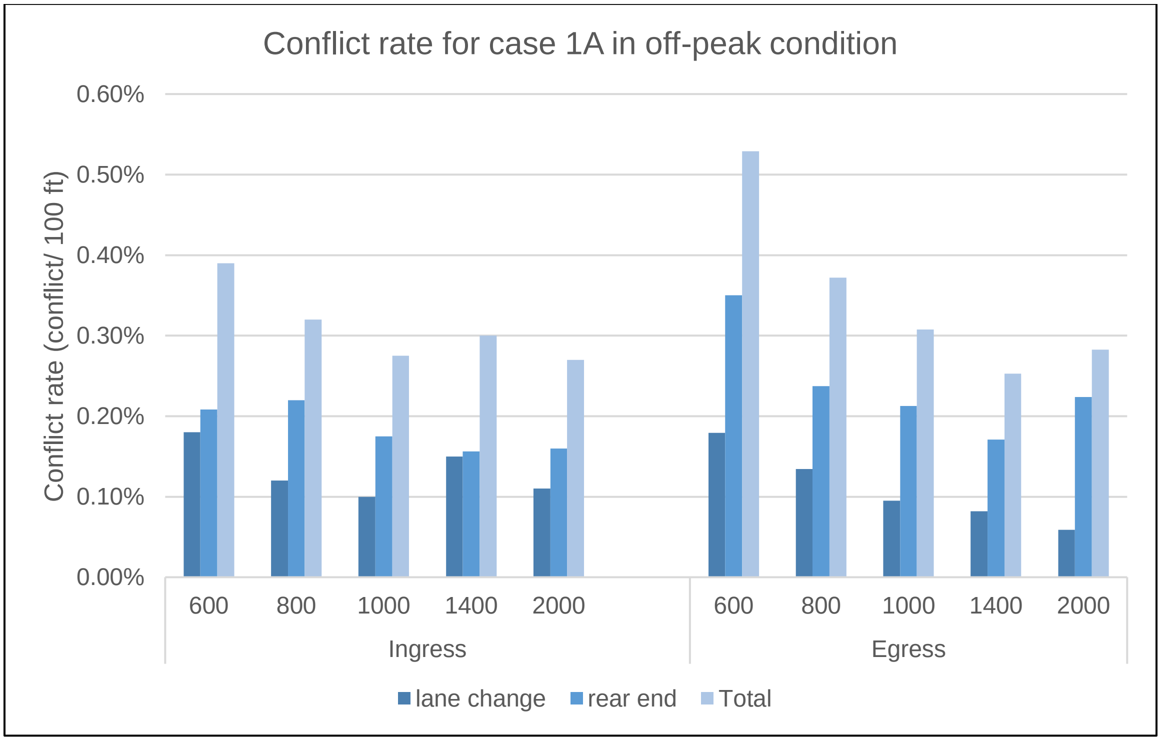


Figure 2.28 - Conflict rate for Case 1A in off-peak condition

For the case with two access zones (two ingresses and two egresses), which are located at the one-third and two-thirds points of the network, Figure 2.29 shows the weaving segments for the two accessibility levels. In the figure, L_1 is the first ingress weaving segment, L_2 is the first egress weaving segment, L_3 is the second ingress weaving section, and L_4 is the second egress weaving segment.

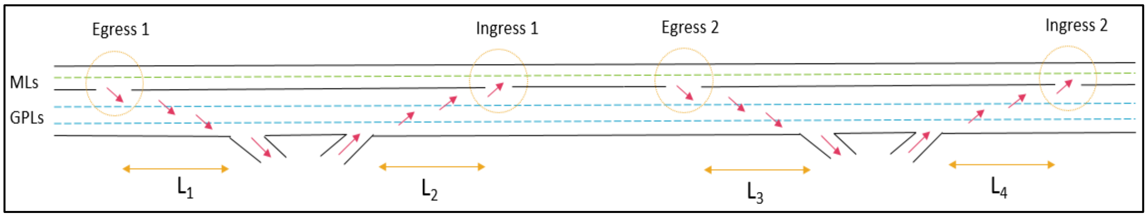


Figure 2.29 - Weaving segments for the two accessibility levels

From the results of the conflict rate, it can be concluded that the suggested optimal weaving length is 1,000 feet per lane change for the first and second access zones. Consequently, for the case with two access zones, the recommended minimum distance is 1,000 feet per lane change for the weaving segments near the access zones. Additionally, in this case, it can be noted that the conflict rate is high at the first ingress and the second egress when the weaving length is 1,400 feet per lane change, as shown in Figure 2.30.

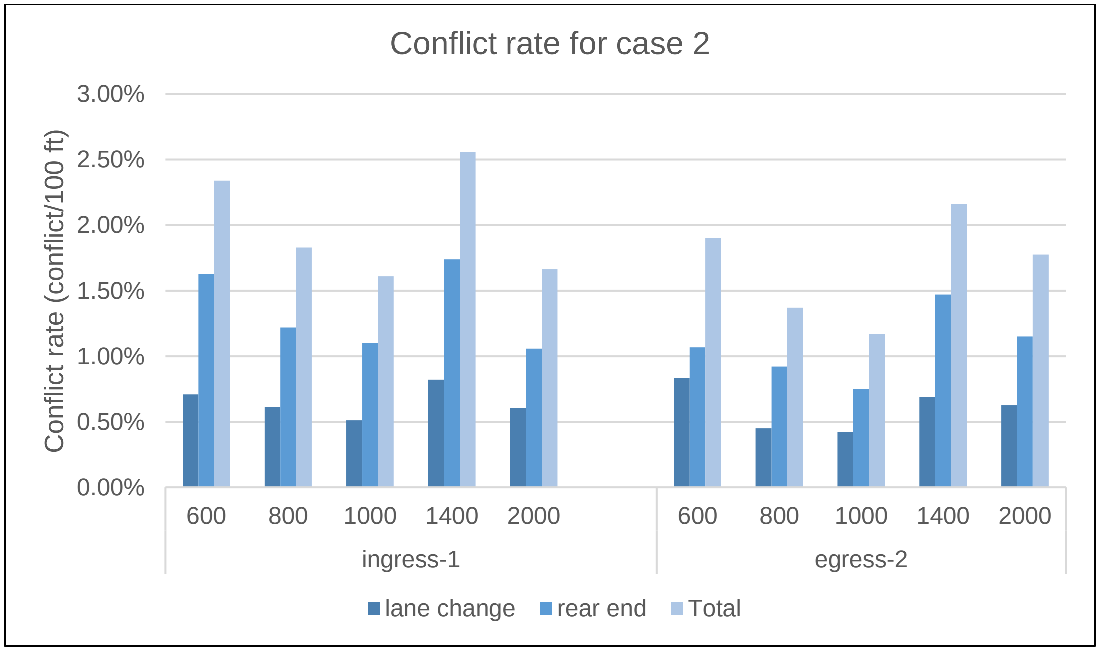


Figure 2.30 - Conflict rate for the first ingress and the second egress for Case 2

This situation happened in the case with 1,400 feet per lane change due to the overlapping between the two weaving segments for the first ingress and the second ingress, which creates a high number of conflicts at this area, as shown in Figure 2.31. The overlapping distance between the two access zones was 0.23 miles (1,200 feet). Hence, a longer lane-change distance does not result in safer conditions because of the overlapping.

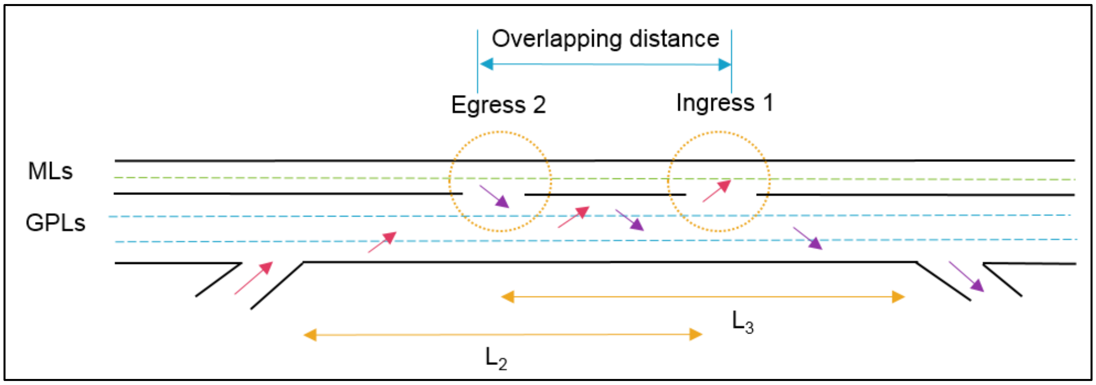


Figure 2.31 - Overlapping between access zones

Similar experiments were conducted for the case with three access zones. The conflict rate of the case with three access zones is the highest among all other accessibility levels for the 9-mile network. Dense access zones would increase the frequency with which vehicles enter and exit MLs. The frequent lane changes, including diverging and merging, would significantly increase crash opportunity. Meanwhile, access zones that are too dense also increase the chance that two adjacent zones will disturb each other. Figure 2.32 shows how conflict rate varies based on the location of the access zone. For example, the last ingress has the lowest conflict rate because of the low volume of traffic entering the MLs at the end of the network. Also, it can be noted that the access zones that overlap had a high conflict rate, especially when the weaving length was more than 1,000 feet. Overall, it is not recommended to have three access zones in a 9-mile network.

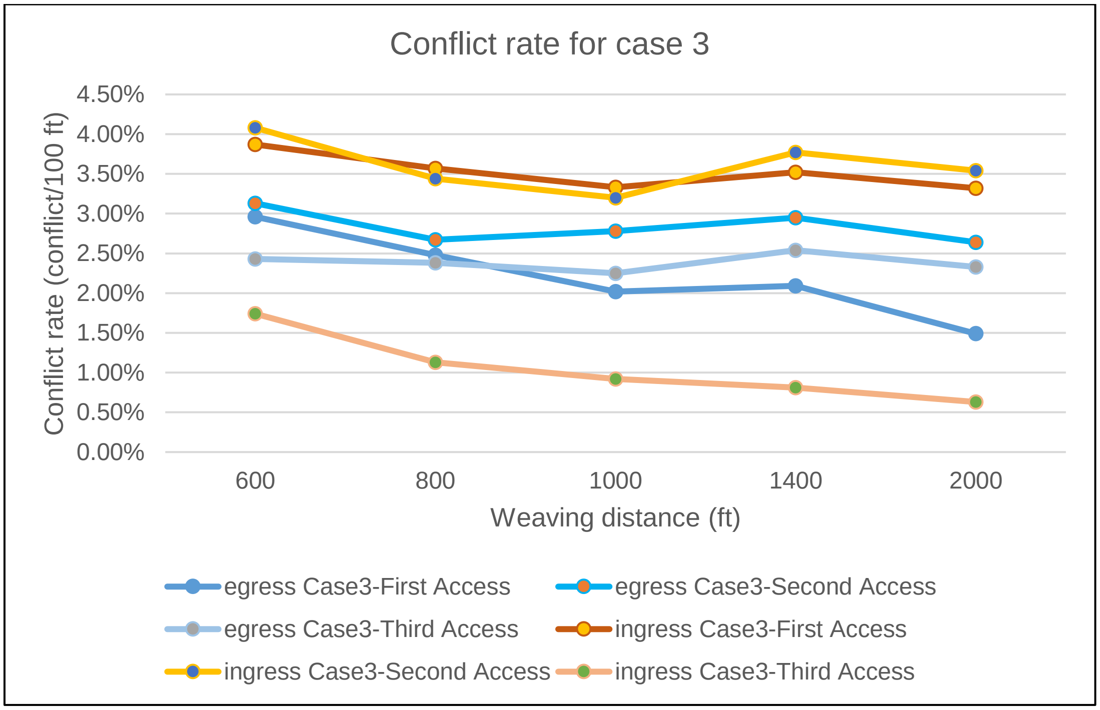


Figure 2.32 - Comparing total conflict rate for Case 3

In conclusion, Case 1A (one accessibility level and egress located upstream) had a lower conflict rate than Case 1B (accessibility level 1 and ingress upstream) and the cases with two and three accessibility levels. The case of three accessibility levels had the highest conflict rate. Moreover, there was a significantly higher conflict rate when the weaving distance was 600 feet per lane change than there was for other weaving distances. On the other hand, the weaving length of 1,000 feet per lane change had the lowest conflict rate when compared to other weaving lengths. Furthermore, in peak traffic conditions, the conflict rate increases by 82% over off-peak conditions, as shown in Figure 2.33.

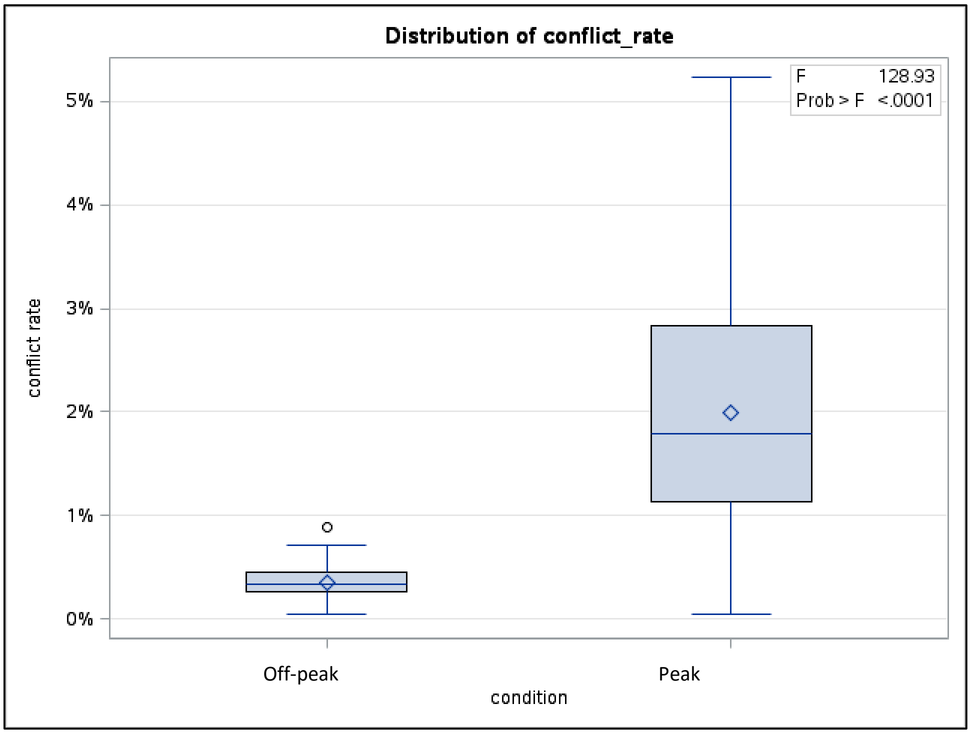


Figure 2.33 - Box plot of the traffic condition for the conflict rate

2.5.1.5 Statistical Modeling

In this step, two statistical modeling were applied to quantify the effect of contributing factors on conflict rate in the weaving segments including Tobit and Log-linear models. Additionally, models were used for identifying the optimal accessibility level and weaving length scenarios that minimized the conflict rate at the studied section.

In the Tobit model, 15 different scenario variables of various access control levels and configurations were included in the model. The statistical analysis software (SAS 9.4) was used for generating the model results. The model formulation takes the following form:

$$y_i = \begin{cases} y_i^*, & if\ y_i^* > 0 \\ 0, & if\ y_i^* \leq 0 \end{cases} \tag{9}$$

$$y_i^* = \beta_0 + \beta_z X + \varepsilon_i \tag{10}$$

Where y_i is the response variable (conflict rate in a weaving segment *i*); y_i^* is a latent variable. The observable variable y_i becomes equal to y_i^* when the latent variable is above zero, and becomes zero otherwise. β_0 is the intercept, β_z represents the coefficient of the independent variables; ε_i is a normally distributed error term with a mean equal to 0 and a variance (α^2); z represents the different scenarios of various accessibility levels and weaving lengths for all studied cases; X is the different scenarios in all cases. The result of the model is shown in Table 2.13.

Table 2.13 - Tobit model for the conflict rate

Parameter	Estimate	Standard Error	t-value	p-value
Intercept	2.285	0.5848	3.91	<.0001
Case 1, 1,000 ft		Reference		
Case 1, 600 ft	1.426	0.4345	3.28	0.001
Case 1, 800 ft	1.043	0.4348	2.4	0.0164
Case 1, 1,400 ft	0.847	0.4348	1.95	0.0392
Case 1, 2,000 ft	0.956	0.4347	2.2	0.0219
Case 2, 600 ft	1.839	0.4351	4.23	<.0001
Case 2, 800 ft	1.474	0.4349	3.39	0.0007
Case 2, 1,000 ft	1.306	0.4348	3.01	0.0027
Case 2, 1,400 ft	1.562	0.4347	3.6	0.0003
Case 2, 2,000 ft	1.482	0.4344	3.41	0.0006
Case 3, 600 ft	3.167	0.4347	7.29	<.0001
Case 3, 800 ft	2.081	0.4349	4.79	<.0001
Case 3, 1,000 ft	1.613	0.4345	3.71	0.0002
Case 3, 1,400 ft	3.073	0.4347	7.07	<.0001
Case 3, 2,000 ft	1.873	0.4348	4.31	<.0001
AIC		145.813		
Log Likelihood		-55.906		
R-squared = 1-(LL (β)/LL (0))		0.18		

The results of the Tobit model revealed that one accessibility level case had a significantly lower conflict frequency than the cases of two and three accessibility levels. Hence, safety analysis showed that one access zone is the optimal level of accessibility in a 9-mile network. It can also be inferred from the model results that 1,000 feet per lane change is the safest weaving length design from the ramps to the access zones.

Additionally, a log-linear model was developed in this study for exploring the interrelationships among the categorical variables. The model was used for identifying the safest access zone design that minimized traffic conflicts at the studied section. Hence, the log linear model was formulated from three variables (x = weaving length, y = accessibility level, and z = traffic condition) and two-way interactions. The statistical analysis software (SAS 9.4) was used for generating the model results utilizing CATMOD procedure. The model formulation is shown as follows:

$$Log\ m_{ijk} = \alpha + \lambda_i^x + \lambda_j^y + \lambda_k^z + \lambda_{ij}^{xy} + \lambda_{jk}^{yz} + \lambda_{ik}^{xz} \quad (11)$$

Where $Log\ m_{ijk}$ is the log of the expected frequency when *i, j,* and *k* are the categories of x, y, and z; α is the overall effect; λ_i^x is the effect due to the *i*th level of the weaving length; λ_j^y is the effect due to the *j*th level of the accessibility level ;λ_k^z is the *k*th level of the traffic condition ;λ_{ij}^{xy} is the interaction of the weaving length at the *i*th level and the accessibility level at the *j*th level; λ_{jk}^{yz} is the interaction of the accessibility level at the *j*th level and the traffic condition at the *k*th level ; λ_{ik}^{xz} is the interaction of the weaving length at the *i*th level and the traffic condition at the *k*th level.

The likelihood ratio (G^2) was used to test the acceptance of the model. The lower value of G^2 and higher p-value (>0.05) indicate better model (the model fits the relationship among the studied variables). The likelihood ratio (G^2=13.279, d.f.=14, p-value=0.1026), as computed from the following formula, implies that the model of two-way interactions was fitted well. Hence, the model can be used to investigate the association between the three categorical variables using the odds multipliers.

The odds multipliers represent the probability of the occurrence of an event relative to another event. It can be calculated from equation (*11*) for main and interaction effects. Equations (*12*) shows the odds multipliers calculation for an event of x=*i*, y=*j*, and z=*k* to the event of x=*i*, y=1, and z=*k*. Similarly, equation (*13*) was formulated when x=*i*, y=*j*, z=*k* instead of z=1. The results of the model are shown in Table 2.14.

$$\frac{m_{ijk}}{m_{i1k}} = \exp[\left(\lambda_j^y - \lambda_1^y\right) + \left(\lambda_{ij}^{xy} - \lambda_{i1}^{xy}\right) + \left(\lambda_{jk}^{yz} - \lambda_{1k}^{yz}\right)] \quad (12)$$

$$\frac{m_{ijk}}{m_{ij1}} = \exp[(\lambda_k^z - \lambda_1^z) + (\lambda_{ik}^{xz} - \lambda_{i1}^{xz}) + \left(\lambda_{jk}^{yz} - \lambda_{j1}^{yz}\right)] \quad (13)$$

Table 2.14 - Comparison of odds multipliers of conflict frequency between various cases (numbers between parentheses are the 90% confidence interval)

Weaving Length (ft)	600	800	1,000	1,400	2,000
Weaving length × Accessibility level:					

Case 1	0.619	0.604	**0.553**	0.569	0.593
	(0.611-0.628)	(0.596-0.615)	(0.545-0.561)	(0.563-0.576)	(0.589-0.602)
Case 2	0.920	0.897	0.871	0.989	0.918
	(0.911-0.930)	(0.887-0.908)	(0.860-0.881)	(0.980-0.998)	(0.914-0.922)
Case 3*	1	1	1	1	1
Weaving length × Traffic period:					
Off-peak	0.341	0.321	**0.292**	0.329	0.334
	(0.338-0.345)	(0.318-0.324)	(0.288-0.297)	(0.326-0.333)	(0.331-0.338)
Peak*	1	1	1	1	1

Note: The odds multiplier more or less than 1 implies higher or lower likelihood of conflict frequency, respectively, than the baseline. *
Base condition

The results of the log-linear model were consistent with the results of the Tobit model. In the log-linear model, the odds multiplier was used for describing the conflict frequency for various scenarios. The first part of the table (Weaving length × Accessibility level) shows the effect of the various weaving lengths on the odds of the accessibility level to the baseline (Case 3). The model results revealed that, one accessibility level case had the lowest odds than both two and three access zones cases. One access zone is the safest level of accessibility in a 9-mile network. Therefore, it can be concluded that the average distance between access zones should not be less than 4.5 miles. This result confirmed the latest guidelines of implementing MLs by NCHRP *(10)* which recommended that spacing between access zones should be designed at 3 to 5 miles. This range was suggested in order to provide safe weaving length between access zones, and to leave sufficient space for signage *(10)*. Additionally, from the second part of Table 1.14 (Weaving length × Traffic condition), it is apparent that the odds multipliers at the off-peak condition are lower than the peak condition. Hence, drivers tend to have lower conflicts in the off-peak conditions than peak conditions.

Furthermore, the results of the table revealed that the weaving length of 1,000 feet per lane change had significant lower odds (α=0.10) compared to all other lengths. Therefore, it can be inferred from the results that the weaving length of 1,000 ft per lane change is the safest access design and it can be used to guarantee a safe lane maneuver from the ramps to the access zones. The result of the weaving distance was confirmed by the findings of the Washington State Department of Transportation (WSDOT) *(21)*. Lastly, from the results, the most dangerous cases, with higher odds multipliers, occurred when the weaving length was 600 feet per lane change. This outcome supports the findings from the California Department of Transportation (Caltrans), which recommends a minimum distance of 800 feet per lane change *(15)*.

2.5.2 Results for Operational Measurements

The traffic operation measurements and revenue were analyzed to assess the operational effects of access control level of the MLs. The evaluation measures for traffic operation included the level of service (LOS), travel speed, time efficiency (time saved by using the MLs), average delay, and revenue.

2.5.2.1 Level of Service (LOS)

LOS is a measurement of the smooth traffic flow in the network. The analysis of LOS was determined based on the methodology identified in Chapter 10 “Freeway Facilities” of the Highway Capacity Manual (HCM) 2010. In this method, the lane density for both GPLs and MLs was used to define the LOS thresholds, as shown in Table 2.15 *(3, 34)*.

