# Safety, Operational, and Design Analyses of Managed Toll and Connected Vehicles' Lanes

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# SAFETY, OPERATIONAL AND DESIGN ANALYSES OF MANAGED TOLL AND CONNECTED VEHICLES' LANES

by

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# ABSTRACT

Managed lanes (MLs) have been implemented as a vital strategy for traffic management and traffic safety improvement. The majority of previous studies involving MLs have explored a limited scope of the impact of the MLs segments as a whole, without considering the safety and operational effects of the access design. Also, there are limited studies that investigated the effect of connected vehicles (CVs) on managed lanes. Hence, this study has two main objectives: (1) the first objective is achieved by determining the optimal managed lanes access design, including accessibility level and weaving distance for an at-grade access design. (2) the second objective is to study the effects of applying CVs and CV lanes on the MLs network. Several scenarios were tested using microscopic traffic simulation to determine the optimal access design while taking into consideration accessibility levels and weaving lengths. Both safety (e.g., standard deviation of speed, time-to-collision, and conflict rate) and operational (e.g., level of service, average speed, average delay) performance measures were included in the analyses. For the first objective, the results suggested that one accessibility level is the optimal option for the 9-mile network. A weaving length between 1,000 feet to 1,400 feet per lane change was suggested based on the safety analysis. From the operational perspective, a weaving length between 1,000 feet and 2,000 feet per lane change was recommended. The findings also suggested that MPR% between 10% and 30% was recommended when the CVs are only allowed in MLs. When increasing the number of MLs, the MPR% could be improved to reach 70%. Lastly, the findings proposed that MPR% of 100% could be achieved by allowing the CVs to use all the lanes in the network.

## EXTENDED ABSTRACT

On freeways, Managed Lanes (MLs) have emerged as an effective dynamic traffic management strategy. MLs have been successfully implemented as an important facility in improving traffic mobility and in generating revenue for transportation agencies. This study has two main objectives. First, the optimal managed lanes access design including accessibility level and weaving distance for an at-grade access design (I-95, South Florida) are determined. Second, the effect of applying connected vehicles on the safety and the operation of the network is explored.

The first goal focuses on studying the effect of access design on the safety and the operation of the MLs. The primary research task of this objective is to use microsimulation to maximize the system-wide efficiency, by determining the optimal accessibility level in conjunction with sufficient length and locations of weaving segments near access zones. 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 segment in its entirety without considering accessibility levels and weaving distance of the access design. In the following study, several scenarios were tested using microscopic traffic simulation to determine the optimal access design while also taking into consideration accessibility levels and weaving lengths. The studied accessibility levels varied from one to three along the studied network. In order to achieve the study's objective, a 9-mile corridor of general purpose lanes (GPLs) and MLs in Miami-Dade County, Florida were replicated in a microsimulation environment in terms of traffic data, geometric design, and driving behavior (i.e., car following, lane changing). Several safety

measures of effectiveness (i.e., speed standard deviation, time-to-collision, and conflict rate) and operational performance measures (i.e., level of service, average speed, average delay) were analyzed using statistical models.

The safety analysis of the access design in MLs was successfully demonstrated. The findings of this study have several important implications for future practice and policy. It is recommended that both access control level and weaving configuration should be taken into account when designing the access openings of MLs for expressways. The study provides recommendations to the transportation agencies for improving the mobility and the efficiency of the MLs. One of the most prominent findings from this study was that the conflict rate on MLs were 48% and 11% lower than that of GPLs in the peak and the off-peak periods, respectively. After comparing the surrogate safety measures between MLs and GPLs, it was found that MLs were safer than GPLs since it had higher time-to-collision, higher post-encroachment-time, and lower maximum deceleration. A log-linear model was developed for investigating the safest access zone design that would minimize traffic conflicts. Analysis of conflicts proposed that one accessibility level is the safest option in a 9-mile corridor. Additionally, it was found that a length of 1,000 ft per lane change is indeed the optimal length for the weaving segments. Furthermore, from the findings of this study, a weaving length of 600 ft per lane change is not recommended near the access zones of the MLs. Additionally, Tobit models were developed for investigating the factors that affect the safety measures. ANOVA and level of service (LOS) calculations were also used to evaluate traffic operation. Tobit models were able to be successfully developed to investigate the optimal MLs access zone design. Analysis of safety measures (i.e., conflict rate, speed SD, and TTC) proposed that a weaving length between 1,000 feet and 1,400 feet per lane change should be considered. Moreover, the operational measurements were investigated, which included the LOS, average speed, and average delay. The results of the operational measures confirmed several findings from the safety results. One access zone was found as the optimal level, with better LOS, higher speed, and less delay. The results of the average speed, average delay, and LOS proposed a weaving length between 1,000 feet and 2,000 feet per lane change for a more efficiently operated network. Lastly, it was found that the off-peak periods had better safety and operational performance (e.g., lower conflict rate, less delay) compared to the peak periods. For future studies, more attention should be allotted to the peak conditions.

The second goal of this study focused on investigating the effect of applying connected vehicles (CVs) and connected vehicle lanes on the safety and the operation of the network. Also, this objective sought to determine the optimal market penetration rate (MPR%) of the CVs by investigating various configurations of CVs and CV lanes in MLs environment. A comparison between the different cases of MLs designs with the presence of CVs with different market penetrations was generated for different traffic conditions. Similar to the first objective, the 9-mile corridor located on Interstate 95 (I-95) in South Florida was used in the second objective. PTV VISSIM 11 microsimulation was used for investigating various scenarios of CV lane design and MPR% in the managed lanes network. The parameters for car following and lane changing models in VISSIM 11 were calibrated and validated using real-world CVs data in a project named CoEXist which is a European Union's Horizon 2020 funded Project (Groves, 2018; PTV, 2018; Sukennik, 2018). In this study, five main cases were considered. The base condition (Case 0) included the I-

95 corridor with one access zones (one ingress and one egress) in the middle of the corridor. In this case, three types of vehicles were considered: passenger cars (PCs), heavy goods vehicles (HGVs), and carpools. It is worth mentioning that connected vehicles are not considered in the base case. A total of 110 scenarios were studied with different lane configuration cases, market penetration rates, and traffic conditions. Six different cases of CV lane configuration in the MLs network were studied. In Case 1, connected vehicles were only allowed in the managed lanes and had the choice to use any of the managed lanes. In Case 2, Connected vehicles could use either the dedicated connected vehicles lane or the managed lanes. In Case 3, connected vehicles were only allowed to use the dedicated Connected vehicles' lane. Case 4 was similar to Case 1 with converting one lane of GPLs to MLs in order to increase the capacity of the MLs. In this case, connected vehicles were only allowed in the managed lanes and had the choice to use any of the managed lanes. Case 5 was similar to Case 1 with adding one lane to the MLs in order to increase the capacity of the network. In Case 6, CVs could use any of the lanes in the network.

The safety and operational analysis of the CVs and CVLs configurations in MLs were successfully represented. Various market penetration rates were studied and compared using three performance measures including: conflict reduction, speed increase, and delay reduction compared to the base case with no connected vehicles. For Case 1, the results of adding connected vehicles to the MLs network revealed that the maximum conflict reduction (compared to the base case) occurred at an MPR% between 20% and 25% for peak conditions. Regarding off-peak conditions, the maximum conflict reduction compared to the base condition happened when the MPR% was between 20% and 30%. For Case 2, it was found that the maximum conflict reduction occurred when the MPR% was between 25% and 30% in peak condition. On the other hand, in off-peak conditions, the best scenarios occurred when the market penetration rate was between 25% and 30%. Moreover, Case 3 (CVs can only use CVLs) was not recommended since it showed lower conflict reduction than other studied cases.

For Case 4, it was found that the maximum conflict reduction occurred when the MPR% was between 40% and 60% for the peak condition. For off-peak conditions, it is worth mentioning that the lowest conflict reduction occurred when the MPR% was between 50% to 70%. For Case 5, it was found that the maximum conflict reduction occurred when the MPR% was between 50% and 70% for the peak condition. For off-peak conditions, it is worth mentioning that the lowest conflict reduction occurred when the MPR% was between 60% to 80%. For Case 6, it was found that the maximum conflict reduction occurred at a higher MPR% between 60% and 100%. There was a positive association between a higher MPR% and the conflict reduction. It is worth noting that the off-peak conditions followed the same conflict reduction distribution as the peak conditions. Hence, a higher MPR% could be recommended for improving the network safety in Case 6. The highest conflict reduction was reached at an MPR% of 100%. It was also noted that at, in all cases of an MPR% of 10% or lower, there was no conflict reduction in the network.

Based on the Tobit and Negative Binomial models, the results highlighted that an MPR% of 10% and lower had no significant improvement than the base case with no CVs. Therefore, an MPR% lower than 10% was not recommended in managed lanes network. The findings suggested that an MPR% between 10% and 30% was recommended when the CVs are only allowed in MLs (Case 1 or Case 2). The MPR% could reach 60% by converting one lane of the GPLs to a lane of

MLs (Case 4). When increasing the number of managed lanes (Case 5), the MPR% could be improved to reach 70%. Lastly, the findings suggested that MPR% of 100% could be achieved by allowing the CVs to use all the lanes in the network (Case 6). In this case, the conflict reduction could reach 70% for an MPR% of 100% and could achieve 60% for an MPR% between 70% and 90%. Moreover, allowing CVs to use only CVLs (Case 3) was not recommended since it showed significant higher conflict frequency, higher delays, and lower speeds than other studied cases. Lastly, it was found that the off-peak periods had better safety and operational performance (e.g., lower conflict frequency, less delay, higher speed) in comparison to the peak periods. For future studies, more attention should be allotted to the peak conditions. It is expected that the outcomes from this study could be used as guidance to establish effective safety and operational plans for managed lanes in a connected vehicles environment.

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# TABLE OF CONTENTS

LIST OF FIGURES ..................... xv

LIST OF TABLES ..................... xix

LIST OF ACRONYMS / ABBREVIATIONS ..................... xxi

CHAPTER 1: INTRODUCTION ..................... 1

1.1 Overview ..................... 1

1.2 Research Objectives ..................... 7

1.3 Dissertation Organization ..................... 9

CHAPTER 2: LITERATURE REVIEW ..................... 11

2.1 Managed Lanes Safety ..................... 11

2.2 Microscopic Simulations ..................... 17

2.3 Conflict Studies ..................... 19

2.4 Connected Vehicles Studies ..................... 20

2.5 Summary ..................... 23

CHAPTER 3: MANAGED LANES ACCESS DESIGN ..................... 24

3.1 Introduction ..................... 24

3.2 Experimental Design ..................... 28

3.2.1 Study Area ..................... 28

3.2.2 Traffic Input ..................... 30

3.2.3 Data Collection Points ..................... 31

3.2.4 Simulation Scenarios ..................... 32

3.2.5 Vehicle Classes ..................... 36

3.2.6 Vehicle Composition ..................... 37

3.2.6.1 Type 1 ..................... 37

3.2.6.2 Type 2 ........................................................................................ 40

3.2.6.3 Type 3 ........................................................................................ 41

3.2.7 Trip Distribution ........................................................................................ 41
3.2.8 Desired Speed Distribution ........................................................................................ 46
3.2.9 Dynamic Toll Pricing ........................................................................................ 47
3.3 Calibration and Validation ........................................................................................ 48
3.3.1 Car Following Model ........................................................................................ 48
3.3.2 Lane Change Parameters ........................................................................................ 50
3.3.3 Traffic Volume and Speed ........................................................................................ 53
3.4 Safety Analysis ........................................................................................ 56
3.4.1 Surrogate Safety Measurements ........................................................................................ 56
3.4.2 Conflict Validation ........................................................................................ 57
3.4.3 SSAM Results ........................................................................................ 60
3.4.4 Conflict Results ........................................................................................ 64
3.3.4.1 Conflict Rate for Base Condition ........................................................................................ 64

3.3.4.2 Conflict Rate for Access design condition ........................................................................................ 64

3.3.4.3 Log-Linear Model ........................................................................................ 66

3.3.4.4 Tobit Model ........................................................................................ 71

3.5 Operation Analysis ........................................................................................ 75
3.5.1 Average Travel Speed ........................................................................................ 75
3.5.1.1 Traffic Speed Data Analysis ........................................................................................ 75

3.5.1.2 Post-Hoc Test for Speed ........................................................................................ 77

3.5.2 Average Delay ........................................................................................ 80
3.5.2.1 Traffic Delay Data Analysis ........................................................................................ 80

3.5.2.2 Post-Hoc Test for Delay ... 81

3.5.3 Time Efficiency ... 83
3.5.4 Level of Service (LOS) ... 84
3.6 Summary and Conclusions ... 86
CHAPTER 4: IMPACT OF CONNECTED VEHICLES ON FREEWAY FACILITIES WITH MANAGED LANES ... 90

4.1 Introduction ... 90
4.2 Experimental Design ... 91
4.2.1 Connected Vehicles Environment ... 91
4.2.2 Dedicated Connected Vehicles Lanes (CVLs) ... 95
4.2.3 Market Penetration Rate (MPR%) ... 96
4.2.4 Vehicle Classes ... 97
4.2.5 Desired Speed Distribution ... 98
4.2.6 Dynamic Toll Pricing ... 99
4.2.7 Scenarios Setup ... 100
4.3 Safety Analysis ... 106
4.3.1 Conflict Frequency ... 106
4.3.2 Conflict Reduction ... 114
4.3.3 Statistical Modeling ... 120
4.4 Operational Analysis ... 123
4.4.1 Average Speed ... 123
4.4.1.1 Speed Increase ... 131

4.4.1.2 Statistical Modeling ... 136

4.4.2 Average Delay ... 139
4.4.2.1 Delay Reduction ... 147

4.4.2.2 Statistical Modeling ... 152

4.5 Summary and Conclusion ........................................................................... 155
CHAPTER 5: CONCLUSIONS ........................................................................... 163

REFERENCES ........................................................................... 171

# LIST OF FIGURES

Figure 1. An example of Dynamic Toll Pricing Lanes, San Diego, California (Source: HNTB) .. 2

Figure 2. An example of High Occupancy Vehicle (HOV) lanes, Nashville, Tennessee ............. 2

Figure 3. An example of Bus Rapid Transit [BRT] lanes, Boston, Massachusetts ........................ 3

Figure 4. Managed lanes growth from 1970 to 2015 (Source: Fitzpatrick, 2017).......................... 4

Figure 5. Priced managed lanes in the U.S. Source: (ATKINS, 2013).......................................... 5

Figure 6. Hybrid Mainline Toll Plaza (HMTP) (Source: Central Florida Expressway Authority, Abuzwidah and Abdel-Aty, 2015) ........................................................................................... 7

Figure 7. Locations of the Potential Conflicts at Weaving Segments. ........................................ 17

Figure 8. Location of the existing MLs in I-95. Source: (FDOT, 2012; Systematics, 2014) ....... 25

Figure 9. Simulation process flow chart ...................................................................................... 27

Figure 10. Study area located on I-95 (Source: (FDOT, 2017), Google maps)........................... 28

Figure 11. Part of the VISSIM network (On-Ramp) ................................................................... 29

Figure 12. Volume Distribution ................................................................................................... 31

Figure 13. Weaving segments near access zones......................................................................... 32

Figure 14. Minimum weaving distance for access zones (min=minimum). Source: California DOT report, 2011 (Caltrans, 2011) ............................................................................................ 33

Figure 15. Ingress and egress details for different cases. Source: (FHWA, 2011) ....................... 34

Figure 16. Weaving Segments for the two accessibility levels case............................................ 34

Figure 17. Accessibility Level Cases ........................................................................................... 35

Figure 18. Car Following Behavior Model Parameters for MLs in VISSIM .............................. 49

Figure 19. Lane Change parameters for Weaving Sections in VISSIM ..................................... 52

Figure 20. Conflict Angle Diagram in SSAM ........................................................................... 57

Figure 21. Comparing simulated conflicts with field crashes..................................................... 60

Figure 22. TTC chart for GPLs and MLs .................................................................................. 63

Figure 23. MaxS chart for GPLs and MLs ................................................................................ 63

Figure 24. Conflict rate for various weaving lengths (conflict/ 1,000 vehicle-mile per hour)..... 66

Figure 25. The overlapping between access openings.............................................................. 70

Figure 26. Travel speed of GPLs and MLs for base condition................................................... 76

Figure 27. Comparing Average Speed among One Access Zone Cases ..................................... 77

Figure 28. Average delay for the base case ............................................................................... 80

Figure 29. Average delay for Case 1 ......................................................................................... 81

Figure 30. Time efficiency for Case 1 ....................................................................................... 84

Figure 31. Interaction objects and vehicles for the all-knowing logic......................................... 92

Figure 32. Interaction objects and vehicles for the Cautious and Normal logics ......................... 92

Figure 33. Connected Vehicles Driving Logics (PTV 2018) ..................................................... 92

Figure 34. Different Driving Logics in VISSIM (PTV, 2018; Sukennik, 2018).......................... 93

Figure 35. Assigning Driving Logic to CV for CV Lanes (Source: VISSIM 11). ....................... 96

Figure 36. The Base Case (Case 0) with No Connected Vehicles in the Network..................... 100

Figure 37. Case 1 with Connected Vehicles in the Managed Lanes.......................................... 101

Figure 38. Case 2 with Connected Vehicles in the CVLs Only................................................. 101

Figure 39. Case 3 with Connected Vehicles in either MLs or CVLs ........................................ 102

Figure 40. Case 4 with CVs in MLs and Converting One GPLs to MLs .................................. 102

Figure 41. Case 5 with CVs in MLs and Increasing the Number of MLs ................................. 103

Figure 42. Case 6 with Connected Vehicles in All Lanes (GPLs, MLs, CVLs) ........................ 103

Figure 43. Conflict Frequency for Peak and Off-peak Conditions in the Base Case ................. 107

Figure 44. Conflict Frequency for peak and off-peak condition in Case 1................................ 110

Figure 45. Conflict Frequency for Peak and Off-peak Conditions in Cases 2 ........................... 111

Figure 46. Conflict Frequency for Peak and Off-peak Conditions in Case 3............................ 111

Figure 47. Conflict Frequency for Peak and Off-peak Conditions in Case 4............................ 112

Figure 48. Conflict Frequency for Peak and Off-peak Conditions in Case 5............................ 113

Figure 49. Conflict Frequency for Peak and Off-peak Conditions in Case 6............................ 114

Figure 50. Conflict Reduction for Peak Conditions in Cases 1, 2, and 3 .................................. 115

Figure 51. Conflict Reduction for Off-Peak Conditions in Cases 1, 2, and 3 ........................... 116

Figure 52. Conflict Reduction for Peak and Off-peak Condition in Case 4.............................. 117

Figure 53. Conflict Reduction for Peak and Off-peak Condition in Case 5.............................. 118

Figure 54. Conflict Reduction for Peak and Off-peak Condition in Case 6.............................. 119

Figure 55. Average Speed for Different MPR% in Case 1....................................................... 126

Figure 56. Average Speed for Peak and Off-peak Conditions in Case 2.................................. 127

Figure 57. Average Speed for Peak and Off-peak Conditions in Case 3.................................. 128

Figure 58. Average Speed for Peak and Off-peak Conditions in Case 4.................................. 129

Figure 59. Average Speed for Peak and Off-peak Conditions in Case 5.................................. 130

Figure 60. Average Speed for Peak and Off-peak Conditions in Case 6 .................................. 131

Figure 61. Speed Increase for Peak Condition in Cases 1, 2, and 3 .......................................... 132

Figure 62. Speed Increase for the Off-peak Condition in Cases 1, 2, and 3 .............................. 133

Figure 63. Speed Increase for Peak and Off-peak Conditions in Case 4 .................................. 134

Figure 64. Speed Increase for Peak and Off-peak Conditions in Case 5 .................................. 135

Figure 65. Speed Increase for Peak and Off-peak Conditions in Case 6 .................................. 136

Figure 66. Average Delay for Peak and Off-peak Conditions in Case 1 .................................. 142

Figure 67. Average Delay for Peak and Off-peak Conditions in Case 2 .................................. 143

Figure 68. Average Delay for Peak and Off-peak Conditions in Case 3 .................................. 144

Figure 69. Average Delay for Peak and Off-peak Conditions in Case 4 .................................. 145

Figure 70. Average Delay for Peak and Off-peak Conditions in Case 5 .................................. 146

Figure 71. Average Delay for Peak and Off-peak Conditions in Case 6 .................................. 147

Figure 72. Average Delay Reduction in Peak Conditions ........................................................ 148

Figure 73. Delay Reduction in Off-Peak Conditions ................................................................. 149

Figure 74. Delay Reduction in Case 4 ........................................................................................ 150

Figure 75. Delay Reduction in Case 5 ........................................................................................ 151

Figure 76. Delay Reduction in Case 6 ........................................................................................ 152

Figure 77. Highest Conflict Reduction and Safest MPR% for All Studied Cases .................... 168

# LIST OF TABLES


Table 1. Weaving distances for MLs ........ 15

Table 2. List of scenarios ........ 36

Table 3. Vehicle Composition: Type 1 ........ 38

Table 4. Vehicle Composition: Type 2 ........ 41

Table 5. Traffic Volumes in VISSIM (Vehicle per Hour) ........ 43

Table 6. Percentages of Vehicles from the Beginning to the Off-Ramps and to the End of the Network ........ 44

Table 7. Percentages of Vehicles from the First On-Ramp to the Off-Ramps and to the End of the Network ........ 45

Table 8. Calibration Results ........ 55

Table 9. Validation Results ........ 56

Table 10. Descriptive Statistics of the Surrogate Safety Measures ........ 62

Table 11. Comparison of Odds Multipliers of Conflict Frequency between Various Cases ........ 69

Table 12. Tobit Models for the Safety Measures ........ 73

Table 13. Post-Hoc Test Results of Average Speed for Accessibility Levels ........ 79

Table 14. Post-Hoc Test Results of Average Speed for Weaving Lengths ........ 79

Table 15. Post-Hoc Test Results of Average Delay for Accessibility Levels ........ 82

Table 16. Post-Hoc Test Results of Average Delay for Weaving Lengths ........ 83

Table 17. Level of Service from Density ........ 85

Table 18. Level of Service for Case 1 .......... 86

Table 19. Car Following Parameters for Different Driving Logics (PTV 2018) .......... 94

Table 20. Lane Change Behavior for Different Driving Logics (PTV 2018) .......... 95

Table 21. List of Scenarios .......... 105

Table 22. Descriptive Statistics of Conflict Frequency for All Studied Cases .......... 108

Table 23. Post Hoc Test of Conflict Frequency between Cases .......... 109

Table 24. Negative Binomial Model for Conflict Frequency .......... 122

Table 25. Descriptive Statistics of Average Speed in All studied Cases .......... 124

Table 26. Post-hoc Test of Average Speed between Cases .......... 125

Table 27. Tobit Model for Average Speed .......... 137

Table 28. Descriptive Statistics for Average Delay in All studied Cases .......... 140

Table 29. Post-hoc Test for Delay between Cases .......... 141

Table 30 Tobit Model for Delay .......... 153

Table 31. Optimal Market Penetration Rates (MPR%) for Different Cases .......... 159

Table 32. CV Lane Design Recommendations for Different MPR% .......... 162

# LIST OF ACRONYMS / ABBREVIATIONS

| | |
|---|---|
| AADT | Annual average daily traffic |
| ACS | American Community Survey |
| AETC | All-Electronic Toll Collection |
| ANOVA | Analysis of variance |
| ASCE | American Society of Civil Engineering |
| AVI | Automatic Vehicle Identification |
| BRT | Bus Rapid Transit |
| CC0 | Stand Still Distance |
| CC1 | Following Headway Distance |
| CC2 | Following Variation Distance |
| CG | Comparison group |
| CMF | Crash Modification Factor |
| COM | Component Objective Model |
| CS | Cross-sectional |

| | |
|---|---|
| CVLs | Connected Vehicles Lanes |
| CVs | Connected Vehicles |
| DeltaS | Difference in vehicle speeds |
| DLC | Discretionary Lane Change |
| DOT | Department of Transportation |
| DR | Deceleration rate |
| DSD | Desired speed distribution |
| EB | Empirical Bayes |
| ELs | Express Lanes |
| ETC | Electronic toll collection |
| ETLs | Express Toll Lanes |
| FDOT | Florida Department of Transportation |
| FHWA | Federal Highway Administration |
| GEH | Geoffrey E. Havers |
| GPLs | General-purpose lanes |
| HMTP | Hybrid Mainline Toll Plazas |

| | |
|---|---|
| HOT | High-Occupancy Toll |
| HOV | High-Occupancy Vehicle |
| HSIS | Highway Safety Information System |
| I-95 | Interstate 95 |
| ITS | Intelligent Transportation System |
| LOS | Level of Service |
| MaxD | Maximum deceleration |
| MaxDeltaS | Maximum difference in vehicle speeds |
| MaxS | Maximum speed |
| MLC | Mandatory Lane Change |
| MLs | Managed lanes |
| MPH | Mile per hour |
| NB | Negative Binomial |
| NCHRP | National Cooperative Highway Research Program |
| ORT | Open Road Tolling |
| PDO | Property damage only |

| | |
|---|---|
| PET | Post Encroachment Time |
| RITIS | Regional Integrated Transportation Information System |
| RLs | Reversible Lanes |
| RM | Ramp Metering |
| S4A | Signal Four Analytics |
| SAS | Statistical Analysis System |
| SPF | Safety Performance Function |
| SR-408 | State Road 408 |
| SSAM | Surrogate Safety Assessment Model |
| TO | Truck Only lanes |
| TOT | Truck Only Toll |
| TTC | Time to Collision |
| V/C | Volume-to-Capacity-Ratio |
| VPH | Vehicles per hour |
| VSL | Variable speed limit |

# CHAPTER 1: INTRODUCTION

## 1.1 Overview

On freeways, managed lanes (MLs) have emerged as an effective dynamic traffic management strategy. They are a vital option for managing time and congestion through tolling while also providing drivers with more choices. They play an important role in improving traffic mobility, efficiency, and safety, in addition to generating revenue for transportation agencies. MLs are designated lanes where the flow of traffic is managed by limiting vehicle eligibility (e.g., High Occupancy Vehicle [HOV], Truck Only lanes [TO]), restricting facility access (e.g., Reversible Lanes [RLs], Express Lanes [ELs]), employing fixed or dynamic price tolls (e.g., toll ways, Express Toll Lanes [ETLs]), pricing and vehicle eligibility (e.g., High-Occupancy Toll [HOT], Truck Only Toll [TOT] lanes), or vehicle eligibility and access control (e.g., Bus Rapid Transit [BRT] lanes, dedicated truck lanes, transit ways) (Fitzpatrick et al., 2016; Fitzpatrick et al., 2017; Perez et al., 2012). Figure 1, 2, and 3 show examples of express lanes with dynamic toll pricing, HOV lanes, and BRT lanes, respectively. In this research, it is also proposed that there might be a new designation for managed lanes as designated Connected Vehicles' lanes.

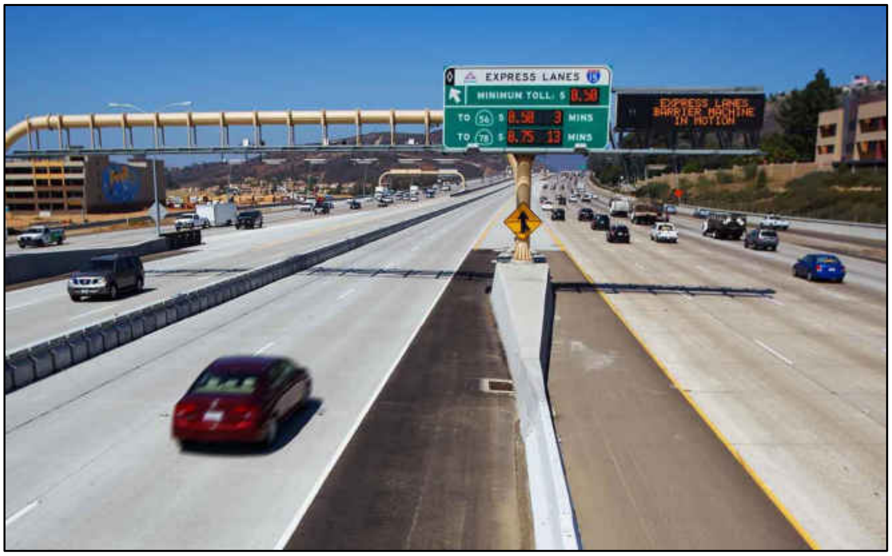



**Figure 1. An example of Dynamic Toll Pricing Lanes, San Diego, California (Source: HNTB)**


**Figure 2. An example of High Occupancy Vehicle (HOV) lanes, Nashville, Tennessee**

**Figure 3. An example of Bus Rapid Transit [BRT] lanes, Boston, Massachusetts**

The route-miles of MLs from 1970 to 2015 is shown in Figure 4. The figure revealed a trend of MLs growth over the years. Since 1995, express lanes have grown drastically. The growth of MLs is expected to continue. 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. By 2020, managed lanes are projected to be expanded throughout the U.S. to reach 6,000 lane-miles.

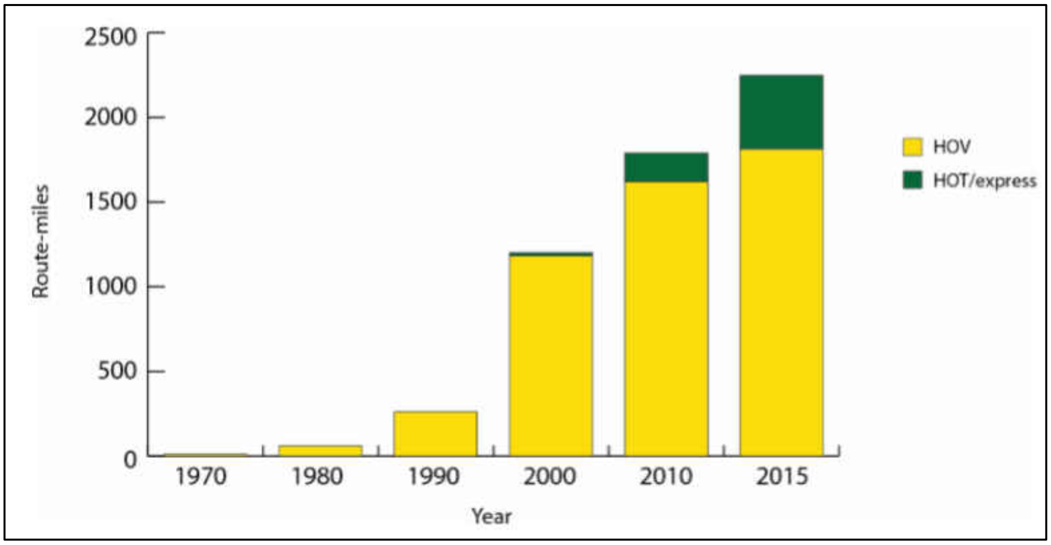



**Figure 4. Managed lanes growth from 1970 to 2015 (Source: Fitzpatrick, 2017)**

Several major cities in the United States, as shown in Figure 5, have introduced managed toll lanes systems such as ETLs and HOT lanes for managing time efficiency and alleviating congestion via tolling systems and providing drivers with more alternative routes. In U.S., 35 states use tolling roads with 6,233 miles of toll roads, bridges and tunnels. There are also more than 50 million transponders along 46 priced managed lanes facilities. In 2016, there were more than 5.7 billion trips token on toll facilities which generated $18 Billion in toll revenues. Currently, there are over 300 MLs facilities in the U.S. The managed toll lanes are thought to be an appropriate option to deal with high congestion while also offering a viable cost-effective model for promoting economic development. Toll revenue has the potential to support half of the costs of the $1 billion asset of the facility (ATKINS, 2013).