Table 2.15 - Level of service from density

Level of Service	Density (pc/mi/ln)
A	≤11
B	>11-18
C	>18-26
D	>26-35
E	>35-45
F	>45 or Any component v/c ratio > 1.00

Source: HCM 2010 *(34)*

Table 2.16 shows the density of the GPLs and MLs for all cases, while Table 2.17 represents the corresponding LOS for all the cases. For the base condition case, the LOS for MLs (B) was better than that of GPLs (C) for the peak period; similarly, in the off-peak conditions, the LOS was better in MLs (A) than in GPLs (B). The LOS in MLs is better than GPLs due to the lower density in MLs and then improving the traffic flow. When comparing LOS for all cases, it was observed that the case of one accessibility level had better LOS and density than the cases of two or three access zones. The most striking results to emerge from the data is that, for the case of one access zone, the LOS improved when the weaving segment length is 1,000 feet or more per lane change.

Table 2.16 - Density for all cases

		Case 1A		Case 1B		Case 2		Case 3	
		GPLs	MLs	GPLs	MLs	GPLs	MLs	GPLs	MLs
Peak	Base	18.83	13.31	18.83	13.31	18.83	13.31	18.83	13.31
	600	27.15	16.09	26.28	16.92	28.69	21.73	34.71	21.44

	800	26.15	17.57	27.21	15.89	27.35	21.55	32.44	19.42
	1,000	24.83	16.57	25.37	17.51	25.00	19.83	28.69	20.33
	1,400	22.53	14.69	23.54	15.64	25.17	20.46	29.82	21.43
	2,000	24.03	15.15	23.31	15.56	24.91	19.64	28.98	21.25
Off-peak	Base	14.28	9.36	14.28	9.36	14.28	9.36	14.28	9.36
	600	19.53	12.82	19.84	14.23	21.39	15.35	22.69	16.77
	800	19.55	11.67	18.47	11.19	20.87	13.44	24.74	16.75
	1,000	17.77	10.97	17.99	10.80	19.03	13.03	23.45	16.93
	1,400	17.50	10.88	17.91	10.22	21.85	13.88	23.24	15.65
	2,000	17.05	10.24	17.53	10.10	21.97	12.03	21.99	15.63

Table 2.17 - Level of service for all cases

		Case 1A		Case 1B		Case 2		Case 3	
		GPLs	MLs	GPLs	MLs	GPLs	MLs	GPLs	MLs
Peak	Base	C	B	C	B	C	B	C	B
	600	D	B	D	B	D	C	D	C
	800	D	B	D	B	D	C	D	C
	1,000	C	B	C	B	C	C	D	C
	1,400	C	B	C	B	C	C	D	C
	2,000	C	B	C	B	C	C	D	C
Off-peak	Base	B	A	B	A	B	A	B	A
	600	C	B	C	B	C	B	C	B
	800	C	B	C	B	C	B	C	B
	1,000	B	A	B	A	C	B	C	B
	1,400	B	A	B	A	C	B	C	B
	2,000	B	A	B	A	C	B	C	B

2.5.2.2 Average Travel Speed

Average travel speed is one of the measurements of effectiveness that was used to evaluate the performance of the network and used for comparing the average travel speeds between different cases in the system. For the base case condition, it can be observed from Figure 2.34 that travel average speed increases dramatically in the MLs in both peak and off-peak conditions by 12.4% and 8.1%, respectively.

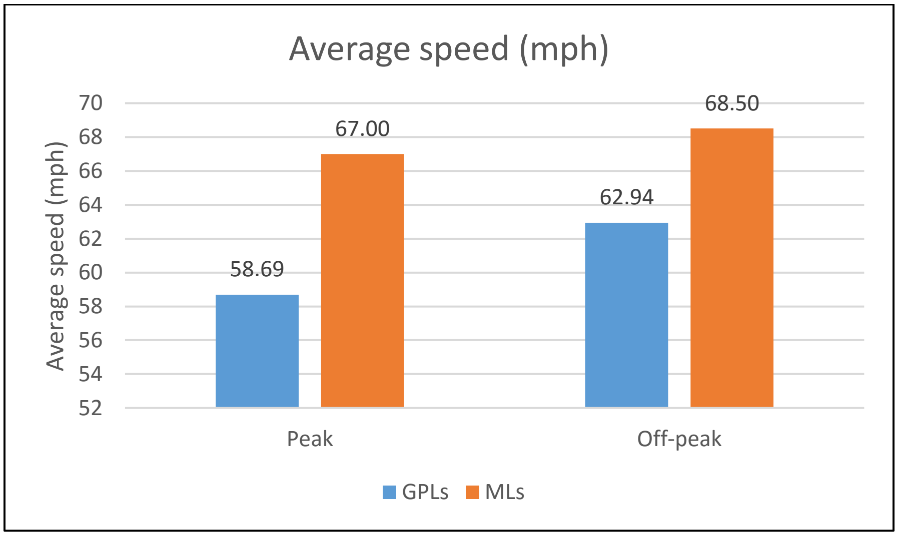


Figure 2.34 - Travel speed of GPLs and MLs in different traffic conditions

The results of travel speed for all cases are shown in Table 2.18. The results are for one hour operating speed for the peak and the off-peak conditions. What stands out in the table is that the average speed in MLs is higher than the GPLs. The highest speed occurred in the case of one accessibility level in both peak and off-peak conditions. Figure 2.35 presents the comparison between travel speed in Case 1A between GPLs and MLs in different traffic conditions. Closer inspection of the figure shows that travel speed was the highest in the MLs (67 mph) in the off-peak conditions. A weaving length of 1,000 feet showed the highest speed among all other weaving distances, followed by the more top weaving lengths of 1,400 and 2,000 feet.

Table 2.18 - Travel speed for all scenarios (mph)

		Case 1A		Case 1B		Case 2		Case 3	
		GPL	ML	GPL	ML	GPL	ML	GPL	ML
Peak	Base	58.69	67.00	58.69	67.00	58.69	67.00	58.69	67.00
	600	55.37	64.58	54.55	63.80	51.12	62.24	50.64	59.81
	800	57.80	65.61	56.62	63.66	52.19	62.27	51.14	59.18
	1,000	57.16	66.77	55.66	64.73	53.19	62.29	51.60	59.64
	1,400	56.99	66.70	56.03	64.73	54.09	63.53	53.30	60.49
	2,000	57.52	66.73	56.71	65.60	54.07	63.54	53.57	62.76
Off-peak	Base	62.94	68.50	62.94	68.50	62.94	68.50	62.94	68.50
	600	60.30	66.41	59.84	65.41	59.58	65.22	58.10	63.32
	800	60.36	66.56	59.81	65.63	59.60	64.87	58.95	63.37
	1,000	61.74	67.73	60.14	66.63	59.62	65.80	58.82	63.34
	1,400	61.36	67.27	60.86	66.96	59.36	64.56	58.39	63.56
	2,000	61.15	67.50	60.94	66.35	59.59	65.69	59.76	63.55

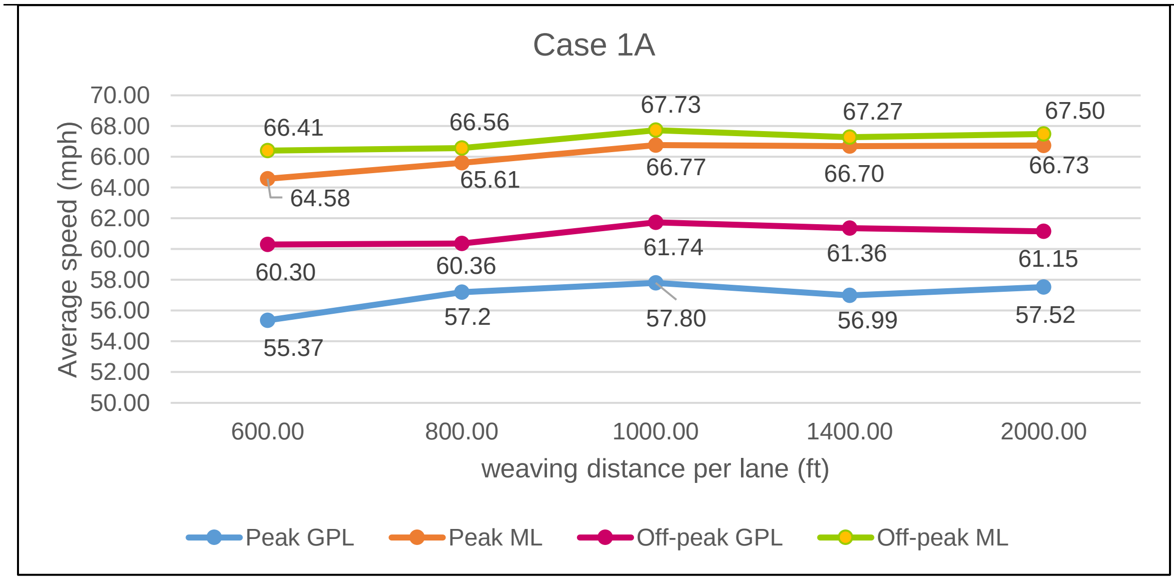
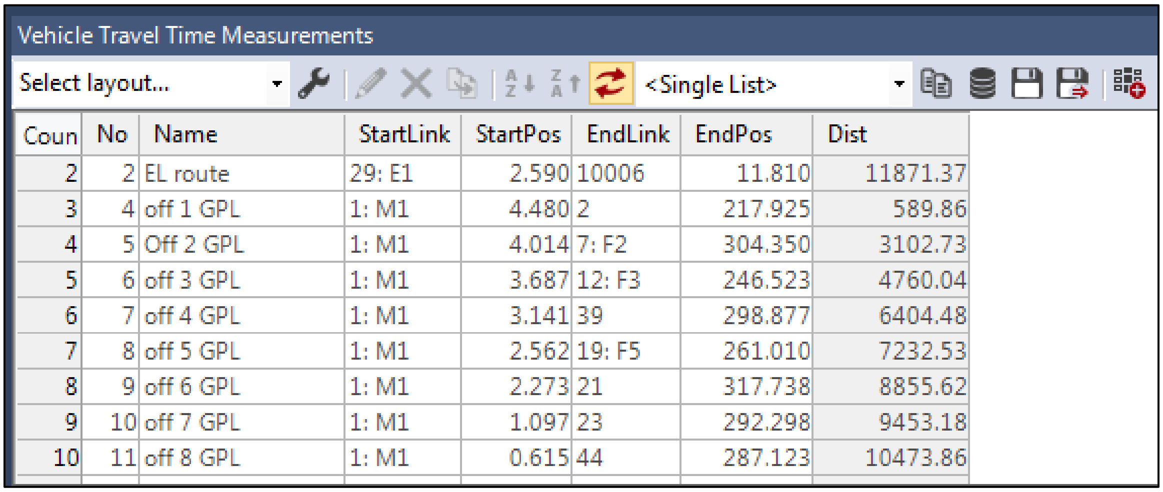


Figure 2.35 - Comparing average speed among one access zone cases

2.5.2.3 Average Delay

The average delay of all vehicles can be measured by subtracting the theoretical travel time from the actual travel time *(27)*. The theoretical travel time is the free flow travel time. The delay measurements were defined in VISSIM by adding vehicle travel time measurements in the network as shown in Figure 2.36. The results showed that for the base case, average delay improved in the MLs markedly by 48% and 41% than GPLs for the peak and the off-peak traffic conditions, respectively, as shown in Figure 2.37.



Coun	No	Name	StartLink	StartPos	EndLink	EndPos	Dist
2	2	EL route	29: E1	2.590	10006	11.810	11871.37
3	4	off 1 GPL	1: M1	4.480	2	217.925	589.86
4	5	Off 2 GPL	1: M1	4.014	7: F2	304.350	3102.73
5	6	off 3 GPL	1: M1	3.687	12: F3	246.523	4760.04
6	7	off 4 GPL	1: M1	3.141	39	298.877	6404.48
7	8	off 5 GPL	1: M1	2.562	19: F5	261.010	7232.53
8	9	off 6 GPL	1: M1	2.273	21	317.738	8855.62
9	10	off 7 GPL	1: M1	1.097	23	292.298	9453.18
10	11	off 8 GPL	1: M1	0.615	44	287.123	10473.86

Figure 2.36 - Delay measurements in VISSIM

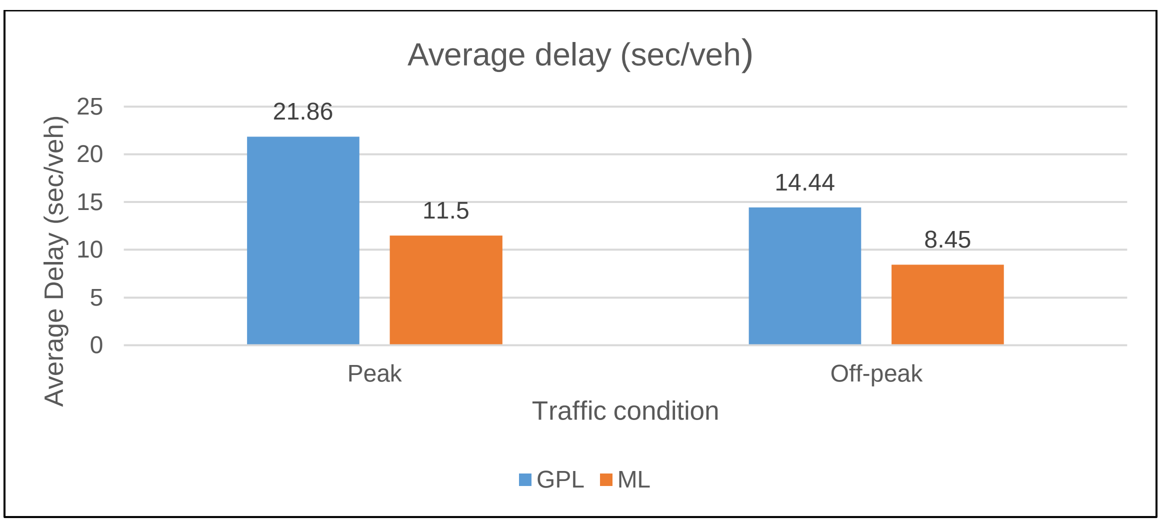


Figure 2.37 - Average delay for the base case

When comparing the delay for the whole network, as shown in Table 2.19, it can be observed that there is a clear trend of average delay declining in the case of one access zone (Case 1A). Also, the lowest delay occurred in the cases of weaving distance of 1,000 feet. Closer inspection of the average delay in Case 1A, as shown in Figure 2.38, it is apparent that the minimum delay happened when the weaving distance was 1,000 feet. In general, the average delay improved in the MLs than the GPLs. The weaving distance of 1,000 feet is the common recommendation among other studied distances.

Table 2.19 - Average delay for all cases (sec/veh)

		Case 1A		Case 1B		Case 2		Case 3	
		GPL	ML	GPL	ML	GPL	ML	GPL	ML
Peak	Base	21.86	11.50	21.86	11.50	21.86	11.50	21.86	11.50
	600	24.20	17.00	26.30	18.70	28.14	20.62	29.50	23.48
	800	23.65	15.40	25.90	17.19	26.65	20.00	29.43	22.00
	1,000	23.05	13.89	25.40	15.83	25.70	19.00	28.17	20.35
	1,400	23.14	14.06	25.76	15.64	26.30	19.31	28.58	20.58
	2,000	23.11	14.03	25.38	15.48	25.90	17.53	28.30	20.11
Off-peak	Base	14.44	8.45	14.44	8.45	14.44	8.45	14.44	8.45
	600	16.70	12.09	16.90	14.21	17.50	15.14	18.50	16.71
	800	15.50	11.45	16.46	13.30	16.50	14.45	19.25	16.45
	1,000	15.15	10.51	15.95	12.83	16.52	14.45	18.37	15.59
	1,400	15.49	10.93	16.12	12.57	16.95	14.75	19.03	15.46
	2,000	15.11	10.68	15.97	12.17	16.60	14.48	18.23	15.66

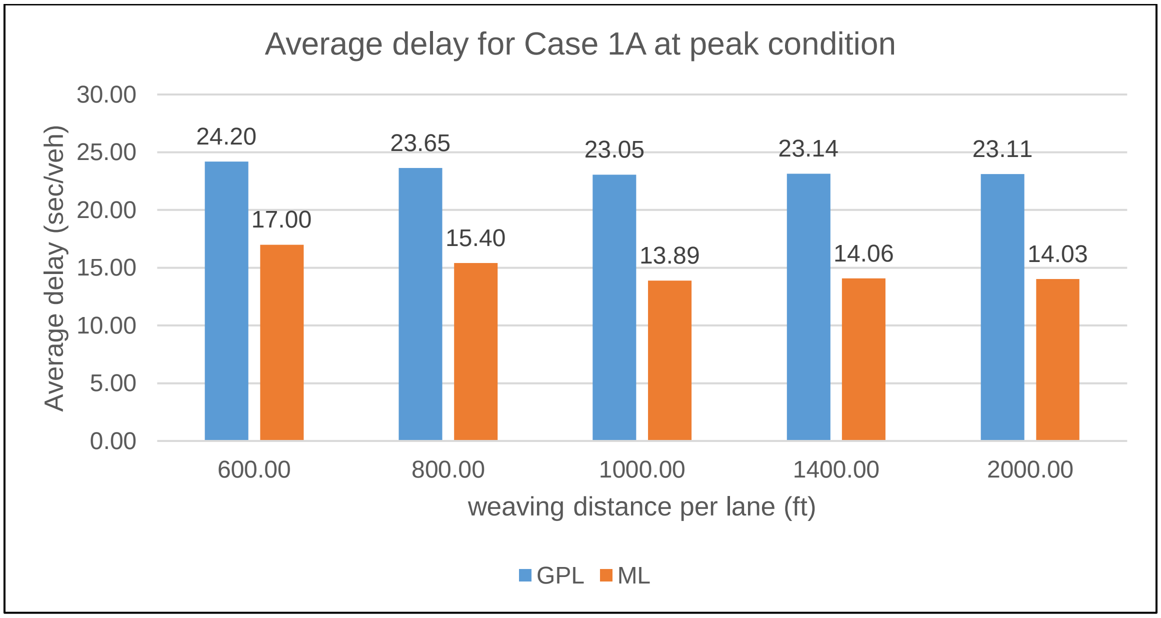


Figure 2.38 - Average delay for Case 1A

2.5.2.4 Time Efficiency

Time efficiency was one of the effectiveness measurements that was used to evaluate the performance of the network for various scenarios. Time efficiency can be explained by the time saved by using MLs. Table 2.20 presents the time efficiency for different cases. The results showed that time efficiency improved in the case of 1A. With respect to weaving length, from the following bar chart in Figure 2.39, it can be concluded that a weaving length of 1,000 is the optimal distance as it can generate maximum time efficiency at both peak and off-peak traffic conditions.

Table 2.20 - Time efficiency for all cases (sec)

		Case 1A	Case 1B	Case 2	Case 3
Peak	Base	64	64	64	64
	600	53	43	43	36
	800	56	45	41	42
	1,000	59	46	36	44
	1,400	56	50	47	45
	2,000	57	43	52	52
Off-peak	Base	55	55	55	55
	600	47	43	46	38
	800	48	45	41	44
	1,000	51	46	42	45
	1,400	48	46	42	41
	2,000	49	45	48	47

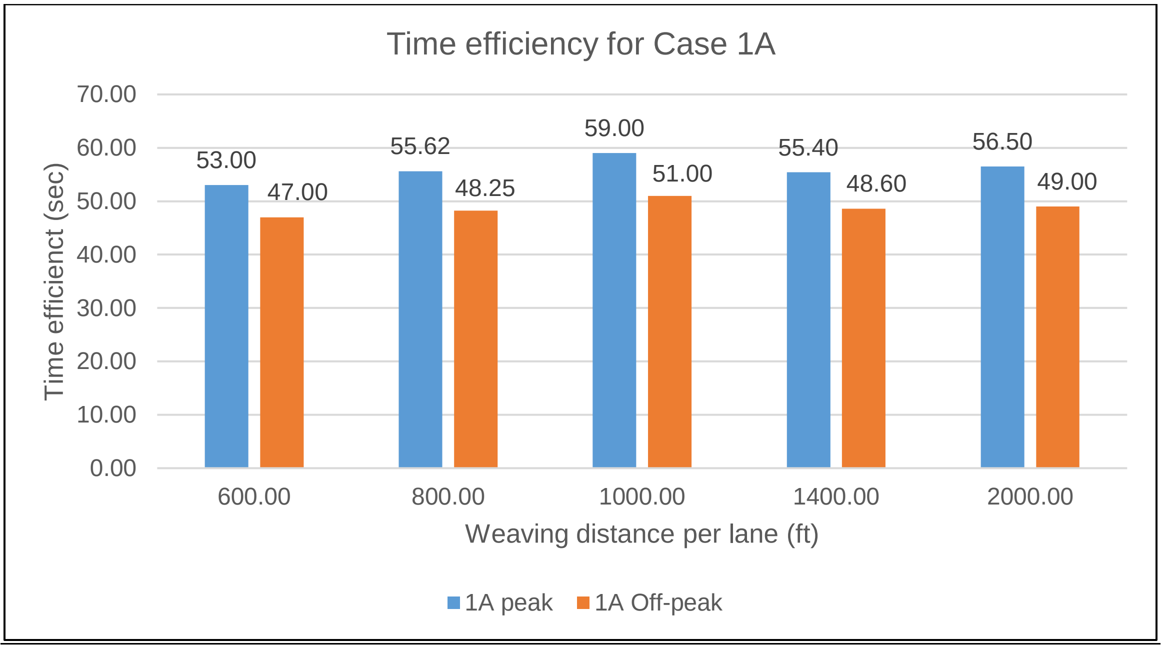


Figure 2.39 - Time efficiency for Case 1A

2.5.2.5 Revenue

The toll pricing of the MLs is one of the main strategies for traffic demand management and for producing revenue. The revenue was calculated, as mentioned before, based on the dynamic toll pricing models, which depend on the speed of the MLs and the time saved by using the MLs (time efficiency). The revenue is higher in the peak hour condition than the off-peak condition by 30%. That can be explained by the higher volume and time efficiency in the peak case.

Table 2.21 shows the revenue of the dynamic toll for all cases. What stands out in the table is that all cases showed higher revenue than the base case with a 30%-65% increase depending on the scenario. For example, the revenue generated from the case of one access zone has about 50% higher than the base case. Case 2 showed the highest revenue for all weaving length cases. The revenue peaks in Case 2 with 1,000 feet weaving length by $2,094 per hour along the network. Contrary to the expected outcome, the case of two access zones was found to have more revenue than the case of three access zones. A possible explanation for this might be due to the location of the access zones. For example, a small number of vehicles use the third access zone because it is near to the end of the network. On the other hand, in the case of two access zones, vehicles use the two access zones and pay two-thirds of the toll in the first access and pay one-third of the toll in the second access zone. Furthermore, it is apparent from the table that revenue is always higher in the peak conditions than the off-peak due to the higher volume in MLs and higher time efficiency. Hence, from the revenue perspective, two access zones case was found to be the optimal case for maximizing the revenue. Moreover, among the weaving lengths cases, the case of 1,000 feet per lane change creates the maximum revenue for both traffic condition cases.

Table 2.21 - Revenue for all scenarios ($/hr)

		Case 1A	Case 1B	Case 2	Case 3
peak	base	820	820	820	820
	600	1,555	1,219	1,881	1,861
	800	1,610	1,200	2,029	1,948
	1,000	1,588	1,223	2,094	1,864
	1,400	1,519	1,293	1,902	1,863
	2,000	1,600	1,217	2,031	1,891
Off-peak	base	580	580	580	580
	600	1,245	956	1,310	1,380
	800	1,241	1,010	1,394	1,297
	1,000	1,236	1,005	1,398	1,341
	1,400	1,246	963	1,420	1,338
	2,000	1,230	957	1,292	1,282

2.5.2.6 Statistical Modeling

Four linear regression models were developed for predicting the factors that affect the traffic operations in the studied network (i.e., average speed, average delay, time efficiency, revenue).