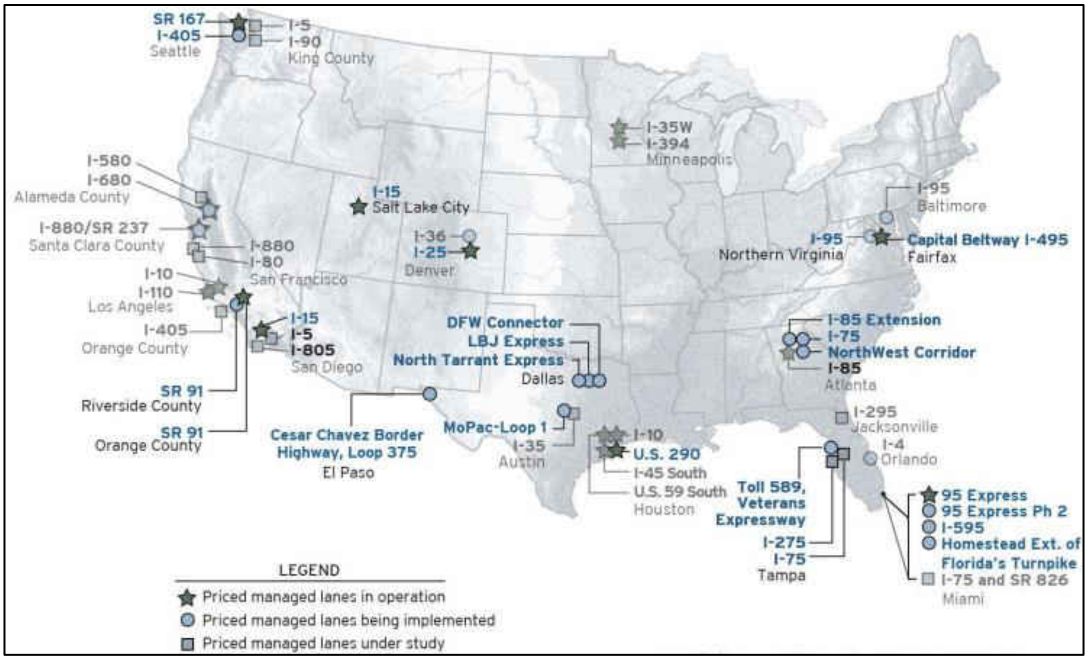



**Figure 5. Priced managed lanes in the U.S. Source: (ATKINS, 2013)**

In recent years, MLs have emerged as an effective dynamic traffic management strategy and are considered a safer option than toll plazas. One of the critical problems of toll plaza areas is the confusion of driver, due to the various lane configurations and the different tolling systems. A study conducted by Saad et al., 2019, evaluated the factors that influence dangerous driving behavior at toll plazas. A hybrid plaza section of SR-408 in Central Florida was used, which consisted of a tollbooth and open road tolling (ORT) systems, as shown in Figure 6. The tollbooth section included cash lanes and electronic toll collection (ETC) lanes. This design requires vehicles to decelerate or stop so drivers can navigate through different fare options including cash toll and electronic toll collection. In the ORT section, drivers can navigate without stopping to pay tolls or changing lanes by using the automatic vehicle identification (AVI) transponders. The studied section included a close on- and off-ramps and was partitioned into four unique segments before, at, and after the plaza. The crash reports of the toll plaza highlighted that the most dangerous locations along the toll plaza segment were the merging and the diverging areas. It was also concluded that the most frequent types of traffic crashes at these locations were the sideswipe and the loss of control crashes. These two categories of traffic crashes were attributed mainly to the unexpected lane changing at these sites. The study used driving simulator to assess driving behavior at hybrid plazas. The random effects models were applied to account for the data from the same participants. Different scenarios have been assessed to test the effect of the potential critical factors on risky driving behavior. The scenario variables included path decision making, signage, pavement marking, extending auxiliary lanes, and traffic conditions. Driver characteristics were also considered in the study. The results revealed that drivers at the open road tolling section performed less risky driving behavior than those who use the tollbooth. It was suggested to convert hybrid toll plazas to open-tolling system (e.g. managed lanes, and all-electronic toll collection system (AETC)) (M. Abdel-Aty et al.; M. Abuzwidah & Abdel-Aty, 2015; M. A. Abuzwidah, 2011; M. Saad, 2016; M. Saad, Abdel-Aty, & Lee, 2018; Xing et al., 2019).

**Figure 6. Hybrid Mainline Toll Plaza (HMTP) (Source: Central Florida Expressway Authority, Abuzwidah and Abdel-Aty, 2015)**

## 1.2 Research Objectives

The ongoing work of this study focuses on the effect of access design on the safety and the operation of the MLs. The primary research objective of this study is to use microsimulation to maximize the system-wide efficiency, by determining the optimal accessibility level, in addition to deciding on the sufficient length and locations of weaving segments near access zones. 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 of the access design.

The detailed objective of this dissertation was achieved by the following main procedures;

1. Determining the optimal managed lanes access design including accessibility level and weaving distance for an at-grade access design (I-95, South Florida).
2. Studying the effect of applying connected vehicles on the safety and the operation of the network.

The first objective was achieved by the following tasks:

1. Conducting a complete literature review related to managed lanes safety, microsimulation studies, and conflict related research.
2. Build a network for 9 mi of dynamic pricing HOT lanes in a South Florida, I-95 section, including MLs, GPLs, and the ramps.
3. Conducting the calibration and the validation of the network. The calibration process includes the parameters of the Car following behavior and lane change behavior models, in addition to the traffic volume calibration of the network. The validation process of the network included comparing the traffic speeds between the simulated data and the field data.
4. Investigating the effects of accessibility levels and weaving on the safety and operation of MLs are investigated. Thirty-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 evaluate the monetary benefits of various strategies.

5. A conflict prediction model was developed for investigating the factors and scenarios that affect traffic conflict frequency. Also, models were developed for analyzing the operation performance measures. Results and conclusions of the first objective are discussed at the end of Chapter 3.

The second objective was accomplished by the following tasks:

1. Conducting a literature review for the connected vehicles studies, which related to applying connected vehicles in the simulated network and determining the optimal market penetration rate.
2. Building a simulation network that considered different configurations of connected vehicles and connected vehicles lanes in the managed lanes environment.
3. Determining the safety and operational impacts of adding connected vehicles and connected vehicles lanes on the MLs network.
4. Studying different market penetration rates of connected vehicles (e.g., 20%, 40%, 60%) and proposing an optimal market penetration rate for different cases of connected vehicles and connected vehicles lanes in the studies network.

## 1.3 Dissertation Organization

This dissertation is composed of five sections. Following this chapter, the second chapter gives a brief review of the previous studies of MLs, a review of studies related to microsimulation and analyzing traffic conflicts, and lastly the previous studies related to connected and automated vehicles. Chapter three describes the microsimulation process for the studied corridor, which mainly included traffic data collection, network building, calibration and validation process, in addition to the connected vehicles scenario design. This part sought to evaluate the operation and safety of different MLs access designs including accessibility level and weaving distance. Chapter four proposed the safety and operational analysis of adding connected vehicles in the managed lanes network. Finally, chapter five provides a summary of the dissertation and a description of the recommendations for future work.

# CHAPTER 2: LITERATURE REVIEW

The literature review part consists of four sections. The first part represents the previous studies of the managed lanes safety. The second section gives a brief review of the microsimulation studies related to traffic safety. The third part shows the studies that utilized simulated conflicts for analyzing safety data. The final section draws together the summary of the literature review.

## 2.1 Managed Lanes Safety

The primary purpose of MLs is to manage and expedite the flow of traffic in a segment through access control (i.e., entrances, and exits), vehicle eligibility (i.e., vehicle type, vehicle occupancy), or pricing strategies (i.e., dynamic tolls) (Kuhn, 2010). As presented by the Federal Highway Administration (FHWA), MLs are a valuable option for transportation agencies to manage traffic congestion (FHWA, 2011, 2017). The priced managed lanes system has risen dramatically in the U.S. in recent years due to improved time reliability, time savings, mobility, congestion management, and revenue generation (HNTB, 2013). The toll revenue is used to fund the facility through the dynamic tolls, which vary based on the time saved and the traffic periods. As the traffic increases in the MLs (i.e., peak period), the toll price increases to maintain the operating speed at the MLs (M. Abuzwidah & Abdel-Aty, 2017, 2018).

Limited research has been conducted on the safety benefits of improving the geometric design of the GPLs 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 of MLs (Fitzpatrick et al., 2017). A recent article conducted by Abuzwidah and Abdel-Aty (2017), analyzed crash data for 156 segments on I-95 over the course of 9 years (2005 to 2013) using three methods: Before-After with the comparison group (CG) method, empirical Bayes (EB) method, and Cross-sectional (CS). The CMFs values were calculated as 1.19 for total crashes and 1.28 for PDO crashes for the whole segment. For the HOT lanes, the CMF for total crashes and PDO crashes are 0.8 and 0.63, respectively. For GPLs, the authors found that the CMF for total crashes is 1.23 while CMF for PDO crashes is 1.35. The results showed that he total crashes in the MLs decreased by 20% and the severe crashes (fatal and injury) were reduced by 30%. Moreover, the total crashes and severe crashes (fatal and injury) increased in GPLs by 19% and 8%, respectively (M. Abuzwidah & Abdel-Aty, 2017)

The latest managed lanes guidelines report from the National Cooperative Highway Research Program (NCHRP) (Fitzpatrick et al., 2017) pointed out that MLs provide better operational and safety performance than GPLs. Access zones are considered to be one of the most dangerous locations on the GPLs segments. Crashes frequently occur near the entrances and the exits of the MLs. One of the countermeasures that were suggested by NCHRP was to appropriately locate the access zones and the traffic control devices (Fitzpatrick et al., 2017). Designated access should be strategically positioned to minimize erratic weaving from or to nearby ramps (Fuhs, 1990). Two types of crashes are common near the access zones, including sideswipe and rear-end crashes. Sideswipe crashes happen due to the lane changing maneuvers upstream from the MLs entrances or exits. Meanwhile, the rear-end crashes occurred as a result of the vehicles that decelerate before entering MLs (Fitzpatrick et al., 2017). Access zones crashes are fundamentally affected by access type, traffic periods, and weaving length upstream or downstream of the facility.

Meanwhile, the high crash frequency is associated with small access length and close access points to the on- or -off-ramps (Caltrans, 2011; Jang et al., 2009; Machumu et al., 2017).

There are multiple approaches for providing access to managed lanes: 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 managed lanes at any point. Experiences from the design of access zones for managed lanes suggest several recommendations (Fuhs, 1990). First, the geometric criteria for access zones should be the same as those that are used for freeway ramps, including locally recognized entrance and exit standards. Second, the location of ingress/egress facilities is influenced by certain factors. For example, direct access ramps to/from local streets should be made with candidate streets that currently do not have freeway access to bettr distribute demand and prevent overloading existing intersections. For at-grade access with the adjacent freeway lanes, designated outlets should be strategically positioned to minimize erratic weaving to reach nearby freeway exits. Third is to locate ingress/egress points associated with street access away from intersections that are operating at or near the 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 (RM) and enforcement should be considered.

Weaving segments are one of the most critical areas on freeways, with more sideswipe and rear-end crashes than other segments (Glad, 2001; Golob et al., 2004; Kim & Park, 2018). Pulugurtha and Bhatt (2010), explained that the high crashes in the weaving segments is likely due to the short weaving distances near the ramps (Pulugurtha & Bhatt, 2010). The weaving length is an important factor that affects the crash count (Bonneson & Pratt, 2008; Cirillo, 1970; Pulugurtha & Bhatt, 2010; Qi et al., 2014) found that longer weaving segments have lower CMF, which indicates a lower number of crashes.  Previous studies have explored the efficient weaving length near the access zones of MLs. One of these studies was conducted by the California Department of Transportation (Caltrans, 2011), which suggested a minimum distance of 800 ft per lane change between the on- or off- ramps and the access zones. Another study conducted by the Washington State Department of Transportation (WSDOT) (Burgess, 2006) proposed the minimum distance between the access zones and the on- or off- ramps to be 500 ft per lane change. Meanwhile, the study recommended that the desired distance be 1,000 ft per lane change. A study conducted by Venglar et al. (2002), suggested that the range of the weaving length varied between 500 ft to 1,000 ft per lane change (Venglar et al., 2002). They provided various cases of the weaving distance as shown in Table 1. They concluded that the minimum distance between the ingress and the egress of the MLs was 2,500 ft.

**Table 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 (ft) |
|---|---|---|---|
| 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 (Venglar et al., 2002)

Yuan et al., conducted a study in the University of Central Florida to investigate the safety effects of weaving length, traffic condition, and driver characteristics on drivers' mandatory lane change behavior based on a driving simulator study. Mixed factorial design with two within-subject factors (traffic volume: off-peak and peak; speed harmonization (SH): SH and Non-SH) and one between-subject factor (weaving length per lane change ($\boldsymbol{L_{LC}}$): 600 feet, 1,000 feet, and 1,400 feet) were employed in this study. Fifty-four licensed drivers were recruited to conduct this driving simulator experiment. Based on the experimental data, three lane change decision metrics (i.e., lane change merging gap, duration, and patience time), three lane change execution metrics (i.e., maximum longitudinal deceleration, lateral acceleration, and steering wheel angle), and two surrogate-safety metrics (i.e., number of conflicts and time exposed time-to-collision) were analyzed. Results indicated that for the ingress of MLs (entrance weaving segment), 1,000 feet $\boldsymbol{L_{LC}}$ would be recommended if the space is limited, otherwise 1,400 feet $\boldsymbol{L_{LC}}$ is more preferable. For the egress of MLs (exit weaving segment), however, only 1,000 feet $\boldsymbol{L_{LC}}$ was recommended since the 1,400 feet $\boldsymbol{L_{LC}}$ was found to be significantly more dangerous than the 600 and 1,000 feet $\boldsymbol{L_{LC}}$. Moreover, the peak traffic condition could significantly increase the difficulty of lane change behavior on the weaving segments, and the speed harmonization could significantly improve the lane change safety on the entrance weaving segment (Yuan & Abdel-Aty, 2018; Jinghui Yuan et al., 2019).

Another work completed in the University of Central Florida by Cai et al., 2018, for investigating the optimal weaving distance in a freeway segment of Interstate 95 (I-95) in Miami, Florida, with four GPLs and two MLs. In the simulation, three weaving lengths (600 ft, 1,000 ft, and 1,400 ft) per lane change were tested under two traffic conditions (Peak off-peak and peak). Three performance measurements were used for the safety evaluation including: speed standard deviation, potential conflict, and time to collision. The results of the speed standard deviation and the potential conflicts revealed that a 1,400 ft per lane change increased the crash risk at the weaving segment. However, no significant difference could be found between the 600 ft and the 1,000 ft length per lane change. Based on the traffic condition results, it was found that better safety performance could be found under the off-peak traffic condition. In addition, variable speed limit (VSL) strategy was tested in the driving simulator experiment and it was found that VSL improved the safety of the studied network. The results of the driving simulator experiments were consistent with the results of the microsimulation with respect to the optimal weaving length. The study suggested that better results could be obtained if the drivers' lane change behavior observed in the driving simulator study could be used as input in the VISSIM simulation using COM interface (Cai, Saad, et al., 2018). Figure 7 shows the locations of the potential conflicts at weaving segments.

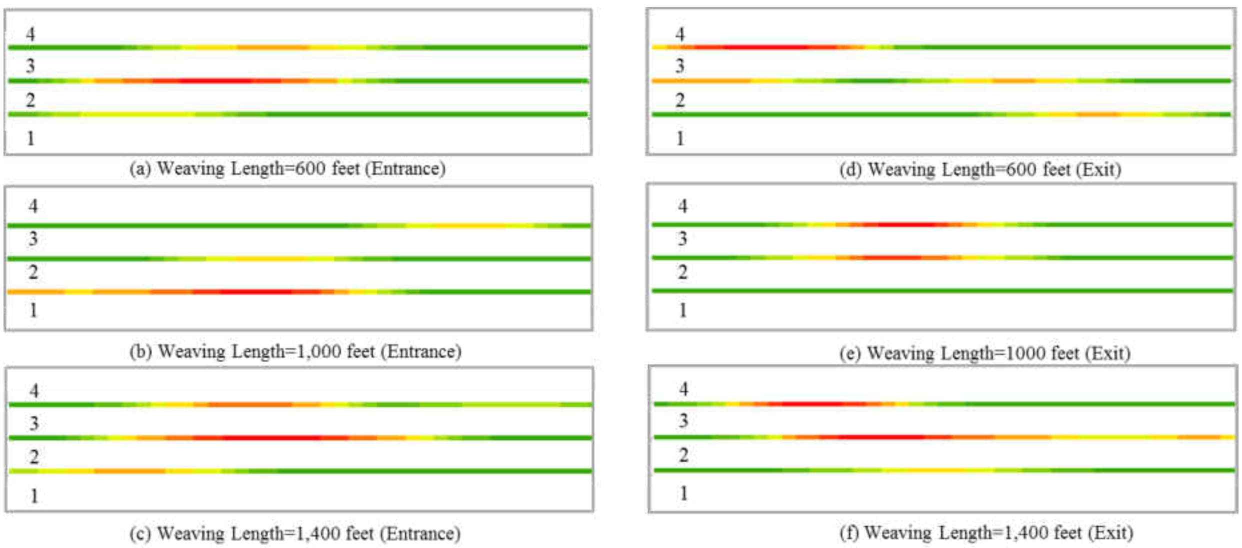



**Figure 7. Locations of the Potential Conflicts at Weaving Segments.**

2.2 <u>Microscopic Simulations</u>

As indicated by Haleem (2007), traffic simulation plays a vital role in better understanding the traffic of the real world and producing accurate, quick results. Using traffic simulation has many advantages. First, we are able to predict the driving behavior due to a specific action. Second, the reason why some events happened in the real world can be explored. Third, studying hotspot areas or regions with problems before carrying out solutions becomes a reality. Fourth, the impact of any modifications on the traffic system would be able to be identified. Fifth, familiarity with all variables could be attainable. Sixth, the drawbacks of the traffic system could be recognized. Lastly, new ideas would be able to be efficiently simulated. Many studies have used simulation experiments to carry out conclusions for traffic safety studies. Lately especially, simulation has been a flexible and efficient tool for improving traffic safety analysis. It is also proven that using simulation in traffic safety studies is a cost-effective way for testing different scenarios which are an accurate representation of the real world in a simulated environment (Haleem, 2007; Shalaby et al., 2003).

According to Nilsson (1993), simulation is one of the most widely used and efficient tools for studying roadway system operation and investigating traffic safety impacts. Compared to other methods, simulation is more efficient, and an easier way for traffic data collection. It is able to test the impact of a treatment before implementation in field. Meanwhile, it is an alternative tool for evaluating different operations and improvements since field data collection is a costly and time-consuming process (Nilsson, 1993). Simulation techniques can be used for analyzing risky driving factors by replicating a similar environment to real life experience (Allen et al., 2011). This method allows testing multiple scenarios applicable to road geometry or traffic control devices (Bham et al., 2010). In conclusion, because of the enormous amount of field data required for studying driver behavior, simulation techniques are the most appropriate tool for conducting this kind of study.

Simulation networks have to be validated with real world data as an attempt to study traffic safety and especially for exploring driving behavior accurately (M. Abdel-Aty et al., 2006). Calibration and validation are the most important steps when utilizing simulation to replicate real world conditions. When studying weaving segments in simulation, several driving behavior parameters for car following and lane change should be adjusted to well calibrate and validate the network (Koppula, 2002; Woody, 2006). The car following model determines the longitudinal movement of the simulated vehicle, while the lane change model decides a vehicle's lateral movement.

## 2.3 Conflict Studies

This study utilized traffic conflicts to evaluate the safety of the studied corridor and to determine the safest access zone design for the toll managed lanes. Traffic conflict is identified as an evasive action (e.g., braking, deceleration, jerking, etc.), when two or more vehicles are approaching each other (Perkins & Harris, 1968; Tageldin et al., 2015; Wang, 2016; Ling Wang et al., 2017). Previous studies have investigated the factors that affect traffic conflicts (El-Basyouny & Sayed, 2013; Sacchi & Sayed, 2015). El-Basyouny and Sayed (2013), conducted a study for developing a safety performance function (SPF) using Negative Binomial model for predicting traffic conflicts by utilizing several variables including hourly volume, geometric design, and area type (El-Basyouny & Sayed, 2013).

Some studies compared simulated conflicts with real crash data and with real conflicts in order to validate the simulation safety data. The use of simulated conflicts is a promising approach for estimating safety performance (Perez et al., 2012). Previous research has investigated the relationship between simulated conflicts and real conflicts in order to validate the simulation safety and recommend countermeasures for reducing crashes (Perez et al., 2012). Gettman et al. (2008), compared the number of simulated conflicts with real crash frequency at intersections. It was concluded that there was a significant correlation between the simulated conflicts and real crash frequencies (Gettman et al., 2008). A study conducted by Saleem et al. (2014), proved that conflict frequency is a significant variable for the various crash types and severities (Saleem et al., 2014). Shahdah et al. (2014), used simulated conflicts for determining crash modification factors (CMFs) and compared it with the real CMFs based on the empirical Bayes (EB) method for the same study area. It was found that CMFs from simulated conflicts are consistent with CMFs from real crashes (Shahdah et al., 2014).

### 2.4 Connected Vehicles Studies

Connected vehicles are quickly expanding in transportation industry. During the coming decade, CVs are expected to be more widespread. CVs are one of the most recent developments in traffic and safety engineering (Ekram & Rahman, 2018; Fagnant & Kockelman, 2015; Papadoulis et al., 2019; M. H. Rahman et al., 2019; Rahman, 2018; Rahman & Abdel-Aty, 2018; M. S. Rahman et al., 2019; Rahman et al., 2018; Wu, 2017; Wu, Abdel-Aty, et al., 2019a, 2019b; Wu, Abdel-Aty, Zheng, et al., 2019). Connected vehicles have the potential to revolutionize safety and efficiency by reducing the number of crashes and fatalities on the road. This technology enables vehicles, roads, traffic signals and other infrastructure to communicate with one another about current road conditions, alerts and signals. This advanced technological correspondence is possible through two methods of communication: (1) vehicle-to-vehicle (V2V) and (2) vehicle - to-infrastructure (V2I). These signals are conveyed mainly by dedicated short-range communication system (DSRC). Intricate networks of communication allow for conveyance of a wide variety of information including position, speed, weather conditions, obstructions, etc. (Ekram & Rahman, 2018; Rahman & Abdel-Aty, 2018).

Every year, over 5 million crashes occur on the road, resulting in over 30,000 fatalities and many serious injuries. Connected vehicles could be the answer to reducing these accidents on the road. While advanced engineering safety controls (e.g., airbags, emergency brakes, anti-lock braking systems etc.) exist to protect drivers, the goal of CVs is to prevent crashes from ever occurring. With the use of V2V and V2I technology, vehicle user errors-which occur in more than 94% of traffic crashes- would occur less frequently, resulting in fewer crashes. Vehicle users would be able to make safer choices regarding acceleration, speed, lane changing and more. Despite the numerous advantageous possibilities, there is very little research on how safe CVs actually are. Previous studies have focused mainly on mobility and traffic operations of CVs instead of the impact they would have on traffic safety (Singh, 2015; Yue et al., 2018). Fyfe and Sayed conducted an experiment that combined VISSIM and Surrogate Safety Assessment Model (SSAM) along with the Cumulative Travel Time (CTT) algorithm. They observed that there was a 40% reduction in frequency of rear-end crashes at a signalized intersection (Fyfe & Sayed, 2017). Another study by Olia et al. paired CV technology with PARAMICS and found that the safety index improved by up to 45% (Olia et al., 2016). PARAMICS were also utilized by Paikari et al., by combining V2V and V2I technologies for enhancing safety and mobility (Paikari et al., 2014).

One of the biggest obstacles of popularizing CVs is related to the market penetration rate (MPR%). Full market penetration of CVs might prove to be difficult. As a result, a mixture of conventional vehicles and CVs would likely be used in simulation models. Rahman and Abdel-Aty et al. used simulation to study the safety effects of managed lane CVs platoons. It was found that longitudinal safety was significantly improved with the implementation of CVs platoons. Additionally, managed lane CVs platoons significantly surpassed non-managed lanes with the same market penetration rate. However, a limitation of the study was the use of high occupancy vehicle (HOV) managed lanes instead of a separated managed lane. Vehicle Platooning is another major feature of CVs that is worth exploring. Vehicle platooning involves a group of cars that are able travel closely to one another as a unit. A leading car would control the speed and direction and the cars following it would respond automatically with appropriate braking and acceleration. The result would be more efficient use of road space and more steady traffic flow. A stochastic model for determining the likelihood of collision for a vehicle platoon was performed by Tian et al. The results indicate a potential for less chain collision occurrences as well as a decrease in severity of chain collisions.

Full implementation of V2V communications could prevent hundreds of thousands of crashes every year. Yue et al. studied how V2V technology affected the safety of vehicles that were involved and found that crashes were reduced by 33% for light vehicles and 41% for heavy trucks. On the other hand, V2I communications have yet to be thoroughly explored. Li et al. performed a simulation study with controlled variable speed and adaptive cruise control. The results indicated that I2V communications overall provide significant safety benefits. Real time traffic collected from CVs communications could be used for improving traffic flow and therefore increase efficiency and reliability of self-driving vehicles. Automated vehicles have been explored as an option for vehicle users in many works of literature. Morando et al. studied fully autonomous vehicles and the findings were a 20-65% decrease in conflicts with penetration rates of 50-100%. To date, there are none to very few studies focusing on a lower level of automation by combining autonomous vehicles with CVs technology. Kockelman et al. conducted a questionnaire survey in the United States and found that most people who took the surveys were interested in lower level automated vehicles. They also anticipate lower level automation technology to be adopted at a rate of more than 90% by 2045 (Kockelman et al., 2016). A key challenge of studying automated vehicles and CVs technology is the task of determining the effects of driving behavior on connected and automated vehicles.

## 2.5 Summary

In general, the literature supports the notion that MLs are an important countermeasure for improving the safety and 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. Micro-traffic simulation was utilized, as it is a valid approach for studying the safety and operational effectiveness of the access zone design and can generate traffic conflict data. Previous studies proved that the simulated conflicts can be used as validated data to represent the real conflicts.

# CHAPTER 3: MANAGED LANES ACCESS DESIGN

## 3.1 Introduction

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 any 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 microsimulation was used for developing the network due to its feature of simulating dynamic priced MLs. PTV VISSIM microscopic simulation, version 9.0, was chosen in this study for its ability to simulate the lane choice process, based on dynamic tolling. The microsimulation network was built for evaluating the safest control level for the MLs. First, the corridor's geometry and traffic were inputted in VISSIM. The simulated area consisted of 9 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 8.

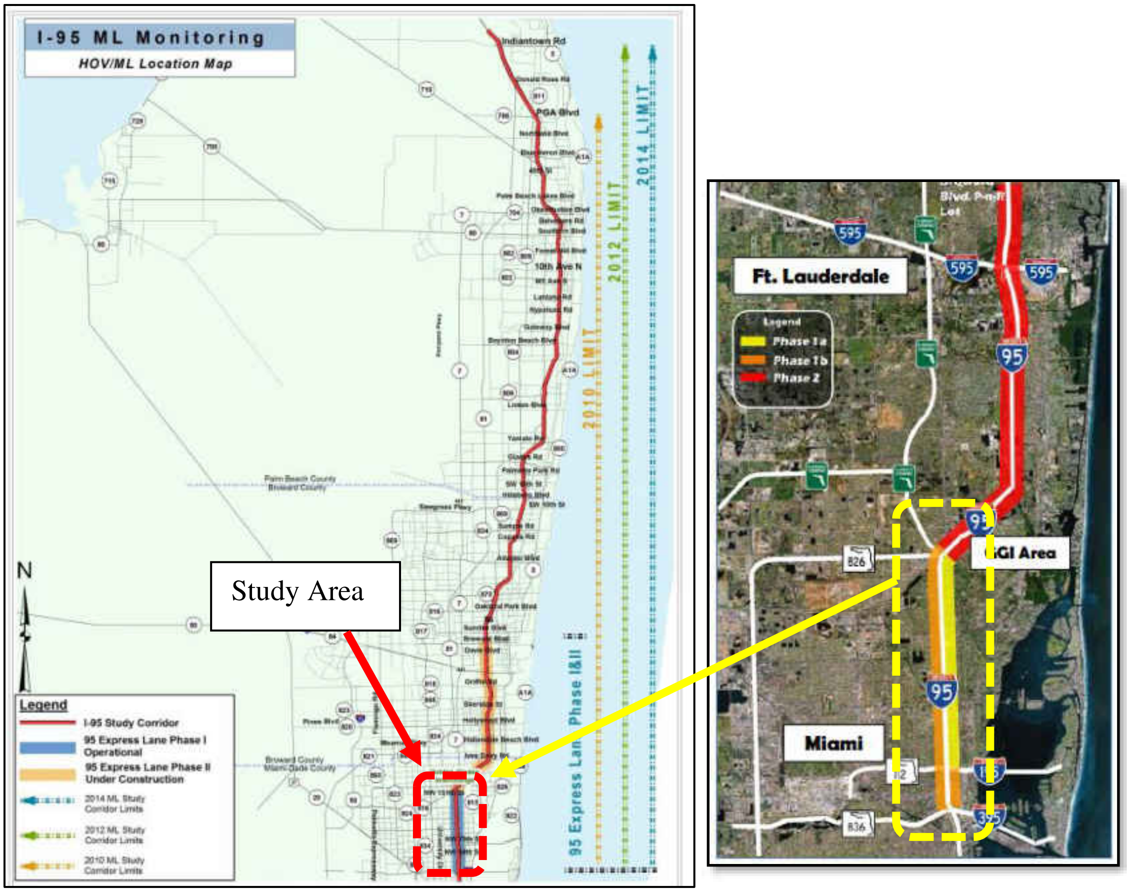



**Figure 8. Location of the existing MLs in I-95. Source: (FDOT, 2012; Systematics, 2014)**

In the simulation, the lane choice replicated drivers' choice behavior at dynamic tolls based on modeling components and algorithms generated in VISSIM. Afterward, the simulated corridor was calibrated and validated using the Regional Integrated Transportation Information System (RITIS) data that was collected from detectors along the corridor. Subsequently, the experimental design was conducted, including various scenarios, which are based on different access levels, access configurations, and traffic periods. The safety performance for various scenarios was evaluated with the Surrogate Safety Assessment Model (SSAM), by analyzing the traffic conflict frequency. 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. Furthermore, the revenue generated by the MLs was also computed. Therefore, the primary objectives of this chapter 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 flow chart of the simulation process is shown in Figure 9. This chapter is composed of six sub-chapters. The first sub-chapter is the introduction. The second sub-chapter is the experimental design and the microsimulation process for the studied network, which mainly includes network building, calibration, and validation. The third sub-chapter shows the principal findings of this safety analysis, while evaluating the operations of different ML designs is introduced in the fourth sub-chapter. Lastly, the final sub-chapter gives a summary and conclusion of the results in addition to discussing the implications of the findings for future research.