A significance level of P≤0.05 was set as a criterion. The linear regression can be represented by the following formula:

$$Y = \beta_0 + \beta_1\,(Lane\ type) + \beta_2\,(Traffic\ condition) + \ \beta_{ij}\,X_{ij} + \varepsilon \qquad (14)$$

Where y is the dependent variable, for example, speed, average delay; β_0 is the intercept; β_1, β_2, and β_{ij} represent the coefficients of the parameters. The independent variables in the model are the lane type (0 for MLs and 1 for GPLs), traffic condition (0 for off-peak condition and 1 for peak condition), and the different scenarios of accessibility level *i* and the weaving distance *j* for all available cases (Xij). Also, the disturbance term is represented by ε. Table 2.22 shows the four linear regression models for the traffic operational data.

Table 2.22 - Linear regression of the operational models

	Speed (mph)		Delay (sec/veh)		Time efficiency (sec)		Revenue ($/hr)	
Parameter	Estimate	t-stat	Estimate	t-stat	Estimate	Estimate	Estimate	t-stat
Intercept	56.15	80.66	16.91	19.42	51.01	25.18	1618.48	18.9
GPL	-7.27	-24.49	5.63	15.16	All segment		All segment	
Peak Condition	-3.75	-12.63	6.80	18.31	2.03	2.30	436.96	11.69
Case 1A-600	Reference		Reference		Reference		Reference	
Case 1A-800	-	-	-	-	-	-	-	-
Case 1A-1,000	1.68	1.80	-1.45	-1.97	5.02	1.79	-	-
Case 1A-1,400	1.42	1.69	-	-	-	-	-	-
Case 1A-2,000	1.56	1.71	-	-	-	-	-	-
Case 1B-600	-	-	1.35	1.84	-7.14	-2.5	-312.50	-2.64
Case 1B-800	-	-	-1.27	-1.73	-5.08	-1.79	-295.25	-2.5
Case 1B-1,000	-	-	-	-	-	-	-286.25	-2.42
Case 1B-1,400	-	-	1.24	1.69	-	-	-272.13	-2.3
Case 1B-2,000	-	-	-	-	-6.64	-2.15	-313.25	-2.65
Case 2-600	-2.12	-2.26	2.85	2.43	-5.52	-1.97	245.50	2.08
Case 2-800	-1.93	-2.06	-	-	-9.33	-3.22	311.12	2.63
Case 2-1,000	-	-	-	-	-11.12	-3.98	395.75	3.35
Case 2-1,400	-	-	1.82	2.48	-5.52	-1.97	316.6	2.68
Case 2-2,000	-	-	-	-	-	-	-	-
Case 3-600	-3.69	-3.94	4.55	3.87	-13.25	-4.74	220.50	1.87
Case 3-800	-3.50	-3.73	4.28	3.65	-6.87	-2.46	222.25	1.88
Case 3-1,000	-3.31	-3.53	3.12	2.66	-5.75	-2.06	202.50	1.71
Case 3-1,400	-2.73	-2.91	3.41	2.91	-6.87	-2.46	200.25	1.69
Case 3-2,000	-1.75	-1.87	3.07	2.62	-	-	-	-
MSE (SSE)	1.762 (102.20)		2.75 (160.02)		7.81 (148)		8997 (530832)	
R-squared	0.91		0.89		0.78		0.91	
Coefficient of Variance	2.1		8.80		5.32		6.72	

The first linear regression model was constructed for modeling the average speed in the studied network. Four independent variables were used in this model including lane type (GPL, ML), traffic condition (Peak, off-peak), and the different scenarios of accessibility level and weaving distance. The model indicated a main effect of the lane type. Vehicles travel at a significantly higher speed in the MLs than GPLs. Another important finding was that, in off-peak conditions, there is a significantly higher speed of 3.75 mph, as shown in Figure 2.40. Moreover, the average travel speed in the network for the one access zone scenarios (Case 1A) is considerably higher than other cases. For example, the travel speed in the three access zones is lower than Case 1A by 3.69 mph when the weaving length is 600 feet. The lowest speeds occurred in the case of three access zones. With respect to the weaving length, travel speed peaks in the case of weaving length of 1,000 feet compared to all other cases. Another finding is that there is no significant difference between the travel speed in the case of weaving length of 600 feet and 800 feet when testing one access zone case.

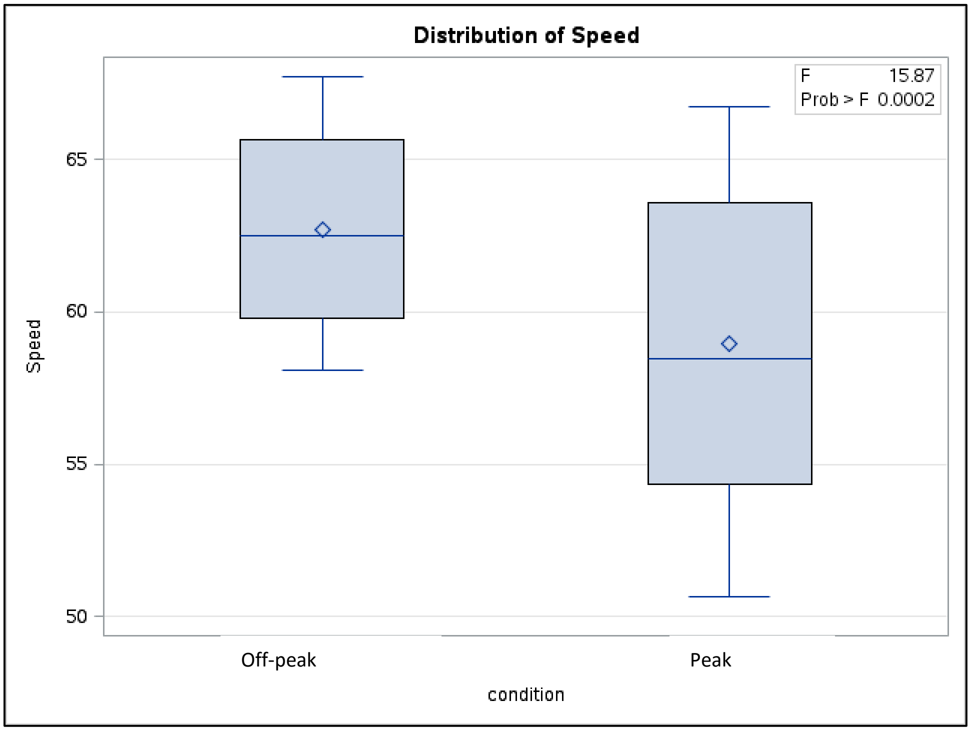


Figure 2.40 - Relation between travel speed in different traffic conditions

The second linear regression model was developed to explore the factors that affect the average delay in the whole network. As Table 2.22 Shows, there is a significant difference between the different lane types. The average delay in GPLs is higher than the MLs by 5.63 sec/veh. The results showed that there is an association between the average delay and the traffic conditions. During peak-hour, the average delay is significantly higher than off-peak conditions by 6.8 sec/veh, as shown in the boxplot in Figure 1.41. Also, from the results, vehicles that drove in scenarios with one access zone (Case 1A) has a significantly lower delay than other cases. Meanwhile, the highest average delay occurred in the case of three access zones with 4.54 sec/veh to 3.07 sec/veh higher than the case of one access zone (Case 1A) for the weaving length of 600 feet to 2,000 feet. Furthermore, the case of weaving length equal to 600 feet always had a significantly higher delay than other lengths.

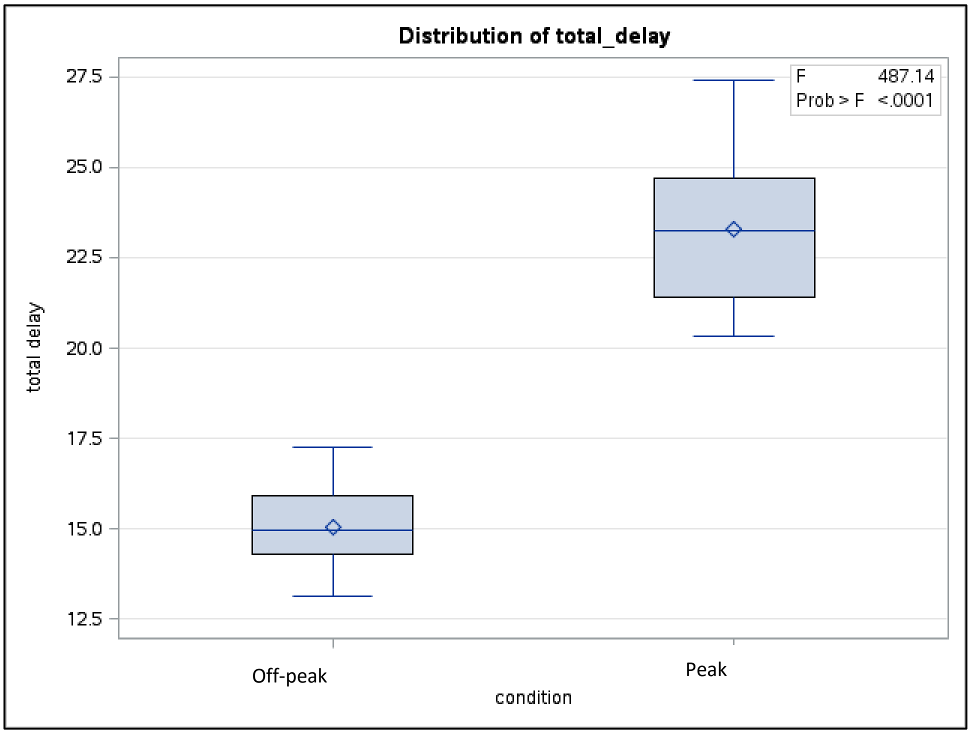


Figure 2.41 - Boxplot of the average delay in different traffic conditions

The third linear regression model was conducted for estimating the factors that influence time efficiency, as shown before in Table 2.22. For the traffic condition, there is a significantly higher time efficiency (2.03 sec) in peak hour condition than an off-peak condition, as shown in Figure 2.42. Moreover, the results offer interesting findings that there is a significantly higher time efficiency in the case of one access zone (Case 1A) than other cases. Weaving length of 600 feet had the lowest time efficiency compared to other weaving distances.

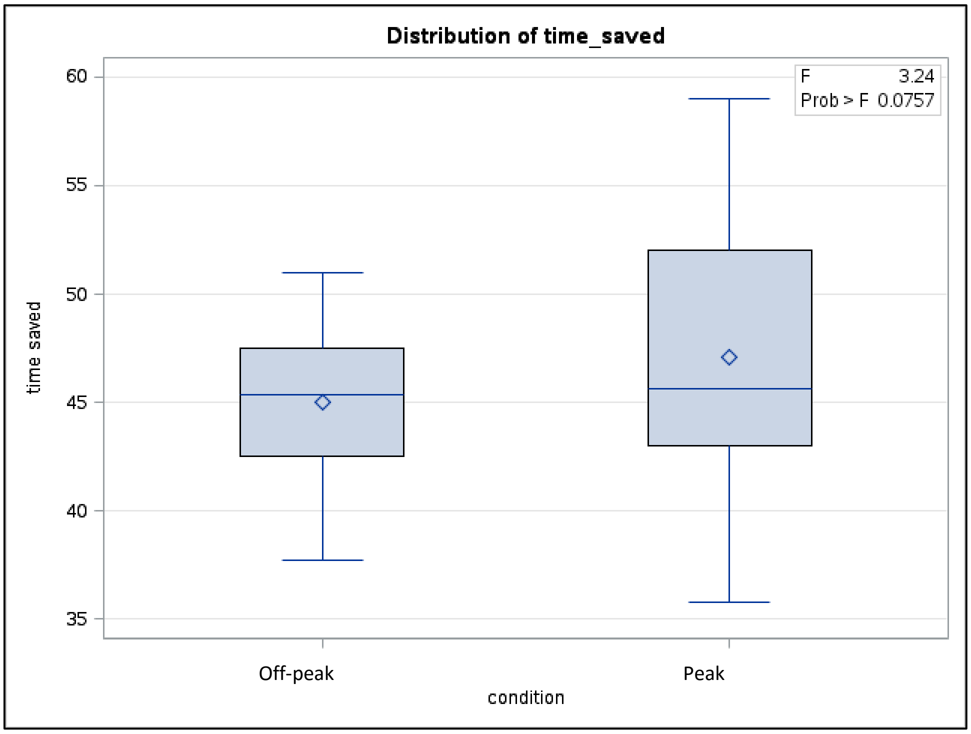


Figure 2.42 - Boxplot of time efficiency in different traffic conditions

The fourth linear regression model was developed for predicting the factors that influence the revenue in the network, as illustrated in Table 1.22. The results showed that revenue is more likely to increase in the peak hour condition by $437 per hour (Figure 2.43). Revenue in Case 1A is higher than Case 1B by an average of $303 per hour for all scenarios. There were no significant differences in the revenue between the various weaving length scenarios in Case 1A. Another significant finding is that the case of two access zones had the highest revenue and Case 1B had the lowest revenue among all cases.

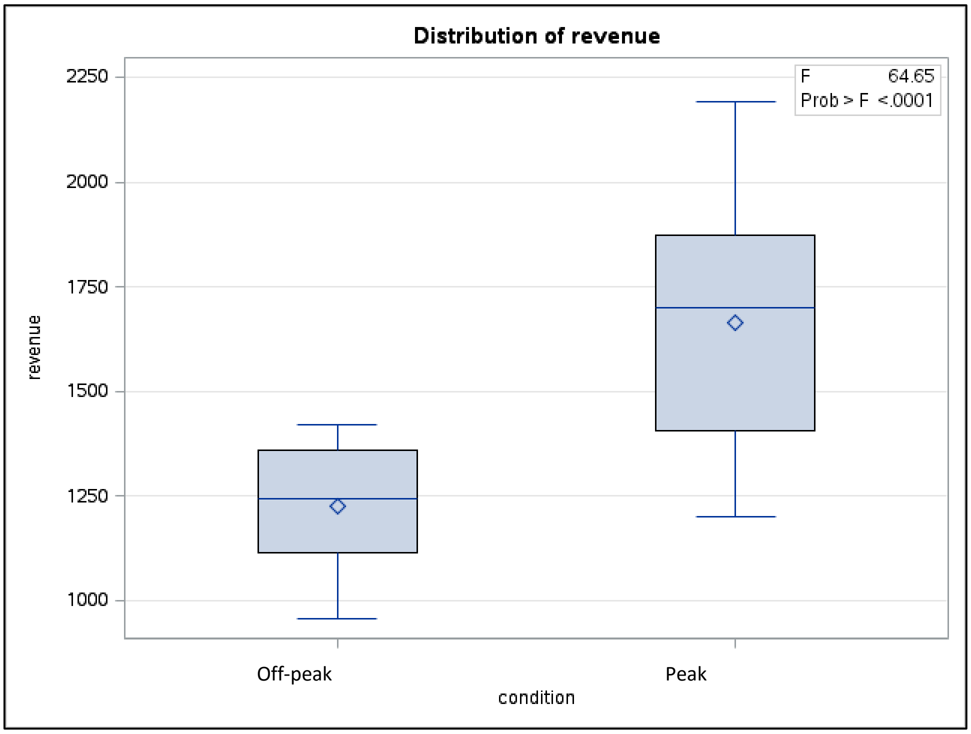


Figure 2.43 - Boxplot of the revenue for different traffic conditions

2.6 Conclusions

On expressways, concerns about traffic safety and operation have been highlighted with a rapid growth of traffic in urban areas. Managed lanes have been implemented as an important facility in improving traffic mobility, efficiency, and safety, in addition to generating revenue for transportation agencies. This study presents the first comprehensive investigation for the access zones of the MLs. Most of the previous studies of the MLs have only explored safety and operational impacts of whole ML segments without consideration of the safety and operational effects of accessibility levels and weaving distance. This research was undertaken to design the accessibility of the MLs and evaluate the safety and the operation of the sections near access zones. Hence, in this investigation, the aim was to determine the optimal accessibility level to maximize system-wide efficiency. The second purpose of the study was for deciding the sufficient length and the location of the weaving access zones. Microscopic state-of-the-art traffic simulation technique was developed and applied to achieve the principal objectives of the research. Extensive data collection was conducted from microsimulation scenarios that included a 9-mile network of a ML segment on an Interstate (I-95) in South Florida. The network was well calibrated and validated by comparing the operational measurements for both simulated and field data.

Subsequently, safety and operational measures of effectiveness were used from the experiments. For the safety measurements, conflict frequency and conflict rate of the weaving segments were used to determine the optimal weaving length at the access zones. With respect to the operational measures of effectiveness, level of service (LOS), travel speed, time efficiency, and average delay were used to determine the optimal accessibility level. Additionally, the revenue was generated to compare the monetary benefits of various strategies. Overall, this study established a quantitative framework for deciding the accessibility level and density for a ML section and nearby on- and off-ramps.

This project illustrated the association between the design of access zones in MLs and both the operational and the safety effectiveness. In general, a logistic regression model was used to compare the surrogate safety measures (i.e., time-to-collision, the maximum speed at the conflict, and conflict type) between MLs and GPLs. From the developed logistic regression model, it was found that MLs were safer since it has higher TTC and lower lane-change conflicts comparing to GPLs. Moreover, the findings of the study indicated that the conflict frequency per vehicle on MLs were 48% and 11% lower than that of GPLs in the peak and off-peak traffic conditions, respectively. One of the most prominent findings from this study was that, under different traffic conditions, the conclusion of safety and operational measurements were different. The proposed linear regression models for the operational measurements indicated that the case of one access zone had a higher speed, a lower delay, and a greater time efficiency than other levels. Overall, the operational performance deteriorates when accessibility level increases. Hence, the operational performance results confirmed the results of the safety analysis that one access zone is the optimal level of accessibility in a 9-mile network. Also, the level of service (LOS) improved with the smaller number of access zones. When comparing the case of no access zones and the case of one access zone, it was observed that the LOS was the same when the access zones length is more than 1,000 feet per lane change. From the monetary point of view, the highest revenue was created from the two access zones case in the studied network.

The findings of this research have a number of important implications for future practice or policy. An implication of these results is that both accessibility level and weaving segment length should be taken into account when designing the access zones of MLs for expressways. The findings can contribute to improving the priced MLs based on the fusion of the safety and operational measurements. The study gives recommendations to the transportation agencies for improving the mobility and the efficiency of the MLs. This study is consistent with previous studies that the implementation of MLs improves the mobility and efficiency of the network besides generating revenue from the dynamic toll *(8)*. The result of the weaving distance is confirmed by the findings of the Washington State Department of Transportation (WSDOT) *(21)* that the recommended weaving length should be 1,000 feet per lane change. For the locations where ramp density is low, the 1,000 feet per lane change might be the minimum. But for locations where ramp density is high, the longer distance might result in plenty of ramp traffic involves in the entering or exiting MLs access maneuvers. Hence, longer distance might result in unsafe situation. Hence, under this condition, 1,000 feet per lane change is the optimal length. Furthermore, from the findings of this study, a weaving length of 600 feet per lane change is not recommended near the access zones of the MLs. This outcome supports the California Department of Transportation, which recommends a minimum distance of 800 feet per lane change *(15)*. The findings proposed distance between access zones should be not less than 4.5

miles. This result confirmed the guidelines for implementing MLs by NCHRP that the recommended spacing between access zones should be between 3 and 5 mile *(10)*

Future research should focus on improving the safety and the operation of the MLs. Recently, several new designs have been established to connect the ramps with the MLs. For example, the direct and slip ramps have been used to connect the ramps to MLs directly without generating weaving segments. Additionally, new technologies and transportation strategies are being proposed for maximizing the traffic performance in MLs. The active traffic management techniques (i.e., variable speed limit, ramp metering, dynamic shoulder lanes) should be tested with MLs using a simulation technology for operational and safety improvement. Wang et al. *(35)* proved that ATM strategies could improve the safety of the weaving segments by generating lower conflict frequency. Also, one of the new applications is examining the impact of autonomous vehicles and connected vehicles to enhance the traffic operation performance simultaneously with the safety benefits at the facility. These new strategies may be excellent approaches for improving the traffic operation and safety at MLs, because it is responsive to real-time traffic. Meanwhile, implementing these strategies at the facility may be used for environmental optimization since it can be efficiently responsive to different environmental conditions *(36)*. Lastly, driving behavior in the risky locations near MLs should be explored utilizing driving simulator technique in the hopes that it would have a significant impact on the facility.

3 Driving Simulator Experiment Approach

3.1 Introduction

Toll managed lanes have been employed for mitigating congestion and improving efficiency of freeway facilities. They allow transportation agencies to allocate parts of the capacity of the freeway facility to special user classes, including high-occupancy vehicles (HOV), high-occupancy tolls (HOT), public transit, truck-only tolling, and express tolling *(37)*. The Federal Highway Administration (FHWA) defines the MLs as highway facilities or a set of lanes in which operational strategies are implemented and managed in real time in response to changing conditions *(38)*. The MLs are distinguished from other traditional forms of lane-management strategies in that they are proactively implemented, managed, and may involve using more than one operational strategy.

In recent years, the TML systems have been introduced on several states in the United States such as HOT (High-Occupancy Toll) lanes on I-15 in California, HOV (High Occupancy Vehicle) lanes on I-25 in Colorado, MLs on I-95 in Florida, etc. The access strategies for the current MLs in different states are quite diverse. For example, there is no access point except the start and end points on I-95 MLs (Phase 1) in Florida, while there are multiple access points on I-15 MLs in Utah and California. Also, the distance between on-ramp (or off-ramp) and the TML entrance (or exit) varies across the different states.

To our knowledge, there is no quantitative conclusion on the safety impacts of the weaving distance between on-ramp (or off-ramp) and the TML entrance (or exit) has been reached in the previous studies. However, the short weaving distance might be very dangerous for the continuous lane change when the vehicles cross the freeway from on-ramp to the entrance of TML *(16)*. In order to efficiently and safely operate the TML systems, it is necessary to determine and provide the optimal weaving distance between on-ramp (or off-ramp) and the TML entrance (or exit).

In this research project, we have tried to investigate the safety impacts of different weaving distance between on-ramp (or off-ramp) and the TML entrance (or exit) based on the driving simulator experiment. There are two major cases we need to consider: first, the distance from an upstream TML exit to the next downstream off-ramp; second, the distance from an upstream on-ramp to the next downstream TML entrance. In addition, the variable speed limit (VSL) strategy might be able to help drivers safely enter or exit MLs. Hence, the required safe length for continuous lane change can be reduced with the implementation of VSL. Therefore, the main research objectives of this project are summarized as follows:

- Identifying optimal weaving distance and location of weaving zones near on- or off-ramps utilizing driving simulator and
- Exploring the impacts of VSL technology on the traffic operation for MLs, and verifying whether it could reduce the required safe length for continuous lane change.

Following the brief introduction and overview in sub-chapter 3.1, sub-chapter 3.2 summarizes literature about TML and safety evaluation. Sub-chapter 3.3 explains the experimental design for the study. Sub-chapter 3.4 explains the experiment development and procedure. Sub-chapter 3.5 presents the analysis and results, and sub-chapter 3.6 concludes the report and provides suggestions.

3.2 Literature Review

3.2.1 Toll managed lane safety

Previous studies about the MLs have mainly focused on the traffic operational performance such as speed, volume, and capacity *(37, 39-43)*. Generally, they found that the volumes in TML and total volumes in all lanes increased during peak hours, the LOS of the MLs was better than that of the GPLs, and the travel speed on the MLs was higher than those on the GPLs in peak hour.

However, only few efforts have been made to investigate the safety effects of the MLs. Golob et al. *(44)* compared the frequency and characteristics of collisions before and after installation of an HOV lane without physical separation (i.e., buffer-separated) by converting the inner shoulder area to an HOV lane on State Route 91 in Los Angeles, California. The study concluded that the installation of HOV lanes did not have an adverse effect on the safety performance of the corridor and the changes in collision characteristics were due to the changes in spatial and temporal attributes of traffic congestion.