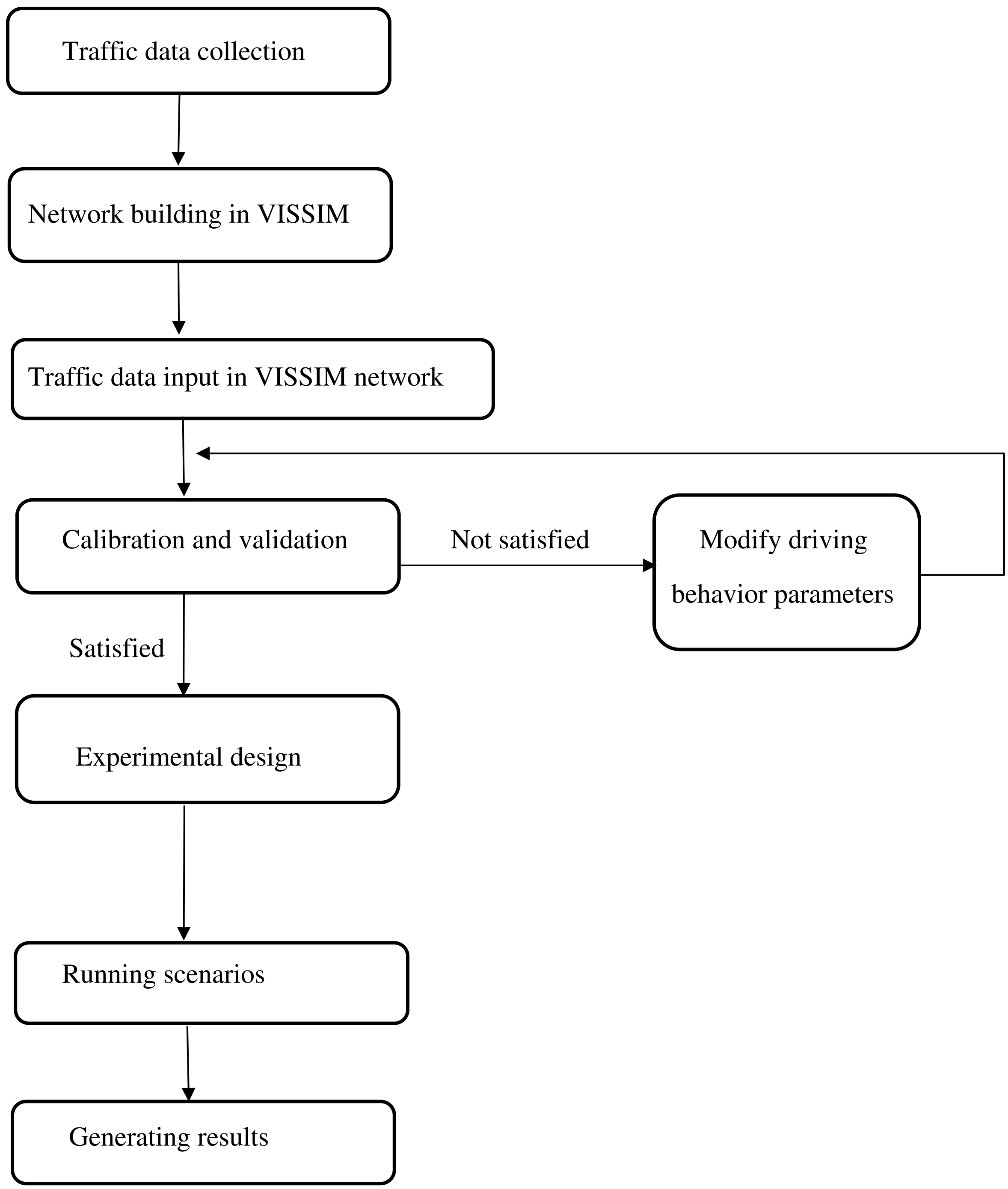



**Figure 9. Simulation process flow chart**

## 3.2 Experimental Design

### 3.2.1 Study Area

The corridor was built in the VISSIM software based on real-world geometric characteristics. The segment that was utilized in VISSIM included 9 mi of GPLs and MLs located on I-95 in Miami, Florida (Figure 10).

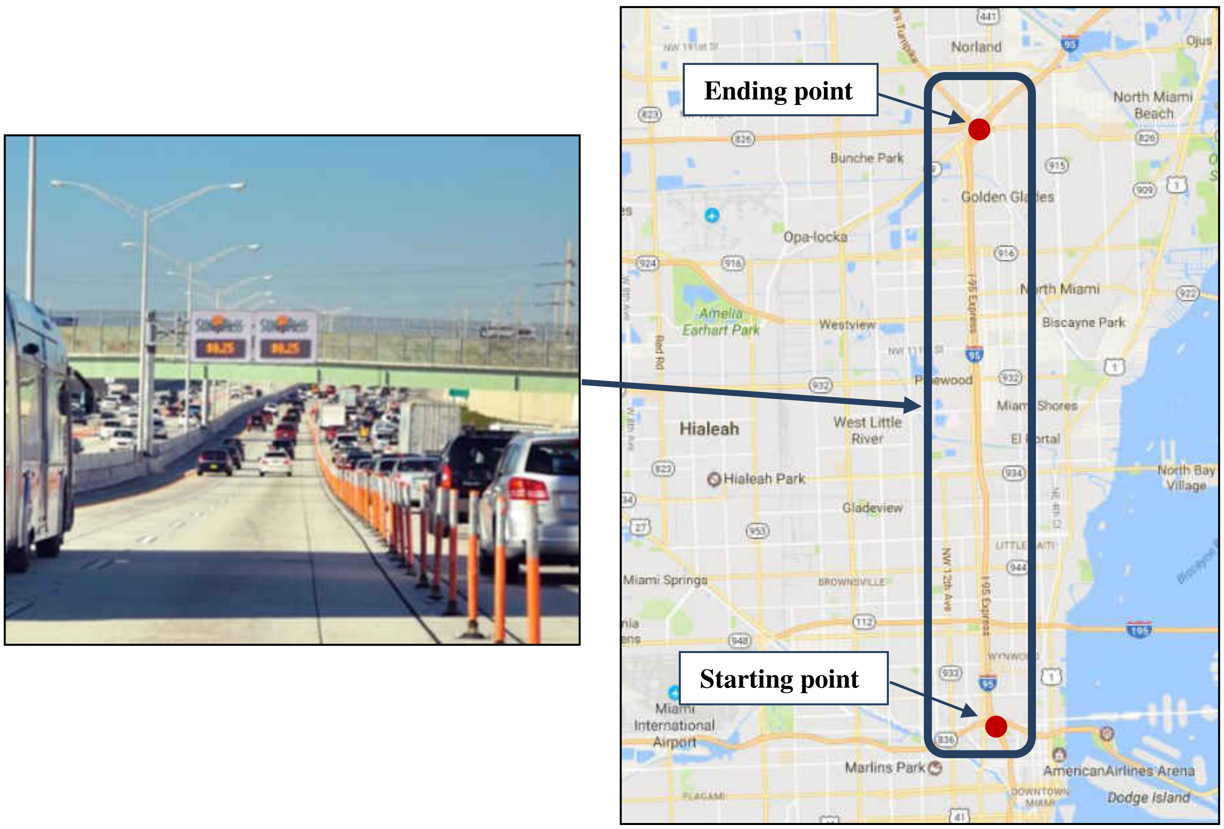



**Figure 10. Study area located on I-95 (Source:** (FDOT, 2017)**, Google maps).**

The network was built in the VISSIM software based on the real-world geometric characteristics. Three types of lanes were built in the VISSIM network, namely, GPLs, MLs, and ramps. Parts of the VISSIM network are shown in Figure 11 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 Figures 11. The geometric properties of each link were adjusted to be consistent with the real network. These properties included link length, the number of lanes, and lane width. Moreover, link behavior type was modified and set to be “Freeway” since the studied segment was on an Interstate.

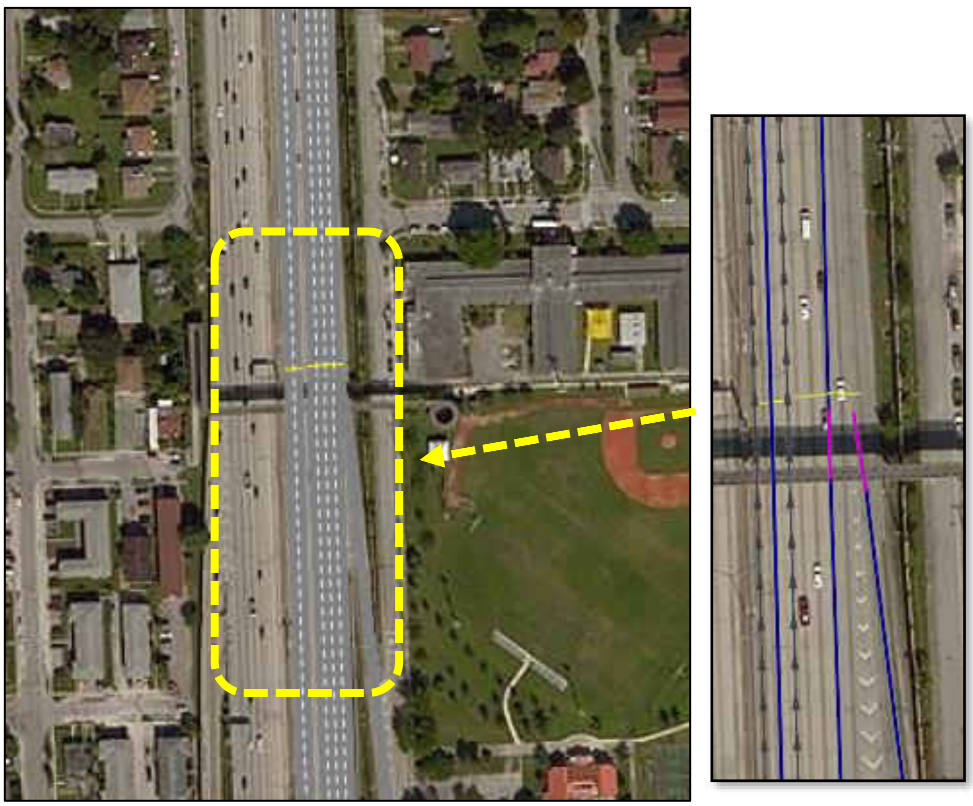


**Figure 11. Part of the VISSIM network (On-Ramp)**

### 3.2.2 Traffic Input

In order to input traffic information to the VISSIM network, the Regional Integrated Transportation Information System (RITIS) data that was collected from detectors along the corridor for April 2016 were used. The data provided detailed traffic information collected by microwave detectors at 20 sec intervals for each lane, including average time, mean speed, volume, and lane occupancy. The traffic data were aggregated to obtain VISSIM traffic input data at 15 min time intervals. Figure 12 shows the average traffic volume for 15 min time intervals for both MLs and GPLs along the studied area. According to the figure, two hours were found to be peak period (7:00 AM to 9:00 AM) and two hours were off-peak period (9:00 AM to 11:00 AM). Traffic data was entered into VISSIM for 15 min time intervals. For both peak and off-peak periods, it was recommended to consider 30 min for a warm-up period at the beginning of the simulation in order to reach a steady-state traffic condition, and 30 min for cool-down period at the end of the simulation (Wang et al., 2017; Wu, 2017; Shalaby et al., 2003). Therefore, after excluding warm-up and cool-down periods, 60 min of simulation was considered in data analyses for each of the peak (7:30 AM to 8:30 AM) and off-peak (9:30 AM to 10:30 AM) periods.

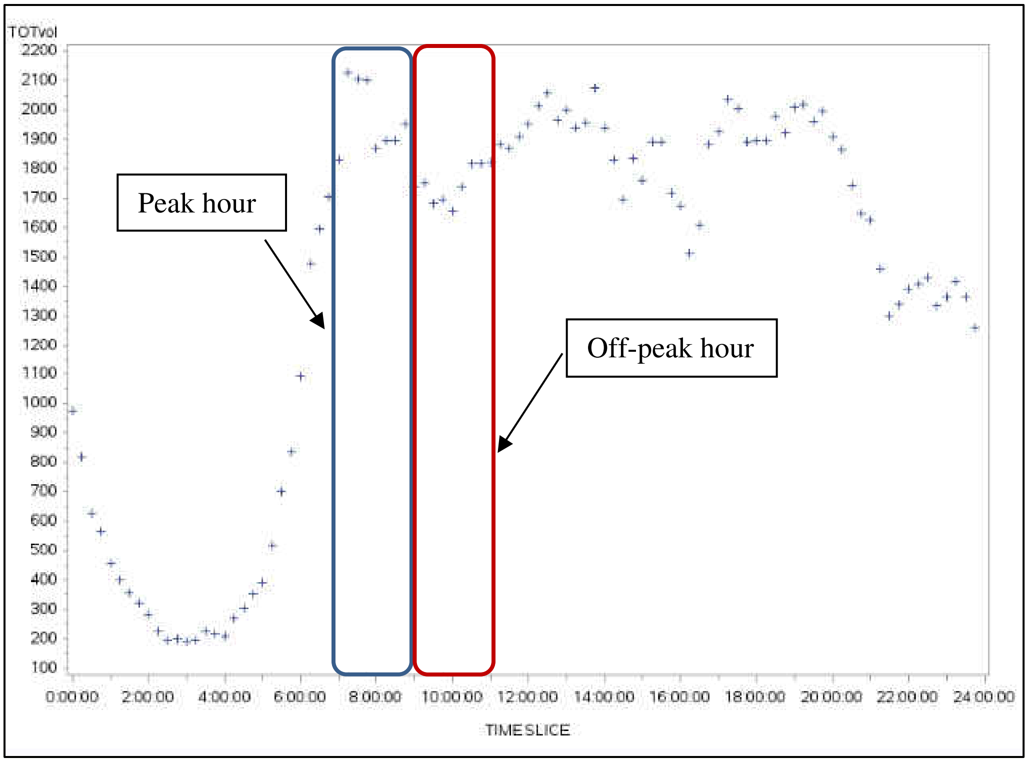



**Figure 12. Volume Distribution**

### 3.2.3 Data Collection Points

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 same exact locations of the Regional Integrated Transportation Information System (RITIS) detectors on I-95. The data collection points were coded in VISSIM for each detector (e.g., G 2763 (1)). 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 following represent the detector name in the RITIS data. Finally, the number in the parentheses is the lane ID. For instance, the lane ID for the right most lane is 1.

### 3.2.4 Simulation Scenarios

The access zones usually form weaving segments, since on-ramp vehicles want to enter the MLs through the ingress and off-ramp vehicles want to exit MLs through the 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. Figure 13 shows the weaving segments where $L_1$ is the ingress weaving segment length and $L_2$ is the length of the egress weaving segment.

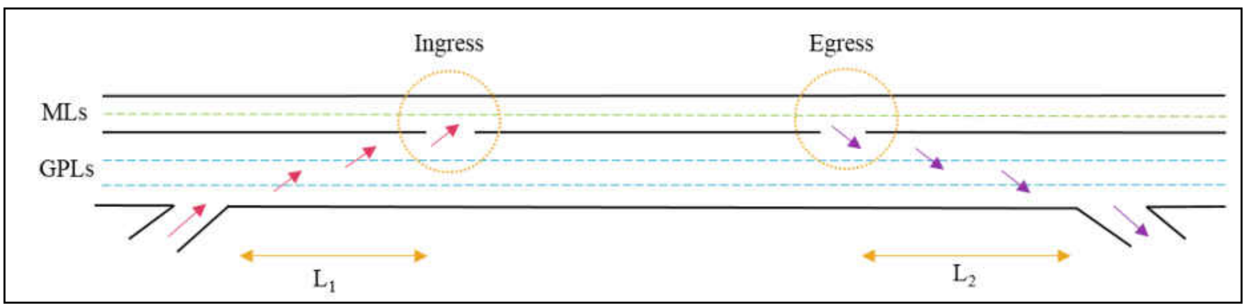



**Figure 13. Weaving segments near access zones**

Previous studies have explored the efficient weaving distance. One of these studies was conducted by the California Department of Transportation (Caltrans, 2011), which suggested a minimum distance of 800 ft per lane change is necessary between the on- or off-ramps and the access zones, as shown in Figure 14.

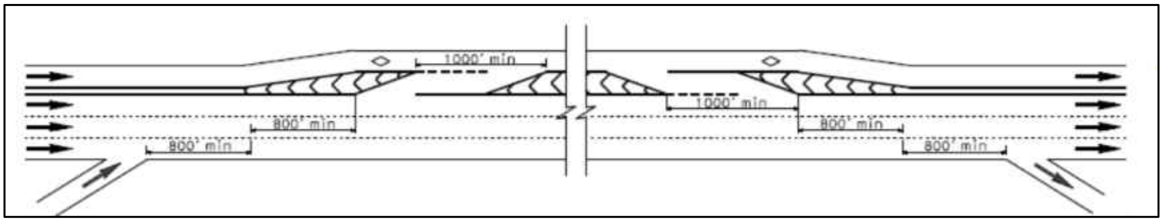



**Figure 14. Minimum weaving distance for access zones (min=minimum). Source: California DOT report, 2011 (Caltrans, 2011)**

Another study conducted by the Washington Department of Transportation (Burgess, 2006) proposed the minimum distance between the access zones and the on- or off-ramps to be 500 ft per lane change. Meanwhile, the study recommended that the desired distance is 1,000 ft per lane change, which is double the minimum distance. Also, another study, conducted by (Venglar et al., 2002), offered that the range of the weaving distance varies between 500 and 1,000 ft. Meanwhile, they concluded that the minimum distance between the ingress and the egress of the MLs was 2,500 ft. Additionally, the NCHRP guidelines for implementing MLs suggested that the spacing between access zones should be between 3 and 5 miles (Fitzpatrick et al., 2017). The ingress and egress design of this study followed the recommendation of the FHWA (FHWA, 2011). The detailed designs for the ingress and egress are shown in Figure 15.

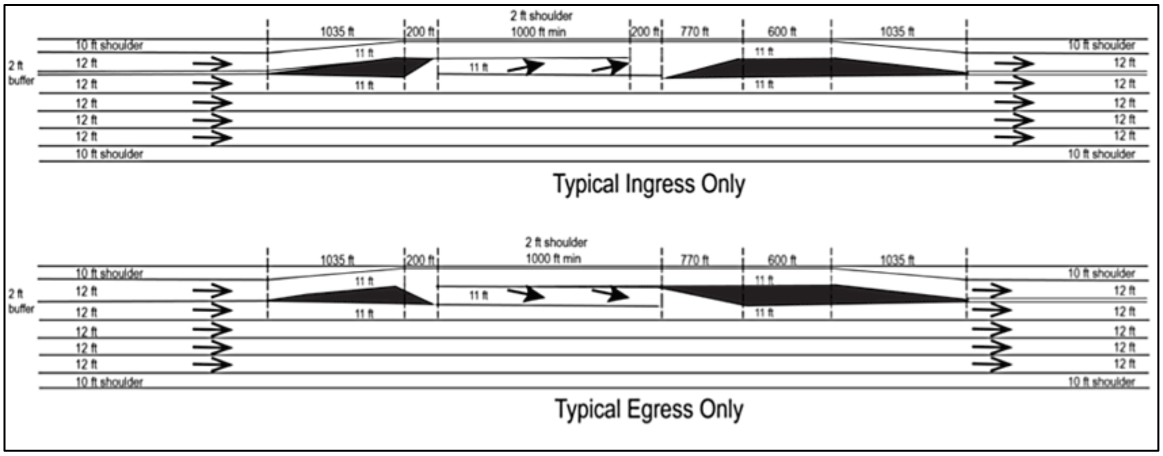



**Figure 15. Ingress and egress details for different cases. Source: (FHWA, 2011)**

Three accessibility levels were tested in this study which included one, two, and three access zones. The base condition is the current situation of the corridor, 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 corridor. Case 2 involves adjusting the corridor to have two ingresses and two egresses, which are located at one-third and two-thirds of the corridor (Figure 16).

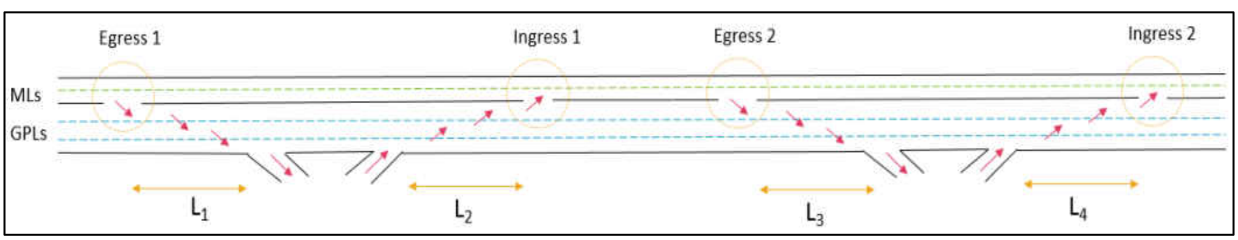



**Figure 16. Weaving Segments for the two accessibility levels case**

Case 3 has three ingresses and three egresses, which are located every quarter of the corridor. The three accessibility cases are shown in Figure 17. In each case, five different weaving lengths were applied including 600 ft, 800 ft, 1,000 ft, 1,400 ft, and 2,000 ft. Meanwhile, two traffic periods (peak and off-peak) were included in the experimental design. Hence, 32 scenarios were tested in VISSIM as shown in Table 2. For each scenario, ten random runs with different random seeds were applied.

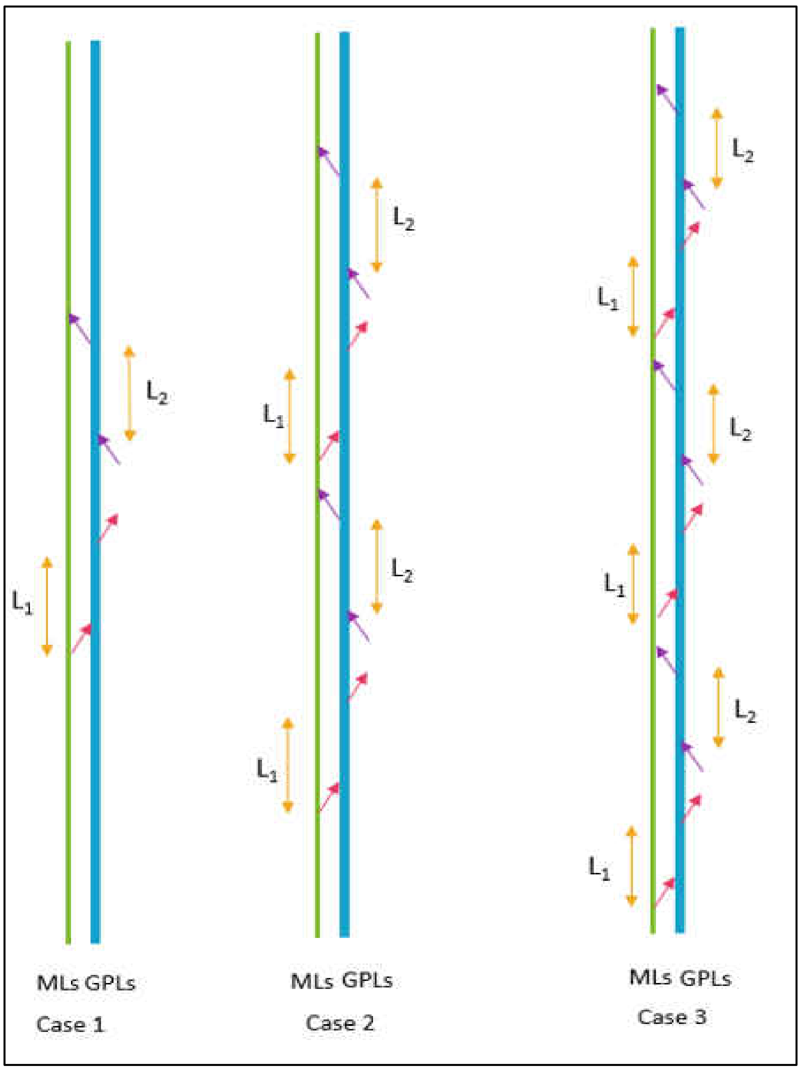



**Figure 17. Accessibility Level Cases**

**Table 2. List of scenarios**

| Cases | Traffic periods | Lane change length between the access zones and the on- or off- ramps (ft)* | | | | | Number of scenarios |
|---|---|---|---|---|---|---|---|
| Base condition | Peak | No access zones | | | | | 1 |
| | Off-peak | No access zones | | | | | 1 |
| Case 1 | Peak | 600 | 800 | 1000 | 1400 | 2000 | 5 |
| | Off peak | 600 | 800 | 1000 | 1400 | 2000 | 5 |
| Case 2 | Peak | 600 | 800 | 1000 | 1400 | 2000 | 5 |
| | Off peak | 600 | 800 | 1000 | 1400 | 2000 | 5 |
| Case 3 | Peak | 600 | 800 | 1000 | 1400 | 2000 | 5 |
| | Off peak | 600 | 800 | 1000 | 1400 | 2000 | 5 |
| Total number of scenarios | | | | | | | 32 |

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

### 3.2.5 Vehicle Classes

Three classes of vehicles were utilized in this simulation: passenger cars (PCs), heavy goods vehicles (HGVs), and carpools. According to (FDOT, 2002), the percentage of HGVs is 5% on freeways. Meanwhile, according to the 2015 U.S. Census American Community Surveys (ACS) for Miami-Dade (ACS, 2015), the percentage of carpools is 10% on freeways. 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 (Joseph, 2013).

### 3.2.6 Vehicle Composition

There are four types of vehicle composition in this study in the trip distribution process. The first type is vehicles that start from the beginning of the corridor and have the choice to use the MLs. The second type is vehicles that start from the on-ramps and 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 type is vehicles that start from the beginning of the corridor and do not have the choice to use the MLs because they exit the network upstream of the access zone. The details of the first three groups are represented in the following sections.

#### 3.2.6.1 Type 1

Type 1 refers to vehicles that come from the beginning of the corridor, which is located upstream of the start of the MLs. This type of vehicle has 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 corridor. The second group is the vehicles that have a choice between the MLs or GPLs and reach the end of the corridor. The third group is the vehicles that use the first MLs egress to exit the corridor 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 3a, 3b, 3c, and 3d. These percentages were calculated and organized based on the field traffic volume (RITIS data), U.S. Census data, and FDOT data.

**Table 3. Vehicle Composition: Type 1**

**(a) Vehicle Composition at the no access zone case**

| | No Access Zones | | |
|---|---|---|---|
| | PCs | Carpools | HGVs |
| Group 1 | 55% | 6% | 3% |
| Group 2 | 30% | 4% | 2% |
| Group 3 | - | - | - |
| Group 4 | - | - | - |
| Group 5 | - | - | - |
| Total | 85% | 10% | 5% |

**(b) Vehicle Composition at the one accessibility level case**

| | One Access Zone | | |
|---|---|---|---|
| | PCs | Carpools | HGVs |
| Group 1 | 47% | 5% | 2% |
| Group 2 | 30% | 4% | 2% |
| Group 3 | 8% | 1% | 1% |
| Group 4 | - | - | - |
| Group 5 | - | - | - |
| Total | 85% | 10% | 5% |

**(c) Vehicle Composition at the two-accessibility level case**

| | Two Access Zones | | |
|---|---|---|---|
| | PCs | Carpools | HGVs |
| Group 1 | 45% | 5% | 2% |
| Group 2 | 30% | 4% | 2% |
| Group 3 | 6% | 0.6% | 0.6% |
| Group 4 | 4% | 0.4% | 0.4% |
| Group 5 | - | - | - |
| Total | 85% | 10% | 5% |

**(d) Vehicle Composition at the three-accessibility level case**

| | Three Access Zones | | |
|---|---|---|---|
| | PCs | Carpools | HGVs |
| Group 1 | 43% | 5% | 2% |
| Group 2 | 30% | 4% | 2% |
| Group 3 | 4% | 0.4% | 0.4% |
| Group 4 | 4% | 0.3% | 0.3% |
| Group 5 | 4% | 0.3% | 0.3% |
| Total | 85% | 10% | 5% |

3.2.6.2 Type 2

Type 2 includes vehicles that come from on-ramps and have the choice of choosing either 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 corridor using the off-ramps; these vehicles do not reach the end of the corridor. The second group is the vehicles that start from the on-ramps, use the MLs, and exit the corridor using the off-ramps. The third group includes the vehicles that reach the end of the corridor and have the choice to use the GPLs or the MLs utilizing the access zones. Table 4 shows the percentages of PCs, carpools, and HGVs for each group.

**Table 4. Vehicle Composition: Type 2**

| 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 corridor
# is the seventh on-ramp that is downstream from the beginning of the corridor.

3.2.6.3 Type 3

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.

## 3.2.7 Trip Distribution

In the trip distribution process, volumes were inputted at the beginning of the network and at each on-ramp, as presented in Table 5. It can be noted from the table that the volumes in the peak hours were higher than the off-peak period. The percentage of vehicles coming from the beginning of the network and heading to the off-ramps is shown in Table 6. Percentages were calculated based on the field volume and used for the static vehicle routes in VISSIM. It can be noted from the table that the volumes of the first off ramp are the highest as it is connected to an interstate (I-195). Additionally, the percentages from the first on-ramp to the off-ramps and to the end of the network is shown in Table 7. The first two off-ramps revealed low percentages (<1%), because not many vehicles enter the highway through an on-ramp and use the following two off-ramps. Similarly, the percentages of the vehicles from the other on-ramps to the network were generated. For the case of access zones, the same percentages were used for the vehicles coming from the beginning of the corridor and from the on-ramps. The vehicles that have the choice to use MLs (i.e., vehicles coming from the beginning of the network or from the on-ramps, vehicles exiting the MLs to the off-ramps or to the end of the corridor, and the carpool vehicles) were controlled based on the logit model of the dynamic toll pricing, as explained in the following section. In VISSIM, managed lanes routing decision panel was used in order to define the routes for the vehicles that have the choice to use the MLs.

**Table 5. Traffic Volumes in VISSIM (Vehicle per Hour)**

| Time | Start of network | On-ramp 1 | On-ramp 2 | On-ramp 3 | On-ramp 4 | On-ramp 5 | On-ramp 6 | On-ramp 7 |
|---|---|---|---|---|---|---|---|---|
| 7:00 | 6621 | 1331 | 373 | 640 | 743 | 902 | 444 | 709 |
| 7:15 | 7009 | 1481 | 436 | 804 | 871 | 1175 | 693 | 867 |
| 7:30 | 7372 | 1716 | 572 | 810 | 788 | 1140 | 570 | 941 |
| 7:45 | 7558 | 1725 | 842 | 995 | 787 | 1224 | 595 | 1181 |
| 8:00 | 7647 | 1683 | 769 | 720 | 697 | 899 | 423 | 1403 |
| 8:15 | 7939 | 1660 | 697 | 740 | 845 | 953 | 407 | 1386 |
| 8:30 | 7526 | 1471 | 547 | 823 | 916 | 935 | 394 | 1128 |
| 8:45 | 7027 | 1688 | 623 | 1019 | 824 | 1045 | 542 | 960 |
| 9:00 | 6357 | 1459 | 363 | 719 | 655 | 866 | 373 | 701 |
| 9:15 | 6570 | 1328 | 339 | 704 | 733 | 833 | 325 | 598 |
| 9:30 | 6272 | 1332 | 337 | 626 | 609 | 735 | 327 | 568 |
| 9:45 | 6185 | 1268 | 301 | 680 | 686 | 819 | 281 | 521 |
| 10:00 | 6460 | 1135 | 309 | 620 | 680 | 770 | 267 | 418 |
| 10:15 | 6623 | 1246 | 268 | 636 | 636 | 847 | 339 | 478 |
| 10:30 | 6689 | 1371 | 310 | 701 | 595 | 899 | 402 | 504 |
| 10:45 | 6831 | 1270 | 405 | 776 | 723 | 922 | 416 | 369 |

**Table 6. Percentages of Vehicles from the Beginning to the Off-Ramps and to the End of the Network**

| | From the beginning of the network to | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Time | Off-ramp 1 | Off - ramp 2 | Off - ramp 3 | Off - ramp 4 | Off - ramp 5 | Off - ramp 6 | Off - ramp 7 | Off - ramp 8 | Off - ramp 9 | End of network |
| 7:00 | 11.61% | 8.11% | 6.42% | 7.24% | 5.14% | 4.46% | 4.55% | 4.06% | 8.58% | 39.83% |
| 7:15 | 10.34% | 8.04% | 6.87% | 7.88% | 5.37% | 6.54% | 3.41% | 5.48% | 9.51% | 36.56% |
| 7:30 | 11.53% | 7.68% | 6.60% | 9.15% | 4.59% | 6.72% | 2.06% | 6.04% | 9.01% | 36.61% |
| 7:45 | 12.08% | 7.60% | 7.09% | 7.45% | 4.53% | 8.23% | 1.84% | 5.37% | 8.98% | 36.83% |
| 8:00 | 12.92% | 8.42% | 6.77% | 8.81% | 4.27% | 5.47% | 0.44% | 3.26% | 9.59% | 40.05% |
| 8:15 | 13.69% | 7.83% | 6.55% | 8.26% | 3.80% | 6.09% | 0.99% | 2.93% | 9.87% | 39.98% |
| 8:30 | 13.18% | 7.90% | 5.99% | 9.50% | 3.69% | 5.94% | 1.35% | 3.10% | 9.79% | 39.55% |
| 8:45 | 12.25% | 7.61% | 6.57% | 8.78% | 4.56% | 7.13% | 1.49% | 4.24% | 9.93% | 37.43% |
| 9:00 | 13.03% | 7.30% | 5.73% | 9.03% | 4.67% | 4.79% | 3.39% | 3.82% | 9.69% | 38.56% |
| 9:15 | 13.40% | 7.04% | 5.09% | 8.85% | 5.21% | 4.46% | 4.09% | 3.43% | 8.94% | 39.49% |
| 9:30 | 12.23% | 7.35% | 5.41% | 9.01% | 4.44% | 4.47% | 4.97% | 4.19% | 8.57% | 39.37% |
| 9:45 | 12.27% | 7.19% | 5.45% | 8.49% | 5.02% | 4.43% | 4.58% | 4.52% | 8.64% | 39.41% |
| 10:00 | 12.88% | 7.13% | 5.19% | 8.65% | 4.94% | 3.89% | 4.56% | 3.58% | 8.31% | 40.86% |
| 10:15 | 12.94% | 7.08% | 5.41% | 7.35% | 6.03% | 4.05% | 4.31% | 3.90% | 8.70% | 40.22% |
| 10:30 | 12.29% | 7.46% | 5.56% | 7.54% | 6.35% | 4.70% | 4.28% | 3.71% | 8.61% | 39.50% |
| 10:45 | 11.75% | 7.21% | 5.82% | 8.59% | 5.52% | 5.48% | 4.25% | 4.21% | 7.92% | 39.25% |

**Table 7. Percentages of Vehicles from the First On-Ramp to the Off-Ramps and to the End of the Network**

| | From the first on-ramp to | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Time | Off - ramp 2 | Off - ramp 3 | Off - ramp 4 | Off - ramp 5 | Off - ramp 6 | Off - ramp 7 | Off - ramp 8 | Off - ramp 9 | End of network |
| 7:00 | 0.07% | 0.19% | 1.34% | 3.06% | 4.20% | 2.70% | 4.27% | 7.02% | 77.15% |
| 7:15 | 0.04% | 0.13% | 1.08% | 2.75% | 4.57% | 3.36% | 5.54% | 8.09% | 74.43% |
| 7:30 | 0.02% | 0.09% | 0.30% | 5.44% | 4.02% | 3.74% | 6.15% | 7.89% | 72.37% |
| 7:45 | 0.02% | 0.26% | 0.35% | 4.12% | 4.10% | 3.07% | 5.75% | 8.14% | 74.19% |
| 8:00 | 0.05% | 0.21% | 0.40% | 5.11% | 3.84% | 2.58% | 3.86% | 8.62% | 75.34% |
| 8:15 | 0.00% | 0.07% | 0.68% | 4.09% | 4.20% | 3.26% | 3.64% | 9.09% | 74.97% |
| 8:30 | 0.02% | 0.12% | 0.95% | 3.89% | 3.91% | 5.18% | 3.63% | 8.54% | 73.77% |
| 8:45 | 0.07% | 0.36% | 1.02% | 4.66% | 3.81% | 3.72% | 4.43% | 8.29% | 73.64% |
| 9:00 | 0.00% | 0.17% | 1.49% | 4.28% | 3.70% | 2.60% | 3.99% | 7.69% | 76.08% |
| 9:15 | 0.00% | 0.07% | 2.07% | 4.18% | 4.16% | 2.36% | 3.70% | 7.14% | 76.31% |
| 9:30 | 0.02% | 0.22% | 1.93% | 4.10% | 3.47% | 2.27% | 4.25% | 6.69% | 77.06% |
| 9:45 | 0.02% | 0.05% | 2.11% | 3.69% | 3.89% | 2.71% | 4.46% | 6.69% | 76.37% |
| 10:00 | 0.05% | 0.15% | 1.88% | 3.91% | 3.91% | 2.59% | 3.81% | 6.57% | 77.13% |
| 10:15 | 0.02% | 0.05% | 1.74% | 2.97% | 4.78% | 2.97% | 4.04% | 6.89% | 76.54% |
| 10:30 | 0.00% | 0.26% | 1.80% | 3.27% | 5.12% | 2.15% | 3.93% | 6.94% | 76.54% |
| 10:45 | 0.00% | 0.12% | 1.67% | 4.34% | 4.60% | 2.23% | 3.50% | 6.59% | 76.96% |

### 3.2.8 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 (PTV, 2015). The DSD has to be inputted 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 by 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 a similar 50th percentile speeds. 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 (Johnson & Murray, 2010) concluded that the average speed difference between cars and trucks was 8.1 miles 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.

### 3.2.9 Dynamic Toll Pricing

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 are as follows:

$$U_{toll} = \beta_{time} \times Time\ gain - \beta_{cost} \times Toll\ rate + Base\ Utility \quad (2)$$

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

The base utility depends on the vehicle class and zero as the default value of the software. The time coefficient ($\beta_{time}$) and the cost coefficient ($\beta_{cost}$) were calculated from the Value of Time (VOT). The ratio of the cost coefficient and the time coefficient ($\beta_{time}$) was utilized to define the VOT as follows:

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

In this study, the VOT was assumed to be \$8.67 per hr based on the result of a multinomial logit model conducted by Jin et al. (2015) (Jin et al., 2015). The time coefficient was assumed to be one min 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 implies an increase in the MLs utility with the decrease of the tolls. The toll price is mainly affected by two components. First, the time saved by using the MLs, which varied from 0 to 8.50 min. Second, the speed in the MLs, which was between 30 mph and 73.50 mph. The dynamic toll prices varied between a minimum value of \$0.50 and a maximum value of \$10.50.

## 3.3 Calibration and Validation

### 3.3.1 Car Following Model

In order to validate the VISSIM network, different CC values were used for GPLs and MLs based on the Wiedemann 99 car following model, which has ten car following parameters (CC0 to CC9). Previous studies defined CC parameters as follows: CC0 is the average standstill distance between two vehicles; CC1 is the following headway time between two vehicles; CC2 defines the following distance variation in the oscillation condition; CC3 is the threshold to enter the following condition; CC4 and CC5 are the parameters that control vehicle speed oscillation; CC6 is the distance influence on speed oscillation; CC7 is the acceleration at the oscillation condition; CC8 represents the standstill acceleration; lastly, CC9 is the acceleration at 50 mph (Koppula, 2002; Sajjadi & Kondyli, 2017; Zhizhou et al., 2005).

Sajjadi et al. (2017), conducted a study for the same study corridor in Miami, South Florida. They proposed CC values for one and two HOT lanes segments separated by flexible pylons. For the GPLs corridor, the CC values were used based on the Wiedemann 99 car following model in VISSIM for freeways including, CC0=1.50, CC1=0.9, CC2=4.00, CC3=-8.00, CC4=-0.35, CC5=0.35, CC6=11.44, CC7=0.25, CC8=3.5, and CC9=1.5. For the MLs corridor, CC values for the two lanes were used as follows: CC0=4.92, CC1=1.9, CC2=39.37, CC4=-0.70, CC5=0.70, and other CC values were similar to the GPLs case (Sajjadi & Kondyli, 2017). The parameters of the car following behavior model in VISSIM is shown in Figure 18.

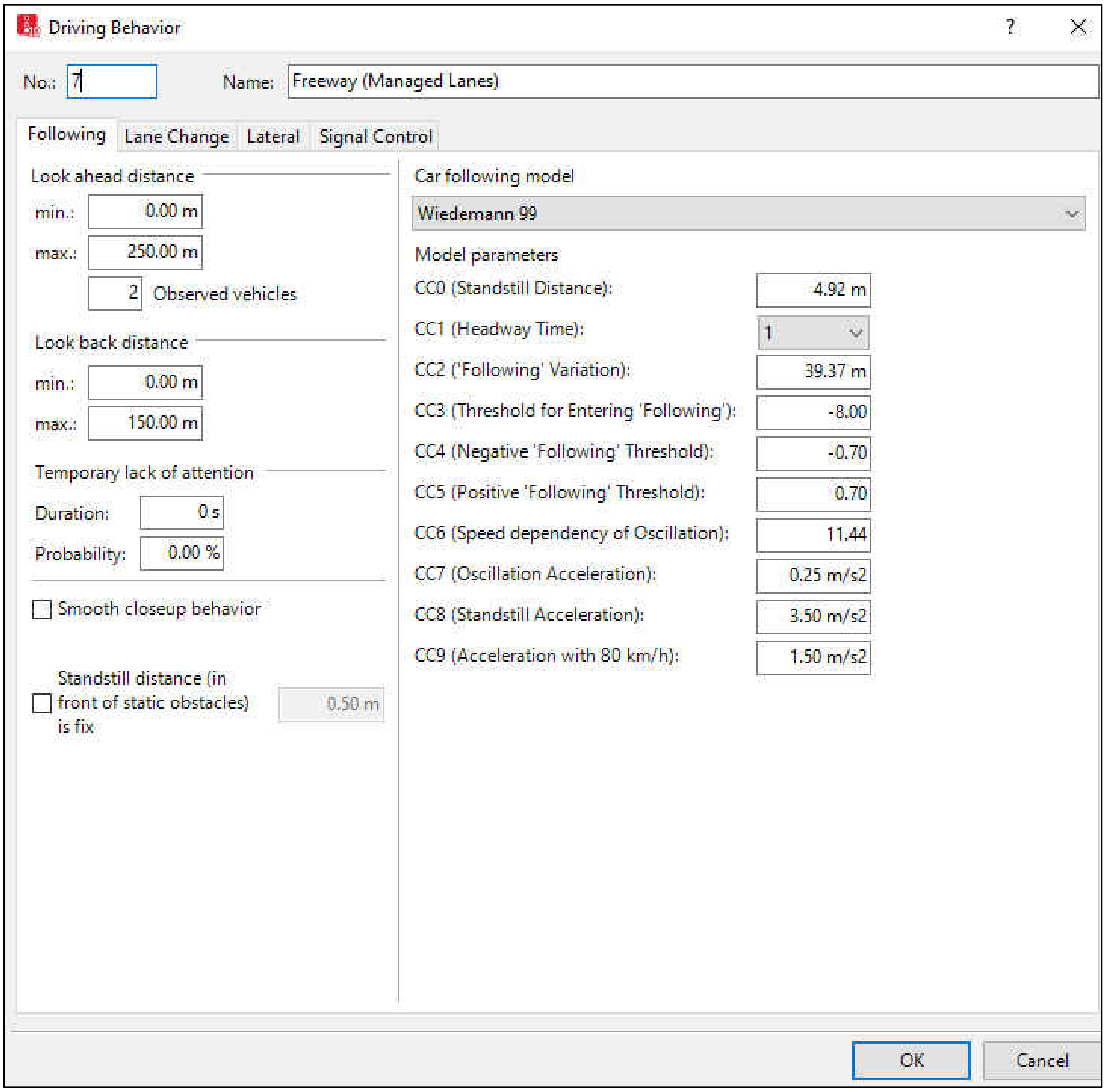



**Figure 18. Car Following Behavior Model Parameters for MLs in VISSIM**

### 3.3.2 Lane Change Parameters

In general, lane changes are classified into two types: discretionary lane change (DLC), and mandatory lane change (MLC). The discretionary lane change (DLC) happens when a vehicle desires to increases its speed to either pass another vehicle or to have more following distance. In this study, vehicles have the choice to use MLs based on the time saved and the toll pricing. According to Zhizhou et al. (2005), DLC is one of the most important parameters when studying weaving segments. The mandatory lane change (MLC) occurs when a vehicle tries to exit the freeway through off-ramps. In the simulation, emergency stop distance was used to determine the distance before off-ramps in order to provide a safe distance for vehicles to start changing lanes to exit the freeway (Zhizhou et al., 2005). The distance was defined as 1,000 m (3,280 ft) before off-ramps for a safe lane change (ACS, 2015). Consequently, vehicles did not stop before the off-ramps to change lanes and exit the freeway.

The discretionary lane change (DLC) in the weaving segments is calibrated using the lane change parameters that are provided in the driving behavior panel in VISSIM. The general behavior was used as a free lane selection, so vehicles may overtake each other on each lane. The maximum deceleration and the accepted deceleration represent the upper and the lower bound of deceleration. The default values were used for both parameters. The maximum deceleration was defined as -4 m/sec$^2$ for the own overtaken vehicle and -3 m/sec$^2$ for the trailing vehicle. The accepted deceleration was defined as -1 m/sec$^2$ for the own overtaken vehicle and -0.5 m/sec$^2$ for the trailing vehicle. The change of deceleration (in m per -1 m/sec$^2$) was defined as 200 m for the own overtaken and the trailing vehicles. The waiting time before diffusion, which is the maximum time for any vehicle to wait before changing lanes, was determined to be 30 sec for weaving segments (Yang et al., 2012). Meanwhile, the minimum headway, which is the minimum distance that must be available between vehicles after changing lanes, was defined as 0.6 m for weaving segments (Jolovic & Stevanovic, 2012). The safety distance reduction factor, which depends on the safety distance of the lane changer and the trailing vehicle, was defined as 0.6. The maximum deceleration for cooperative braking was defined as the default value (-3 m/sec$^2$). The option of the advanced merging was selected so vehicles can change lanes earlier. Hence, vehicles were less likely to stop for a gap before a lane change and therefore would increase the capacity. Also, the vehicle routing decisions look ahead was activated so vehicles can identify new routing decisions on the same segment and take all choices into account when changing lanes. The parameters of the lane change behavior model in VISSIM is shown in Figure 19.

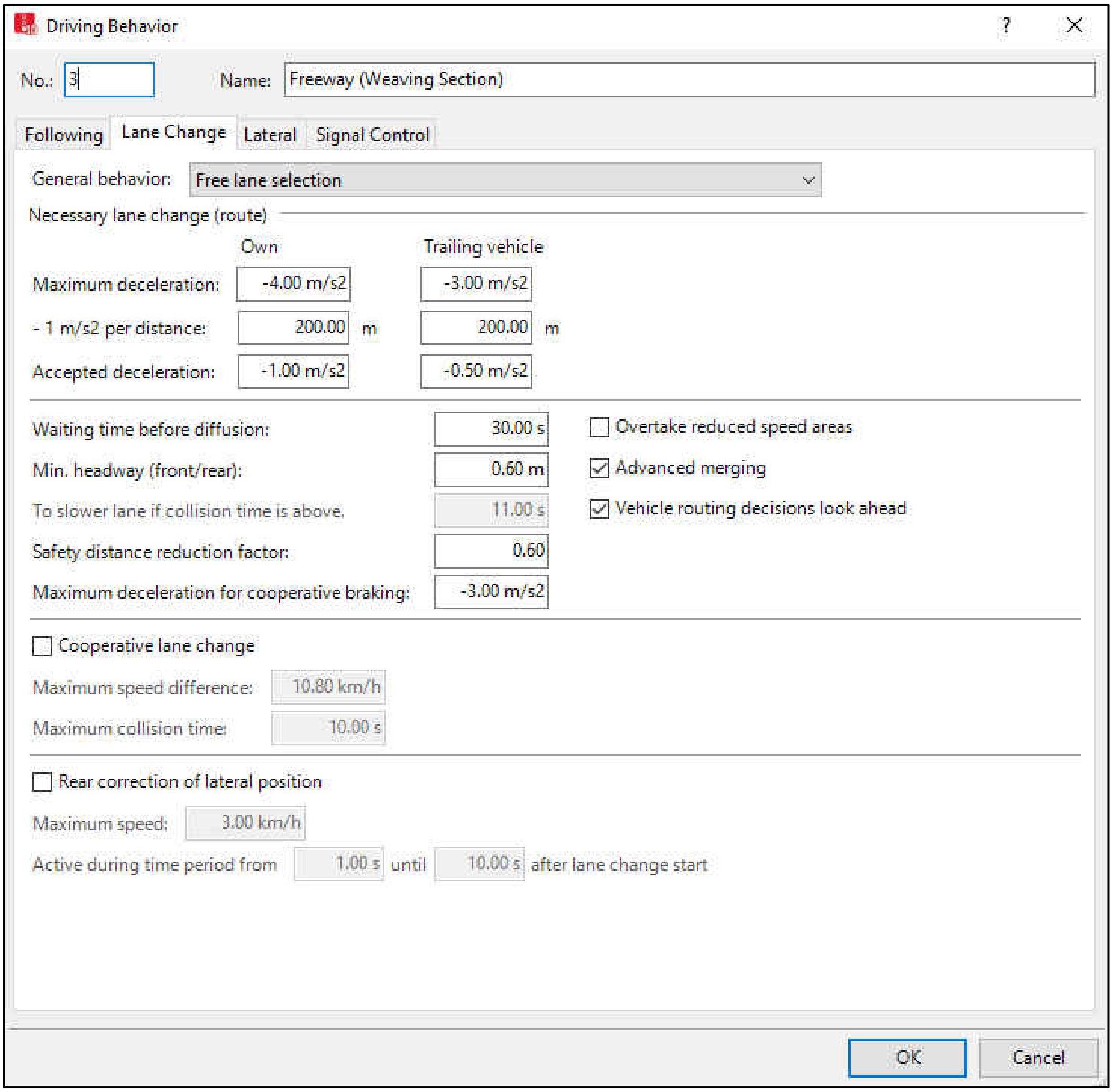



**Figure 19. Lane Change parameters for Weaving Sections in VISSIM**

Previous studies utilized traffic data for validating VISSIM networks that analyzed weaving segments (M. Abdel-Aty & Wang, 2017; Jolovic & Stevanovic, 2012; Sajjadi & Kondyli, 2017; Ling Wang et al., 2017; Wei & Wanjing, 2013; Williams et al., 2010; Yang et al., 2012). A study completed by Jolovic and Stevanovic (2012), investigated the lane changes of weaving segments in VISSIM. The authors calibrated the lane change parameters according to waiting time before diffusion, minimum headway, necessary lane change distance and accepted deceleration (Jolovic & Stevanovic, 2012). In the validation process, the authors compared traffic flows and average speeds, which were used at 15-min time intervals, between the simulation data and the field data. Another study conducted by Abdel-Aty and Wang (2017), studied the safety of weaving segments after implementing active traffic management techniques (i.e., variable speed limit, ramp metering) (M. Abdel-Aty & Wang, 2017). The authors validated the simulated network using the field traffic speeds. Another microsimulation study was conducted by Yang et al. (2012), for investigating the capacity of weaving segments on urban expressways. The network was calibrated using several parameters for car following behavior (i.e., average standstill distance, safety distance, etc.) and lane change behavior (i.e., maximum deceleration, minimum headway, etc.) (Yang et al., 2012). The authors validated the weaving segments in VISSIM using the maximum traffic flow of the expressway. Hence, due to the difficulties of validating the lane change behavior with the field lane change data, previous studies utilized traffic data for the validation process of the weaving segments. In this study, the validation of the VISSIM network was implemented using traffic data, as shown in the following section.

### 3.3.3 Traffic Volume and Speed

After the construction of the VISSIM network, calibration and validation are crucial to the process. A comparison between the VISSIM simulated traffic and the field traffic was conducted. If the difference between the two sets of data was found to be significant, the simulation network could not be utilized to represent the field corridor. Therefore, only after the successful calibration and validation of the simulation network, could the simulation network be employed for further applications. A total of 180 min (from 7:30 AM to 10:30 AM) of VISSIM data were used in the calibration and validation process after excluding 30 min of warm-up time and 30 min 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 adapted (Dowling et al., 2004). 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 corridor. The formula of GEH value is as follows:

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

where E is the traffic volume for the simulated network and V is the traffic volume of the field corridor. If the value of GEH is less than 5, it indicates that the difference between the simulated volume and the field volume 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 (Chu & Yang, 2003; Yu & Abdel-Aty, 2014). 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 (Bhouri et al., 2013; Nezamuddin et al., 2011).

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 8 and 9, respectively. For the calibration process, the average GEH was 2.39 and the average percentage of GEHs less than five was 91.08%. For the validation process, the average absolute speed difference was 1.9 mph, and the average percentage of absolute speed differences lower than 5 mph was 95.56%. Consequently, the VISSIM network was satisfactorily calibrated and validated.

**Table 8. Calibration Results**

| 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 9. Validation Results**

| 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 |

## 3.4 Safety Analysis

### 3.4.1 Surrogate Safety Measurements

The Surrogate Safety Assessment Model (SSAM) was adopted to determine the potential conflict frequency, which is highly correlated with the crash frequency in the field (Shahdah et al., 2014). The main objective of SSAM could be to either evaluate the safety performance of the current roadway designs or used as a new strategy for monitoring theoretical roadway designs before implementation (Gettman et al., 2008). Three types of conflicts can be extracted from SSAM, which include: rear-end, lane change, and crossing conflicts. Two types of conflicts were used in this paper: rear-end and lane-change conflicts. As provided by SSAM, the rear-end conflicts were considered when the conflict angle was between 0 and 30 degrees, while the lane-change conflicts were defined as when the conflict angle was between 30 and 80 degrees. The crossing conflicts were excluded from this study since the percentage of crossing conflicts was less than 1%, and crossing crashes are less likely to happen on freeways. Figure 20 shows the conflict angle diagram in SSAM.

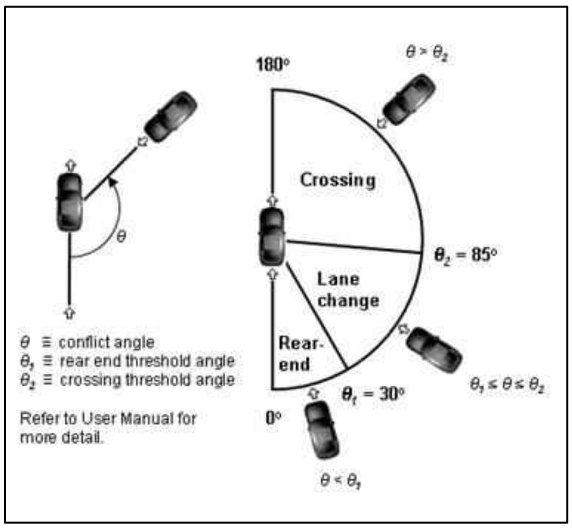



**Figure 20. Conflict Angle Diagram in SSAM**

### 3.4.2 Conflict Validation

Traffic conflict is identified as an evasive action (e.g., braking, deceleration, jerking, etc.) that occurs when two or more vehicles approach each other (Perkins & Harris, 1968; Tageldin et al., 2015). The use of simulated conflicts is a promising approach for estimating safety performance (Gettman et al., 2008; Sayed & Zein, 1999; Shahdah et al., 2014). In order to validate the simulated conflicts, several studies compared the conflict frequency with either the crash data or the field conflicts. Previous research has investigated the relationship between simulated conflicts and field conflicts for the purpose of validating the safety data of the simulation and to recommend countermeasures for reducing crashes (Shahdah et al., 2014; van der Horst et al., 2014). Some studies validated the simulated conflict frequency from SSAM using field conflict data in order to evaluate the accuracy of SSAM. It was proven that there is a significant correlation between the observed conflicts and the simulated conflicts extracted by SSAM (Huang et al., 2013; Roach et al., 2015; Vitale et al., 2017).

On the other hand, several studies validated the simulated conflicts with field crashes (Al-Ghandour et al., 2011; Caliendo & Guida, 2012; Gettman et al., 2008; Saleem et al., 2014; Shahdah et al., 2014). Gettman et al. (2008) by comparing the number of simulated conflicts versus actual crash frequencies occurring at intersections. It was concluded that there was a significant correlation (Spearman rank correlation coefficient was 0.463) between the simulated conflicts and the field crash frequencies (Gettman et al., 2008). Al-Ghandour et al. (2011), tested the relationship between the simulated conflicts and the actual crashes by utilizing the goodness of fit coefficients (Al-Ghandour et al., 2011). The $R^2$ was equal to 0.7, which supported the significant relationship between simulated conflicts and field crashes. Similarly, Caliendo and Guida (2012), developed a model for estimating crashes for each peak hour as a function of simulated conflicts; the results showed a significant correlation between the conflicts from microsimulation and field crashes (Caliendo & Guida, 2012). Another study conducted by Saleem et al. (2014), proved that conflict frequency is a significant variable when analyzing crash types and crash severities (Saleem et al., 2014). Shahdah et al. (2014), used simulated conflicts for determining crash modification factors (CMFs) and compared it with the actual CMFs based on the empirical Bayes (EB) method for the same study area. It was found that CMFs from simulated conflicts are consistent with CMFs from field crashes (Shahdah et al., 2014).

In this study, the traffic conflict in VISSIM was validated using actual crash data which was collected from Signal Four Analytics (S4A) for three years (2015-2017) due to the rarity of crash events during the studied period. The crashes were collected from 7:00 AM to 11:00 AM. Along the studied corridor, forty-four segments were identified including GPLs, MLs, and ramps. A Spearman rank correlation coefficient of 0.663 and a P-value<0.0001 at a 95% confidence interval strongly suggests that there is a significant correlation between the field crashes and the simulated conflicts along the studied corridor. Figure 21 shows the comparison between the simulated conflicts and the actual crashes for the segments of the studied corridor.

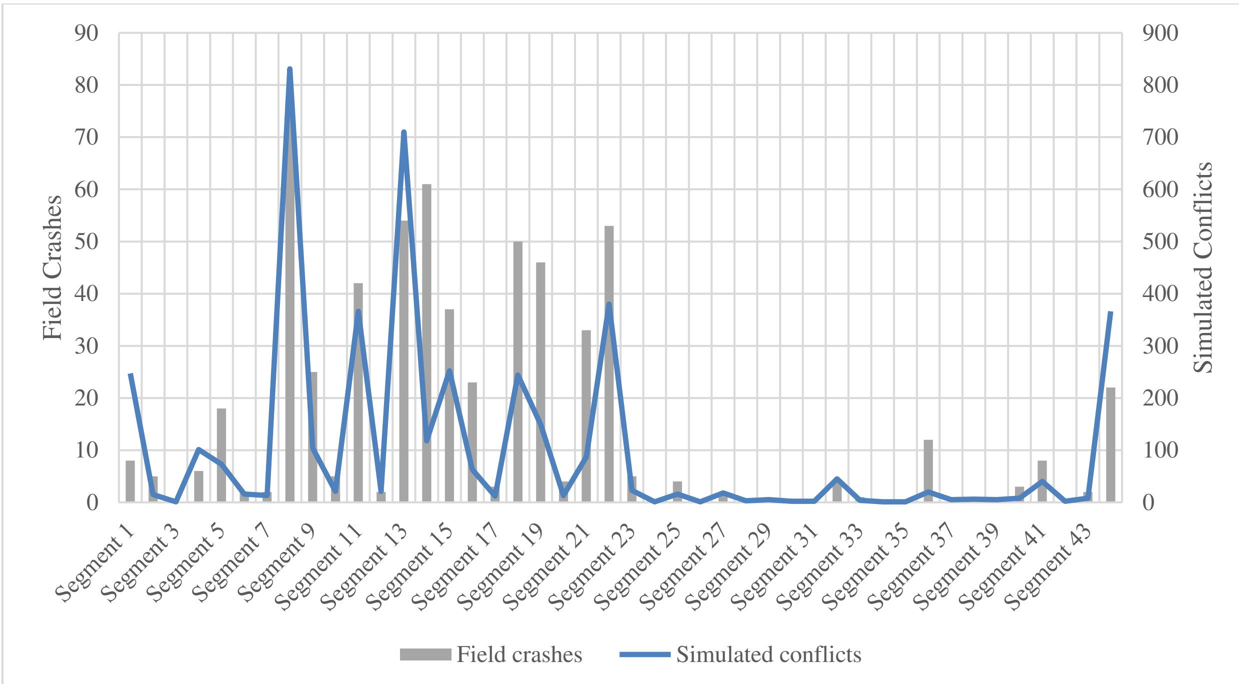



**Figure 21. Comparing simulated conflicts with field crashes**

### 3.4.3 SSAM Results

The vehicle trajectory files from VISSIM were imported into SSAM to obtain the detailed information of the conflicts. In each simulation run, there were “virtual” crashes with a time-to-collision (TTC) of zero. These observations might lead to the inaccuracy of the simulation models (Gettman et al., 2008). Consequently, the cases in which the TTC=0 (crash) were excluded before implementing statistical analysis. Five surrogate measurements were extracted from SSAM to evaluate the safety of the corridor including TTC, Post-Encroachment Time (PET), Maximum Speed (MaxS), the difference in vehicle speeds (DeltaS), and Maximum Deceleration (MaxD).