Lee et al. *(45)* developed negative binomial models to estimate the effects of several lane-specific factors such as AADT and the managed-lane strategy (that allows drivers to use the right shoulders as travel lanes while the inner left lanes are open to only HOV traffic during peak hours) on crash frequency. They found that the managed-lane strategy was not significantly correlated with the crash frequency on the inner left lanes for HOV, GPLs, and right shoulders. Cooner and Ranft *(46)* examined the safety effects of Dallas's buffer-separated concurrent-flow HOV lanes on I-35 East and I-635. They found that both corridors had an increase in crash rates after implementation of the HOV lane, and the increase was primarily attributed to the speed difference between the HOV and the GPLs.

Besides, Jang et al. *(16)* investigated the collision rates on HOV lanes with respect to shoulder width, length of access, and proximity of access to neighboring ramps for the HOV facilities with both continuous access and limited access. It was shown that limited-access HOV facilities with a combination of short ingress–egress length and proximity distance to the nearest on- or off-ramp have markedly higher collision rates than other limited-access freeway segments.

To be sure, a sufficient road length is required for drivers to cross the freeway when they merge into the TML from the on-ramp or exit the TML to the off-ramp. To our knowledge, the study about the safety impacts of different length for continuous lane-change maneuvers for the TML has not been conducted until this point. Thus, this study aims to figure out the sufficient length and location of weave zones near on- or off-ramps utilizing driving simulator.

3.2.2 Crash-Prone Traffic Condition

In order to suggest appropriate road length for driver to make lane change, the crash-prone traffic condition should be considered. The previous studies about the relationship between traffic condition and crash risk are summarized in Table 3.1.

Table 3.1 - Literature Review on Freeway Crash-Prone Traffic Condition

No.	Analysis framework	Crash type	Crash-Prone traffic condition	Reference
1	Disaggregate crash risk analysis	Rear-end	Congestion index >=0.075 & 5-minute volume >175 & average speed <67mph	Shi and Abdel-Aty *(47)*
2	Disaggregate crash risk analysis	Total	Level of Service E	Xu et al *(48)*
3	Aggregate number of vehicles involved in crashes for each traffic state (speed)	Total	The crash involvement rates in Congested traffic (CT), bottleneck front (BN) and back of queue (BQ) are approximately 5 times higher than Free Flow	Yeo et al *(49)*
4	Disaggregate crash risk analysis	Total	Upstream occupancy is 25% (average 30-second detector occupancy)	Xu et al *(50)*
5	Disaggregate crash risk analysis	Fatal and incapacitating injury (KA), Non-incapacitating and possible injury crashes (BC), and PDO.	Low severity crashes (PDO) tended to occur in congested traffic flow condition; The injury crashes (KA and BC) were found to occur more often in less congested traffic flow conditions.	Xu et al *(51)*
6	Disaggregate crash risk analysis	Total	Traffic state 4 (situation in which the upstream traffic is in free flow while downstream traffic is in congested flow)	Xu et al *(52)*

As we can see from Table 3.1, almost all of the studies achieved similar conclusions about the crash-prone traffic condition which occurred when the traffic was slight congested or congested.

3.2.3 Impacts of Variable Speed Limit (VSL)

The impacts of VSL on weaving distance between GPLs and MLs will also be investigated in this study. The VSL should have significant effects on the traffic flow on both GPLs and MLs Thus, before the experiment design, the impacts of VSL should be firstly explored. Generally, for the free flow condition, VSL implementation could result in a significant reduction in the mean and variance of speed within each lane and the speed difference across lanes. For the pre-congestion traffic flow, VSL implementation could result in a slight improvement in the mean speed, whereas it will reduce both the speed variance within each lane and across adjacent lanes. The studies about the effects of VSL on traffic are summarized in Table 3.2.

Table 3.2 - Literature Review on the Impacts of VSL on Traffic Flow

Studies	Non-VSL					VSL				
	Speed Limits (mph)	Lane Type	Observed Speed (mph) Mean	Observed Speed (mph) SD	Traffic flow	Speed Limits (mph)	Lane Type	Observed Speed (mph) Mean	Observed Speed (mph) SD	Traffic flow
(McMurtry et al., 2009)	65	Total	60	8.36	N/A	65	Total	62.5	5.64	N/A
						55	Total	56.4	6.08	
(Lucky, 2014)	80	Total	70.84	11.42	Headway (s)	68	Total	64.56	8.23	Headway (s)
		Shoulder lane	56.55#	14.9*	3		Shoulder lane	54.06#	5.59*	3
		Middle lane	75.81#	9.32*	2		Middle lane	72.08#	7.46*	2
		Fast lane	78.29#	8.08*	2		Fast lane	73.94#	8.08*	2
	62	Total	59.46	5.06	N/A	55	Total	60.83	7.05	Headway (s)
	50	Total	47.53	5.12	N/A		Shoulder lane	52.2#	4.35*	2
							Middle lane	64.62#	9.32*	2
							Fast lane	70.84#	11.19*	1
(Knoop et al., 2010)	75	Total	N/A	N/A	Lane flow rate (%)	37	Total	N/A	N/A	Lane flow rate (%)
		Shoulder lane	N/A	N/A	40		Shoulder lane	N/A	N/A	37.5
		Middle lane	N/A	N/A	40		Middle lane	N/A	N/A	37.5

		Fast lane	N/A	N/A	20		Fast lane	N/A	N/A	25
(van Nes et al., 2010)	50	Total	N/A	longitudinal driving speed SD: 3.6	N/A	44	Total	N/A	longitudinal driving speed SD: 2.86	N/A
(Duret et al., 2012)	80	Total	N/A	N/A	Lane flow rate (%)	68	Total	N/A	N/A	Lane flow rate (%)
		Shoulder lane	55.92	N/A	22.35		Shoulder lane	55.92	N/A	19.77
		Middle lane	68.35	N/A	N/A		Middle lane	62.14	N/A	N/A
		Fast lane	77.67	N/A	N/A		Fast lane	68.35	N/A	N/A
(Chang et al., 2011)	55	Total	28.7	N/A	Total flow: 3713veh/h (2 lanes)	35	Total	30.3	N/A	Total flow: 3980veh/h (2 lanes)
(Weikl et al., 2013)	75	Total	N/A	N/A	Lane flow rate (%)	37, 50, 62, 75	Total	N/A	N/A	Lane flow rate (%)
		Shoulder lane	N/A	N/A	25		Shoulder lane	N/A	N/A	27
		Middle lane	N/A	N/A	37		Middle lane	N/A	N/A	34
		Fast lane	N/A	N/A	38		Fast lane	N/A	N/A	40
(Hoogen ; and Smulders,	75	Total	N/A	N/A	N/A	44/56	Total	N/A	N/A	N/A
		Shoulder lane	54.06	3.11	N/A		Shoulder lane	50.33	3.73	N/A

1994)		Middle lane	57.79	4.35	N/A		Middle lane	53.44	4.35	N/A
		Fast lane	60.89	4.97	N/A		Fast lane	55.92	4.97	N/A
(Kang and Chang, 2011)	NO VSL	Total	20.8	9.5	1670vphpl	VSL	Total	28.2	7.4	1750vph pl
(Kwon et al., 2007)	NO VSL	Total	47.21	14.15	3595vehph	VSL	Total	48.46	10.43	3852veh ph

Note: the sign of # indicates the Upper-Lower quartile; the sign of * indicates the Upper-Lower quartile.

3.2.4 Surrogate Safety Measures

Traffic safety evaluation is one of the most important processes in the analysis of transportation systems performance. Most of traditional analyses of traffic safety measures are carried out based on the observed crash data. However, since it is difficult to obtain the crash data based on the driving simulator experiment, the surrogate safety measures have been widely used in previous studies. For example, time-to-collision (TTC) was used as the crash risk indicator in previous driving simulator studies *(53-55)*. The commonly used surrogate safety measurements that are summarized as shown in Table 3.3.

Table 3.3 - Literature Review on Surrogate Safety Measurements

Surrogate safety measurements	Description
Time-to-Collision (TTC)	The TTC has been widely employed to analyze rear end collision: (a) conflict with the preceding vehicle, (b) conflict with the following vehicle in the merge lane, (c) conflict with the leading vehicle in the target lane, and (d) conflict with the lagging vehicle in the target lane.
Time-exposed time-to-collision (TET)	The total time spent in safety critical situations, characterized by the TTC value below the threshold value TTC*.
Time-integrated time-to-collision (TIT)	The TIT evaluates the entity of the TTC lower than the threshold and allows expression of the severity associated with the conditions of approach that take place in time.
Headway distance (time)	Headway distance is the distance between the merging vehicle and the leading vehicle.
Post-Encroachment-Time (PET)	PET is the time difference between when the first vehicle leaves a potential point of conflict to the moment the second vehicle subsequently arrives at the same point.
Deceleration rate difference	Deceleration rate difference measures the difference between leading and following vehicles' deceleration rates.
Deceleration Rate to Avoid Crash (DRAC)	It was defined by Cooper and Ferguson (1976) as the minimum deceleration rate required by the following vehicle to come to a timely stop (or match the leading vehicle's speed) and hence avoid a crash

- **Time-to-Collision (TTC)**

The idea of computing a time-to-collision (TTC) was first suggested by Hayward *(56)*. The TTC is defined as “the time required for two vehicles to collide if they continue at their present speed and on the same path”. For the merging or lane-changing vehicle, four kinds of conflicts should be considered for the TTC: (a) conflict with the preceding vehicle, (b) conflict with the following vehicle in the merge lane, (c) conflict with the leading vehicle in the target lane, and (d) conflict with the lagging vehicle in the target lane *(57)*. The merging or lane-changing vehicle and the neighboring vehicles are depicted in Figure 3.1.

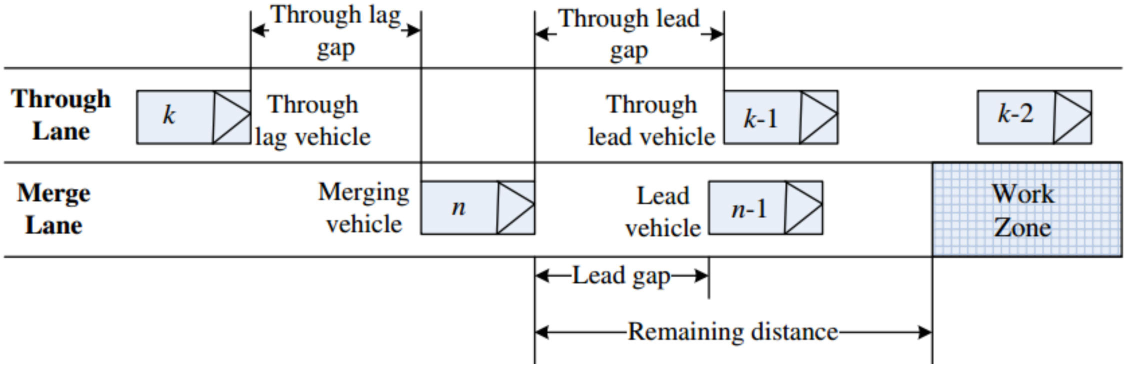


Figure 3.1 - Merging/Lane-changing Vehicle and the Neighboring Vehicles *(57)*.

If the merging vehicle decides to move into the through lane, there will be a potential of collision with the through lead and/or lag vehicles for the merging vehicle. The TTC between the merging vehicle and the through lag vehicle k can be calculated as *(58)*.

$$\mathrm{TTC}_n^k(t) = \frac{d_n^k(t)}{v_k(t) - v_n(t)}$$

Where $d_n^k(t)$ is the through lag gap between the through lag vehicle k and the merging vehicle n at time t, and $v_k(t)$ is the speed of the through lag vehicle k at time t.

The TTC between the merging vehicle and the through lead vehicle k-1 can be calculated as:

$$\mathrm{TTC}_n^{k-1}(t) = \frac{d_n^{k-1}(t)}{v_n(t) - v_{k-1}(t)}$$

Where $d_n^{k-1}(t)$ is the through lead gap between the merging vehicle n and the through lead vehicle k-1 at time t, and $v_{k-1}(t)$ is the speed of the through lead vehicle k at time t.

In terms of the threshold of TTC, large values are not included because they are not safety critical. A value of 6 s was the threshold applied following Vogel's research in which vehicles with a headway of more than 6 s chose their speed independently of the leading vehicle *(59)*. Furthermore, no research points to a TTC larger than 6 s as affecting safety; instead, some studies have suggested an even smaller TTC threshold of 4 s.

- **Time-exposed time-to-collision (TET)**

The TET indicator expresses the total time spent in safety critical situations, characterized by the TTC value below the threshold value TTC* *(54, 60)* (Figure 3.2).

$$\mathrm{TET}^* = \sum_{j=1}^{T} \delta_j\,(t) \cdot \tau_{sc} \quad \delta_j(t) = \begin{cases} 1 & \forall\ 0 \le TTC_i(t) \le TTC^* \\ 0 & else \end{cases}$$

The superscript * indicates that the parameter has been calculated with respect to a prefixed threshold value.

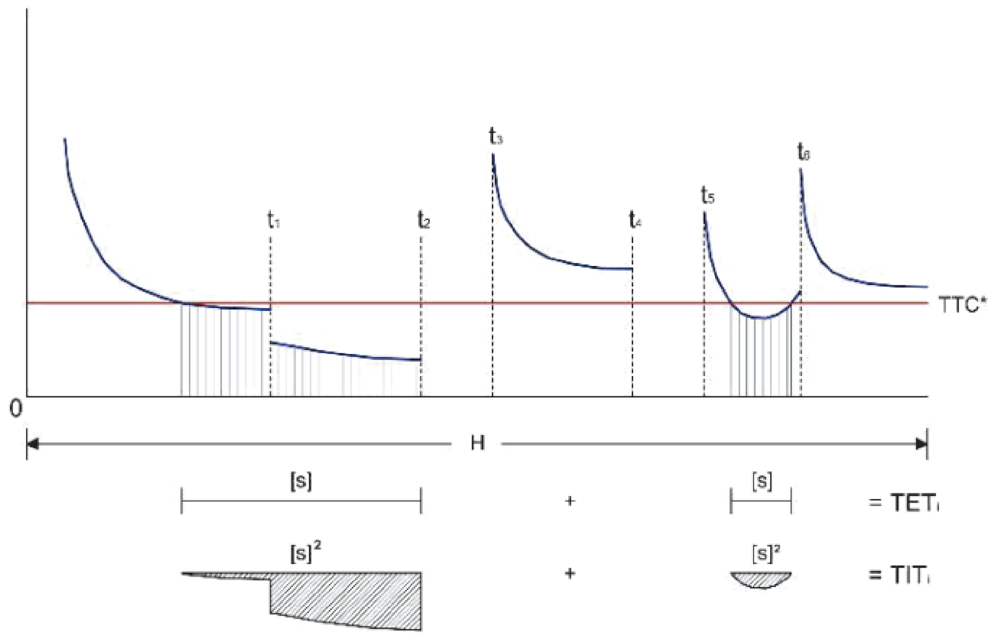


Figure 3.2 TTC Profile and Corresponding TTC-based Safety Indicators *(54)*.

- **Time-integrated time-to-collision (TIT)**

The TIT indicator, which evaluates the entity of the TTC lower than the threshold, allows expression of the severity associated with the conditions of approach that take place in time. The most efficient values of threshold of TTC were considered to be 2.5 and 3 s *(54)*.

$$TIT^* = \sum_{j=1}^{T} [TTC^* - TTC_i(j)]\tau_{sc} \qquad \forall 0 \leq TTC_i(j) \leq TTC^*$$

- **Headway distance (time)**

Headway distance serves as a buffer zone of an urgent stop for the hazardous situation. For a safety concern, headway time is often required to be sufficient for drivers to stop without crash, which is subject to vehicles' dynamic driving speed. For a safe following distance, the U.S. National Safety Council recommended 3-seconds rule for the dry and straight road situations.

- **Post-Encroachment-Time (PET)**

Post-Encroachment-Time (PET) is used as a surrogate measure for conflict severity. PET is the time difference between the time when the first vehicle leaves a potential point of conflict and the moment when the second vehicle subsequently arrives at the same point. The advantage of PET is that it considers both the speed and the acceleration of vehicles involved in conflicts. Small values of PET indicate high severity levels of the expected crashes *(61)*.

- **Deceleration rate difference**

The deceleration rate difference (DRD) is the difference between leading and following vehicles' deceleration rates. In certain cases, when the deceleration rate of the leading vehicle is not very high and the deceleration rate difference is within a reasonable range, the following vehicle probably does not need to take an aggressive braking maneuver *(62)*. It was found that the deceleration rate differences for all the non-crash vehicles were less than 15 ft/sec^2.

- **Deceleration Rate to Avoid Crash (DRAC)**

DRAC is another widely used surrogate measure. It was defined by Cooper and Ferguson *(63)* as the minimum deceleration rate required by the following vehicle to come to a timely stop (or match the leading vehicle's speed) and hence avoid a crash, which can be denoted as:

$$\mathrm{DRAC} = \frac{V_2 - V_1}{TTC}$$

DRAC is recognized as an effective measure of safety performance *(64)*. A higher value indicates a more dangerous car-following scenario. The above equation expresses the relationship between DRAC and TTC. In general, TTC is negatively related to DRAC. The American Association of State Highway and Transportation Officials suggests that a given vehicle is in conflict if its DRAC exceeds a threshold of 3.4 m/s^2 *(65)*. Archer *(66)* recommends a slightly lower threshold of 3.35 m/s^2 for most drivers.

3.3 Experiment Design

A driving simulator study was conducted to evaluate the safety impacts of the weaving distance between on-ramp/off-ramp and MLs entrance/exit. Also, the effects of the implementation of VSL technology were evaluated in this study. Considering all the factors need to be explored in this study, the experiment is firstly designed. Overall three parts were included in the experiment design including geometric design, traffic flow setting, and design of scenarios.

3.3.1 Geometric Design

The layout design is as shown in Figure 3.3. A segment with two four-lane GPLs and one two-lane TML are employed in this study. Meanwhile, two ramps are used as transition lanes (acceleration and deceleration lanes) to connect the GPL and the TML.

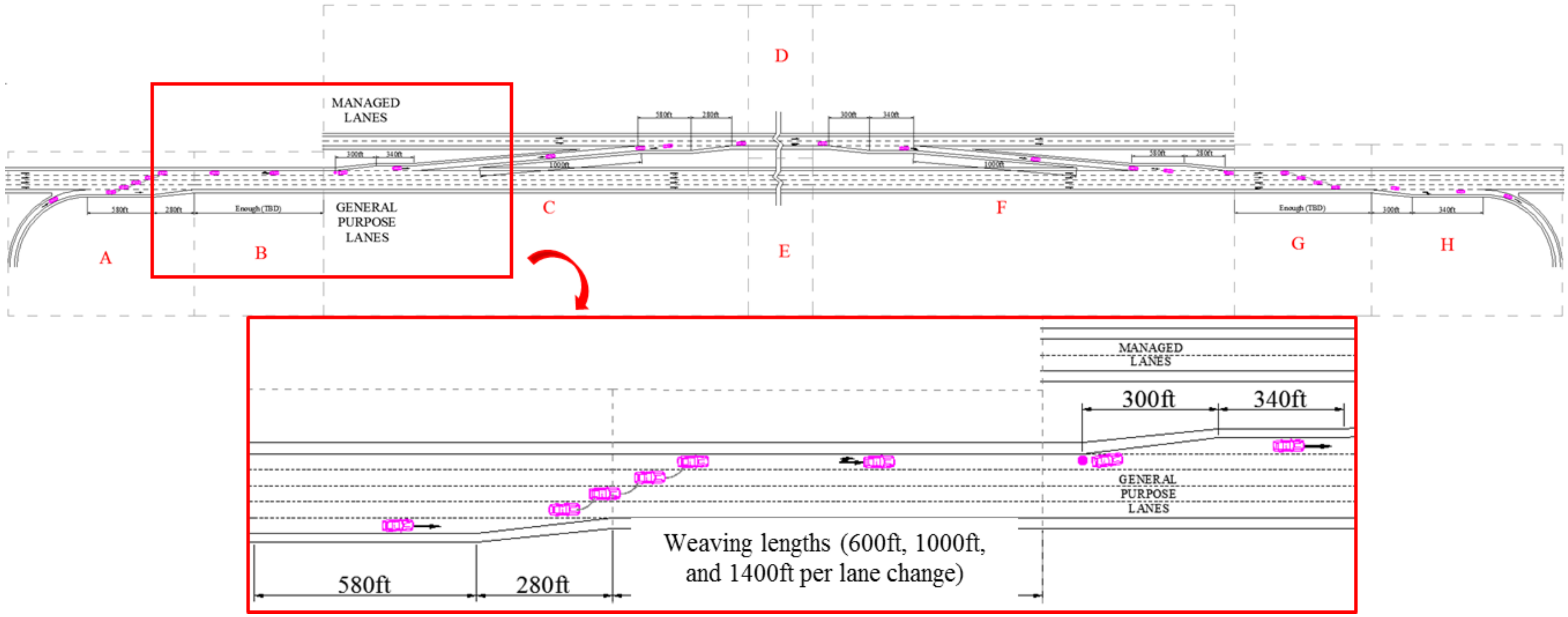


Figure 3.3 - Layout of the I-95 Study Area

All the lengths of acceleration lanes and deceleration lanes are determined by the design speed and the adjacent lane according to the standard of Florida Green book *(67)* (Table 3.4).

Table 3.4 - Designed Length of Acceleration Lanes and Deceleration Lanes

Type		Designed Length (ft)
Acceleration (40mph-60mph)	Total length	860
	Taper	280
Deceleration (60mph-40mph)	Total length	640
	Taper	300

As for the weaving distance per lane change, the recommended values are different across different states. For examples, in California, they recommended that the weaving distance per lane should be larger than 500 ft *(68)*. In New York, it was suggested that the weaving distance per lane should be larger than 500 ft and 1,000 ft is desired *(14)*. In Texas, different weaving distances per lane change were used for different traffic condition. For the serious condition, the weaving distance per lane should be larger than 950 ft and smaller than 1,200 ft *(69)*. Based on the above recommendations, we will try to investigate the safety impacts of three different weaving distances for each lane-change maneuver (i.e., 600 ft, 1,000 ft, and 1,400 ft).

3.3.2 Pavement Marking and Gantry Sign

The pavement marking and gantry sign employed in this experiment were developed based on the existing I-95 TML and guidelines from the MUTCD *(70)*.

(1) Pavement Marking

Figure 3.4 shows the marking sign (express only) at the entrance of TML. This marking sign specifies the distinct usage of TML.

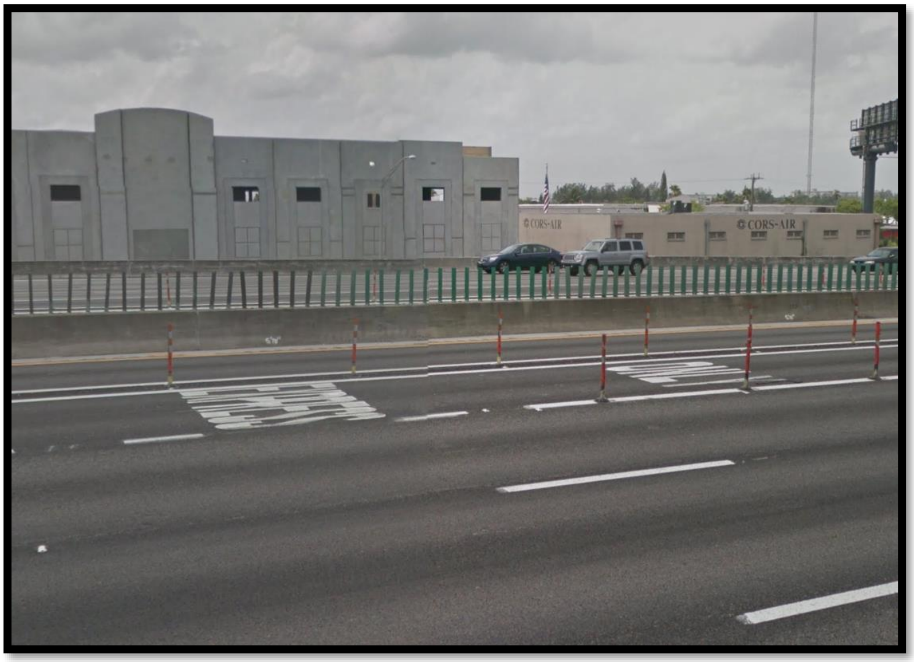


Figure 3.4 - Pavement Marking in the Entrance of TML (Source: Google Earth)

Figure 3.5 shows the pavement marking (merge) at the exit of TML. After this marking sign, drivers should merge to the GPLs.

Figure 3.5 - Pavement Marking in the Exit of TML (Source: Google Earth)

(2) Gantry Sign

Figure 3.6 shows the gantry sign at the ½ mile upstream of TML entrance that reminds drivers is the existence of a TML entrance at the ½ mile downstream.

Figure 3.6 - Gantry Sign in ½ Mile Upstream of the Entrance of TML

(Source: Google Earth)

Figure 3.7 shows the first gantry sign in front of the entrance, which reminds the drivers to change lanes to the inner-most lane and prepare for entrance into TML.

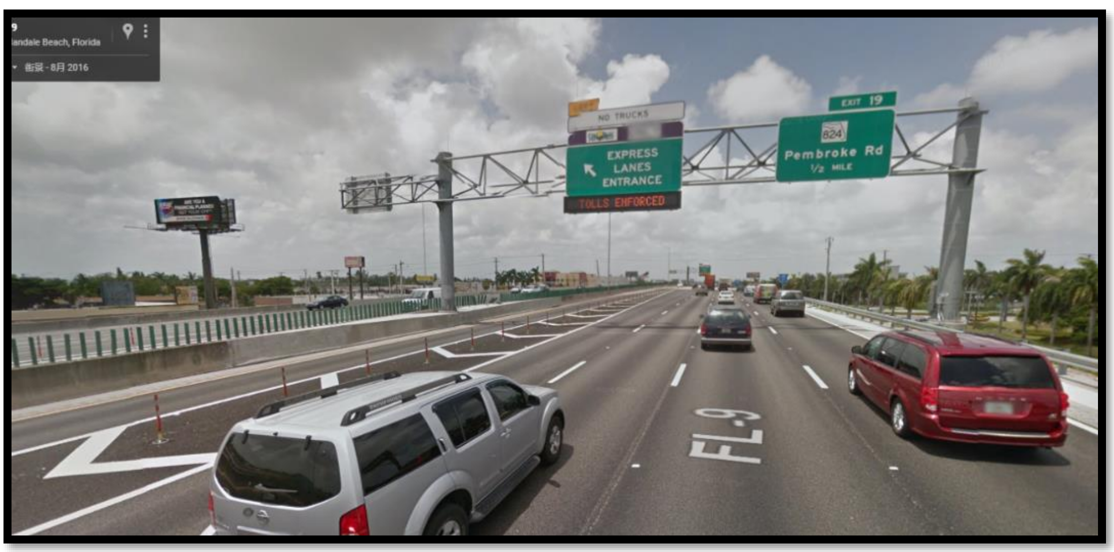


Figure 3.7 - Gantry Sign in the Entrance of TML (Source: Google Earth)

Figure 3.8 shows the second gantry sign in front of the entrance. The function of this gantry sign is the same as the pavement marking "Express Only".

Figure 3.8 - Half Gantry Sign in the Entrance of TML (Source: Google Earth)

Figure 3.9 shows the sign at the exit of TML, suggesting drivers to prepare to exit.

Figure 3.9 - Half Gantry Sign in the Exit of TML (Source: Google Earth)

3.3.3 Traffic Flow Setting

The 20-seconds radar based traffic data of April 6, 13, 20 and 27, 2016 (Wednesday) on I-95 for both MLs and GPLs were collected.

The traffic flow on GPLs during the period 7:00 AM to 9:00 AM was selected for the input of peak volume, and the period of 9:00 AM to 11:00 AM was selected for the input of off-peak volume. Meanwhile, the corresponding traffic flow on MLs was employed for the setting of traffic flow on MLs.

Based on the field data and the previous study about the impacts of VSL on traffic flow, the traffic conditions for VSL could be also determined (Table 3.5).

Table 3.5 - Parameters of Traffic Flow Setting

Lane Type	Without VSL				With VSL			
	Off-Peak		Peak		Off-Peak (60Mph)		Peak (50Mph)	
GPL	Average speed	Volume	Average speed	Volume	Average speed	Volume	Average speed	Volume
1	60	1385	53	1659	60	1511	50	1810
2	54	1372	47	1667	60	1497	50	1819
3	63	1384	56	1679	60	1510	50	1832
4	59	1115	52	1524	60	1200	50	1624
	Off-Peak		Peak		Off-Peak (60Mph)		Peak (60Mph)	
ML	Average speed	Volume	Average speed	Volume	Average speed	Volume	Average speed	Volume
1	65	1317	63	1478	65	1317	63	1478
2	62	1208	59	1522	62	1208	59	1522

3.3.4 Design of Scenarios

Generally, there are three types of scenario designs which have been commonly used by the researchers. The detailed definition of these methods is as shown in Table 3.6.

Table 3.6 - Summary of Different Scenario Design Methods

Scenario design	Number of factors (number of levels in each factor)	Number of scenarios for each subject	Description
Full Factorial Design	K (a)	a^K	-
Fractional (Partial) Factorial Design	K (a)	a^{K-I}	I is the number of main effects which have been confounded
Mixed Factorial Design	K (a)	a^{K-J}	J is the number of between group factors

(1) Full factorial design

A full factorial design considers all possible combinations of all factors levels. A full factorial design may also be called a fully crossed design. Such an experiment design allows the investigator to study the effect of each factor on the response variable as well as the effects of interactions between factors on the response variable *(71)*. However, such experiment design can have many experiment conditions and thus can be quite expensive and time-consuming.

(2) Fractional (Partial) Factorial Design

The fractional factorial design is an alternative that offers many of the advantages of a full factorial design with considerably fewer experimental conditions *(72)*. The fractional factorial design is a variation upon factorial design, involving the use of a carefully chosen subset of the experimental conditions of a complete factorial design. In other words, only certain conditions from the complete factorial are implemented *(73)*.

(3) Mixed Factorial Design

A mixed factorial design involves two or more independent variables, of which at least one is a within-subjects (repeated measures) factor and at least one is a between-groups factor. In the simplest case, there will be one between-groups factor and one within-subjects factor. A within-subjects design is an experiment in which the same group of subjects serves in more than one treatment. However, in a between-subjects design, the various experimental treatments are given to different groups of subjects. This kind of method has been used by several driving simulator studies *(74-77)*.

In this study, the mixed factorial design was employed in order to reduce the number of scenarios for each participant. The factors for this experiment included distance of lane change (three levels: 600 ft, 1,000 ft, and 1,400 ft), traffic volume (off-peak and peak), and VSL technology (Non-VSL and VSL) (Table 3.7).

Table 3.7 - Descriptions and Levels of the Two Factors

Factor	Description	Factor Levels
Length	Length of the weaving section (for continuous lane change)	1. 600 ft per lane change 2. 1,000 ft per lane change 3. 1,400 ft per lane change
Traffic Flow	Traffic flow condition	1. Off-peak 2. Peak
VSL	VSL implementation	1. Non-VSL 2. VSL

Note: The base length for lane change is based on the recommendations by (Kuhn et al., 2005);

The VSL will coordinate the traffic flow around the driver.

All the subjects will be randomly separated into three groups based on the three different distances and each participant will be randomly assigned to four scenarios (Off-peak with non-VSL, Off-peak with VSL, Peak with non-VSL, and Peak with VSL) to reduce the order effects (Figure 3.10).

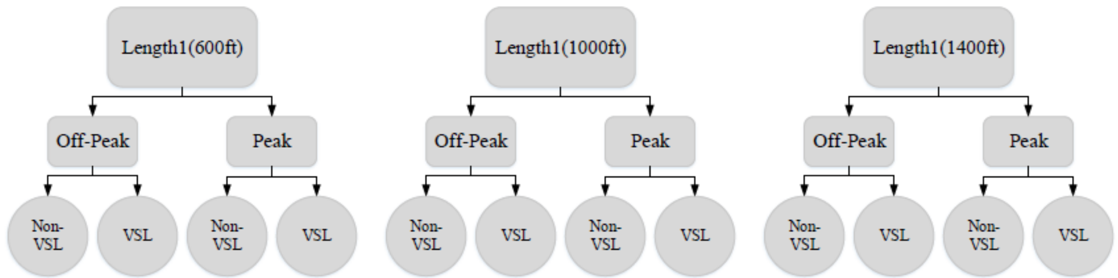


Figure 3.10 - Schematic Diagram of Experiment Design

3.4 Experiment Development

3.4.1 Scenarios Development

NADS MiniSim^{TM} driving simulator was employed in this study, and three software including Tile Mosaic Tool (TMT), Interactive Scenario Authoring Tools (ISAT), and MiniSim were used among the procedure of scenario development.

(1) Driving Simulator Equipment

This experiment is conducted with NADS MiniSim^{TM} at University of Central Florida (UCF), which is a highly flexible PC-based driving simulator system and designed for research, development, clinical and training applications (Figure 3.11).

Figure 3.11 - NADS MiniSim^{TM} at the UCF

(2) Simulation Software

At first, the Tile Mosaic Tool (TMT) was used to assemble the road network based on the established road tile files (Figure 3.12).

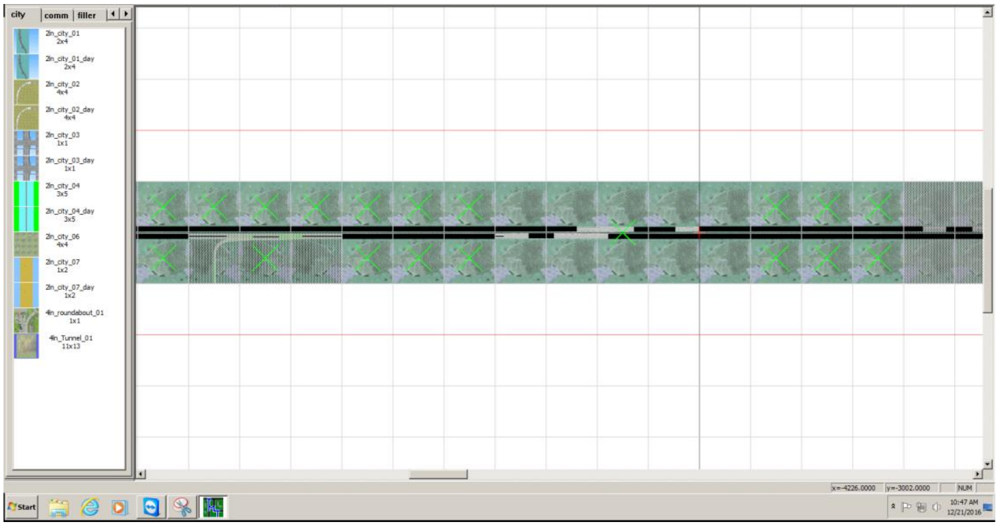


Figure 3.12 GUI of Tile Mosaic Tool (TMT)

Secondly, the Interactive Scenario Authoring Tools (ISAT) was used to implement the Triggers, vehicles and objects in the scenarios (Figure 3.13).

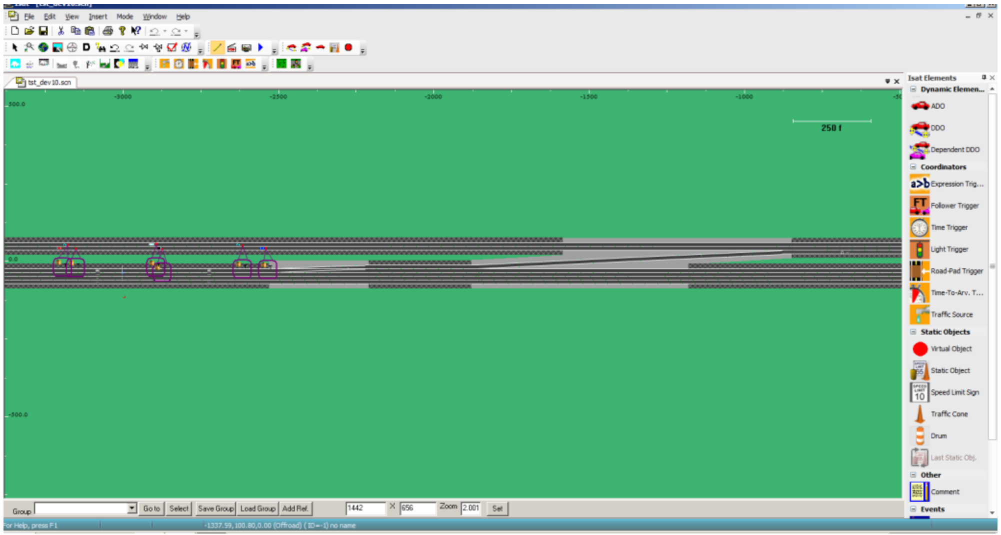


Figure 3.13 - GUI of Interactive Scenario Authoring Tools (ISAT)

After establishing the scenario, the experiment will be conducted by MiniSim^{TM}. It provides different setting for the data collection mode (Figure 3.14).

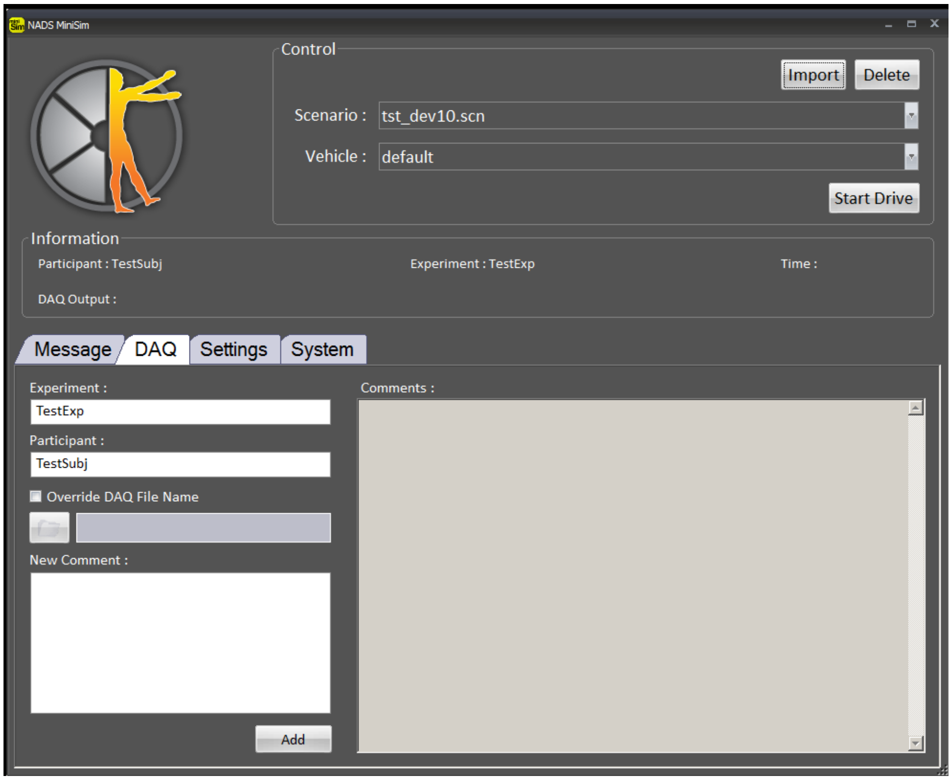


Figure 3.14 GUI of MiniSim^{TM}

As previously noted, 54 participants are needed to complete this study. The general criteria required participants to be in the age range of 18 to 65 with a valid driver's license and they must not have a history of motion sickness, which ensures the safety and comfort of the participants.

3.4.2 Participants

able 3.8 shows the statistical summary of the participants have been recruited. In total, 54 participants were recruited to meet the requirements of different gender and age group. However, there are 9 old participants (7 female and 2 male) suffered motion sickness when they were doing the experiment. Finally, the data of 45 participants were used for the analysis.

able 3.8 - Descriptive Statistics of Participants Recruitment

Participant Type	Gender	Age	Number of Participants Required	Number of Participants Recruited
1	F	YOUNG	9	9
2	F	MIDDLE	9	9
3	F	OLD	9	9 (7 sickness)
4	M	YOUNG	9	9
5	M	MIDDLE	9	9
6	M	OLD	9	9 (2 sickness)

3.4.3 Experiment Procedure

Upon arrival, each participant would be informed about the requirements of the experiment and asked to read and sign an informed consent form. Afterwards, they would be required to complete a questionnaire about the personal information (e.g. age, education, driving experiences etc.) before the experiment. The participants would be advised to drive as they normally do in real-life situations. Before the formal test, each participant would be asked to perform a practice driving of at least 5 minutes to make sure the participant becomes familiar with the driving simulator (with automatic transmission). In this practice session, the participants would exercise maneuvers including straight driving, acceleration, deceleration, lane changing, and other basic driving behaviors. In addition, participants would be notified that they could quit the experiment at any time in case of motion sickness or any kind of discomfort.

During the experiment, each participant would be randomly assigned to a group of scenarios with the same weaving length. For each group, the participant would be asked to drive in four scenarios with different traffic condition: off-peak, peak, off-peak with VSL and peak with VSL. Besides, in order to eliminate the experiment order effect, participants would be assigned to the four scenarios in a random sequence. After all the scenarios, the participants would be required to complete a questionnaire about all the feedbacks and suggestions.

3.5 Result Analysis

In each experiment, participants need to enter from GPLs to MLs and then exit from MLs to GPLs (see Figure 3.15). Since this study focuses on the safety impacts of varied Toll Lane Configuration (i.e. weaving length), only two parts of the trajectory were collected for the analysis, i.e., GPLs entrance, and GPLs exit.

The trajectory and speed data of the participant vehicle and other vehicles were recorded by the NADS MiniSim™ and then were processed by using MATLAB®. Based on the processed data, speed distribution, lane-change duration, and two surrogate safety measures were chosen as dependent variable to conduct repeated measures ANOVA for five independent variables (i.e., weaving length, volume, variable speed limit, gender, age).

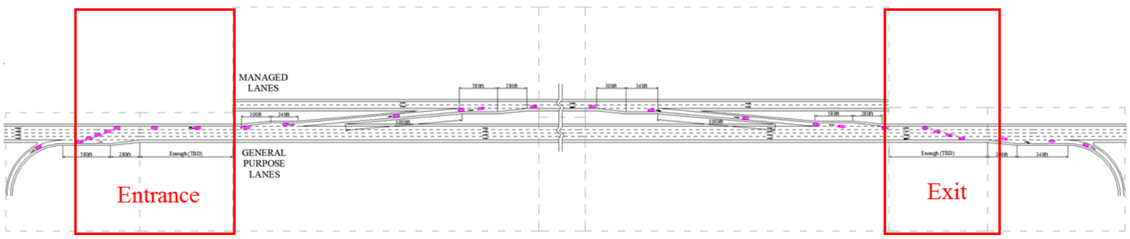


Figure 3.15 - Illustration of the Study Area

3.5.1 Average speed

A repeated measures ANOVA was carried out with weaving length (600 ft vs. 1,000 ft vs. 1,400 ft) as between subjects variable and gender (female vs. male), age (young vs. middle vs. old), volume (non-peak vs. peak) and VSL (non-VSL vs. VSL) as within subjects variables.

Table 3.9 - Results of Repeated Measures ANOVA (Average Speed)

Effect	Entrance		Exit	
	F Value	Pr > F	F Value	Pr > F
Weaving Length	F(2, 42)=2.02	0.145	F(2, 42)=0.01	0.986
Gender	F(1, 43)=0.07	0.786	F(1, 43)=0.18	0.674
Age	F(2, 42)=0.34	0.713	F(2, 42)=1.28	0.290
Volume	F(1, 44)=443.87	**<.0001**	F(1, 44)=318.3	**<.0001**
VSL	F(1, 44)=2.54	0.118	F(1, 44)=0.63	0.433

(1) Entrance

As presented in Table 3.9**Error! Reference source not found.**, the weaving length is insignificant at the 0.05 level, which means there is no significant difference in average speed between three types of weaving lengths. However, the post hoc test results in Table 3.10 indicate that the difference between 600 ft and 1,000 ft, is significant at the 0.1 level. The same result has been also observed for the difference between 600 ft and 1,400 ft.

Table 3.10 - Results of Post Hoc Test for Weaving Length (Entrance)

Weaving Lengths		Estimate	Standard Error	DF	t Value	Pr > |t|
600	**1000**	-2.0166	1.1937	42	-1.69	**0.0986**
600	**1400**	-2.0622	1.1545	42	-1.79	**0.0813**
1000	**1400**	-0.04563	1.1755	42	-0.04	0.9692

Figure 3.16 shows that the average speed of the scenario with the weaving length of 600 ft (mean: 50.51 mph) is much lower than 1,000 ft (mean: 52.52 mph) and 1,400 ft (mean: 52.57 mph), while the difference between 1,000 ft and 1,400 ft is very small.

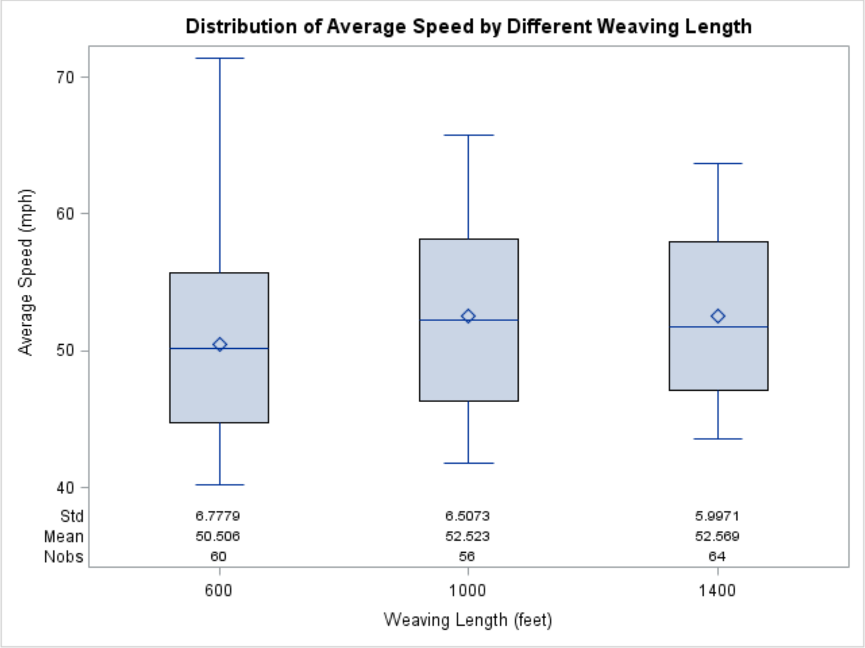


Figure 3.16 - Distribution of Average Speed by Different Weaving Length (Entrance)

Volume (F(1, 44)=443.87, p<0.0001) was found to have significant effect on average speed. The average speed (mean: 46.42 mph) of peak traffic condition is much lower than the off-peak (mean: 57.31 mph) traffic condition (Figure 3.17).