According to FHWA (Gettman et al., 2008), TTC is the minimum time-to-collision, which is calculated based on the speed and location of vehicles. The FHWA report recommended maximum critical value for TTC as 1.5 sec. It was stated that conflicts with TTC values larger than 1.5 sec are not recognized as a severe condition. As the TTC value increased, the conflict risk was found to decline (Sayed & Zein, 1999). Additionally, the FHWA report suggested a minimum TTC value of 0.1 sec. Several studies used the same threshold (0.1 sec to 1.5 sec) as severe conflicts (Saleem et al., 2014; Saulino et al., 2015; Ling  Wang et al., 2017). PET is the minimum post-encroachment time, which is defined as the time between two vehicles to occupy the same point. The maximum value of PET was determined to be 5.0s for identifying a conflict. The high risk occurred when the PET value decreased (Jeffrey Archer & Kosonen, 2000). MaxS is the maximum speed for any of the two vehicles that participated in the conflict. DeltaS is the difference in speed between the vehicles in the conflict. MaxD is the maximum deceleration of a vehicle to avoid the conflict with the other vehicle (Gettman et al., 2008). The high risk is associated with lower MaxD (Jeffery Archer, 2004).

The descriptive statistics of the surrogate measures for the base condition are shown in Table 10 for both peak and off-peak periods. An ANOVA test was carried out to compare the surrogate measures in MLs and GPLs for the whole segment. The results showed that TTC (F-value=13.24, p-value=0.0003) and PET (F-value=35.66, p-value<0.0001) were higher in the MLs, which indicated that MLs were safer than GPLs. The percentage of different values of TTC for MLs and GPLs is shown in Figure 22. Meanwhile, the maximum speed of any vehicle participated in the conflict was higher in the MLs than the GPLs (F-value=61.98, p-value<0.0001). Figure 23 presents the percentage of different values of maximum speed for MLs and GPLs. Compared to MLs, GPLs had lower conflict risk with higher MaxD (F-value=6.75, p-value=0.0096). Another significant result was that GPLs had higher conflict angle than MLs (F-value=18.8, p-value<0.0001). This result could be due to the higher number of lane-change conflicts to rear-end conflicts in GPLs than MLs. Additionally, the results showed no significant difference in DeltaS (F-value=0.04, p-value=0.8476) between MLs and GPLs.

**Table 10. 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 |

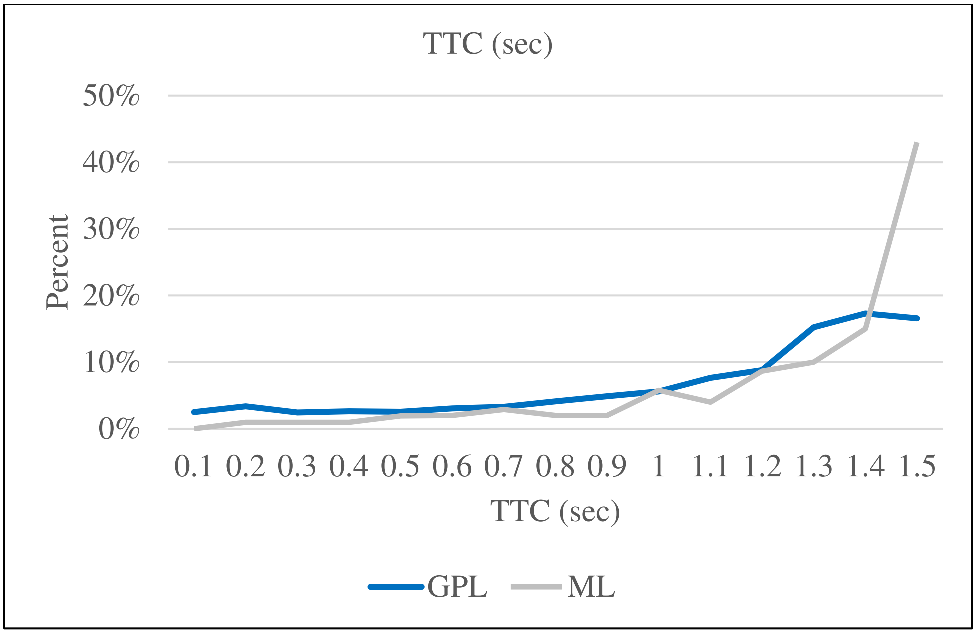



**Figure 22. TTC chart for GPLs and MLs**

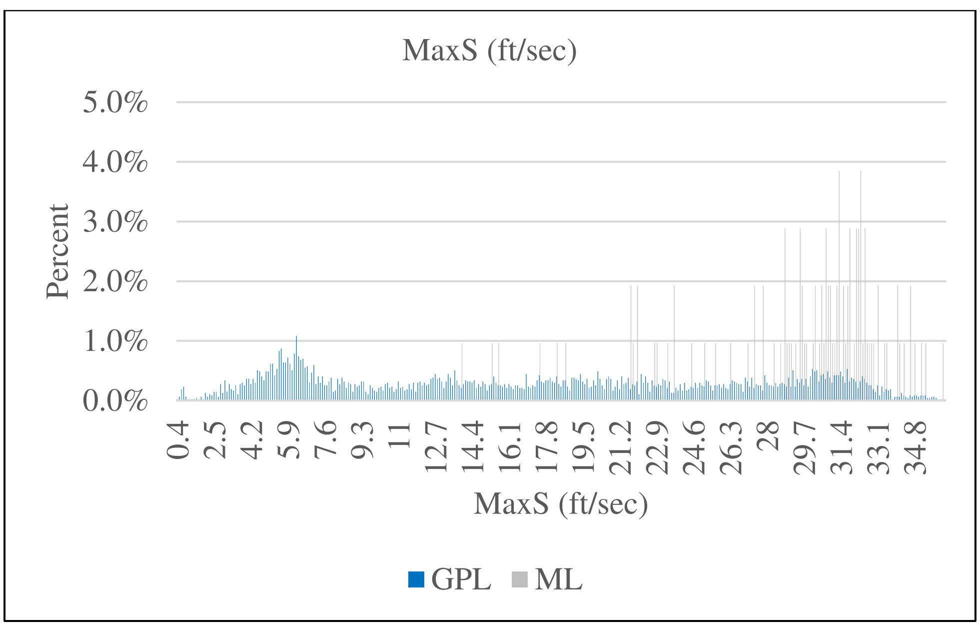



**Figure 23. MaxS chart for GPLs and MLs**

### 3.4.4 Conflict Results

#### 3.3.4.1 Conflict Rate for Base Condition

In the peak period, the conflict frequency in GPLs is higher than the MLs by 78% (82% higher for lane-change conflicts and 72% higher for rear-end conflicts). In the off-peak period, the conflict frequency in GPLs is higher than the MLs by 54% (80% higher for lane-change conflicts and 33% higher for rear-end conflicts).

When taking the volume of GPLs and MLs into account, conflict rate can be calculated by dividing the number of conflicts over the total number of vehicles. It was found that the conflict rate in GPLs was higher than MLs by 48% and 11% in the peak and the off-peak periods, respectively. This higher conflict frequency and conflict rate in GPLs as compared to MLs is due to the frequent lane changing of vehicles near the access zone area on GPLs, which can generate both lane-change and rear-end crashes. Also, the conflict rate is higher in the peak period than the off-peak period by 68% in GPLs and 45% in MLs.

#### 3.3.4.2 Conflict Rate for Access design condition

In the case of access design, the conflict rate was identified to compare the safety effectiveness among different scenarios with various accessibility levels and weaving lengths. Conflict rate was calculated for weaving segments near access zones as follows:

$$Conflict\ Rate = \frac{Conflict\ Frequency}{(Volume\ (vehicle/hour) * Weaving\ Segment\ Length\ (mile))} \tag{6}$$

Figure 24 shows the comparison of conflict rate among various cases of weaving lengths and accessibility levels for the peak and the off-peak periods. The mean of the conflict rate was found to be 14.77 and 9.10 conflicts per 1,000 vehicle-mi per hr for the peak and the off-peak periods, respectively. It is worth mentioning that the conflict rate decreased in the off-peak period compared to the peak period. Closer inspection of the figure shows that the weaving length of 1,000 ft per lane change had the lowest conflict rate among all other lengths in both peak and off-peak periods. For the locations where ramp density is low, the 1,000 ft 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 involved in the entering or exiting MLs. Hence, longer distance might result in an unsafe situation. Additionally, the highest conflict rate occurred when the weaving length was 600 ft.

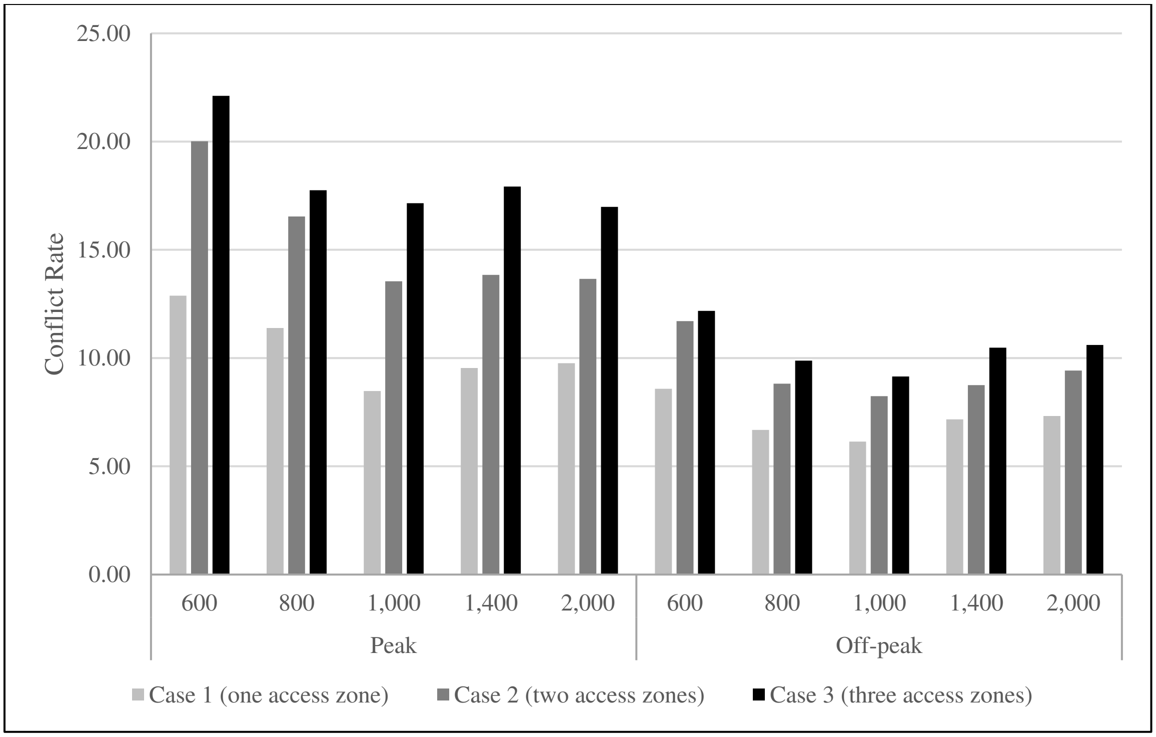



**Figure 24. Conflict rate for various weaving lengths (conflict/ 1,000 vehicle-mile per hour)**

3.3.4.3 Log-Linear Model

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 design that would minimize 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 periods) and two-way interactions. The statistical analysis software (SAS 9.4) was used for generating the model results employing CATMOD procedure. The model formulation is as follows:

$$Log\, m_{ijk} = \alpha + \lambda_i^x + \lambda_j^y + \lambda_k^z + \lambda_{ij}^{xy} + \lambda_{jk}^{yz} + \lambda_{ik}^{xz} \qquad (7)$$

where $Log\ m_{ijk}$ is the log of the expected frequency when *i, j,* and *k* are the categories of x, y, and z; $\alpha$ is the overall effect; $\lambda_i^x$ is the effect due to the *i*th level of the weaving length; $\lambda_j^y$ is the effect due to the *j*th level of the accessibility level ;$\lambda_k^z$ is the *k*th level of the traffic periods ;$\lambda_{ij}^{xy}$ is the interaction of the weaving length at the *i*th level and the accessibility level at the *j*th level; $\lambda_{jk}^{yz}$ is the interaction of the accessibility level at the *j*th level and the traffic periods at the *k*th level ; $\lambda_{ik}^{xz}$ is the interaction of the weaving length at the *i*th level and the traffic periods 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 a 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) implies that the model of two-way interactions was well-fitted. Additionally, traffic volume was used as an exposure measure in the conflict frequency model. The model was developed and compared with the conflict frequency model. The results showed that the conflict frequency model with exposure has higher likelihood ratio ($G^2$=19.331, d.f.=14, p-value=0.096) than the studied model. Hence, the conflict frequency model provides better results than the exposure based model and it can be used to investigate the association between the three categorical variables using the odds multipliers (M. A. Abdel-Aty et al., 1998).

The odds multipliers represent the probability of the occurrence of an event relative to another event. It can be calculated from Eq. (6) for main and interaction effects. Eq. (7) 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, Eq. (8) was formulated when x=$i$, y=$j$, z=$k$ instead of z=1. The results of the model are shown in Table 11.

$$\frac{m_{ijk}}{m_{i1k}} = \exp[\left(\lambda_j^y - \lambda_1^y\right) + \left(\lambda_{ij}^{xy} - \lambda_{i1}^{xy}\right) + \left(\lambda_{jk}^{yz} - \lambda_{1k}^{yz}\right)] \tag{8}$$

$$\frac{m_{ijk}}{m_{ij1}} = \exp[(\lambda_k^z - \lambda_1^z) + (\lambda_{ik}^{xz} - \lambda_{i1}^{xz}) + \left(\lambda_{jk}^{yz} - \lambda_{j1}^{yz}\right)] \tag{9}$$

The results of the log-linear model for the access design safety were consistent with both the Tobit model and the Negative Binomial model. The odds multiplier was used in the log-linear model 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 (one ingress and one egress) had lower odds multipliers than the cases of two and three access densities. One accessibility level was shown to be the safest option in a 9 mi corridor. Additionally, from the second part of Table 11 (Weaving length × Traffic periods), it is apparent that the odds multipliers at the off-peak period are lower than that of the peak period. Hence, drivers tend to have lower conflicts in the off-peak period than peak period.

**Table 11. Comparison of Odds Multipliers of Conflict Frequency between Various Cases**

| 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: An odds multiplier more or less than 1 implies higher or lower likelihood of conflict frequency, respectively, than the baseline (Numbers between Parentheses Are The 90% Confidence Interval).

* Base condition.

Furthermore, the results of the table revealed that the weaving length of 1,000 ft per lane change had significantly lower odds multipliers ($\alpha$=0.10), compared to all other weaving lengths. Therefore, it can be inferred 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 length was confirmed by the findings of the Washington State Department of Transportation (WSDOT) (Burgess, 2006). Lastly, from the results, the most dangerous cases, with higher odds multipliers, occurred when the weaving length was 600 ft per lane change. This outcome supports the findings from the California Department of Transportation (Caltrans), which recommends a minimum distance of 800 ft per lane change (Caltrans, 2011).

In the cases of two and three accessibility levels, the weaving lengths of 1,400 ft and 2,000 ft per lane change had higher conflicts than the other cases. This situation occurred due to the overlap between weaving segments, which created a considerable number of conflicts at this area, as shown in Figure 25. Hence, longer lane change distance does not necessarily guarantee safer conditions due to the effects of the overlapping.

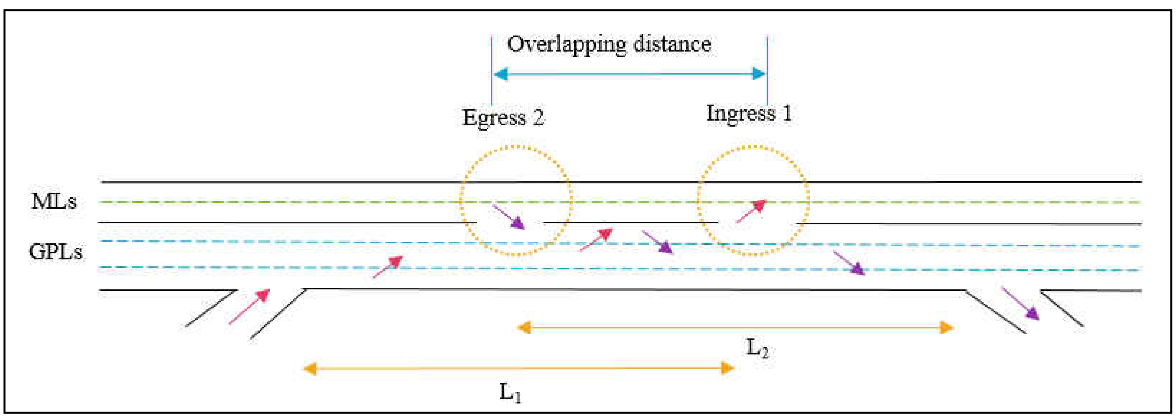



**Figure 25. The overlapping between access openings**

The study suggested one accessibility level (one ingress and one egress) for a 9 mi of MLs. However, there was insufficient evidence to decide the optimal spacing between access openings. The major problem of the MLs opening design is that each agency had different considerations (i.e., policy decisions, operational effectiveness, and safety performance). The latest guidelines of implementing managed lanes by NCHRP (Fitzpatrick et al., 2017) suggested that similar considerations must be applied for the access design. The NCHRP report defined the spacing between access openings to be 3 to 5 mi, based on the existing locations of MLs. Additionally, the NCHRP report recommends following the same principals of the AASHTO Green Book (AASHTO, 2011; Highway & Officials, 2011) for the spacing between freeway access points, which can be applicable for MLs. Chapter 10 of the Green Book suggested a minimum spacing of 1 mi in order to provide a safe weaving length and to leave a sufficient space for the signage. The Nevada Department of Transportation 2014 provides a desirable minimum distance of 2 mi between access openings, based on weaving analysis (NDOT, 2014). Another study by NCHRP (Ray et al., 2011) recommended analyzing the sequence of access points using microsimulation tools. Hence, further research needs to be done in order to provide a comprehensive, definitive conclusion for the access spacing design in MLs.

3.3.4.4 Tobit Model

In this step, statistical modeling was applied to quantify the effect of contributing factors on access zone safety effectiveness measures (i.e., conflict rate, speed SD, and TTC) in the weaving segments (Anastasopoulos et al., 2008; Cai, Abdel-Aty, et al., 2018; Cai, Saad, et al., 2018; McDonald & Moffitt, 1980; M. Saad, Abdel-Aty, Lee, & Cai, 2019; M. Saad, Abdel-Aty, et al., 2018a, 2018b; M. Saad, Abdel-Aty, Lee, & Wang, 2019; M. A.-A. Saad, Mohamed; Lee, Jaeyoung; Wang, Ling 2018). A series of Tobit models were used for identifying the optimal accessibility level and weaving length scenarios that maximize the safety performance at the studied section. In the Tobit model, 15 different scenario variables of various access control levels and configurations were included in the model. In addition, traffic condition (peak, off-peak) and the location of access zones (i.e., entrance, exit) were included in the analysis. The statistical analysis software (SAS 9.4) was used for generating the model results. The model formulation takes the following form:

$$y_i = \begin{cases} y_i^*, & if\ y_i^* > 0 \\ 0, & if\ y_i^* \le 0 \end{cases} \tag{10}$$

$$y_i^* = \beta_0 + \beta_z X + \varepsilon_i \tag{11}$$

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. $\beta_0$ is the intercept, $\beta_z$ represents the coefficients of the independent variables; $\varepsilon_i$ is a normally distributed error term with a mean equal to 0 and a variance ($\alpha^2$); 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 results of the models are shown in Table 12.

**Table 12. Tobit Models for the Safety Measures**

| Parameter | Conflict rate | | Speed SD | | TTC | |
|---|---|---|---|---|---|---|
| | Estimate | p-value | Estimate | p-value | Estimate | p-value |
| Intercept | 22.387 | <0.0001 | 6.981 | <0.0001 | 0.978 | <0.0001 |
| Case 1, 600 feet | -5.371 | 0.0005 | -4.128 | <0.0001 | 0.218 | <0.0001 |
| Case 1, 800 feet | -6.251 | <0.0001 | -4.404 | <0.0001 | 0.232 | <0.0001 |
| Case 1, 1000 feet | -9.442 | <0.0001 | -5.254 | <0.0001 | 0.243 | <0.0001 |
| Case 1, 1,400 feet | -8.206 | <0.0001 | -5.111 | <0.0001 | 0.296 | <0.0001 |
| Case 1, 2,000 feet | -7.756 | <0.0001 | -4.881 | <0.0001 | 0.1832 | <0.0001 |
| Case 2, 600 feet | -3.093 | 0.0449 | -1.581 | 0.0031 | 0.068 | 0.0246 |
| Case 2, 800 feet | -3.313 | 0.0317 | -1.466 | 0.0059 | 0.103 | 0.0007 |
| Case 2, 1,000 feet | -4.779 | 0.0019 | -1.471 | 0.0057 | 0.101 | 0.0009 |
| Case 2, 1,400 feet | -2.993 | 0.0523 | -0.795 | 0.1353 | 0.041 | 0.1771 |
| Case 2, 2,000 feet | -2.706 | 0.0792 | -0.598 | 0.2606 | 0.036 | 0.2368 |
| Case 3, 600 feet | 1.226 | 0.4266 | -0.573 | 0.2814 | 0.041 | 0.1738 |
| Case 3, 800 feet | -0.661 | 0.6683 | -0.762 | 0.1519 | 0.046 | 0.126 |
| Case 3, 1,000 feet | -1.521 | 0.3242 | -0.455 | 0.3922 | 0.046 | 0.1279 |
| Case 3, 1,400 feet | 0.0348 | 0.9821 | -0.318 | 0.5497 | 0.025 | 0.3984 |
| Case 3, 2,000 feet | Reference | | | | | |
| Entrance (v.s. Exit) | -1.465 | 0.0093 | -0.467 | 0.0161 | -0.006 | 0.5598 |
| Off-peak (v.s. Peak) | -9.528 | <0.0001 | -0.871 | <.0001 | 0.0573 | <0.0001 |
| α | 2.181 | <0.0001 | 0.7523 | <.0001 | 0.0431 | <0.0001 |
| R-squared | 0.31 | | 0.57 | | 0.55 | |

Based on the results of the Tobit models for the safety measures, one access zone case had a significantly lower conflict rate, lower speed SD, and higher TTC than other cases. The cases of two and three accessibility levels had higher conflict rate, speed SD, and TTC since more openings created considerably more conflicts in weaving segments. Therefore, safety analysis showed that one access zone was the optimal level of accessibility in a 9-mile network. It can also be inferred from the results of the conflict rate, and SD speed models that 1,000 feet per lane change was the optimal weaving length design from the ramps to the access zones. The results of TTC recommends a weaving distance of 1,400 feet per lane change. Hence, a weaving length between 1,000 feet and 1,400 feet per lane change is recommended, which would maintain a safe lane maneuver from the ramps to the access zones. For the locations where ramp density is low, a weaving distance of 1,000 feet per lane change should be the minimum. But for locations where ramp density is high, the longer distance might result in plenty of ramp traffic involved in entering or exiting the MLs. Hence, longer distances may result in more traffic conflicts. The result of the weaving length confirmed the findings of the Washington State Department of Transportation (WSDOT) *(Burgess, 2006),* which recommended a weaving distance of 1,000 feet per lane change.

It is also worth noting that the highest conflict rate occurred when the weaving length was 600 feet. Similarly, for the case of one access zone, the highest speed and SD and lowest TTC occurred at the case of a weaving length of 600 feet per lane change. This outcome supports the findings from the California Department of Transportation (Caltrans), which recommended a minimum distance of 800 feet per lane change (Caltrans, 2011). Regarding the traffic condition, it is worth mentioning that the off-peak period had a significantly lower conflict rate, lower speed

SD, and higher TTC compared to the peak period. Hence, more attention should be paid to the peak conditions. Lastly, it is apparent from the table that the weaving segments after the exit are more likely to have less conflict rate and less speed SD than the weaving segments near the entrance of MLs. Nevertheless, there is no significant difference between the TTC at weaving segments near the entrances and the exits of MLs (M. Saad, 2016; M. Saad, Abdel-Aty, & Lee, 2018).

## 3.5 Operation Analysis

The traffic operation measurements 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), and average delay.

### 3.5.1 Average Travel Speed

#### 3.5.1.1 Traffic Speed Data Analysis

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 26 that travel average speed increases dramatically in the MLs in both peak and off-peak conditions by 12.4% and 8.1%, respectively.

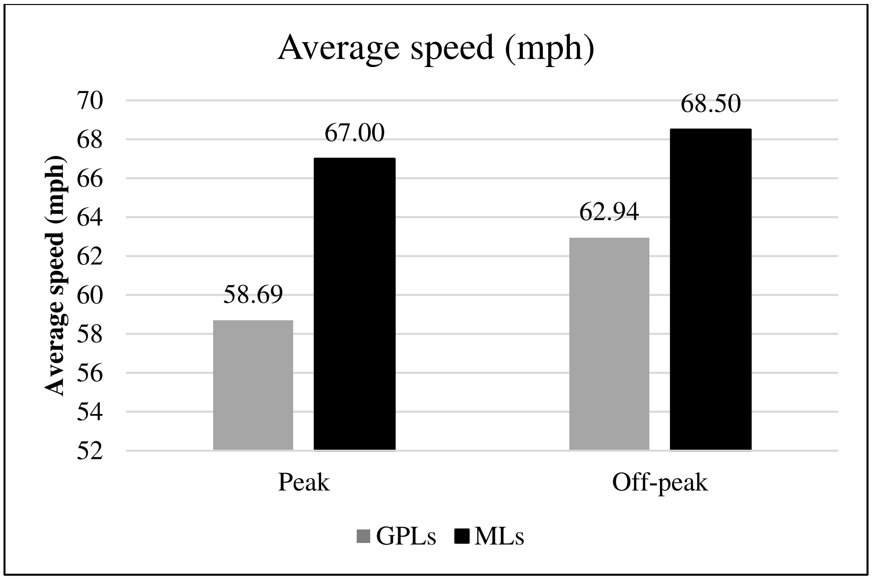



**Figure 26. Travel speed of GPLs and MLs for base condition**

The results of travel speed for the different access zone designs speed for the peak and the off-peak conditions stand out 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. The results also showed that the case of one access zone has higher speeds that the cases of two and three access zones. Figure 27 presents the comparison between travel speed in Case 1 between GPLs and MLs in different traffic conditions. Closer inspection of the figure shows that travel speed was the highest in the MLs in the off-peak conditions. Also, it can be noticed that the average speeds increase when the weaving length is 1,000 ft per lane change or more in the GPLs for the peak and off-peak condition. Similarly, for the MLs case, the average speed increases when the weaving length is 1,000 per lane change or more in both peak and off-peak condition.

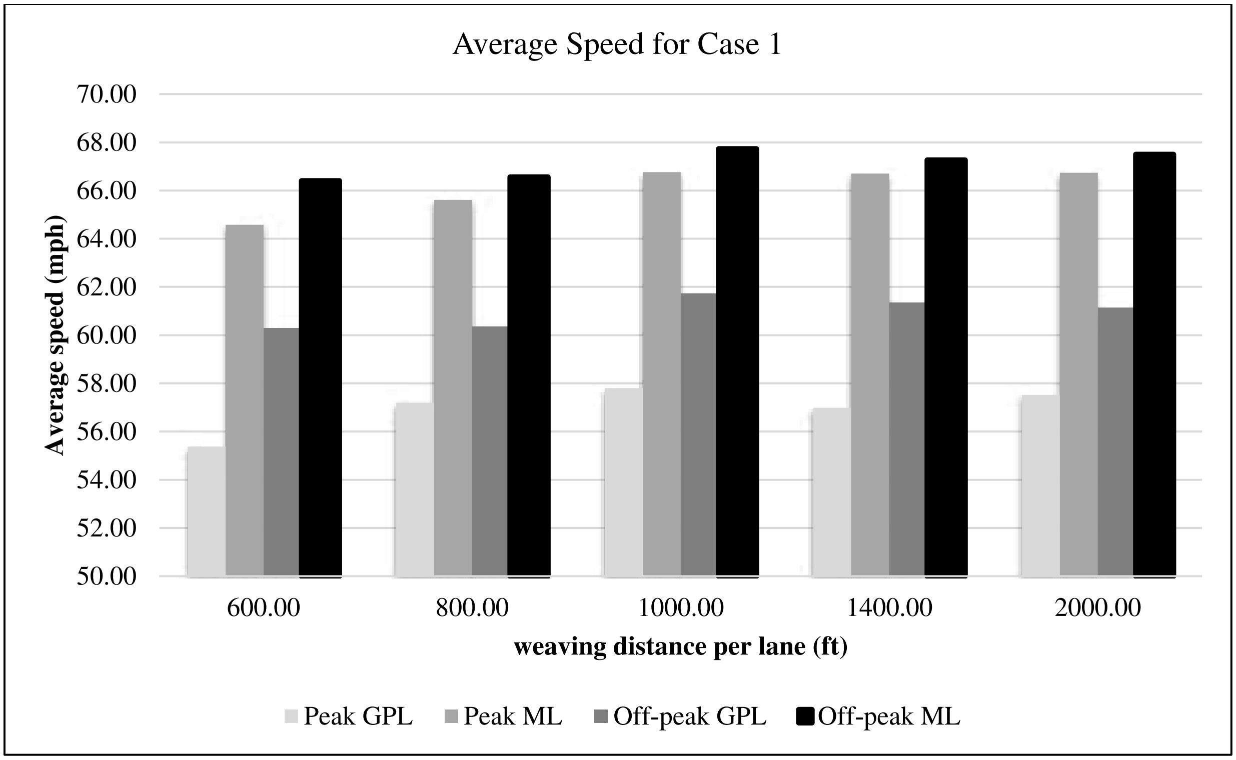



**Figure 27. Comparing Average Speed among One Access Zone Cases**

3.5.1.2 Post-Hoc Test for Speed

Average travel speed was one of the measurements of effectiveness that was used to evaluate the performance of the network and was used to compare the average travel speeds between different cases in the system. For the access zone cases, one-way analysis of variance (ANOVA) was conducted to test the effect of the accessibility level for average speeds at weaving segments. The results showed that there was a significant difference between accessibility levels (F-value=18.43, p-value<0.0001). A post-hoc test was conducted to test the significant difference between the accessibility levels, as shown in Table 13. The results showed that there was a significant difference in travel speeds between all cases. Specifically, one access zone cases had significantly higher travel speeds than cases of two or three access zones. Also, it was found that the case of two access zones had significantly higher speeds than the three access zones case. Hence, one access zone is the most recommended accessibility level among all cases. Regarding the weaving length, the ANOVA analysis was carried out, and it was found that there was a significant difference in the weaving lengths in the case of one access zone (F-value=4.56, p-value=0.0131). The results of post-hoc test (Table 14) showed that the weaving length of 600 feet had a significantly lower speed than the cases of 1,000, 1,400, and 2,000 feet per lane change. Also, it can be noted that a weaving length of 800 feet had a significantly lower speed than the cases of weaving lengths of 1,000, 1,400, and 2,000 feet per lane change. Also, it was found that there was no significant difference between cases of 1,000, 1,400, and 2,000 feet per lane change. Moreover, from the ANOVA analysis, it was found that travel speeds were significantly higher in off-peak conditions than peak conditions (F-value=9.75, P-value=0.0028).

**Table 13. Post-Hoc Test Results of Average Speed for Accessibility Levels**

| Parameter | Attributes | | Average Speed Estimate | Average Speed p-value |
|---|---|---|---|---|
| Case | One access zone | Two access zones | 5.714 | 0.0015 |
| | One access zone | Three access zones | 10.373 | <0.0001 |
| | Two access zones | Three access zones | 4.659 | 0.0086 |

**Table 14. Post-Hoc Test Results of Average Speed for Weaving Lengths**

| Parameter | Attributes | | Average Speed Estimate | Average Speed p-value |
|---|---|---|---|---|
| Length (feet) | 600 | 800 | -3.667 | 0.2814 |
| | 600 | 1,000 | -10.385 | 0.0064 |
| | 600 | 1,400 | -11.168 | 0.0039 |
| | 600 | 2,000 | -10.061 | 0.0079 |
| | 800 | 1,000 | -6.717 | 0.0586 |
| | 800 | 1,400 | -7.501 | 0.0373 |
| | 800 | 2,000 | -6.392 | 0.0704 |
| | 1,000 | 1,400 | -0.782 | 0.8148 |
| | 1,000 | 2,000 | 0.325 | 0.9224 |
| | 1,400 | 2,000 | 1.107 | 0.7405 |

## 3.5.2 Average Delay

3.5.2.1 Traffic Delay Data Analysis

The average delay of all vehicles can be measured by subtracting the theoretical travel time from the actual travel time. The theoretical travel time is the free flow travel time. 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 28.

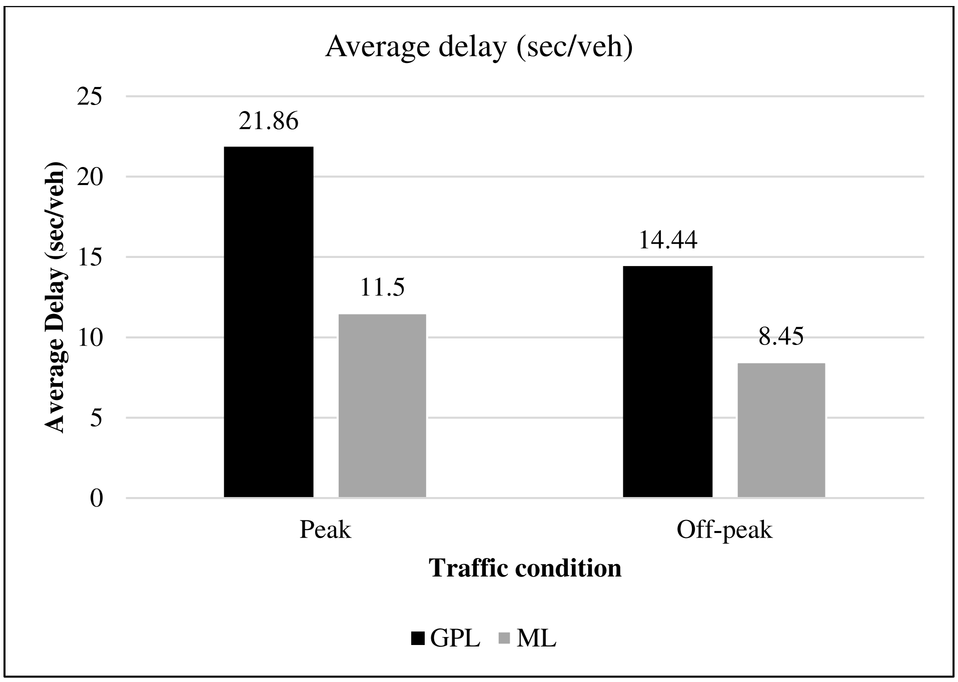



**Figure 28. Average delay for the base case**

When comparing the delay for the whole network it can be concluding that there is a clear trend of average delay declining in the case of one access zone. Also, the lowest delay occurred in the cases of weaving distance of 1,000 ft. Closer inspection of the average delay in Case 1, as shown in Figure 29, it is apparent that the minimum delay happened when the weaving distance was 1,000 ft or more. In general, the average delay improved in the MLs than the GPLs. One access zone with a minimum weaving distance of 800 ft per lane change is suggested and a distance of 1,000 ft per lane change is the common recommendation among other studied distances.

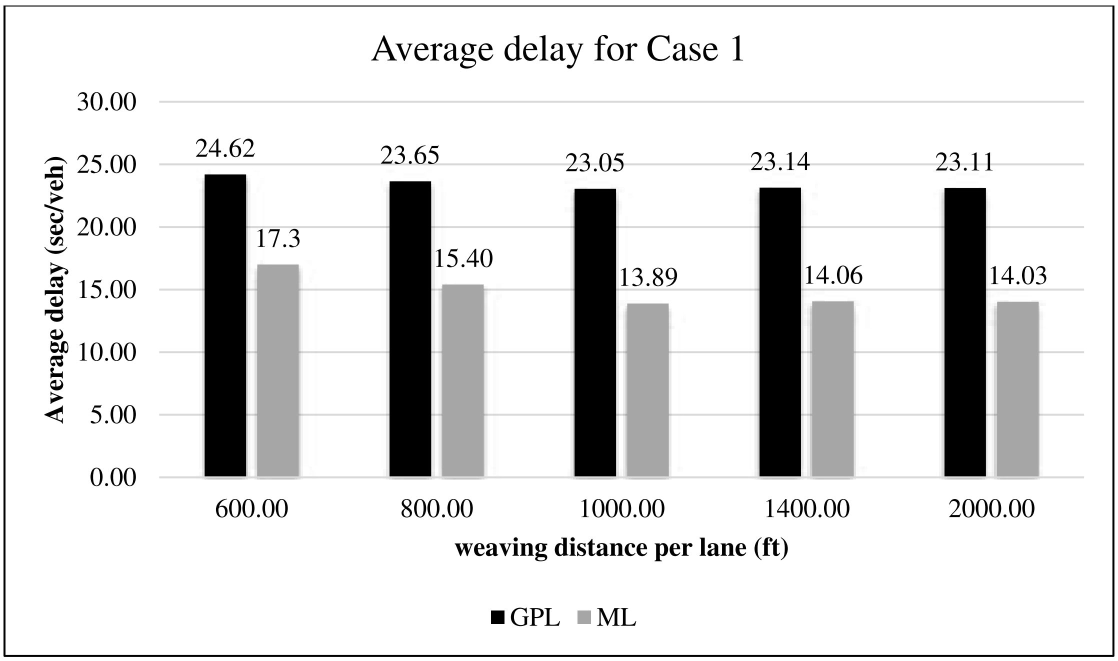



**Figure 29. Average delay for Case 1**

<u>3.5.2.2 Post-Hoc Test for Delay</u>

The average delay of all vehicles can be measured by subtracting the theoretical travel time from the actual travel time. The theoretical travel time is the free flow travel time. Similar to average speed, an ANOVA analysis was conducted to compare average delay for different accessibility levels. The results showed that there was a significant difference in the average delay between accessibility levels (F-value=29.15, p-value<0.0001). Table 15 shows the post-hoc results for the accessibility levels. The results concur with the results of average speed that the case of one access zone had the lowest delay compared to the other accessibility levels. In addition, from the ANOVA analysis, it was found that there is a significant difference between weaving lengths for the case of one access zone (F-value=7.26, p-value=0.0018). The results of post-hoc test for the average delay is shown in Table 16. It is worth mentioning that the results of average delay confirmed the findings of average speed in the case that there are no significant differences between the weaving lengths of 1,000 feet, 1,400 feet, and 2,000 feet. Also, from the ANOVA analysis, it was concluded that peak periods had a significantly higher delay than off-peak periods (F-value=4.65, P-value=0.0351).

**Table 15. Post-Hoc Test Results of Average Delay for Accessibility Levels**

| Parameter | Attributes | | Average Delay | |
|---|---|---|---|---|
| | | | Estimate | p-value |
| Case | One access zone | Two access zones | -9.226 | <0.0001 |
| | One access zone | Three access zones | -16.039 | <0.0001 |
| | Two access zones | Three access zones | -6.813 | 0.002 |

**Table 16. Post-Hoc Test Results of Average Delay for Weaving Lengths**

| Parameter | Attributes | | Average Delay | |
|---|---|---|---|---|
| | | | Estimate | p-value |
| Length (feet) | 600 | 800 | 1.927 | 0.3975 |
| | 600 | 1,000 | 8.332 | 0.0019 |
| | 600 | 1,400 | 9.752 | 0.0005 |
| | 600 | 2,000 | 6.932 | 0.0069 |
| | 800 | 1,000 | 6.405 | 0.0111 |
| | 800 | 1,400 | 7.825 | 0.0031 |
| | 800 | 2,000 | 5.005 | 0.0391 |
| | 1,000 | 1,400 | 1.421 | 0.5308 |
| | 1,000 | 2,000 | -1.401 | 0.5366 |
| | 1,400 | 2,000 | -2.821 | 0.222 |

### 3.5.3 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. The results showed that time efficiency improved in the case of one access zone. With respect to weaving length, from the following bar chart in Figure 30, it can be concluded that weaving length of 800 ft per lane change is recommended for generating maximum time efficiency at both peak and off-peak traffic conditions. Also, it can be noted that time efficiency in peak condition is higher that the off-peak condition, which indicates higher difference in speeds between MLs and GPLs in Peak condition.

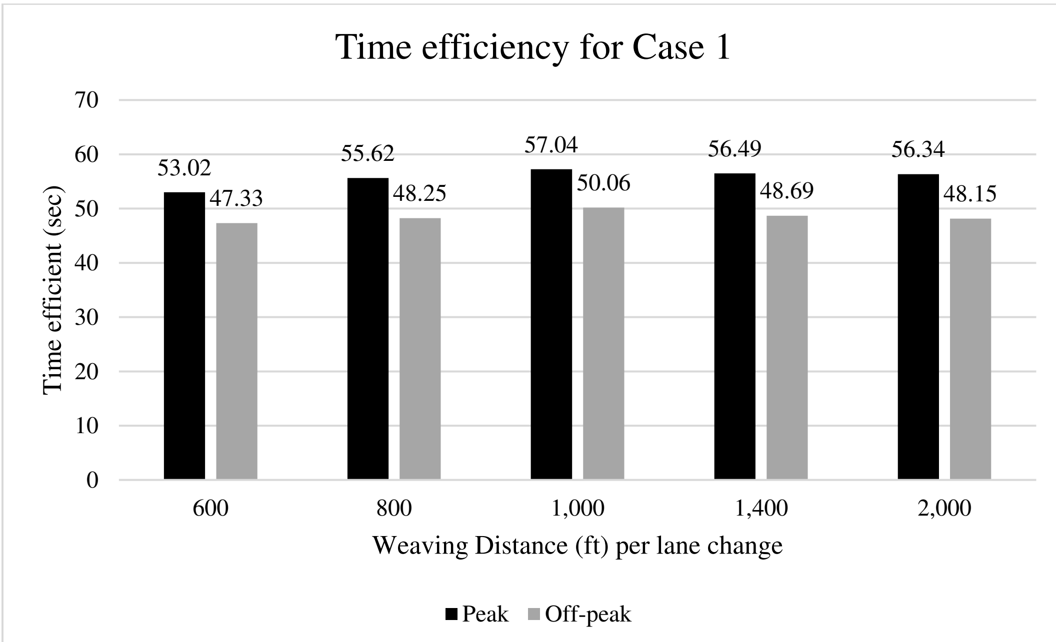



**Figure 30. Time efficiency for Case 1**

### 3.5.4 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 17 (Manual, 2010; Systematics, 2014).

**Table 17. 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<br>any component v/c ratio > 1.00 |

Source: HCM 2010 (Manual, 2010)

Table 18 represents the LOS for all cases. For the base condition case, the LOS for MLs (A) 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, which has LOS ranges between D to E for peak conditions and B to C for off-peak conditions. The striking results to emerge from the data is that, for the case of one access zone, the LOS improved when the weaving segment length are 1,000 ft, 1,400 ft, and 2,000 ft per lane change for peak conditions. Hence, a minimum weaving distance of 1,000 ft is recommended (Saad et al., 2019).

**Table 18. Level of Service for Case 1**

| | Peak | | Off-Peak | |
|---|---|---|---|---|
| Length (ft) | GPLs | MLs | GPLs | MLs |
| 600 | D | B | B | A |
| 800 | C | B | B | A |
| 1,000 | C | A | B | A |
| 1,400 | C | A | B | A |
| 2,000 | C | A | B | A |

## 3.6 Summary and Conclusions

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 research was undertaken for analyzing the safety and operation of the sections near the access zones of MLs with the intention of maximizing system-wide efficiency. Microscopic traffic simulation techniques were developed and applied including 9 mi corridor of MLs segment on Interstate (I-95) in South Florida. The corridor was satisfactorily calibrated and validated by comparing the operational measurements for both simulated and field data.

The safety and operational analysis of the access design in MLs were successfully demonstrated. The findings of this study have several important implications for future practice or policy. It is recommended that both access control level and weaving configuration should be taken into account when designing the access openings of MLs for expressways. The study gives recommendations to the transportation agencies for improving the mobility and the efficiency of the MLs. One of the most prominent findings from this study was that the conflict rate on MLs were 48% and 11% lower than that of GPLs in the peak and the off-peak periods, respectively. After comparing the surrogate safety measures between MLs and GPLs using ANOVA test, it was found that MLs were safer than GPLs since it had higher time-to-collision, higher post-encroachment-time, and lower maximum deceleration.

A log-linear model was developed for investigating the safest access zone design that would minimize traffic conflicts. Analysis of conflicts proposed that one accessibility level is the safest option in 9 mi corridor. Additionally, it was found that a length of 1,000 ft per lane change is indeed the optimal length for the weaving segments. Furthermore, from the findings of this study, a weaving length of 600 ft per lane change is not recommended near the access zones of the MLs. It was also concluded that better safety performance could be found under the off-peak traffic condition. Moreover, the operation measurements were investigated including level of service, average speed, average delay, and time efficiency. The results of the operational measures confirmed several findings from the operational results. The one access zone was found as the optimal level, with better LOS, higher speed, lower delay, and higher time efficiency than the other cases. Also, the off-peak condition showed better operational measurements than the peak condition. The results of the average speed and LOS proposed a minimum weaving distance of 1,000 ft per lane change near the access zones for a more efficient operation of the MLs and GPLs. However, a minimum weaving distance of 800 ft per lane change was recommended for generating maximum time efficiency and minimum delay. Lastly, it was revealed that peak condition has more speed difference between MLS and GPLs than off-peak condition. Hence, MLs has more time efficiency in peak conditions.

Additionally, Tobit models were able to be successfully developed to investigate the optimal MLs access zone design. Analysis of safety measures (i.e., conflict rate, speed SD, and TTC) proposed that one accessibility level is the optimal option in a 9-mile corridor. Additionally, it was found that a weaving length between 1,000 feet and 1,400 feet per lane change should be considered. In contrast, 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. Moreover, the operational measurements were investigated, which included the level of service, average speed, and average delay. The results of the operational measures confirmed several findings from the safety results. One access zone was found as the optimal level, with better LOS, higher speed, and less delay. The results of the average speed, average delay, and LOS proposed a weaving length between 1,000 feet and 2,000 feet per lane change for a more efficiently operated network. Lastly, it was found that the off-peak periods had better safety and operational performance (e.g., lower conflict rate, less delay) compared to the peak periods. For future studies, more attention should be allotted to the peak conditions.

General conclusions and recommendations can be generated based on the results of the safety and operation measurements. The study recommended one access zone (one ingress and one egress) in a 9 mi corridor for achieving better safety, operation, and efficiency compared to two or three accessibility levels. The findings of the safety measures recommended a distance of 1,000 ft per lane change as the safest distance based on a log-linear model under 90% confidence interval. However, the operation measurements suggested a minimum distance of 800 ft per lane change based on the time efficiency and average delay results and 1,000 ft per lane change based on the results of LOS and average speed. Taken together, a minimum weaving length of 1,000 ft per lane change is recommended, and the distance of 1,000 ft per lane change is preferable. If the space is limited, a minimum weaving length of 800 per lane change is suggested. The weaving distance of 600 ft per lane change is not recommended near the access zones. Lastly, more attention should be paid under the peak periods.

# CHAPTER 4: IMPACT OF CONNECTED VEHICLES ON FREEWAY FACILITIES WITH MANAGED LANES

## 4.1 Introduction

Connected vehicles (CVs) are one of the most recent developments in traffic and safety engineering. Connected vehicles have the potential to revolutionize safety and efficiency by reducing the number of crashes and fatalities on the road. This technology enables vehicles, roads, traffic signals, and other infrastructure to communicate with one another about current road conditions, alerts and signals.

The objective of the research presented in this chapter is to analyze the safety and operational effect of adding CVs and CV lanes to the managed lanes network. Several tasks were determined to achieve the goal of the study. The first objective is to build networks for the managed lanes in a connected vehicles environment. The second objective is to study the effect of different cases of CV lanes and CVs on the safety and operation of the whole network. The third objective is to determine the optimal market penetration of the CV lanes by investigating different market penetration rates (MPR%) for different cases. A comparison between the different cases of MLs designs with the presence of CVs with different market penetration rates is generated for different traffic conditions.

## 4.2 Experimental Design

### 4.2.1 Connected Vehicles Environment

In PTV VISSIM 11, CVs could be added and tested in the managed lanes network. The driving behavior models of CVs were ready to use since it was already calibrated and validated using real-world CVs data in a project named CoEXist which is a European Union's Horizon 2020 funded Project (Groves, 2018; PTV, 2018; Sukennik, 2018). In the software, there are three types of driving logics of connected vehicles including cautious, normal, and all-knowing driving logic. In the cautious driving logic, vehicles always respect the road code and safe behavior. Regarding the normal driving logic, vehicles have the capability of measuring speeds and gaps with the surrounding vehicles with its sensors. The all-knowing driver logic predicts all other road users' behavior with V2V or V2I technologies (Sukennik, 2018). In the all-knowing logic, the number of interaction objects and the number of interaction vehicle can be more than one (Figure 31). The figure shows one interaction objective and two interaction vehicles. However, in the cautious and normal logics, the vehicle can only have one interaction vehicle (Figure 32). Figure 33 shows the different vehicles' gaps between different driving logics. The cautious driving logic has the largest gap compared to other driving logics. The normal driving logic has gaps similar to human drivers but with higher safety. The all-knowing driving logic has smaller gaps but is still relatively safe. Figure 34 shows the different driving logic in PTV VISSIM (PTV 2018). In this study, CVs followed the normal driving logic provided by PTV VISSIM 11.

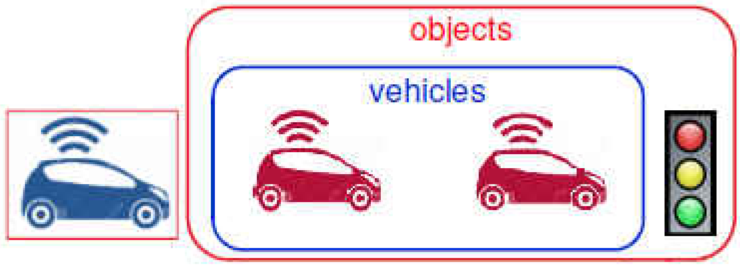



**Figure 31. Interaction objects and vehicles for the all-knowing logic**

**Figure 32. Interaction objects and vehicles for the Cautious and Normal logics**

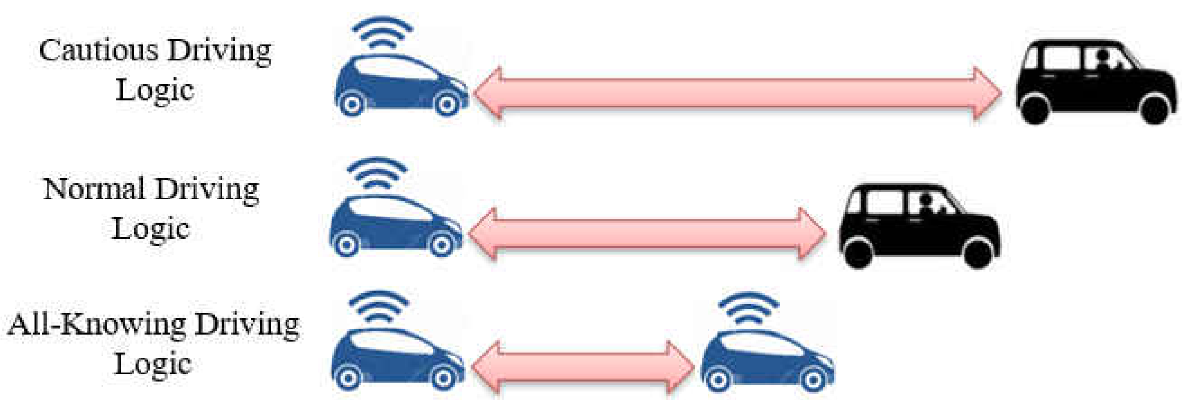



**Figure 33. Connected Vehicles Driving Logics (PTV 2018)**

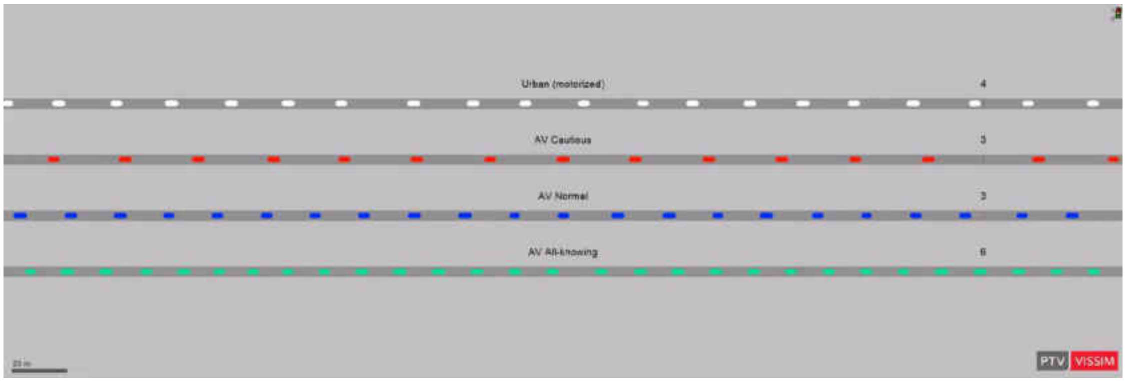



**Figure 34. Different Driving Logics in VISSIM (PTV, 2018; Sukennik, 2018)**

The parameters of car following, and lane change models for all driving logics of CVs were calibrated and validated using real-world connected vehicles data (Groves, 2018; PTV, 2018; Sukennik, 2018). Table 19 shows the calibrated car following parameters in PTV VISSIM 11, which has ten car following parameters (CC0 to CC9). The CC parameters are defined in the presented table. The calibrated lane changing parameters for CVs are shown in Table 20.

**Table 19. Car Following Parameters for Different Driving Logics (PTV 2018)**

| Car following parameter | Description | Human Driving Behavior (Default) | All Knowing Driving Logic | Normal Driving Logic | Cautious Driving Logic |
|---|---|---|---|---|---|
| CC0 | The average standstill distance (meter) | 1.50 | 1.00 | 1.50 | 1.50 |
| CC1 | The headway time (seconds) | 0.90 | 0.600 | 0.90 | 1.50 |
| CC2 | The distance difference in the oscillation condition (meter) | 4.00 | 0 | 0 | 0 |
| CC3 | Controls the deceleration process | -8.00 | -6.00 | -8.00 | -10.00 |
| CC4 | Defines negative speed difference | -0.35 | -0.10 | -0.10 | -0.10 |
| CC5 | Defines positive speed difference | 0.35 | 0.10 | 0.10 | 0.10 |
| CC6 | The distance influence on speed oscillation | 11.44 | 0 | 0 | 0 |
| CC7 | The acceleration at the oscillation condition ($m/s^2$) | 0.25 | 0.10 | 0.10 | 0.10 |
| CC8 | The desired standstill acceleration ($m/s^2$) | 3.50 | 4.00 | 3.50 | 3.00 |
| CC9 | The desired acceleration at 50 mph ($m/s^2$) | 1.50 | 2.00 | 1.50 | 1.20 |

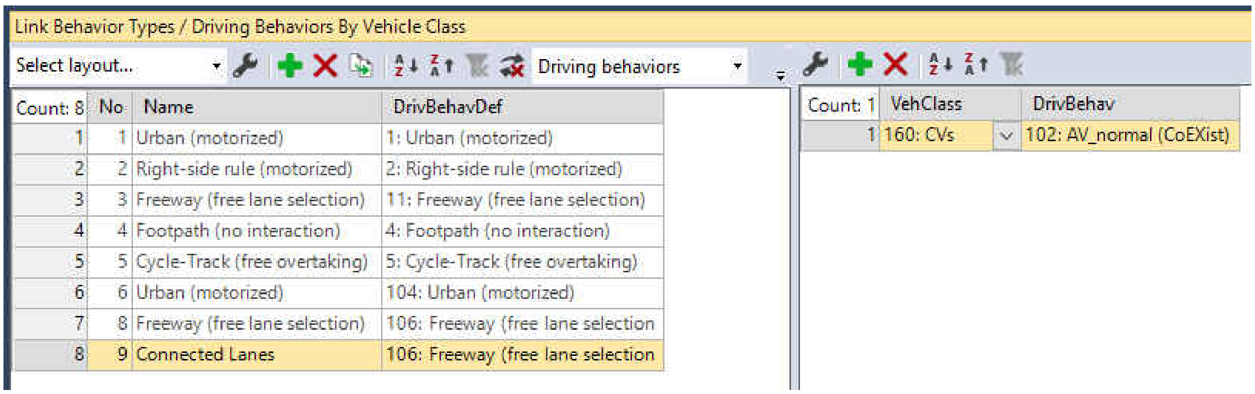
**Table 20. Lane Change Behavior for Different Driving Logics (PTV 2018)**

| | All Knowing Driving Logic | | Normal Driving Logic | | Cautious Driving Logic | |
|---|---|---|---|---|---|---|
| | Own | Trailing Vehicle | Own | Trailing Vehicle | Own | Trailing Vehicle |
| Maximum Deceleration ($m/s^2$) | -4.00 | -4.00 | -4.00 | -3.00 | -3.50 | -2.50 |
| -1 m/s per distance | 100 | 100 | 100 | 100 | 80 | 80 |
| Accepted deceleration | -1.00 | -1.50 | -1.00 | -1.00 | -1.00 | -1.00 |
| Waiting time per diffusion (sec) | 60 | | 60 | | 60 | |
| Min. net headway (front to rear) (m) | 0.5 | | 0.5 | | 0.5 | |
| Safety distance reduction factor | 0.75 | | 0.6 | | 0.6 | |
| Maximum deceleration for cooperative braking ($m/s^2$) | -6.00 | | -3.00 | | -2.50 | |

### 4.2.2 Dedicated Connected Vehicles Lanes (CVLs)

Dedicated connected vehicle lanes (CVLs) were utilized in this study to investigate the impact of CVs in the managed lanes network with the presence of dedicated CV lanes. In this study, several scenarios were studied with the presence of CVLs. For instance, some scenarios allowed CVs to use either CVLs or ML while other scenarios restricted CVs to use only CVLs.

These scenarios were important for deciding the effect of CVLs presence in the managed lanes network. In order to assign CVs in a dedicated lane in VISSIM, the normal behavior was used as shown in Figure 35.



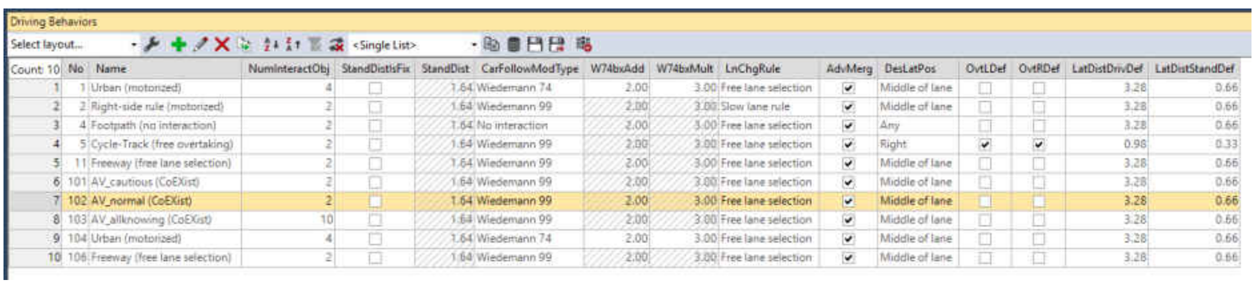




**Figure 35. Assigning Driving Logic to CV for CV Lanes (Source: VISSIM 11).**

### 4.2.3 Market Penetration Rate (MPR%)

The percentage of connected vehicles in the network is represented by the market penetration rate (MPR%). One of the goals of this study is estimating the potential MPR% of CVs when evaluating multiple lane configurations in a connected vehicle environment. The latest report of evaluating connected and automated vehicles on freeways and dedicating lanes by NCHRP

(Project number: 20-102(08)) (NCHRP 2018) showed that network efficiency improved with CVs. The report also showed that the dedicated CV lanes have a significant impact on the network with a low MPR%. Moreover, MPR% increases when the CVs are allowed to use all lanes in the network (i.e., GPLs, MLs, and connected vehicle lanes). Hence, the level of service of GPLs increases with the increase of the capacity and the result is an improvement in the system performance (NCHRP 2018).

In this study, different market penetration rates were taken into consideration in the experimental design (e.g., 10%, 20%, 30%, etc.). From previous studies, the full market penetration of CVs might not be accomplished in the near future. Therefore, traffic flow will likely be composed of a mixture of conventional vehicles and CVs (Talebpour et al., 2017).

### 4.2.4 Vehicle Classes

Four classes of vehicles were utilized in this simulation: passenger cars (PCs), heavy goods vehicles (HGVs), connected vehicles (CVs), and carpools. According to (FDOT, 2002), the percentage of HGVs is 5% on freeways. Meanwhile, according to the 2015 U.S. Census American Community Surveys (ACS) for Miami-Dade (ACS, 2015), the percentage of carpools is 10% on freeways. Considering carpool percentage in this study was important because the policy of the FDOT is that carpools are allowed to use the MLs without paying tolls (Joseph, 2013). The percentage of PCs and CVs were depending on the studies MPRs for each scenario.