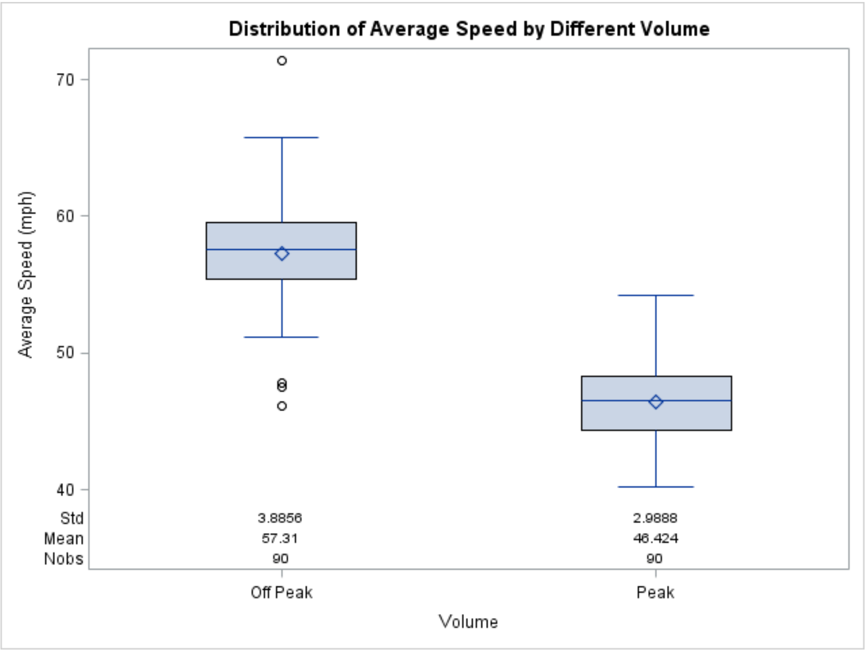


Figure 3.17 - Distribution of Average Speed by Different Volume (Entrance)

(2) Exit

As seen from Table 3.9, only volume is significant at the 0.05 level for the exit segment. Similar to the entrance segment, the average speed (mean: 51.55 mph) of peak traffic condition is much lower than the off-peak (mean: 59.06 mph) traffic condition (Figure 3.18).

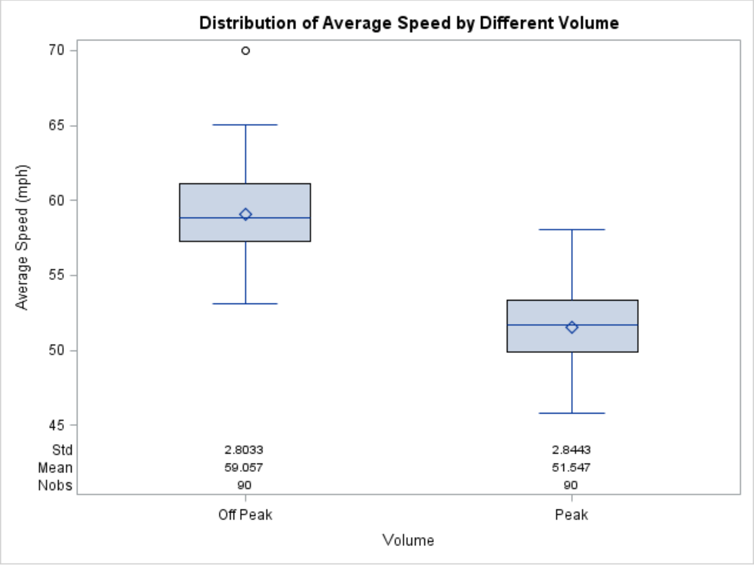


Figure 3.18 - Distribution of Average Speed by Different Volume (Exit)

3.5.2 Speed standard deviation

As presented in Table 3.11, age (F(2, 42)=3.09, p=0.056), volume (F(1, 44)=11.49, p=0.002) and VSL (F(1, 44)=3.50, p=0.068) were found to have significant effects on the speed standard deviation on the entrance segment, while only weaving length (F(2, 42)=4.62, p=0.015) was significant on the exit segment.

Table 3.11 - Results of Repeated Measures ANOVA (Speed Standard Deviation)

Effect	Entrance		Exit	
	F Value	Pr > F	F Value	Pr > F
Weaving Length	F(2, 42)=2	0.1482	F(2, 42)=4.62	**0.0153**
Gender	F(1, 43)=0.32	0.5744	F(1, 43)=1.51	0.2261
Age	F(2, 42)=3.09	**0.0561**	F(2, 42)=1.72	0.1907
Volume	F(1, 44)=11.49	**0.0015**	F(1, 44)=1.21	0.2782
VSL	F(1, 44)=3.5	**0.0682**	F(1, 44)=2.58	0.1151

(1) Entrance

To further investigate the effects of the different weaving lengths and age groups, the post hoc test was applied and the results are shown in Table 3.12 and 3.13.

Table 3.12 - Results of Post Hoc Test for Weaving Length (Entrance)

Weaving Lengths		Estimate	Standard Error	DF	t Value	Pr > |t|
600	**1000**	-0.00484	0.3031	42	-0.02	0.9873
600	**1400**	-0.5103	0.2932	42	-1.74	**0.0891**
1000	**1400**	-0.5054	0.2985	42	-1.69	**0.0978**

Table 3.13 - Results of Post Hoc Test for Age (Entrance)

Age Groups		Estimate	Standard Error	DF	t Value	Pr > |t|
Middle	**Old**	0.4940	0.3310	42	1.49	0.1431
Middle	**Young**	-0.3255	0.2703	42	-1.20	0.2352
Old	**Young**	-0.8195	0.3310	42	-2.48	**0.0174**

As seen from Figure 3.19, in the scenario with the weaving length of 1,400 ft, drivers tend to have higher speed standard deviation (mean: 4.65 mph) compared with the scenarios with the weaving length of 600 ft (mean: 4.14 mph) and 1,000 ft (mean: 4.15 mph).

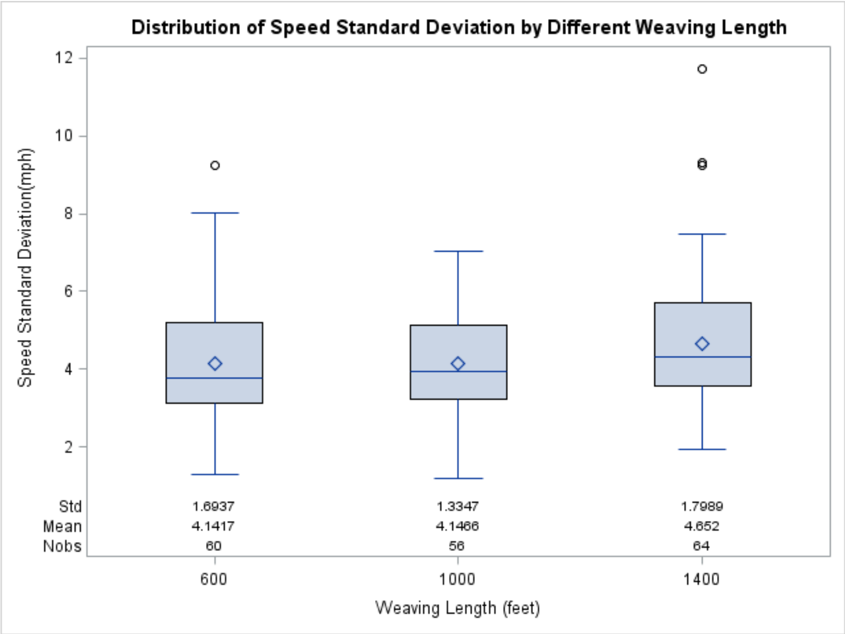


Figure 3.19 - Distribution of Speed Standard Deviation by Different Weaving Length (Entrance)

In terms of age group, Figure 3.20 implies that old drivers are more likely to have lower speed standard deviation (mean: 3.80 mph) while the young drivers prone to have higher speed standard deviation (mean: 4.62 mph). The result implies that young drivers are more aggressive and may increase the crash risk.

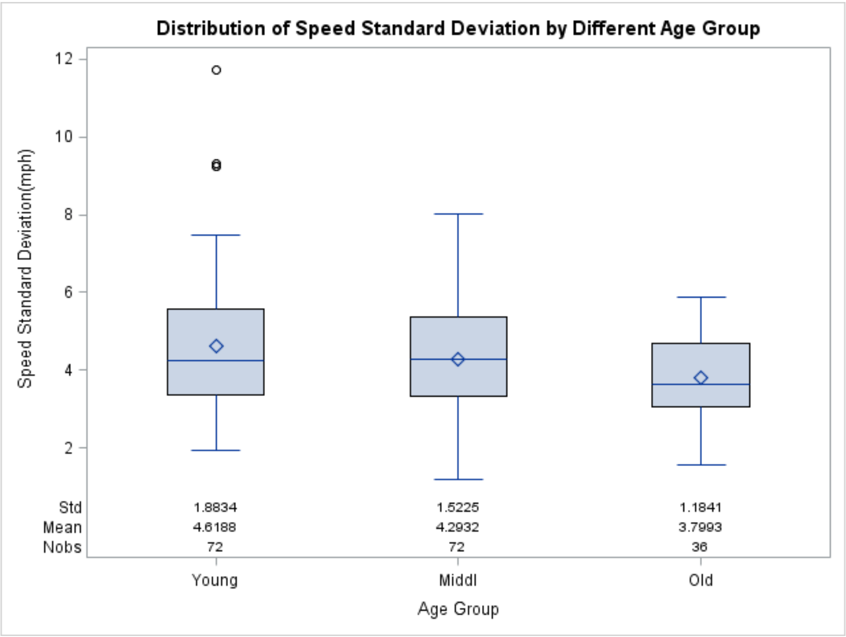


Figure 3.20 - Distribution of Speed Standard Deviation by Different Age Group (Entrance)

Comparing to the off-peak scenarios (mean: 3.92 mph), the peak scenarios tend to cause higher speed standard deviation (mean: 4.73 mph). Peak scenario is more complex than off-peak scenario, which may increase the frequency of acceleration and deceleration maneuvers (Figure 3.21).

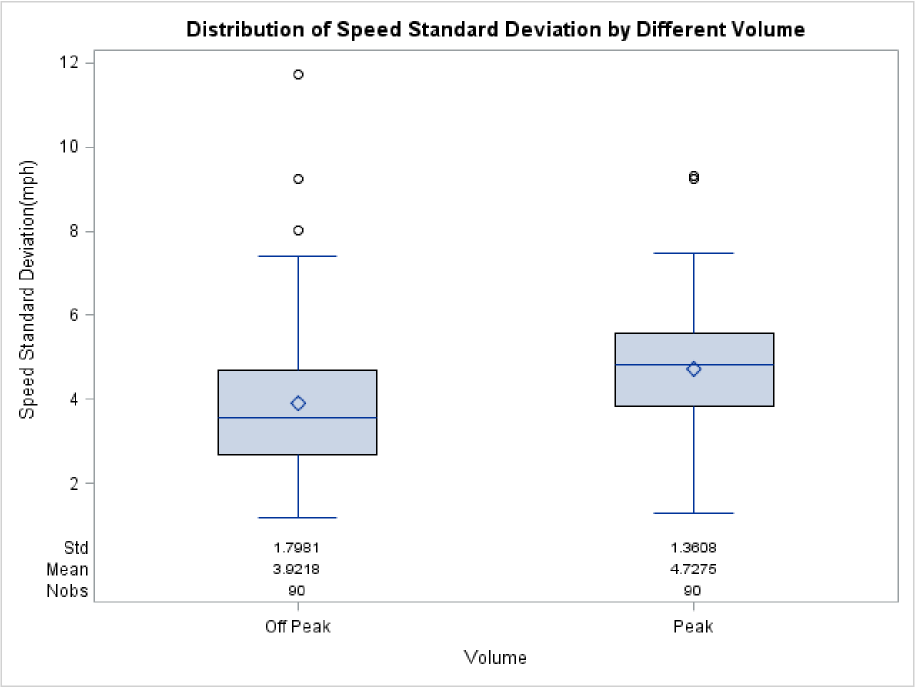


Figure 3.21 - Distribution of Speed Standard Deviation by Different Volume (Entrance)

The speed standard deviation of the scenarios having VSL (mean: 4.10 mph) is significantly lower than that of the scenarios without VSL (mean: 4.55 mph). This result indicates that the implementation of VSL strategy could effectively improve the traffic safety by harmonizing speed (Figure 3.22).

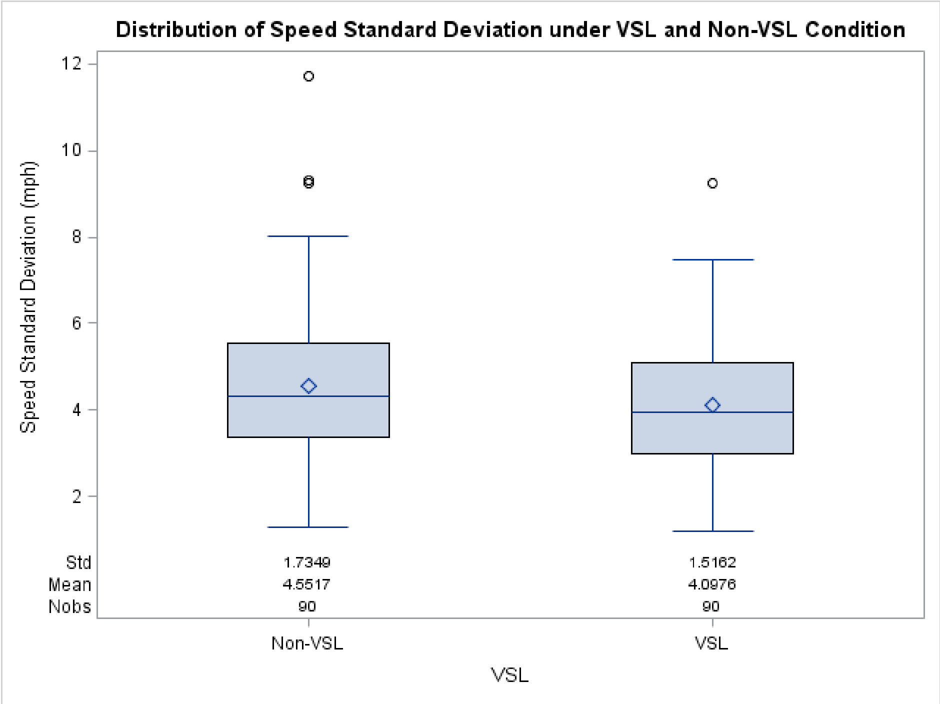


Figure 3.22 - Distribution of Speed Standard Deviation under VSL and Non-VSL Condition (Entrance)

(2) Exit

As shown in Table 3.11, weaving length ($F(2, 42)=4.62$, $p=0.0153$) was found to have significant effects on speed standard deviation. To investigate the significance of the difference between varied weaving lengths, the post hoc test was applied and the results are shown in Table 3.14.

Table 3.14 - Results of Post Hoc Test for Weaving Length (Exit)

Weaving Lengths		Estimate	Standard Error	DF	t Value	Pr > |t|
600	**1000**	0.03127	0.2673	42	0.12	0.9074
600	**1400**	-0.6653	0.2585	42	-2.57	**0.0137**
1000	**1400**	-0.6966	0.2632	42	-2.65	**0.0114**

As indicated from Figure 3.23, the speed standard deviation (mean: 3.84 mph) in the scenario with the weaving length of 1,400 ft is significantly higher than the scenarios with the weaving length of 600 ft (mean: 3.17 mph) and 1,000 ft (mean: 3.14 mph).

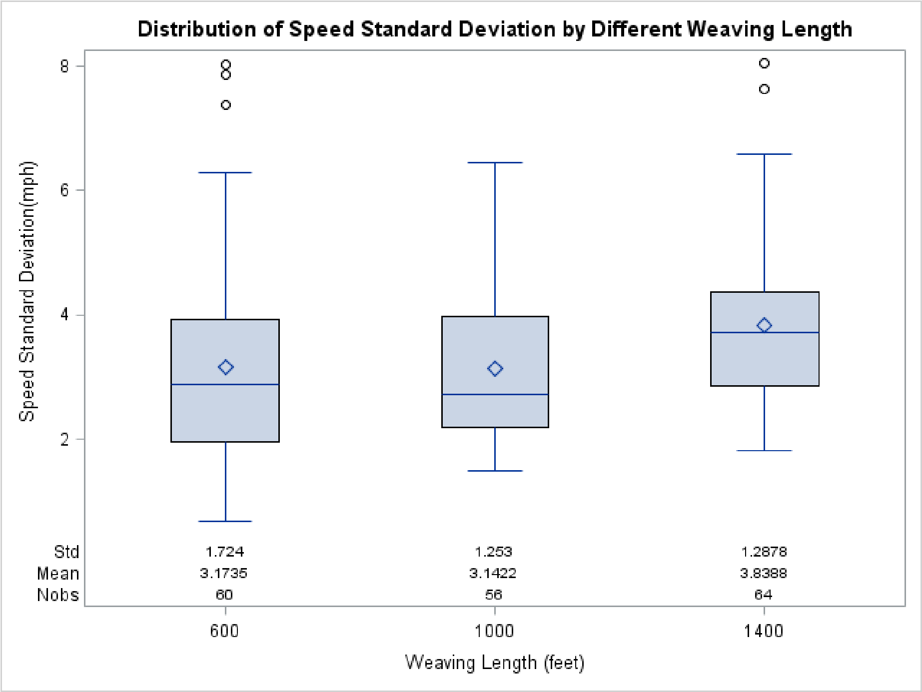


Figure 3.23 - Distribution of Speed Standard Deviation by Different Weaving Length (Exit)

3.5.3 Lane-change duration

As shown in Table 3.15, weaving length ($F(2, 42)=5.09$, $p=0.010$) and Lane Change ($F(2, 88)=12.88$, $p<0.0001$) were found to have significant effects on the lane-change duration in the entrance segment. However, for the exit segment, the significant variables are totally different. Gender ($F(1, 43)=17.06$, $p=0.0002$), age ($F(2, 42)=17.31$, $p<0.0001$), and lane change ($F(2, 88)=6.5$, $p=0.0023$) were found to have significant effects on the lane-change duration in the exit segment.

Table 3.15 - Results of Repeated Measures One-Way ANOVA (Lane-Change Duration)

Effect	Entrance		Exit	
	F Value	Pr > F	F Value	Pr > F
Weaving Length	F(2, 42)=5.09	**0.0104**	F(2, 42)=0.96	0.3906
Gender	F(1, 43)=1.65	0.2065	F(1, 43)=17.06	**0.0002**
Age	F(2, 42)=1.36	0.2675	F(2, 42)=17.31	**<.0001**
Volume	F(1, 44)=0.36	0.5503	F(1, 44)=0.54	0.4646
VSL	F(1, 44)=0.14	0.7103	F(1, 44)=0.4	0.5324
Lane Change	F(2, 88)=12.88	**<.0001**	F(2, 88)=6.5	**0.0023**

(1) Entrance

To further investigate the significance of the difference between varied weaving lengths, the post hoc test was applied and the results are shown in Table 3.16. Besides, the post hoc test was also conducted for lane-change behaviors between different lanes (

Table 3.17).

Table 3.16 - Results of Post Hoc Test for Weaving Length (Entrance)

Weaving Lengths		Estimate	Standard Error	DF	t Value	Pr > |t|
600	**1000**	-0.1971	0.1386	42	-1.42	0.1624
600	**1400**	-0.4270	0.1340	42	-3.19	**0.0027**
1000	**1400**	-0.2300	0.1365	42	-1.69	**0.0994**

Table 3.17 - Results of Post Hoc Test for Lane Change (Entrance)

Lane-Change Maneuvers	Estimate	Standard Error	DF	t Value	Pr > |t|

Lane-Change Maneuvers		Estimate	Standard Error	DF	t Value	Pr > |t|
Lane 1 to Lane 2	Lane 2 to Lane 3	0.2198	0.1343	88	1.64	0.1052
Lane 1 to Lane 2	Lane 3 to Lane 4	0.6685	0.1343	88	4.98	<.0001
Lane 2 to Lane 3	Lane 3 to Lane 4	0.4487	0.1343	88	3.34	0.0012

Note: Lane 1 is the outer lane; Lane 2 is the middle lane close to the outer lane; Lane 3 is the middle lane close to the inner lane; and Lane 4 is the inner lane.

As seen from Table 3.16 and Figure 3.24, the lane change duration (mean: 2.55s) in the scenario with the weaving length of 1400 feet is significantly higher than the scenarios with the weaving length of 600 feet (mean: 2.12s).

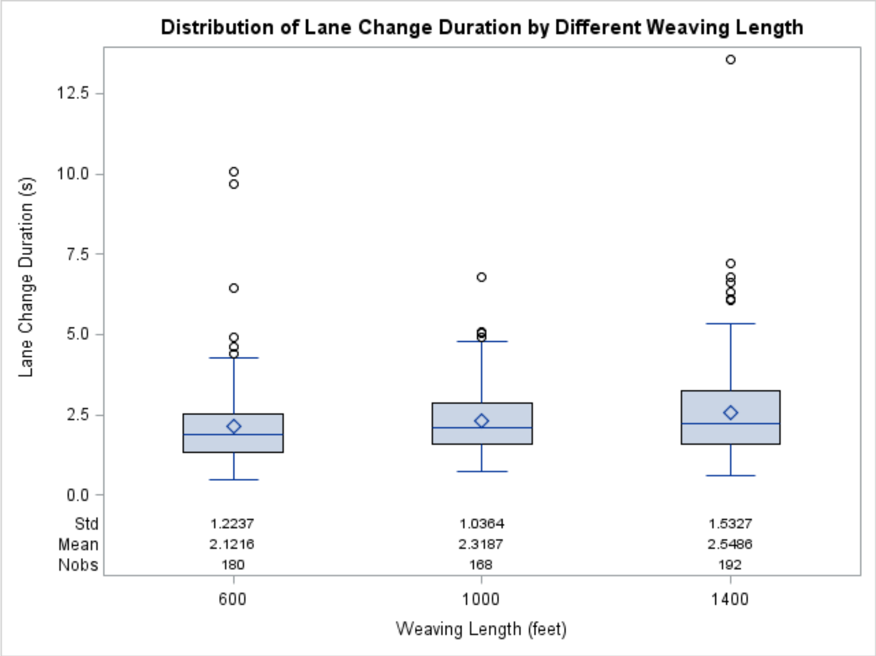


Figure 3.24 - Distribution of Lane-Change Duration by Different Weaving Length (Entrance)

As presented in

Table 3.17 and Figure 3.25, the lane-change duration (mean: 1.96s) from lane 3 to lane 4 is significantly lower than that from lane 1 to lane 2 (mean: 2.63s) and lane 2 to lane 3 (mean: 2.41s).

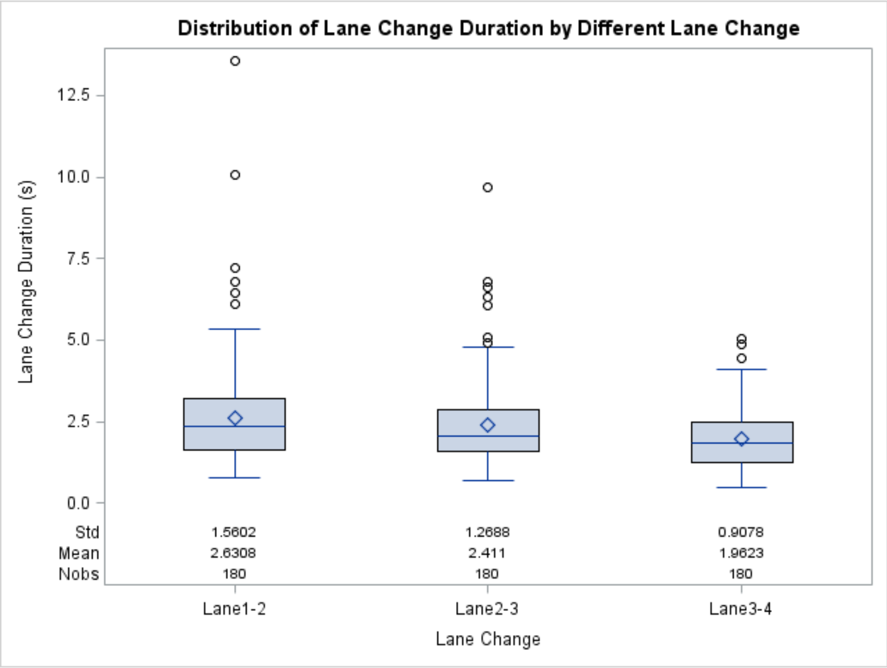


Figure 3.25 - Distribution of Lane-Change Duration by Different Lane Change (Entrance)

(2) Exit

Tables 3.18 and 3.19 summarized the results of post hoc test of lane-change durations between different age groups and different lanes.