### 4.2.5 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 (PTV, 2015). The DSD has to be inputted in VISSIM for different types of vehicles (i.e., PCs, CVs, 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, CVs or carpools, their speed distributions were the same and were divided into four groups. The groups were determined by 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 speeds. 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 inferred from the speed distributions of PCs, CVs and carpools. Johnson and Murray (Johnson & Murray, 2010) concluded that the average speed difference between cars and trucks was 8.1 miles per hour. The HGV percentage is 5%. Suppose x is the speed of PCs, CVs or carpools, then the speed for HGV is equal to (x-8.1), the average speed is y, which is provided by RITIS, and

$$\mathrm{Y} = 0.95 \times \mathrm{PC} + 0.05 \times (\mathrm{PC} - 8.1) \tag{12}$$

From the equation, the speed of the PC, CVs 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.

### 4.2.6 Dynamic Toll Pricing

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 are as follows:

$$U_{toll} = \beta_{time} \times Time\ gain - \beta_{cost} \times Toll\ rate + Base\ Utility \quad (13)$$

$$P_{toll} = 1 - \frac{1}{1+e^{a \times U_{toll}}} \quad (14)$$

The base utility depends on the vehicle class and zero as the default value of the software. The time coefficient ($\beta_{time}$) and the cost coefficient ($\beta_{cost}$) were calculated from the Value of Time (VOT). The ratio of the cost coefficient and the time coefficient ($\beta_{time}$) was utilized to define the VOT as follows:

$$VOT = \frac{\beta_{time}}{\beta_{cost}}\ (\$/hr) \quad (15)$$

In this study, the VOT was assumed to be \$8.67 per hr based on the result of a multinomial logit model conducted by Jin et al. (2015). The time coefficient was assumed to be one min 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 implies an increase in the MLs utility with the decrease of the tolls. The toll price is mainly affected by two components. First, the time saved by using the MLs, which varied from 0 to 8.50 min. Second, the speed in the MLs, which was between 30 mph and 73.50

mph. The dynamic toll prices varied between a minimum value of $0.50 and a maximum value of $10.50.

### 4.2.7 Scenarios Setup

In order to study the effect of CVs and CVLs, four different cases were studied. The base condition (Case 0) included the I-95 corridor with one access zones (one ingress and one egress) in the middle of the corridor. In this case, three types of vehicles were considered: passenger cars (PCs), heavy goods vehicles (HGVs), and carpools. It is worth mentioning that connected vehicles are not considered in the base case (Case 0). Figure 36 displays Case 0 with no CVs in the network.

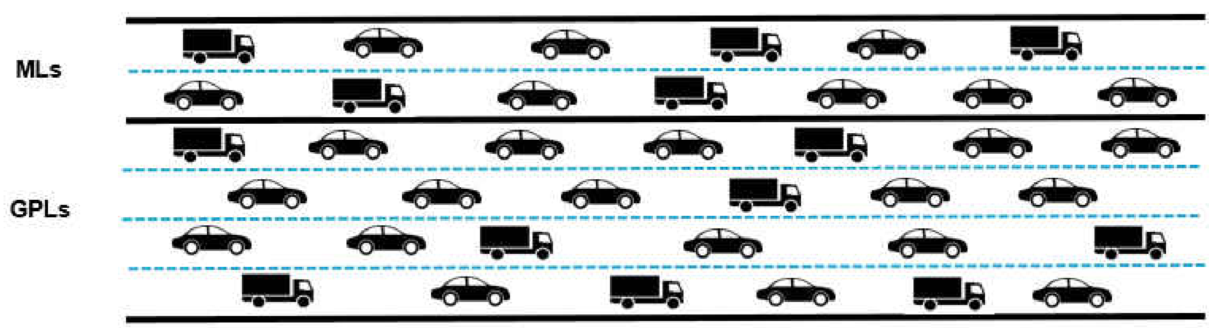



**Figure 36. The Base Case (Case 0) with No Connected Vehicles in the Network**

In Case 1, four types of vehicles were studied including PCs, HGVs, carpools, and connected vehicles. In this case, connected vehicles are only allowed in the managed lanes and have the choice to use any of the managed lanes. Figure 37 provides Case 1 with the configuration of the different types of vehicles in the network.

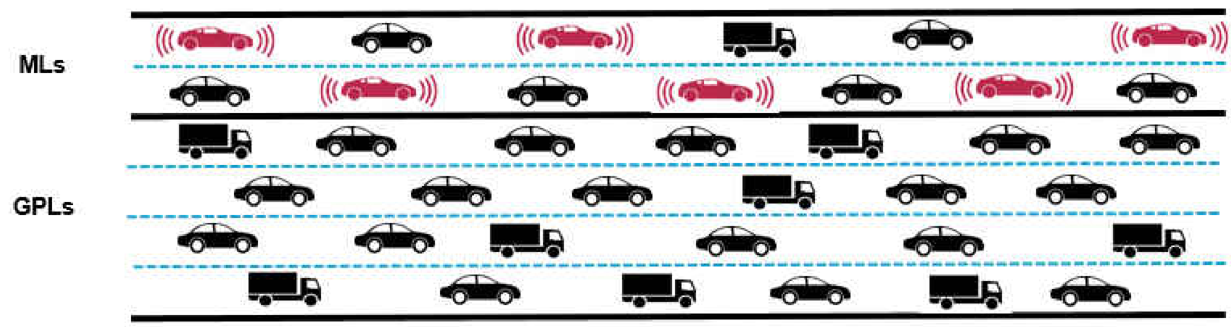



**Figure 37. Case 1 with Connected Vehicles in the Managed Lanes**

Regarding Case 2, four types of vehicles were used in this case, similarly to the previous case. In Case 2, a dedicated connected vehicles lane was studied in the left side of the network Therefore, connected vehicles can use either the connected vehicles lanes (CVLs) or the managed lanes. Figure 38 presents the configuration of the different types of vehicles in case 2.

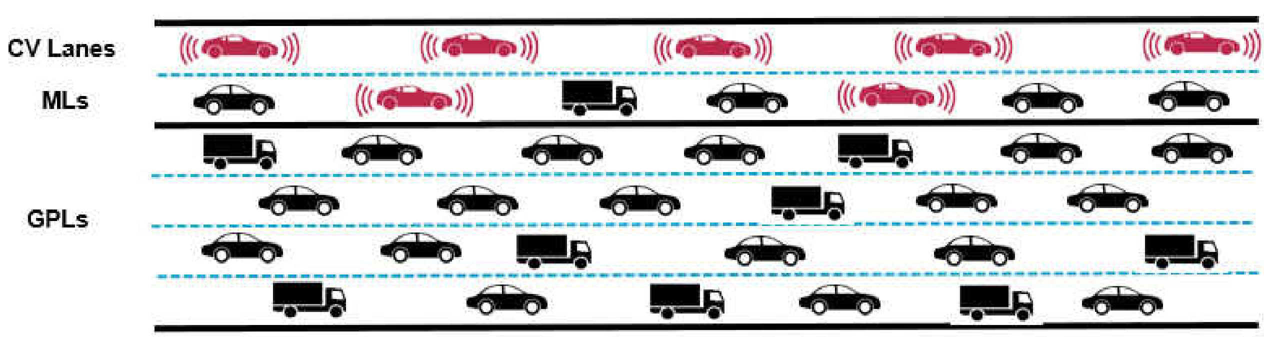



**Figure 38. Case 2 with Connected Vehicles in the CVLs Only**

Case 3 also includes four types of vehicles (i.e., PCs, HGVs, carpools, and CVs). Dedicated connected vehicle lanes were also studied in this case on the left side of the network. In this case, CVs were only allowed to use the CVLs, as shown in Figure 39.

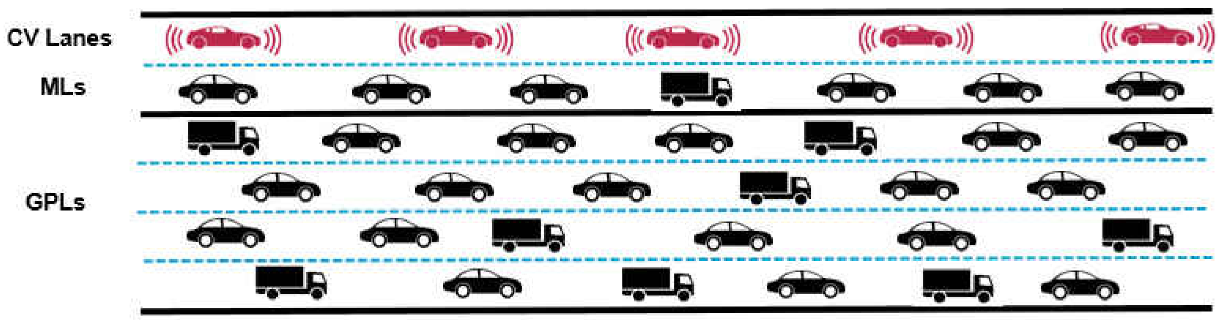



**Figure 39. Case 3 with Connected Vehicles in either MLs or CVLs**

Case 4 is similar to Case 1 with converting one lane of the GPLs to a lane of MLs in order to increase the capacity of the MLs. In this case, connected vehicles were only allowed in the MLs and had the choice to use any of the MLs. Figure 40 provides Case 4 with the configuration of the different types of vehicles in the network.

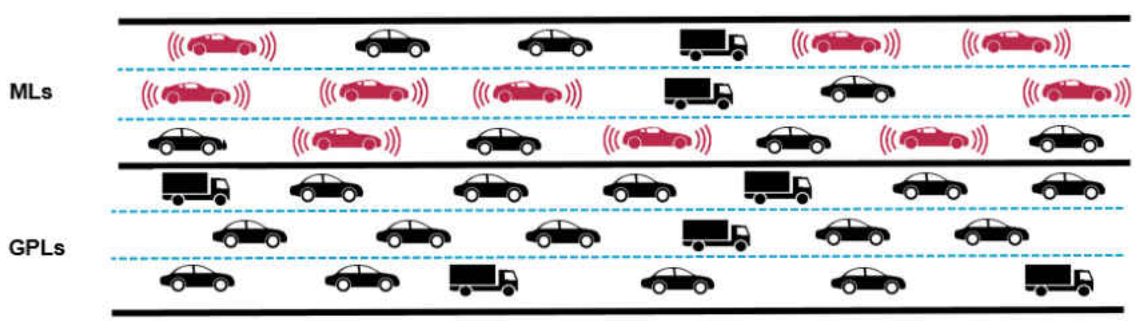



**Figure 40. Case 4 with CVs in MLs and Converting One GPLs to MLs**

Case 5 is similar to Case 1 with adding one lane to the MLs in order to increase the capacity of the network. In this case, connected vehicles were only allowed in the managed lanes and had the choice to use any of the managed lanes. Figure 41 provides Case 5 with the configuration of the different types of vehicles in the network.

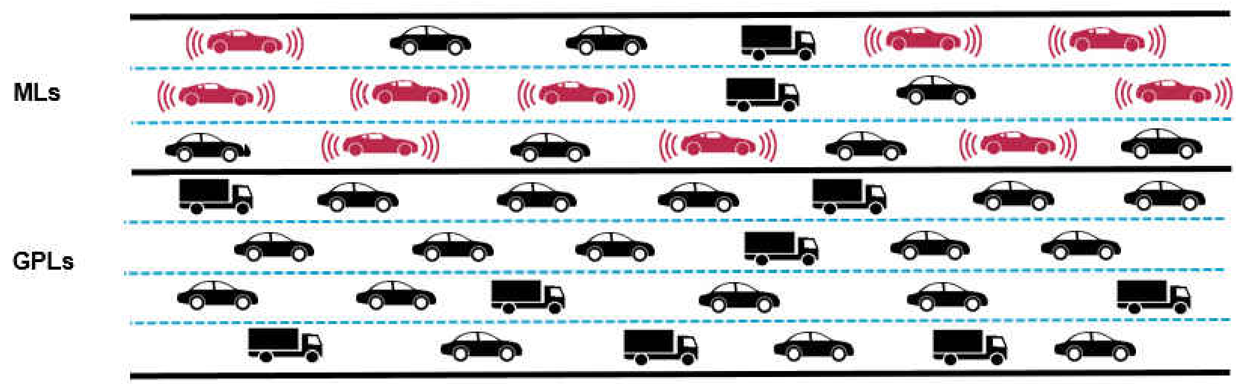



**Figure 41. Case 5 with CVs in MLs and Increasing the Number of MLs**

Similar to the previous three cases, Case 6 considered four different types of vehicles. In Case 6, CVs had the choice to use any of the lanes in the network: CVLs, MLs, or GPLs. Figure 42 shows the configuration of the different vehicle types in Case 6.

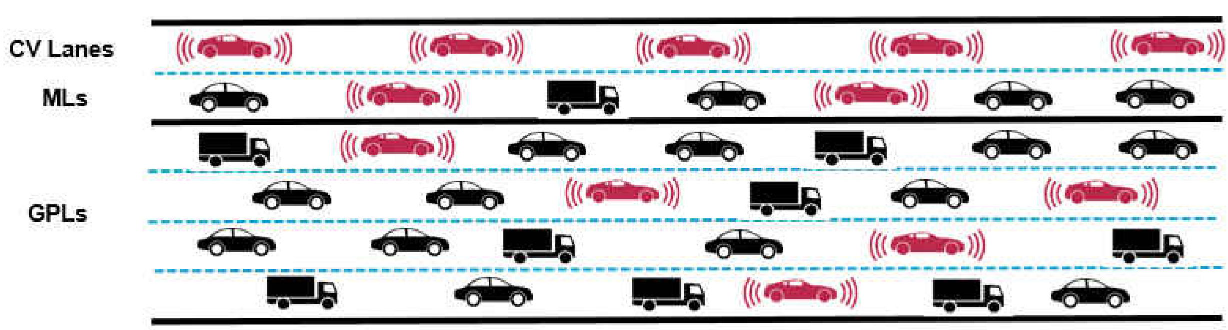



**Figure 42. Case 6 with Connected Vehicles in All Lanes (GPLs, MLs, CVLs)**

There was of total of 110 scenarios, including the base case for peak and off-peak conditions, were tested in this study with different CV lane configurations in managed lanes network (i.e., Case 1, Case 2, Case 3, Case 4, Case 5, Case 6) in both peak and off-peak conditions. Various market penetration rates (MPR%) were also being considered in the scenarios design (e.g., 10%, 20%, 30%, etc.,). Table 21 shows the list of the 110 studied scenarios. For each scenario, ten random runs with different random seeds were applied. It is worth noting that in Cases 1, 2, and 3, the maximum studied MPR% was 40%. This can be explained by when the MPR% is over 40%, the MLs have reached their capacity. In cases 4 and 5, the configurations of lanes were changed in order to increase the capacity of the network. Hence, in cases 4 and 5, the studied MPR% reached 100%. Similarly, in Case 6, the studied MPR% reached 100% because CVs were allowed to use any of the lanes in the network.

**Table 21. List of Scenarios**

| Case | Traffic Condition | Market Penetration Rate | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Case 0 | Peak | | | | | 0% | | | | | |
| (Base Condition) | Off-peak | | | | | 0% | | | | | |
| Case 1 | Peak | 5% | 10% | 15% | 20% | 25% | 30% | 35% | 40% | | |
| (CVs in MLs with no CVLs) | Off-peak | 5% | 10% | 15% | 20% | 25% | 30% | 35% | 40% | | |
| Case 2 | Peak | 5% | 10% | 15% | 20% | 25% | 30% | 35% | 40% | | |
| (CVs in CVLs and MLs) | Off-peak | 5% | 10% | 15% | 20% | 25% | 30% | 35% | 40% | | |
| Case 3 | Peak | 5% | 10% | 15% | 20% | 25% | 30% | 35% | 40% | | |
| (CVs in CVLs only) | Off-peak | 5% | 10% | 15% | 20% | 25% | 30% | 35% | 40% | | |
| Case 4 (Converting one GPLs to MLs) | Peak | 10% | 20% | 30% | 40% | 50% | 60% | 70% | 80% | 90% | 100% |
| | Off-peak | 10% | 20% | 30% | 40% | 50% | 60% | 70% | 80% | 90% | 100% |
| Case 5 | Peak | 10% | 20% | 30% | 40% | 50% | 60% | 70% | 80% | 90% | 100% |
| (Increasing number of managed lanes) | Off-peak | 10% | 20% | 30% | 40% | 50% | 60% | 70% | 80% | 90% | 100% |
| Case 6 | Peak | 10% | 20% | 30% | 40% | 50% | 60% | 70% | 80% | 90% | 100% |
| (CVs in all lanes) | Off-peak | 10% | 20% | 30% | 40% | 50% | 60% | 70% | 80% | 90% | 100% |

## 4.3 Safety Analysis

### 4.3.1 Conflict Frequency

The Surrogate Safety Assessment Model (SSAM) was adopted to determine the potential conflict frequency, which is associated with the number of crashes in the field (Shahdah et al., 2014). The main objective of SSAM could be to either evaluate the safety performance of the current roadway designs or as a new strategy for monitoring theoretical roadway designs before implementation (Gettman et al., 2008). Three types of conflicts can be extracted from SSAM, which include: rear-end, lane change, and crossing conflicts. Two types of conflicts were used in this paper: rear-end and lane-change conflicts. As provided by SSAM, the rear-end conflicts were considered when the conflict angle was between 0 and 30 degrees, while the lane-change conflicts were defined as when the conflict angle was between 30 and 80 degrees. The crossing conflicts were excluded from this study, since the percentage of crossing conflicts was less than 1%, and crossing crashes are less likely to happen on freeways.

The vehicle trajectory files (.trj file) from VISSIM were imported into SSAM to obtain the detailed information of the conflicts. A time-to-collision (TTC) of zero implies "virtual" crashes that might lead to the inaccuracy of the simulation models (Gettman et al., 2008). Consequently, the cases in which the TTC=0 (crash) were excluded before implementing further analysis. According to FHWA (Gettman et al., 2008), TTC is the minimum time-to-collision, which is calculated based on the speed and location of vehicles. The FHWA report recommended a maximum critical value for TTC as 1.5 sec. It was stated that conflicts with TTC values larger than

1.5 sec are not recognized as a severe condition. As the TTC value increased, the conflict risk was found to decline (Sayed & Zein, 1999). Additionally, the FHWA report suggested a minimum TTC value of 0.1 sec. Several studies used the same threshold (0.1 sec to 1.5 sec) as severe conflicts (Wang et al. 2017; Saleem et al. 2014; Saulino et al. 2015). In this study, a TTC threshold between 0.1 sec and 1.5 sec was used.

For the base case with no CVs, it was found that, for peak conditions, 77.87% were rear-end conflicts and 22.15% were lane change conflicts. It was also found that in off-peak conditions, 65.57% of conflicts were rear-end and 34.43% were lane change conflicts, as shown in Figure 43.

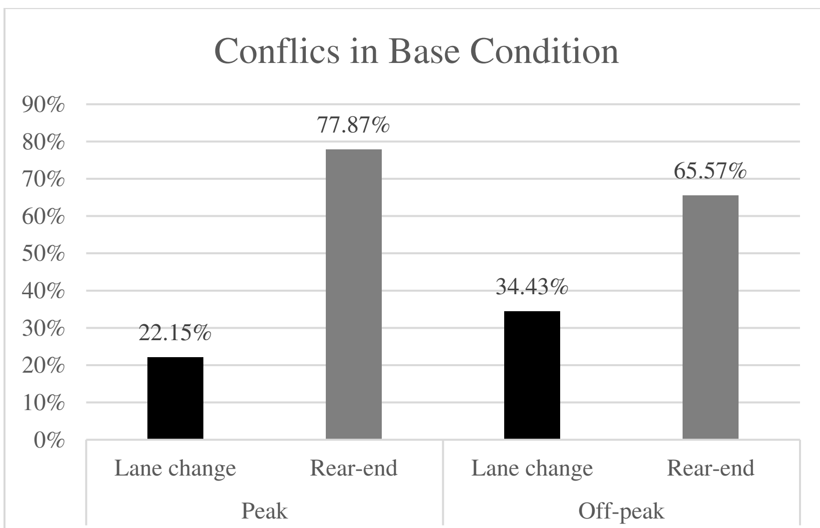



**Figure 43. Conflict Frequency for Peak and Off-peak Conditions in the Base Case**

The descriptive statistics of the conflict frequency for all studied case are shown in Table 22 for both peak and off-peak periods. The results of the table indicated that Cases 4 and 5 had the lowest conflict frequency among all cases. Meanwhile, Case 3 showed the highest conflict frequency. An ANOVA test was carried out to compare the conflict frequency in various CV lane design cases, MPR%, and traffic conditions. The results showed that there was a significant difference in conflicts between cases (F-value=12.86, p-value<0.0001). The results also showed significant differece in conflicts between different MPR% (F-value=35.09, p-value=0.0003). Additionally, the results showed that conflicts (F-value=51.87, p-value<0.0001) were higher in the peak conditions than the off-peak conditions. A post-hoc test was conducted to test the significant difference between different cases, as shown in Table 23. The results revealed that there was no significant difference between Cases 1, 4, 5, and 6. Case 3 had significant higher conflicts than all other cases.

**Table 22. Descriptive Statistics of Conflict Frequency for All Studied Cases**

| Case | Traffic Condition | Mean | Standard Deviation | Minimum | Maximum |
|---|---|---|---|---|---|
| **Base** | **Peak** | 1687.7 | - | 1687.7 | 1687.7 |
| | **Off-peak** | 408 | - | 408 | 408 |
| **Case 1** | **Peak** | 6258.39 | 12546.35 | 556.1 | 36721.1 |
| | **Off-peak** | 266.35 | 72.06 | 190.7 | 390.86 |
| **Case 2** | **Peak** | 9090.06 | 17501.07 | 716.78 | 51704.8 |
| | **Off-peak** | 360.92 | 129.59 | 199.80 | 560.33 |
| **Case 3** | **Peak** | 26479.1 | 27241.1 | 2155 | 72846.1 |
| | **Off-peak** | 1104.68 | 938.02 | 369.4 | 2846.1 |
| **Case 4** | **Peak** | 3490.8 | 4936.44 | 490 | 15472 |
| | **Off-peak** | 298.8 | 135.567 | 112 | 573 |
| **Case 5** | **Peak** | 788.4 | 429.31 | 435.5 | 1754 |
| | **Off-peak** | 261.87 | 125.43 | 146 | 521 |
| **Case 6** | **Peak** | 1064.9 | 503.81 | 487 | 2102 |
| | **Off-peak** | 256.5 | 98.32 | 156 | 420 |

**Table 23. Post Hoc Test of Conflict Frequency between Cases**

| **Case** | | Estimate | P-Value |
|---|---|---|---|
| **Case 1** | **Case 2** | -0.3937 | 0.3882 |
| **Case 1** | **Case 4** | 0.5105 | 0.2392 |
| **Case 1** | **Case 5** | 0.3487 | 0.4204 |
| **Case 1** | **Case 6** | 0.0437 | 0.9168 |
| **Case 4** | **Case 5** | -0.1618 | 0.6913 |
| **Case 4** | **Case 6** | -0.5288 | 0.1991 |
| **Case 5** | **Case 6** | -0.3483 | 0.3966 |
| **Case 2** | **Case 4** | 0.9042 | 0.0388 |
| **Case 2** | **Case 5** | 0.7424 | 0.0884 |
| **Case 2** | **Case 6** | 0.3011 | 0.4902 |
| **Case 3** | **Case 1** | 1.5726 | 0.0008 |
| **Case 3** | **Case 2** | 1.1789 | 0.0111 |
| **Case 3** | **Case 4** | 2.0831 | <0.0001 |
| **Case 3** | **Case 5** | 1.9213 | <0.0001 |
| **Case 3** | **Case 6** | 1.7038 | <0.0001 |

In Case 1 (which allow CVs to use any of the MLs), the lowest conflicts occurred when the MPR% was 20% for peak conditions and 30% for off-peak conditions. Figure 44 shows the conflict counts for Case 1.

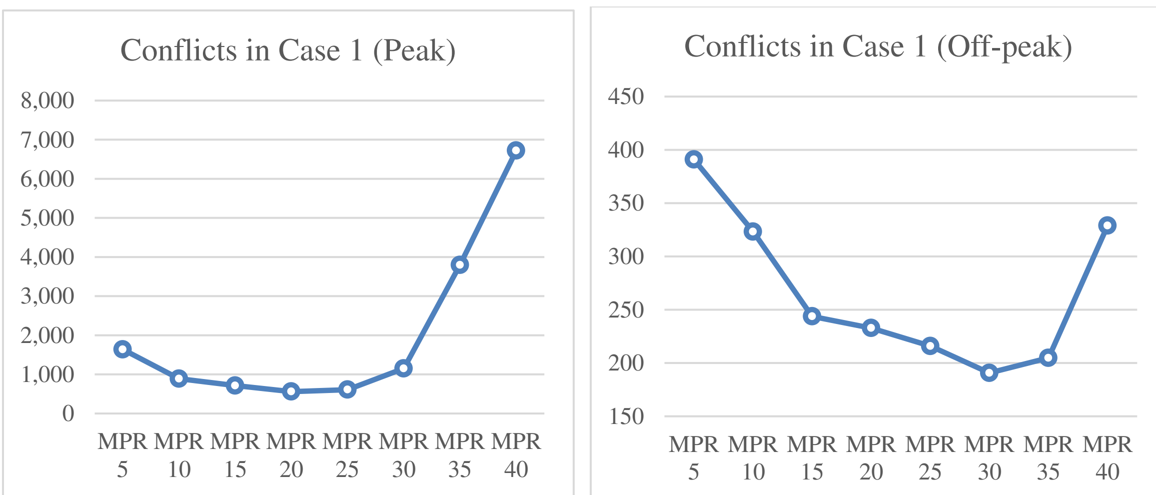



**Figure 44. Conflict Frequency for peak and off-peak condition in Case 1**

Similarly, the lowest conflict frequency happened in Case 2 (which allows CVs to use either dedicated CV lanes or MLs) when the MPR% was 25% for peak conditions and 30% for off-peak conditions. Also, the results showed that traffic conflicts increase dramatically after a market penetration rate of 40%. Figure 45 shows the conflict counts for Case 2.

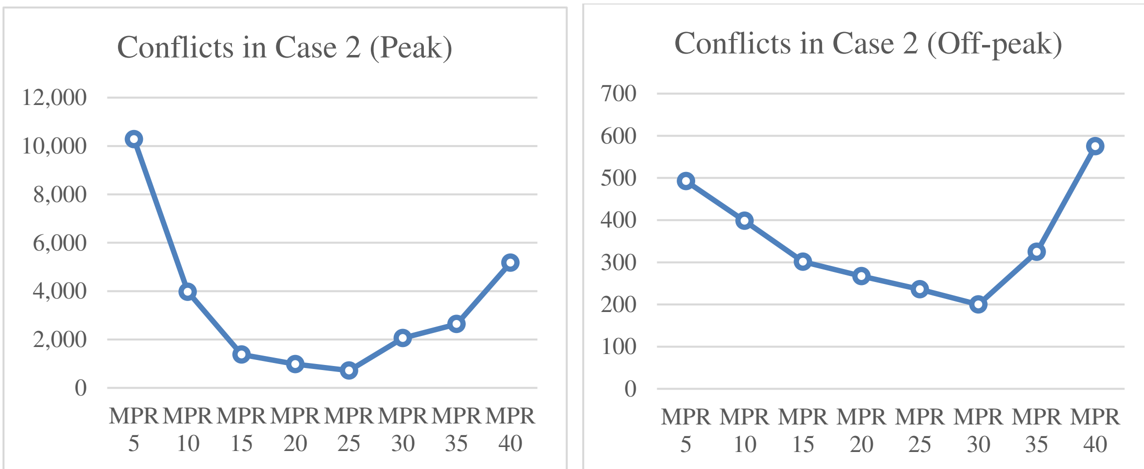



**Figure 45. Conflict Frequency for Peak and Off-peak Conditions in Cases 2**

It was also revealed that Case 3 (which allows CVs to use only dedicated CV lanes) has the highest conflict frequency among all other cases as shown in Figure 46. The lowest conflicts happened when the MPR% was 15% for peak conditions and 20% for off-peak conditions.

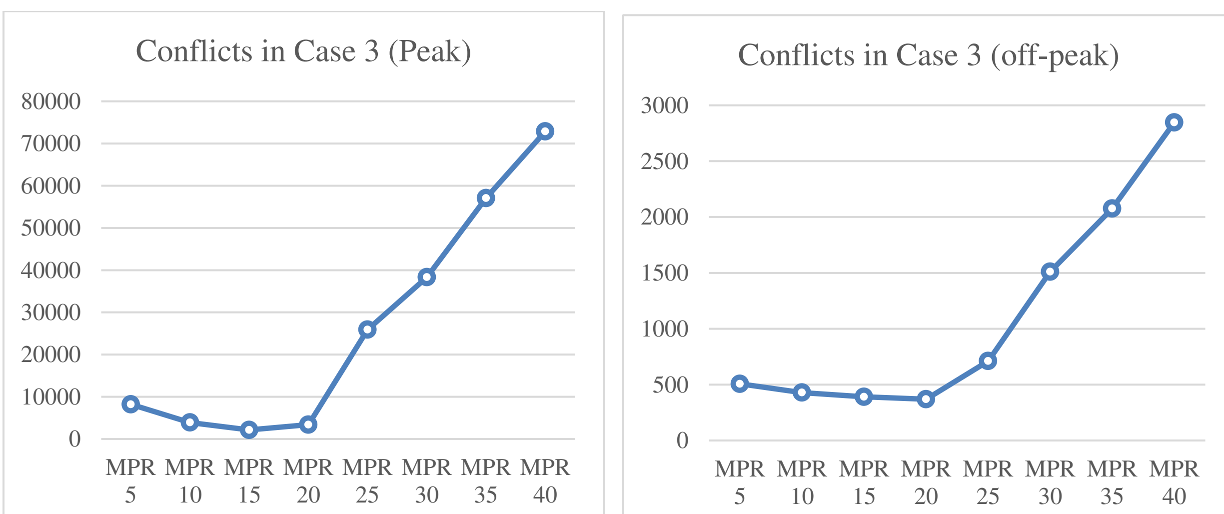



**Figure 46. Conflict Frequency for Peak and Off-peak Conditions in Case 3**

For Case 4 (which is similar to Case 1 with converting one GPLs to MLs), it was found that in peak conditions, the lowest conflicts occurred at an MPR of 50%. It is worth mentioning that the conflicts were reduced when the MPR% was between 40% and 60%. In off-peak conditions, the lowest conflicts occurred at an MPR% of 60%. The conflicts frequency was the lowest when the MPR% was between 50% and 70%. Figure 47 shows the distribution of conflict frequency in Case 4.