Table 3.18 - Results of Post Hoc Test for Age Group (Exit)

Age Groups		Estimate	Standard Error	DF	t Value	Pr > |t|
Middle	**Old**	-0.6727	0.1273	42	-5.28	**<.0001**
Middle	**Young**	0.02176	0.1040	42	0.21	0.8353
Old	**Young**	0.6944	0.1273	42	5.45	**<.0001**

Table 3.19 - Results of Post Hoc Test for Lane Change (Exit)

Lane Change Maneuvers		Estimate	Standard Error	DF	t Value	Pr > |t|
Lane 1 to Lane 2	**Lane 2 to Lane 3**	-0.1953	0.1161	88	-1.68	**0.0962**
Lane 1 to Lane 2	**Lane 3 to Lane 4**	-0.4183	0.1161	88	-3.60	**0.0005**
Lane 2 to Lane 3	**Lane 3 to Lane 4**	-0.2231	0.1161	88	-1.92	**0.0580**

Figure 3.26 shows that the lane-change duration of male drivers (mean: 2.24s) is significantly higher than the female drivers (mean: 1.85s).

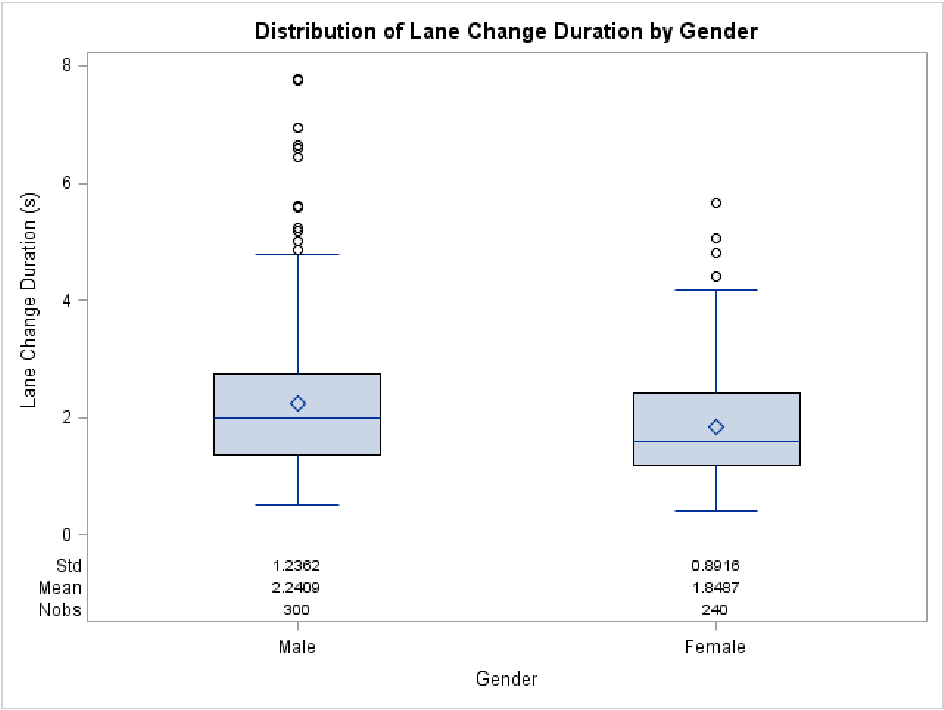


Figure 3.26 - Distribution of Lane-Change Duration by Gender (Exit)

As seen from Figure 3.27, together with the post hoc test table, it can be concluded that the lane-change duration of old drivers (mean: 2.61s) is significantly higher than the young (mean: 1.92s) and middle age drivers (mean: 1.95s).

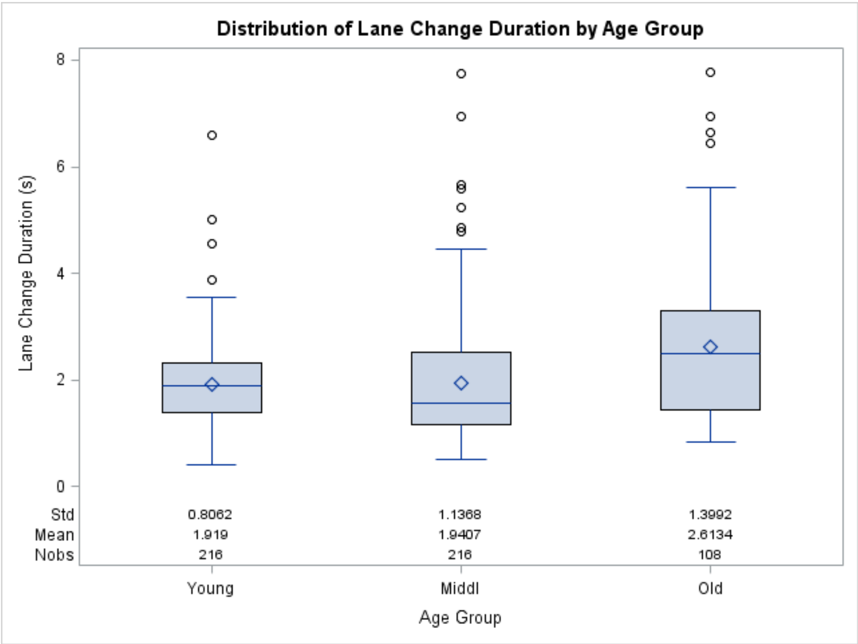


Figure 3.27 - Distribution of Lane-Change Duration by Age Group (Exit)

As shown in Figure 3.28, it can be concluded that the lane-change duration from lane 1 to lane 2 (mean: 1.86s) is significantly lower than lane 2 to lane 3 (mean: 2.06s), and the lane-change duration from lane 2 to lane 3 (mean: 2.06s) is significantly lower than lane 3 to lane 4 (mean: 2.28s).

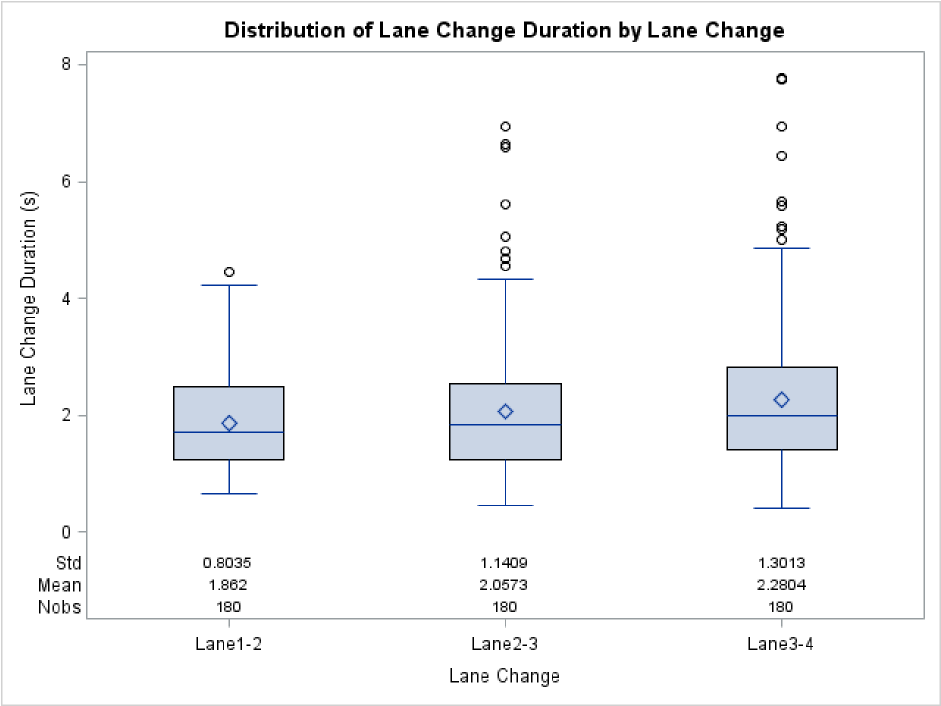


Figure 3.28 - Distribution of Lane-Change Duration by Lane Change (Exit)

3.5.4 Minimum TTC

(1) Entrance

Both one-way and two-way repeated measures ANOVA were conducted for three between-subjects factors (weaving length, gender, and age) and two within-subjects factors (volume and VSL), the results are as shown in

Table 3.20.

Table 3.20 - Results of Repeated Measures ANOVA (Entrance)

Effect	Num DF	Den DF	F Value	Pr > F
Weaving Length	2	39	0.33	0.7177
Gender	1	40	4.04	**0.0513**
Age	2	39	0.45	0.6405
Volume	1	21	0.02	0.8990
VSL	1	28	2.74	0.1092
Weaving Length*Gender	2	36	4.60	**0.0166**
Weaving Length* Age	4	33	0.28	0.8860
Weaving Length* Volume	2	19	0.05	0.9484
Weaving Length* VSL	2	26	1.01	0.3792
Gender* Age	2	36	1.59	0.2187
Gender* Volume	1	20	0.79	0.3857
Gender* VSL	1	27	0.01	0.9187
Age* Volume	2	19	0.12	0.8874
Age* VSL	2	26	0.23	0.7969

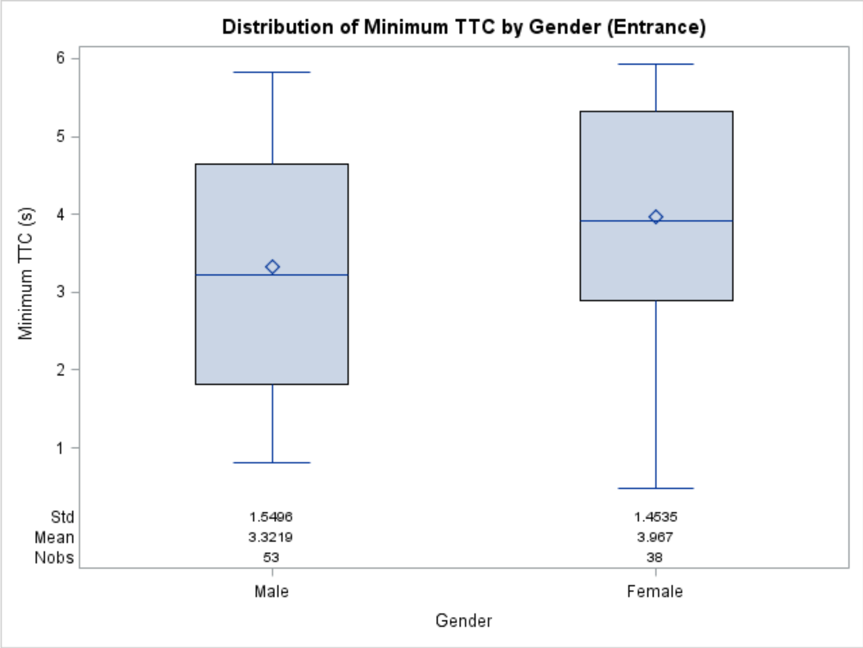


Figure 3.29 - Distribution of Minimum TTC by Gender (Entrance)

As seen from Figure 3.29, the minimum TTC of female driver (mean: 3.97s) is significantly higher than the male driver (mean: 3.32s). This indicates that male drivers are more aggressive and they are more likely to be involved in dangerous situation when they are driving on the entrance weaving segment.

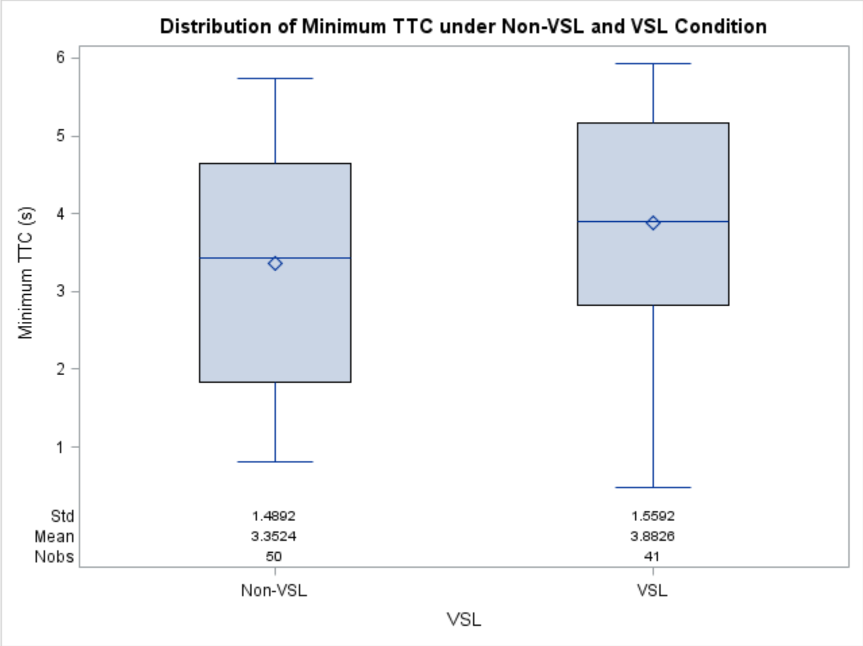


Figure 3.30 - Distribution of Minimum TTC under Non-VSL and VSL Condition (Entrance)

The p-value of VSL strategy is 0.1092, which indicates that there is a weakly significant difference of Minimum TTC between non-VSL and VSL condition. As shown in Figure 5-16, the scenarios having VSL strategies (3.88s) could have higher mean value of minimum TTC compared with the scenarios without VSL strategies (3.35s), suggesting that the implementation of VSL could improve traffic safety. To break down the significant two-way interaction (weaving length*gender), two separated repeated measures ANOVA were carried out for female drivers and male drivers.

Female driver

As presented in Table 3.21, the weaving length (F(2, 15)=4.54, p=0.029) was found to have significant effects on the minimum TTC.

Table 3.21 - Results of Repeated Measures ANOVA (Entrance-Female)

Type 3 Tests of Fixed Effects				
Effect	**Num DF**	**Den DF**	**F Value**	**Pr > F**
Weaving Length	2	15	4.54	**0.0288**
Age	2	15	1.80	0.1994
Volume	1	9	0.30	0.5995
VSL	1	11	0.98	0.3436

Table 3.22 - Results of Post Hoc Test for Weaving Length (Entrance-Female)

Weaving Lengths		Estimate	Standard Error	DF	t Value	Pr > |t|
600	**1000**	-0.4852	0.5456	15	-0.89	0.3878
600	**1400**	1.0260	0.5366	15	1.91	**0.0751**
1000	**1400**	1.5113	0.5129	15	2.95	**0.0100**

Table 3.23 - Results of Post Hoc Test for Age Group (Entrance-Female)

Age Groups		Estimate	Standard Error	DF	t Value	Pr > |t|
Middle	**Old**	1.3319	0.7023	15	1.90	**0.0773**

Age Groups		Estimate	Standard Error	DF	t Value	Pr > |t|
Middle	Young	0.4045	0.5122	15	0.79	0.4420
Old	Young	-0.9275	0.6664	15	-1.39	0.1843

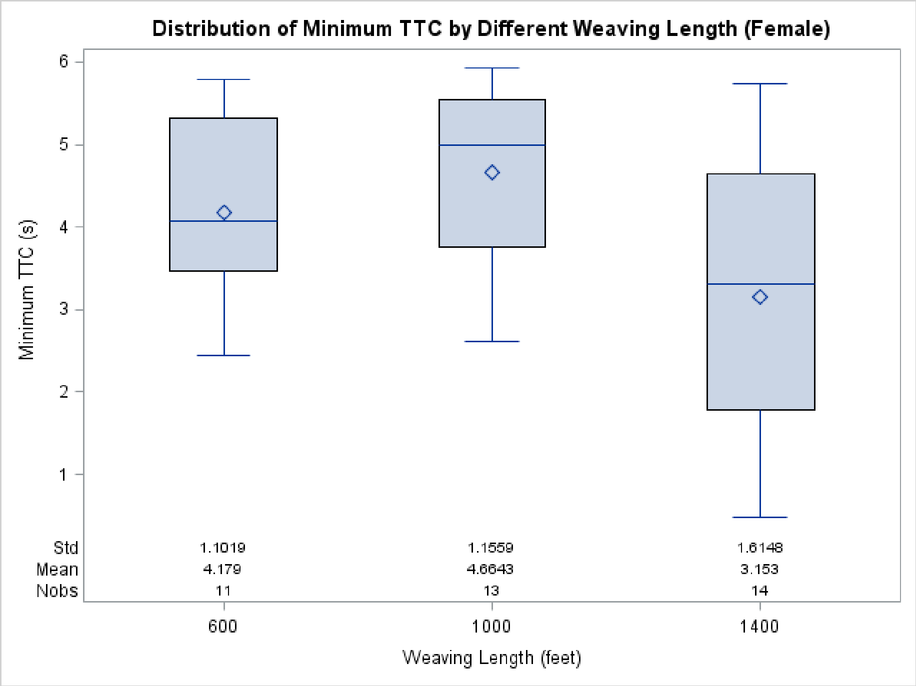


Figure 3.31 - Distribution of Minimum TTC by Different Weaving Length (Entrance-Female)

Based on the results of post hoc test and Figure 3.31, it can be concluded that the minimum TTC in the scenario with the weaving length of 1,400 ft (mean: 3.15s) is significantly lower than the scenario with the weaving length of 600 ft (mean: 4.18s) and 1,000 ft (mean: 4.66s) for female drivers. In addition, the minimum TTC in the scenario with the weaving length of 1,000 ft is the highest among three types of weaving length, even though the difference between 600 ft and 1,000 ft is insignificant. Above all, it can be concluded that the weaving length of 1,000 ft is the safest weaving length for female driver.

As shown in Table 3.23, the difference in minimum TTC between middle age and old drivers is significant at the 0.1 level. Based on the boxplot of different age group, we can reach the conclusion that the minimum TTC of old drivers is significantly lower than the middle age drivers, which implies that old drivers are more likely to be involved in dangerous situations. This might be explained as that old drivers always need more reaction time to take action to avoid potential conflicts (Figure 3.32).

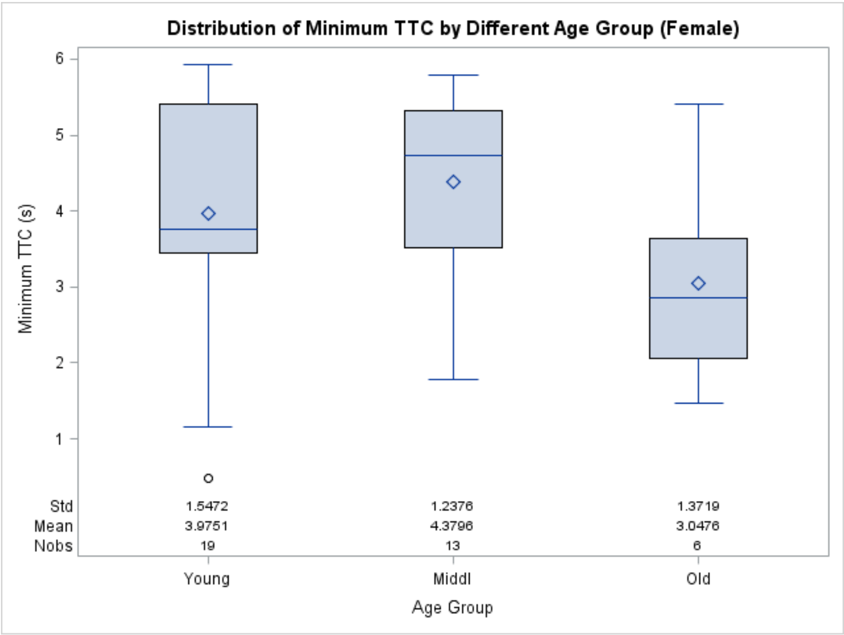


Figure 3.32 - Distribution of Minimum TTC by Different Age Group (Entrance-Female)

Male driver

As presented in Table 3.24, all the factors for minimum TTC are insignificant.

Table 3.24 - Results of Repeated Measures ANOVA (Entrance-Male)

Type 3 Tests of Fixed Effects				
Effect	**Num DF**	**Den DF**	**F Value**	**Pr > F**
Weaving Length	2	21	1.12	0.3461
Age	2	21	0.29	0.7504
Volume	1	11	0.53	0.4813
VSL	1	16	1.56	0.2294

(2) Exit

As shown from Table 3.25, there is no significant relationship between these factors and minimum TTC in the exit segment.

Table 3.25 - Results of Repeated Measures ANOVA (Exit)

Type 3 Tests of Fixed Effects

Effect	Num DF	Den DF	F Value	Pr > F
Weaving Length	2	30	0.58	0.5648
Gender	1	31	0.30	0.5895
Age	2	30	0.68	0.5153
Volume	1	16	0.02	0.8973
VSL	1	15	0.50	0.4893

3.5.5 Number of conflicts (TTC<3s)

Table 3.26 presents the statistical summary of conflict frequency for different factors. Based on the statistical results of conflict frequency, several conclusions could be drawn for each factor. For weaving length, 1,000 ft has the lowest conflicts, followed by 600 ft. The weaving length of 1400 ft was shown to have the highest frequency of conflicts, this might be explained in that when the weaving length becomes more sufficient, drivers maybe more relaxed at the beginning of changing lanes while they may need to change lane urgently when they are approaching the entrance or exit. Besides, drivers would be more likely to get involved in hazardous situation with longer driving distance. For genders, male drivers are more likely to be involved into dangerous situations than female drivers, which is consistent with previous studies that male driver is more aggressive than female driver. As to the age groups, the number of conflicts increases when the age of driver decreases, which indicate that the young drivers are the most aggressive drivers when they change lanes. This finding about the young drivers is consistent with the well-known fact that young drivers prone to be involved in crashes due to the lack of driving experience.

In terms of traffic volume, the conflict frequency of peak scenario is much higher than the off-peak scenario. Moreover, the scenarios with the implementation of VSL strategy are found to have fewer conflicts than the scenarios with Non-VSL. For the segment type, the entrance segments are more likely to have traffic conflict than the exit segment. In order to figure out whether there exists difference between different lane-change maneuver, three lane-change maneuver (i.e. lane 1 to lane 2, lane 2 to lane 3, and lane 3 to lane 4) were compared. The results indicate that the lane-change maneuver which is more close to the entrance or exit of MLs tend to have more traffic conflicts.

Table 3.26 - Statistical Summary of Conflict Frequency by Different Factors

Variables	Frequency	Percent	Cumulative Frequency	Cumulative Percent
Weaving Lengths				
600	21	31.34	21	31.34

Variables	Frequency	Percent	Cumulative Frequency	Cumulative Percent
1000	17	25.37	38	56.72
1400	**29**	43.28	67	100.00
Gender				
Female	20	29.85	20	29.85
Male	**47**	70.15	67	100.00
Age				
Young	**29**	43.28	67	100.00
Middle	25	37.31	25	37.31
Old	13	19.40	38	56.72
Volume				
Off Peak	19	28.36	19	28.36
Peak	**48**	71.64	67	100.00
VSL				
Non-VSL	**39**	58.21	39	58.21
VSL	28	41.79	67	100.00
Segment Type				
Entrance	**40**	59.70	40	59.70
Exit	27	40.30	67	100.00
Lane Change				
Lane 1 to Lane 2	18	26.87	18	26.87
Lane 2 to Lane 3	23	34.33	41	61.19
Lane 3 to Lane 4	**26**	38.81	67	100.00

Note: Lane 1 indicates the shoulder lane; Lane 2 and 3 indicate the middle lane; Lane 4 indicates the inner-most lane.

The cross table of conflict frequency in terms of the interaction between weaving length and gender indicates that the female drivers tend to have much more traffic conflicts in the scenario with the weaving length of 1,400 ft. However, the male drivers didn't show any significant difference among all the weaving lengths (Table 3.27).

Table 3.27 - Cross Tabulation of Conflict Frequency by Weaving Length*Gender

Table of Weaving Length by Gender

Weaving Length	Gender		
	Female	Male	Total
600	4	**17**	21
1,000	3	14	17
1,400	**13**	16	29
Total	20	47	67

As to the interaction between weaving length and age, Table 3.28 shows the cross tabulation of conflict frequency between different weaving lengths and age groups. The statistical results indicate that old drivers tend to have much more conflicts in the scenario with the weaving length of 1,400 ft. However, for the other age group drivers, there seems have no huge difference between different weaving lengths.

Table 3.28 - Statistical Summary of Conflict Frequency by Weaving Length*Age Group

	Table of Weaving Length by Age			
Weaving Length	Age			
	Young	Middle	Old	Total
600	**11**	8	2	21
1,000	9	6	2	17
1,400	9	**11**	**9**	29
Total	29	25	13	67

In terms of the interaction between segment type and lane-change maneuver, Table 3.29 shows that: in the entrance segment, traffic conflicts are more likely to occur during the lane change from lane 3 to lane 4 than the other lane change maneuvers. However, in the exit segment, traffic conflicts are more likely to occur during the lane change from lane 2 to lane 3 than the other lane-change maneuvers.

Table 3.29 - Cross Tabulation of Conflict Frequency by Segment Type*Lane Change

Table of Part by Lane Change				
Part	Lane Change			
	Lane 1 to Lane 2	Lane 2 to Lane 3	Lane 3 to Lane 4	Total

Table of Part by Lane Change				
Part	**Lane Change**			
	Lane 1 to Lane 2	**Lane 2 to Lane 3**	**Lane 3 to Lane 4**	**Total**
Entrance	12	8	**20**	40
Exit	6	**15**	6	27
Total	18	23	26	67

3.6 Conclusions

The driving simulator approach was adopted at the UCF as a part of the second phase, which aimed at suggesting the optimal weaving length by considering traffic safety at the weaving section between the general purpose lane and toll lane. Also, the effects of the implementation of variable speed limit (VSL) strategy were evaluated in this study. Three different weaving lengths per each lane change were considered: 600 ft, 1,000 ft, and 1,400 ft. Besides, two types of traffic conditions with/without the VSL strategy were included in the experiment. Totally twelve scenarios were developed and fifty-four participants were recruited in this experiment.

Two weaving zones, toll lane entrance and exit zones were considered as two potential dangerous zones since drivers need to change lanes to merge into/out of toll lanes. Drivers' speed controlling and lane-changing maneuvers were analyzed and used for the evaluation of weaving length and VSL operation strategy. Repeated measures ANOVA and post hoc test were adopted for the analysis.

It was found that drivers were prone to have a higher speed if the weaving length is shorter (600 ft) as compared with 1,000 ft and 1,400 ft at the entrance zone. The participants in the scenario with weaving length per lane of 600 ft would drive around 2 mph faster than in the scenarios with weaving length per lane of 1,000 ft and 1,400 ft, whereas no significant difference has been found between scenarios with weaving length of 1,000 ft and 1,400 ft weaving length. The result indicates that drivers may become more anxious when the weaving length is shorter. Meanwhile, the speeds at exit zone were similar for the three different weaving lengths. Larger speed standard deviation was found for the scenario with the weaving length of 1,400 ft at both entrance and exit zones. Further, drivers' lane-changing duration became longer in the scenario with weaving length of 1,400 ft.

When the VSL strategy was implemented, drivers would drive at a lower speed and have a lower speed standard deviation at the entrance zone while the effects of VSL on the speed control at the exit zone were not significant. Besides, drivers' lane-changing duration would not change when the VSL was used.

The safety measures, time to collision (TTC) and number of collisions, were employed to evaluate the safety performance when drivers change lanes under different conditions. The scenario with weaving length of 1,400 ft would have smaller TTC compared with the scenarios with other two weaving lengths, which indicated that more potential dangerous situations could occur for the weaving length of 1,400 ft. Besides, less potential conflicts were observed in the scenarios with weaving length 1,000 ft. When the VSL strategy was implemented, longer TTC and less potential conflicts could be observed.

Considering the results of drivers' speed controlling and lane-changing maneuvers, it was recommended that 1,000 ft would be the optimal weaving length for lane change since drivers could have best driving performance in the scenarios with the weaving length of 1,000 ft. Besides, the experiment results further validated the usefulness of implementation of VSL. Hence, the VSL strategy should be adopted when close to the entrance and exit of toll lane.

4 Summary and Conclusions

Managed lanes have emerged as a dynamic traffic management strategy, which has efficiently improved traffic mobility and enhanced traffic safety, in addition to generating revenue for transportation agencies. The current study contributed to suggest the optimal geometric design for a popular type of ML, which converting the existing facilities to access the ML. The most effective accessibility level and the optimal weaving length between the general purpose lane and the ML were investigated through two simulation approaches: microscopic simulation and driving simulator.

The microscopic simulation study had two major study objectives: first, determining the optimal accessibility level to maximize system-wide efficiency; second, suggesting the optimal weaving length for vehicles to enter and exit from the ML based on the traffic flow characteristics. VISSIM microscopic simulation were developed based on a nine-mile network of a ML segment on the interstate (I-95) in South Florida. Three accessibility levels with one, two and three ingresses and egresses were tested. For each accessibility level, five different weaving lengths (600 feet, 800 feet, 1,000 feet, 1,400 feet, and 2,000 feet) under two traffic flow conditions (peak and off-peak) were included in the experiment. The experiment results suggested the Level 1 condition which had one ingress and one egress could have a higher speed, a low delay, and a greater time efficiency compared with other accessibility levels. In addition, the 1,000 feet weaving length could provide comparatively less conflict rates for different accessibility levels and traffic flow conditions. Beside the major study intentions, the monetary benefits based on different accessibility levels were also evaluated for transportation agencies. It was suggested that the highest revenue could be obtained if two access zones were implemented in the studied network.

The driving simulator experiment also had two major study tasks: first, suggesting the optimal weaving length at the weaving segment between the general purpose lane and the ML by considering different driving behaviors; second, evaluating the effectiveness of variable speed limit (VSL) in enhancing the traffic safety on the weaving segment. Totally twelve scenarios based on a 3× 2 × 2 mixed factor experiment design with weaving length (600 ft, 1,000 ft, and 1,400 ft) as a within-subject variable and traffic flow (peak and off-peak) and VSL strategy (without VSL strategy and with VSL strategy) as between-subject variables. The experiment results indicated that drivers would drive faster to merge into the ML under the 600 feet condition, which indicated that drivers could become more anxious with the limited length. Besides, larger speed standard deviation and smaller time to collision (TTC) could be found at both entrance and exit zones under 1,400 feet condition, which suggested drivers could become too relaxed with the excess length. In addition, less potential conflicts were observed in the scenarios with weaving length 1,000 feet. Further, the result suggested that, when the VSL strategy was implemented, drivers would drive at a lower speed, have a lower speed standard deviation, and experience longer TTC and less potential conflicts. It implies that the VSL strategy could effectively enhance traffic safety at the weaving segment between the general purpose lane and the ML.

In summary, based on the results from the microscopic simulation study and the driving simulator experiment, it is suggested 1,000 feet as the optimal length for lane change at the weaving segment. It is interesting that both microsimulation and driving simulator experiment approaches drew a consistent conclusion. In addition, one accessibility level is the safest option in the nine-mile network based on the microscopic simulation result. Further, variable speed limit (VSL) control is recommended to enhance traffic safety for the weaving segment between the general purpose lane and ML based on the driving simulator experiment result.

The current research could be also extended to further improve the efficiency and safety for the ML in the future. First, the direct and slip ramps have been used to connect the ramp directly to MLs without generating weaving segments. Additional study efforts need to be made to explore the effectiveness of such design. Second, the operation and safety benefits of connected vehicles and autonomous vehicles along with the ML should be also investigated.

References

1. Perez, B. G., C. Fuhs, C. Gants, R. Giordano, and D. H. Ungemah (2012). Priced managed lane guide (No. FHWA-HOP-13-007).
2. ATKINS (2017). *The Road Less Traveled*. Retrieved May 9, 2017 from

   http://www.atkinsglobal.com/en-GB/angles/all-angles/the-road-less-travelled.
3. Cambridge Systematics, Inc (2014). I-95 Managed Lanes Monitoring Report.
4. FDOT (2012). 95 Express Phase 1 Fiscal Year 2012 Annual UPA Evaluation Report. File Code: 424 Doc ID#21796.
5. Kuhn, B (2010). Efficient use of highway capacity summary.
6. The Federal Highway Administration (FHWA) (2017): Freeway Management Program. Retrieved May 20, 2017 from

   https://ops.fhwa.dot.gov/freewaymgmt/managed_lanes.htm.
7. Ely, J (2013), HNTB: Priced Managed Lanes in America.
8. Abuzwidah, M., and M. Abdel-Aty (2017). Effects of Using High Occupancy Vehicle Lanes on Safety Performance of Freeways (No. 17-06894).
9. Cho, Y., R. Goel, P. Gupta, G. Bogonko, and M. W. Burris (2011). What Are I-394 HOT Lane Drivers Paying for? *In Transportation Research Board 90th Annual Meeting* (No. 11-0496).
10. Fitzpatrick, K., M. A. Brewer, S. Chrysler, N. Wood, B. Kuhn, G. Goodin, C. Fuhs, D. Ungemah, B. Perez, and V. Dewey (2017). *Guidelines for Implementing Managed Lanes.* (No. Project 15-49).
11. Jang, K., S. Kang, J. Seo, and C.-Y. Chan (2012). Cross-section designs for the safety performance of buffer-separated high-occupancy vehicle lanes. *Journal of Transportation Engineering,* Vol. 139, No. 3, pp. 247-254.
12. Srinivasan, S., P. Haas, P. Alluri, A. Gan, and J. Bonneson (2015). Crash Prediction Method for Freeway Facilities with High Occupancy Vehicle (HOV) and High Occupancy Toll (HOT) Lanes.
13. Schroeder, B., S. Aghdashi, N. Rouphail, X. Liu, and Y. Wang (2012). Deterministic approach to managed lane analysis on freeways in context of Highway Capacity Manual. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2286, pp. 122-132.
14. Fuhs, C. A (1990). High-occupancy vehicle facilities: A planning, design, and operation manual.
15. California Department of Transportation (Caltrans) (2011): Updated Managed Lane Design: Traffic Operations Policy Directive 11-02. Caltrans. Sacramento, CA.

16. Jang, K., K. Chung, D. Ragland, and C.-Y. Chan (2009). Safety performance of high-occupancy-vehicle facilities: Evaluation of HOV lane configurations in California. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2099, pp. 132-140.

17. Gomes, G., A. May, and R. Horowitz (2004). Congested freeway microsimulation model using VISSIM. *Transportation Research Record: Journal of the Transportation Research Board*, No. 1876, pp. 71-81.

18. Park, B., and H. Qi (2005). Development and Evaluation of a Procedure for the Calibration of Simulation Models. *Transportation Research Record: Journal of the Transportation Research Board*, No. 1934, pp. 208-217.

19. Gettman, D., L. Pu, T. Sayed, and S. G. Shelby (2008). *Surrogate safety assessment model and validation: Final report* (No. FHWA-HRT-08-051).

20. Shahdah, U., F. Saccomanno, and B. Persaud (2004). Integrated traffic conflict model for estimating crash modification factors. *Accident Analysis & Prevention,* Vol. 71, pp. 228-235.

21. Burgess, C (2006). HOT lane buffer and mid-point access design review report. *Washington State Department of Transportation, Olympia, WA*.

22. Venglar, S., D. Fenno, S. Goel, and P. Schrader (2002). Managed Lanes-Traffic Modeling.In.

23. Federal Highway Administration (FHWA) (2017): Tolling and Pricing Program. Retrieved May 29, 2017 from http://ops.fhwa.dot.gov/publications/fhwahop13007/pmlg6_0.htm.

24. Florida Department of Transportation (FDOT) (2002): Project traffic forecasting handbook.

25. US Census American Community Survey's (ACS) for Miami-Dade (2015).

26. Joseph, R (2013). Managed Lanes Case Studies-A companion to the preliminary Investigation-Impacts of Increasing Vehicle-Occupancy Requirements on HOV/HOT Lanes- Caltrans Division of Research and Innovation.

27. PTV (2015). PTV VISSIM 7 User Manual. Karlsruhe, Germany: PTV AG.

28. Johnson, S., and D. Murray (2010). Empirical analysis of truck and automobile speeds on rural interstates: Impact of posted speed limits.In *Transportation Research Board 89th Annual Meeting* (No. 10-0833).

29. Jin, X., M. S. Hossan, and H. Asgari (2015). *Investigating the Value of Time and Value of Reliability for Managed Lanes*.

30. Park, B., and J. Schneeberger (2003). Microscopic Simulation Model Calibration and validation: case study of VISSIM simulation model for a coordinated actuated Signal system. *Transportation Research Record: Journal of the Transportation Research Board*, No. 1856, pp. 185-192.

31. Dowling, R., A. Skabardonis, and V. Alexiadis (2004). Traffic analysis toolbox volume III: guidelines for applying traffic microsimulation modeling software.

32. Yu, R., and M. Abdel-Aty (2014). An optimal variable speed limits system to ameliorate traffic safety risk. *Transportation research part C: Emerging Technologies,* Vol. 46, pp. 235-246.

33. Nezamuddin, N., N. Jiang, T. Zhang, S. T. Waller, and D. Sun (2011). *Traffic operations and safety benefits of active traffic strategies on TXDOT freeways*. Report: FHWA/TX-12/0-6576-1.

34. National Research Council. (2010). *HCM 2010: Highway Capacity Manual*. Washington, DC: Transportation Research Board. Manual,

35. Wang, L., M. Abdel-Aty, and J. Lee (2017). Implementation of Active Traffic Management Strategies for Safety of a Congested Expressway Weaving Segment. *Transportation Research Board 96th Annual MeetingTransportation Research Board*, No. 17-00248 (10.3141/2635-04).

36. Intelligent Transportation Systems (ITS) (2017): *Aeris operational scenarios and applications eco-lanes* . Retrieved May 10, 2017 from https://www.its.dot.gov/research_archives/aeris/pdf/AERIS_Operational_Scenarios020216.pdf

37. Schroeder, B., S. Aghdashi, N. Rouphail, X. Liu and Y. Wang. Deterministic Approach to Managed Lane Analysis on Freeways in Context Ofhighway Capacity Manual. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2286, 2012, pp. 122-132.

38. Obesberger, J. Managed Lanes: Combining Access Control, Vehicle Eligibility, and 12 Pricing Strategies Can Help Mitigate Congestion and Improve Mobility on the Nation’s 13 Busiest Roadways. *Journal of Public Roads*, Vol. 14, 2004, pp. 15.

39. Liu, X., B. Schroeder, T. Thomson, Y. Wang, N. Rouphail and Y. Yin. Analysis of Operational Interactions between Freeway Managed Lanes and Parallel, General Purpose Lanes. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2262, 2011, pp. 62-73.

40. Liu, X., Y. Wang, B. Schroeder and N. Rouphail. Quantifying Cross-Weave Impact on Capacity Reduction for Freeway Facilities with Managed Lanes. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2278, 2012, pp. 171-179.

41. Liu, X., G. Zhang, Y. Lao and Y. Wang. Modeling Traffic Flow Dynamics on Managed Lane Facility: Approach Based on Cell Transmission Model. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2278, 2012, pp. 163-170.

42. Liu, X. C., Y. Wang, B. J. Schroeder and N. M. Rouphail. An Analytical Framework for Managed Lane Facility Performance Evaluation. *ITE Journal*, Vol. 82, No. 10, 2012, pp.

43. Thomson, T., X. Liu, Y. Wang, B. Schroeder and N. Rouphail. Operational Performance and Speed-Flow Relationships for Basic Managed Lane Segments. *Transportation*

Research Record: Journal of the Transportation Research Board, No. 2286, 2012, pp. 94-104.

44. Golob, T. F., W. W. Recker and D. W. Levine. Safety of High-Occupancy Vehicle Lanes without Physical Separation. *Journal of Transportation Engineering*, Vol. 115, No. 6, 1989, pp. 591-607.

45. Lee, J.-T., R. Dittberner and H. Sripathi. Safety Impacts of Freeway Managed-Lane Strategy: Inside Lane for High-Occupancy Vehicle Use and Right Shoulder Lane as Travel Lane During Peak Periods. *Transportation Research Record: Journal of the Transportation Research Board*, 2008, pp.

46. Cooner, S. and S. Ranft. Safety Evaluation of Buffer-Separated High-Occupancy Vehicle Lanes in Texas. *Transportation Research Record: Journal of the Transportation Research Board*, No. 1959, 2006, pp. 168-177.

47. Shi, Q. and M. Abdel-Aty. Big Data Applications in Real-Time Traffic Operation and Safety Monitoring and Improvement on Urban Expressways. *Transportation Research Part C: Emerging Technologies*, Vol. 58, 2015, pp. 380-394.

48. Xu, C., P. Liu, W. Wang and Z. Li. Identification of Freeway Crash-Prone Traffic Conditions for Traffic Flow at Different Levels of Service. *Transportation Research Part A: Policy and Practice*, Vol. 69, 2014, pp. 58-70.

49. Yeo, H., K. Jang, A. Skabardonis and S. Kang. Impact of Traffic States on Freeway Crash Involvement Rates. *Accident Analysis & Prevention*, Vol. 50, 2013, pp. 713-723.

50. Xu, C., W. Wang and P. Liu. Identifying Crash-Prone Traffic Conditions under Different Weather on Freeways. *J Safety Res*, Vol. 46, 2013, pp. 135-144.

51. Xu, C., A. P. Tarko, W. Wang and P. Liu. Predicting Crash Likelihood and Severity on Freeways with Real-Time Loop Detector Data. *Accid Anal Prev*, Vol. 57, 2013, pp. 30-39.

52. Xu, C., P. Liu, W. Wang and Z. Li. Evaluation of the Impacts of Traffic States on Crash Risks on Freeways. *Accid Anal Prev*, Vol. 47, 2012, pp. 162-171.

53. Aust, M. L., J. Engström and M. Viström. Effects of Forward Collision Warning and Repeated Event Exposure on Emergency Braking. *Transportation Research Part F: Traffic Psychology and Behaviour*, Vol. 18, 2013, pp. 34-46.

54. Bella, F. and G. D'Agostini. Combined Effect of Traffic and Geometrics on Rear-End Collision Risk: Driving Simulator Study. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2165, 2010, pp. 96-103.

55. van Driel, C. J., M. Hoedemaeker and B. van Arem. Impacts of a Congestion Assistant on Driving Behaviour and Acceptance Using a Driving Simulator. *Transportation Research Part F: Traffic Psychology and Behaviour*, Vol. 10, No. 2, 2007, pp. 139-152.

56. Hayward, J. C. Near Miss Determination through Use of a Scale of Danger. 1972, pp.

57. Yang, H. and K. Ozbay. Estimation of Traffic Conflict Risk for Merging Vehicles on Highway Merge Section. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2236, 2011, pp. 58-65.

58. Weng, J. and Q. Meng. Rear-End Crash Potential Estimation in the Work Zone Merging Areas. *Journal of Advanced Transportation*, Vol. 48, No. 3, 2014, pp. 238-249.

59. Sun, C., Z. Qing, P. Edara, B. Balakrishnan and J. Hopfenblatt. Driving Simulator Study of J-Turn Acceleration–Deceleration Lane and U-Turn Spacing Configurations. *Transportation Research Record: Journal of the Transportation Research Board*, No. 2638, 2017, pp. 26-34.

60. Minderhoud, M. M. and P. H. Bovy. Extended Time-to-Collision Measures for Road Traffic Safety Assessment. *Accident Analysis & Prevention*, Vol. 33, No. 1, 2001, pp. 89-97.

61. Habtemichael, F. G. and L. de Picado Santos. Crash Risk Evaluation of Aggressive Driving on Motorways: Microscopic Traffic Simulation Approach. *Transportation Research Part F: Traffic Psychology and Behaviour*, Vol. 23, 2014, pp. 101-112.

62. Park, B., Y. Chen and J. Hourdos. Opportunities for Preventing Rear-End Crashes: Findings from the Analysis of Actual Freeway Crash Data. *Journal of Transportation Safety & Security*, Vol. 3, No. 2, 2011, pp. 95-107.

63. Cooper, D. F. and N. Ferguson. Traffic Studies at T-Junctions. 2. A Conflict Simulation Record. *Traffic Engineering & Control*, Vol. 17, No. Analytic, 1976, pp.

64. Meng, Q. and J. Weng. Evaluation of Rear-End Crash Risk at Work Zone Using Work Zone Traffic Data. *Accident Analysis & Prevention*, Vol. 43, No. 4, 2011, pp. 1291-1300.

65. AASHTO, *Guide for High Occupancy Vehicle Facilities*. Washington, D.C., 2004.

66. Archer, J. Methods for the Assessment and Prediction of Traffic Safety at Urban Intersections and Their Application in Micro-Simulation Modelling. *Royal Institute of Technology*, 2004, pp.

67. FDOT. *Manual of Uniform Minimum Standards for Design, Construction and Maintenance for Streets and Highways*. Florida, 2013.

68. Operations, C. D. o. T. *High Occupancy Vehicle Guidelines for Planning, Design and Operations*. Department of Transportation, 2003.

69. Venglar, S., D. Fenno, S. Goel and P. Schrader *Managed Lanes: Traffic Modeling*. the Institute, 2001.

70. FHWA. *MUTCD*. 2009.

71. Box, G. E., J. S. Hunter and W. G. Hunter *Statistics for Experimenters: Design, Innovation, and Discovery*. Wiley-Interscience New York, 2005.

72. Wu, C. J. and M. S. Hamada *Experiments: Planning, Analysis, and Optimization*. John Wiley & Sons, 2011.

73. Dziak, J. J., I. Nahum-Shani and L. M. Collins. Multilevel Factorial Experiments for Developing Behavioral Interventions: Power, Sample Size, and Resource Considerations. *Psychological methods*, Vol. 17, No. 2, 2012, pp. 153.

74. Boyle, L. N. and F. Mannering. Impact of Traveler Advisory Systems on Driving Speed: Some New Evidence. *Transportation Research Part C: Emerging Technologies*, Vol. 12, No. 1, 2004, pp. 57-72.

75. Cummings, M. L., R. M. Kilgore, E. Wang, L. Tijerina and D. S. Kochhar. Effects of Single Versus Multiple Warnings on Driver Performance. *Human factors*, Vol. 49, No. 6, 2007, pp. 1097-1106.

76. Strand, N., J. Nilsson, I. M. Karlsson and L. Nilsson. Semi-Automated Versus Highly Automated Driving in Critical Situations Caused by Automation Failures. *Transportation Research Part F: Traffic Psychology and Behaviour*, Vol. 27, 2014, pp. 218-228.

77. Wikman, A.-S., T. Nieminen and H. Summala. Driving Experience and Time-Sharing During in-Car Tasks on Roads of Different Width. *Ergonomics*, Vol. 41, No. 3, 1998, pp. 358-372.