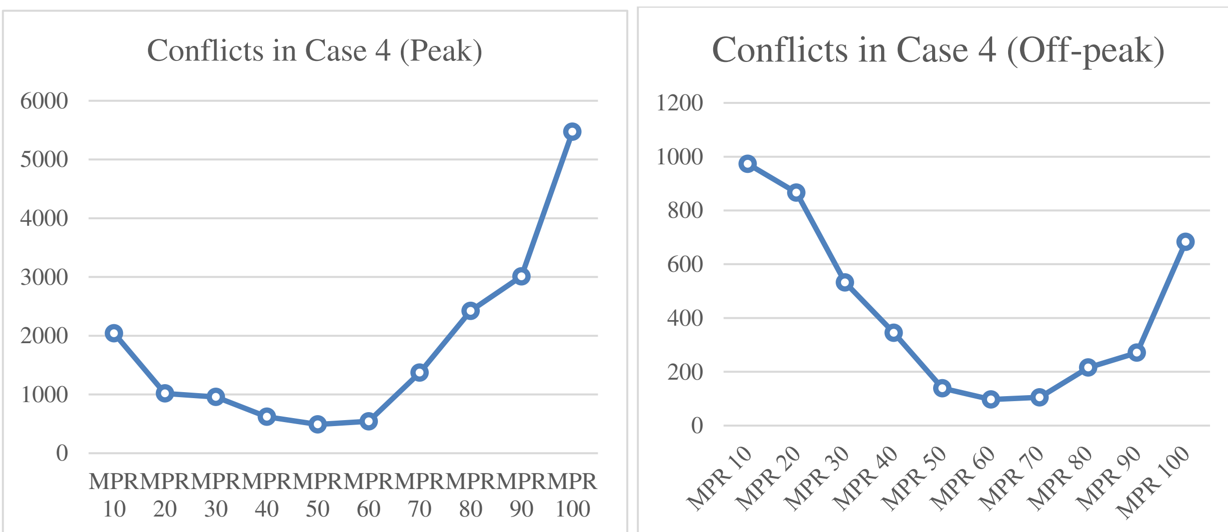



**Figure 47. Conflict Frequency for Peak and Off-peak Conditions in Case 4**

Regarding Case 5 (which is similar to Case 1 with an increase in the number of MLs), it was found that in peak conditions, the lowest conflicts occurred at an MPR of 60%. It is worth mentioning that the conflicts were reduced when the MPR% was between 50% and 70%. In off-peak conditions, the lowest conflicts occurred at an MPR of 70%. The conflicts frequency was the lowest when the MPR% was between 70% and 80%. Figure 48 shows the distribution of conflict frequency in Case 5.

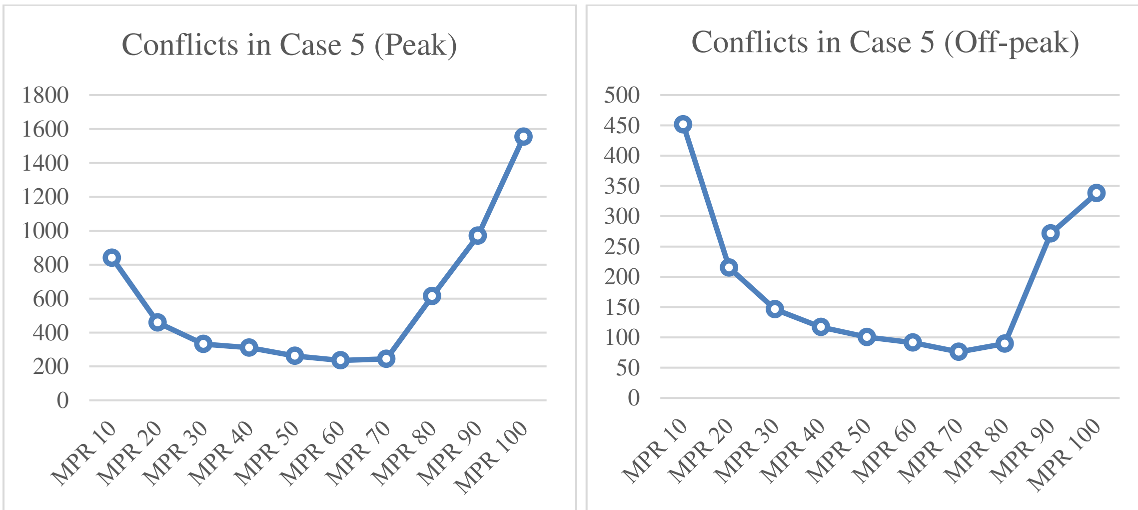



**Figure 48. Conflict Frequency for Peak and Off-peak Conditions in Case 5**

Figure 49 shows the distribution of conflict frequency for each MPR% for Case 6 (which allows CVs to use any of the CVLs, MLs, or GPLs) for both peak and off-peak conditions. Looking at the figure, it is apparent that the conflict frequency reduced with the increase of MPR%. In peak conditions, the lowest conflict frequency occurred when the MPR% was 100%. The highest conflicts appeared when the MPR% was 10%. In off-peak conditions, it is worth noting that the conflict distribution followed the same trend as the peak conditions. The lowest conflict frequency occurred at an MPR% of 100%. Hence, a higher MPR% could be recommended in Case 6.

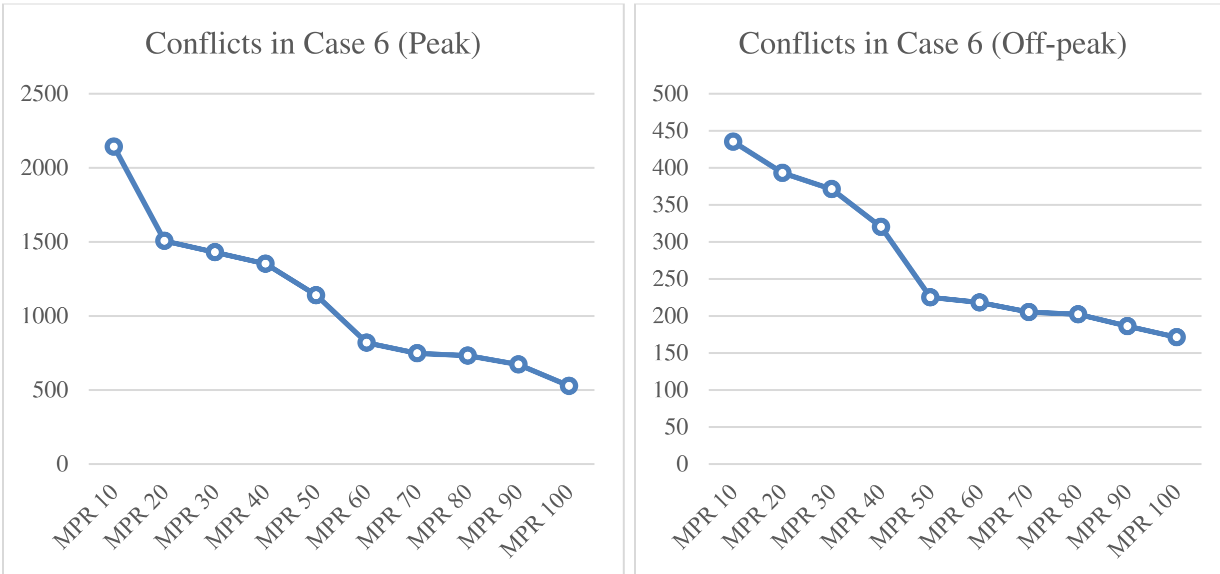



**Figure 49. Conflict Frequency for Peak and Off-peak Conditions in Case 6**

4.3.2 Conflict Reduction

Conflict reduction was calculated based on the difference between the traffic conflicts of any case of connected vehicles (i.e., Case 1, Case 2, Case 3, Case 4, Case 5, and Case 6) and the conflicts of the base case with no connected vehicles as shown in the following equation.

$$\text{Conflict Reduction} = \frac{\textit{Conflicts in base case} - \textit{Conflicts in cases of CVs}}{\textit{Conflicts in base case}} \tag{16}$$

For Case 1 (which allow CVs to use any of the MLs), the results of adding CVs to the MLs network revealed that the maximum conflict reduction (compared to the case of no CVs) occurred at an MPR% of **20%** during peak conditions. The conflict reduction reached 66.87% more than any other cases. Regarding off-peak conditions, the maximum conflict reduction was 53.23% and it happened when the MPR% was **30%.** For Case 2 (which allows CVs to use either dedicated CV lanes or MLs), it was found that the maximum conflict reduction (57.53%) occurred when the MPR% was **25%** during peak condition. On the other hand, in off-peak conditions, it was found that at an MPR% of **30%**, the maximum conflict reduction occurred, which was 51.03%. For Case 3 (which allows CVs to use only dedicated CV lanes), it was found that there was no conflict reduction in the case of peak condition. The safest MPR was 15%, which had an increase of conflicts by 21.68%. However, in the off-peak condition, there was a conflict reduction of 9.46% at the safest MPR%, which was 20%. Figures 50 and 51 show the conflict reduction (value more than zero) and conflict increase (value less than zero) for Cases 1, 2, and 3 for peak and off-peak conditions, respectively.

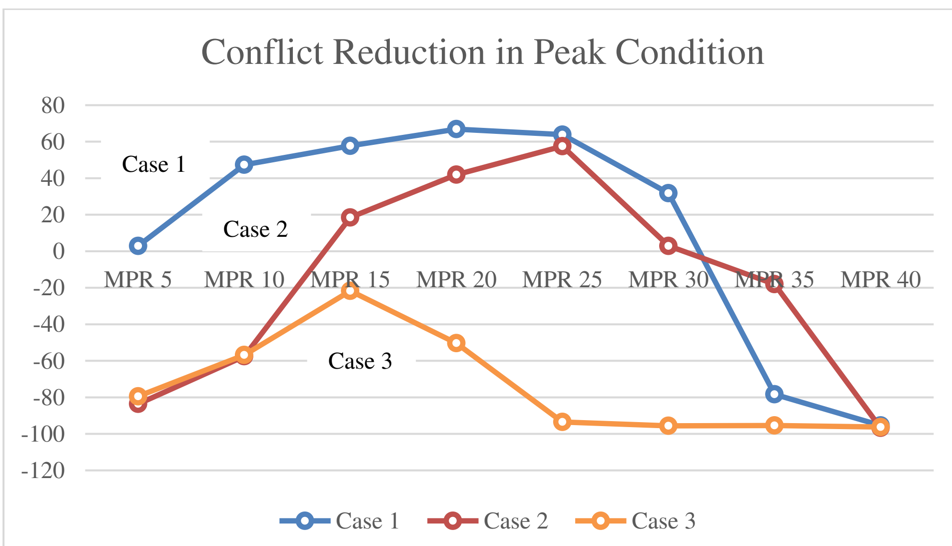



**Figure 50. Conflict Reduction for Peak Conditions in Cases 1, 2, and 3**

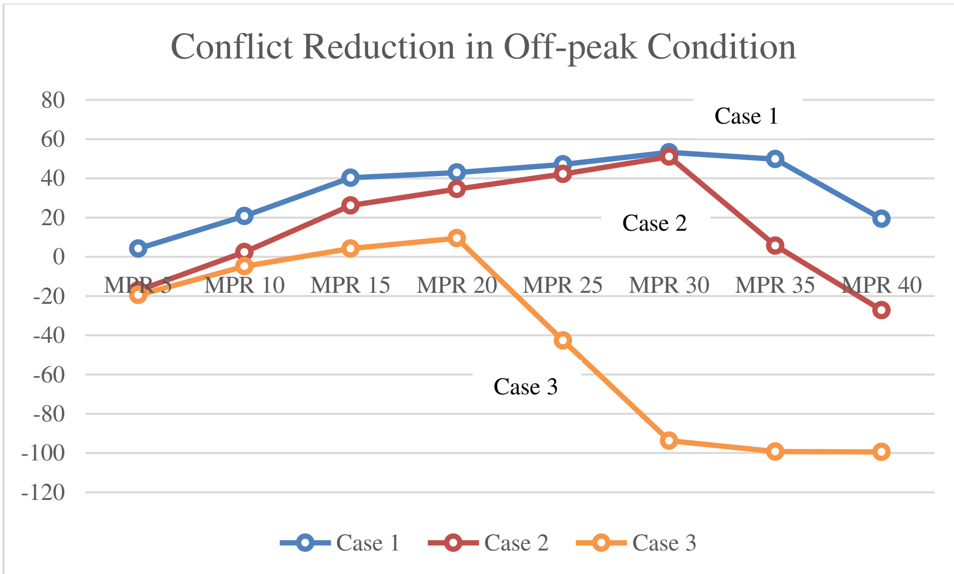



**Figure 51. Conflict Reduction for Off-Peak Conditions in Cases 1, 2, and 3**

According to the conflict reduction results for Case 4 (which is similar to Case 1 with converting one lane of GPLs to MLs), it was found that the maximum conflict reduction occurred when the MPR% was between **40% and 60%** for the peak condition. The maximum conflict reduction occurred at an MPR% of 50% with a value of 70.67%. The conflict reduction decreased when the MPR% reached 80% or more. For off-peak conditions, it is worth mentioning that the lowest conflict reduction occurred when the MPR% was between **50% to 70%.** The maximum reduction occurred when the MPR was 60% with a value of 61.02%. Figure 52 shows the conflict reduction for Case 4.

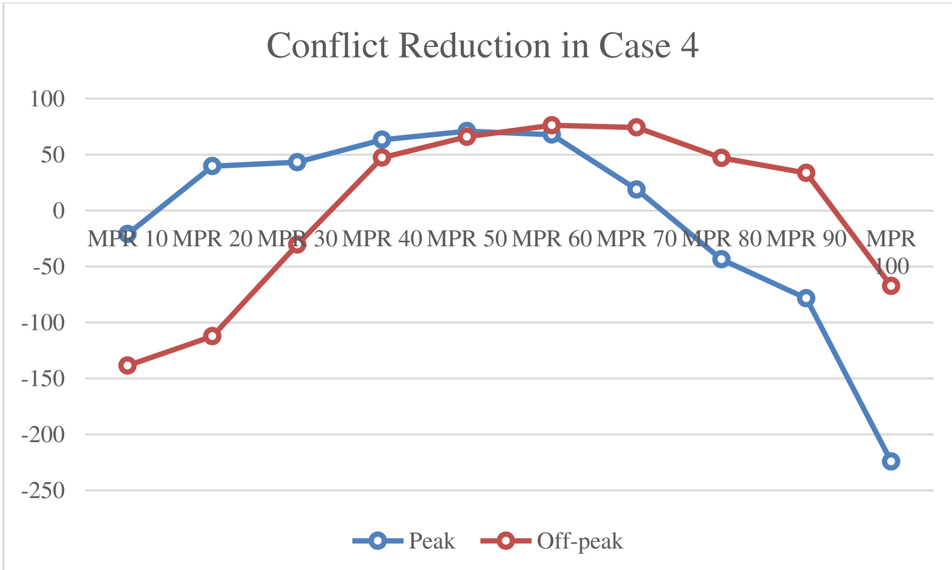



**Figure 52. Conflict Reduction for Peak and Off-peak Condition in Case 4**

For Case 5 (which is similar to Case 1 with an increase in the number of MLs), it was found that the maximum conflict reduction occurred when the MPR% was between **50% and 70%** for the peak condition. The maximum conflict reduction occurred at an MPR% of 60% with a value of 74.19%. The conflict reduction decreased when the MPR% reached 80% or more. For off-peak conditions, it is worth mentioning that the lowest conflict reduction occurred when the MPR% was between **60% to 80%.** The maximum reduction occurred when the MPR was 70% with a value of 64.21%. Figure 53 shows the conflict reduction for Case 5.

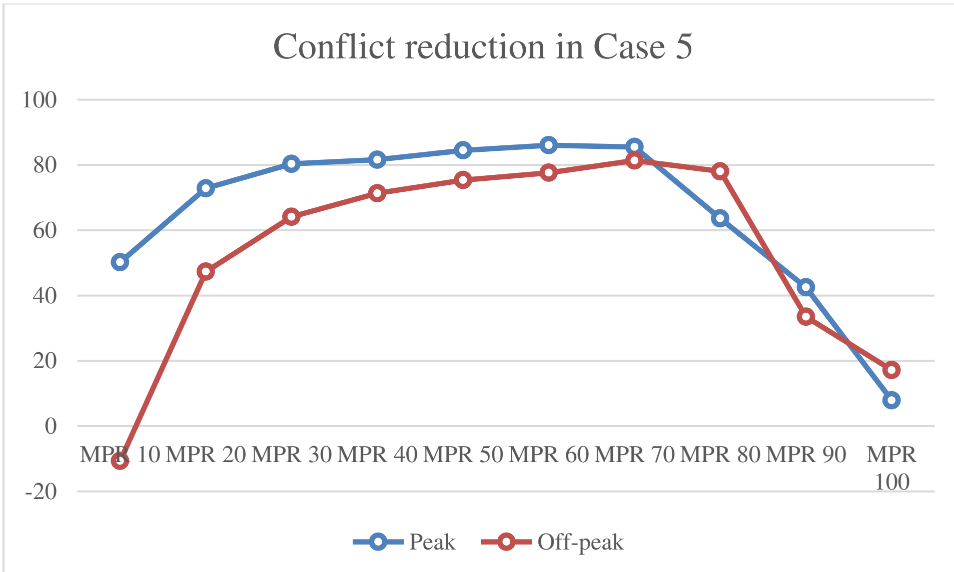



**Figure 53. Conflict Reduction for Peak and Off-peak Condition in Case 5**

Figure 54 shows the conflict reduction (compared to the base case with no CVs) for Case 6 (which allows CVs to use any of CVLs, MLs, or GPLs) in all studied MPR%. In peak conditions, it was found that the maximum conflict reduction occurred at a higher MPR%. There was a positive association between higher MPR% and the conflict reduction. The highest conflict reduction occurred at an MPR% of 100% with a conflict reduction of 71.14%. With an MPR% between **60% and 100%,** the conflict reduction could reach between 52% and 70%. Also, the conflict reduction could reach 10% to 20% when the MPR% was at 20% to 40%. It was also noted that at an MPR% of 10%, there was no conflict reduction in the network. It is also worth noting that the off-peak conditions followed the same conflict reduction distribution as the peak conditions. Therefore, a higher MPR% could be recommended for improving the network safety in Case 6. The highest conflict reduction was reached at an MPR% of 100% with a reduction of 62.74% in off-peak conditions.

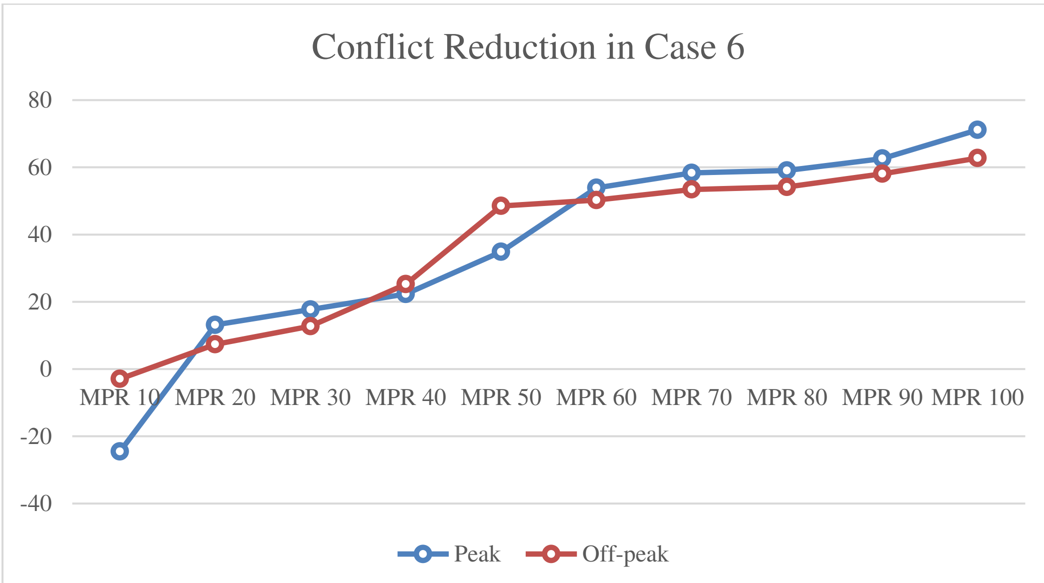



**Figure 54. Conflict Reduction for Peak and Off-peak Condition in Case 6**

### 4.3.3 Statistical Modeling

Negative Binomial (NB) attempt to quantify the effect of contributing factors on conflict frequencies in the managed lanes network. The conflict frequency was considered as the dependent variable. The lane configuration cases, market penetration rates, and traffic conditions were served as the independent variables. The model formulation takes the following form:

$$\lambda = \exp(\beta_0 + \beta_z X + \varepsilon) \tag{17}$$

Where $\lambda$ is the response variable (conflict frequency); $\beta_0$ is the intercept; $X$ represents the different scenarios in all of the cases and $\beta_z$ are corresponding coefficients to be estimated; z represents the different scenarios of various cases and MPR%; $\varepsilon$ is the gamma-distributed error term with a mean equal to 1 and variance α (i.e., over-dispersion parameter). The results of the models are shown in Table 25. In the model, the base case with no CVs in the network was set as the baseline.

The Results of the Negative Binomial model confirmed the results of the Tobit model. According to the NB model results, it can be inferred that, for Case 1 (CVs can use any of the MLs), an MPR% of 20% and 25% had a significantly lower conflict frequency than the base condition. Specifically, an MPR% of 25% is the safest option compared to all other MPR%'s in Case 1. On the other hand, an MPR% of 35% or higher was not recommended since it had a significantly higher conflict frequency than the base case. Moreover, it is apparent from the table that an MPR% of 25% was the safest option for Case 2 (CVs can use either MLs or CVLs), with the lowest conflict frequency among all studied rates. A range of **25% to 30%** could be recommended as the safest MPR% in Case 2 with the lowest conflict frequencies. Furthermore, an inspection of the results in the previous table revealed that an MPR% of Case 3 (CVs only allowed in CVLs) had the highest conflict frequency among all other studied rates. Hence, Case 3 was not recommended in this study. It is also apparent from the table that an MPR% of 25% and higher had significantly higher conflicts than the base condition.

Interestingly, for Case 4, (same as Case 1 with converting one lane of GPLs to MLs), it was found that an MPR% between 4**0% and 60%** had a significantly lower conflict frequency than the base case. Specifically, an MPR% of 50% had the lowest conflict frequency with the lowest estimate among all rates. For Case 5 (same as Case 1 with an increase in the number of MLs), it was found that an MPR% between **30% and 70%** had a significantly lower conflict frequency than the base case. Specifically, an MPR% of 60% had the lowest conflict frequency with the lowest estimate among all rates. For Case 6 (CVs can use any lane in the network), it was found that the maximum conflict reduction occurred at a higher MPR%. There was a significant positive association between a higher MPR% and the reduction of conflict frequency. Specifically, an MPR% between 60% and 100% had a significantly lower conflict frequency than the base case. An MPR% of 100% had the lowest conflict frequency with the lowest estimate among all rates. Also, it can be concluded that an MPR% between **60% and 100%** is recommended, since it generated the lowest number of conflicts in the network in Case 6. Furthermore, it is apparent from the traffic conditions that peak conditions had significantly higher conflicts than off-peak conditions.

**Table 24. Negative Binomial Model for Conflict Frequency**

| Parameter | Estimate | P-value | Parameter | Estimate | P-value | Parameter | Estimate | P-value |
|---|---|---|---|---|---|---|---|---|
| **Intercept** | 9.216 | <.0001 | | | | | | |
| **Case 1 MPR 5%** | -0.037 | 0.933 | **Case 3 MPR 15%** | 0.094 | 0.831 | **Case 5 MPR 30%** | -0.835 | 0.060 |
| **Case 1 MPR 10%** | -0.395 | 0.373 | **Case 3 MPR 20%** | 0.333 | 0.451 | **Case 5 MPR 40%** | **-0.946** | **0.033** |
| **Case 1 MPR 15%** | -0.656 | 0.139 | **Case 3 MPR 25%** | **1.342** | **0.002** | **Case 5 MPR 50%** | **-1.042** | **0.019** |
| **Case 1 MPR 20%** | **-0.870** | **0.049** | **Case 3 MPR 30%** | **2.492** | **<0.0001** | **Case 5 MPR 60%** | **-1.123** | **0.012** |
| **Case 1 MPR 25%** | -0.841 | 0.058 | **Case 3 MPR 35%** | **2.877** | **<0.0001** | **Case 5 MPR 70%** | **-1.155** | **0.009** |
| **Case 1 MPR 30%** | -0.575 | 0.194 | **Case 3 MPR 40%** | **3.133** | **<0.0001** | **Case 5 MPR 80%** | -0.739 | 0.096 |
| **Case 1 MPR 35%** | 0.843 | 0.057 | **Case 4 MPR 10%** | 0.631 | 0.169 | **Case 5 MPR 90%** | -0.258 | 0.561 |
| **Case 1 MPR 40%** | **2.308** | **<0.0001** | **Case 4 MPR 20%** | 0.384 | 0.403 | **Case 5 MPR 100%** | 0.017 | 0.969 |
| **Case 2 MPR 5%** | **1.213** | **0.006** | **Case 4 MPR 30%** | -0.015 | 0.975 | **Case 6 MPR 10%** | 0.198 | 0.655 |
| **Case 2 MPR 10%** | 0.461 | 0.297 | **Case 4 MPR 40%** | -0.806 | 0.080 | **Case 6 MPR 20%** | -0.071 | 0.873 |
| **Case 2 MPR 15%** | -0.259 | 0.559 | **Case 4 MPR 50%** | **-0.953** | **0.029** | **Case 6 MPR 30%** | -0.126 | 0.776 |
| **Case 2 MPR 20%** | -0.476 | 0.283 | **Case 4 MPR 60%** | **-1.026** | **0.018** | **Case 6 MPR 40%** | -0.234 | 0.597 |
| **Case 2 MPR 25%** | -0.808 | 0.068 | **Case 4 MPR 70%** | -0.696 | 0.130 | **Case 6 MPR 50%** | -0.553 | 0.212 |
| **Case 2 MPR 30%** | -0.767 | 0.084 | **Case 4 MPR 80%** | 0.163 | 0.723 | **Case 6 MPR 60%** | -0.765 | 0.085 |
| **Case 2 MPR 35%** | 0.020 | 0.964 | **Case 4 MPR 90%** | **0.985** | **0.032** | **Case 6 MPR 70%** | -0.832 | 0.061 |
| **Case 2 MPR 40%** | **2.659** | **<0.0001** | **Case 4 MPR 100%** | **1.598** | **0.0005** | **Case 6 MPR 80%** | -0.851 | 0.055 |
| **Case 3 MPR 5%** | **1.045** | **0.018** | **Case 5 MPR 10%** | -0.015 | 0.972 | **Case 6 MPR 90%** | **-0.920** | **0.038** |
| **Case 3 MPR 10%** | 0.476 | 0.282 | **Case 5 MPR 20%** | -0.578 | 0.192 | **Case 6 MPR 100%** | **-1.097** | **0.012** |
| **Base Condition** | | | | | **Reference** | | | |
| **Peak (v.s. off- peak)** | 1.659 | <0.0001 | | | | | | |
| **Over-dispersion** | 0.194 | <0.0001 | | | | | | |
| **R-Square** | | | | | 0.354 | | | |

## 4.4 Operational Analysis

The traffic operation measurements were analyzed to assess the operational effects of adding CVs and CV lanes on freeway facilities with managed lanes. The evaluation measures for traffic operation included the average travel speed, and average delay.

### 4.4.1 Average Speed

Average travel speed was one of the measurements of effectiveness used to evaluate the performance of the network and to compare the average travel speeds between different cases in the system. The descriptive statistics of the average speed for all studied case are shown in Table 25 for both peak and off-peak periods. The results of the table indicated that Cases 5 and 6 had the highest average speed among all cases. Case 3 showed the lowest average travel speed. An ANOVA test was carried out to compare the average speed in various CV lane design cases, MPR%, and traffic conditions. The results showed that there was a significant difference in average speed between cases (F-value=21.45, P-value<0.0001). The results also showed significant differeces in average speed between different MPR% (F-value=8.71, P-value<0.0001). Additionally, the results showed that speeds (F-value=84.79, P-value<0.0001) were lower in the peak conditions comparing to the off-peak conditions. A post-hoc test was conducted to test the significant difference between different cases, as shown in Table 26. The results revealed that there was no significant difference of speed between Cases 1, 2, 4, 5, and 6. Case 3 had significant lower speed that all other cases.

**Table 25. Descriptive Statistics of Average Speed in All studied Cases**

| Case | Traffic Condition | Mean | Standard Deviation | Minimum | Maximum |
|---|---|---|---|---|---|
| **Base** | **Peak** | 58.286 | - | 58.286 | 58.286 |
| | **Off-peak** | 59.924 | - | 59.924 | 59.924 |
| **Case 1** | **Peak** | 59.621 | 3.709 | 53.681 | 63.701 |
| | **Off-peak** | 62.479 | 2.609 | 58.515 | 65.049 |
| **Case 2** | **Peak** | 58.680 | 1.994 | 55.795 | 61.187 |
| | **Off-peak** | 63.029 | 2.826 | 57.244 | 66.127 |
| **Case 3** | **Peak** | 54.027 | 2.667 | 50.719 | 57.799 |
| | **Off-peak** | 58.665 | 3.269 | 53.622 | 62.127 |
| **Case 4** | **Peak** | 59.408 | 4.231 | 52.144 | 64.286 |
| | **Off-peak** | 62.726 | 1.921 | 59.343 | 65.123 |
| **Case 5** | **Peak** | 60.450 | 3.427 | 52.645 | 63.671 |
| | **Off-peak** | 63.016 | 1.778 | 59.721 | 65.847 |
| **Case 6** | **Peak** | 59.940 | 2.582 | 56.519 | 64.021 |
| | **Off-peak** | 63.353 | 2.311 | 60.030 | 66.408 |

**Table 26. Post-hoc Test of Average Speed between Cases**

| Case | | Estimate | P-Value |
|---|---|---|---|
| Case 1 | Case 2 | 0.2079 | 0.8536 |
| Case 1 | Case 4 | -0.6592 | 0.5378 |
| Case 1 | Case 5 | -0.7673 | 0.4735 |
| Case 1 | Case 6 | -0.1736 | 0.8705 |
| Case 4 | Case 5 | -0.1081 | 0.9146 |
| Case 4 | Case 6 | 0.4855 | 0.6290 |
| Case 5 | Case 6 | 0.5937 | 0.5548 |
| Case 2 | Case 4 | -0.8671 | 0.4181 |
| Case 2 | Case 5 | -0.9753 | 0.3627 |
| Case 2 | Case 6 | -0.3816 | 0.7203 |
| Case 3 | Case 1 | -4.5481 | 0.0001 |
| Case 3 | Case 2 | -4.3401 | 0.0002 |
| Case 3 | Case 4 | -5.2073 | <0.0001 |
| Case 3 | Case 5 | -5.3154 | <0.0001 |
| Case 3 | Case 6 | -5.1342 | <0.0001 |

In Case 1, compared to all studied market penetration rates, the average speed peaked when the MPR% was 25% in peak conditions. The lowest speed occurred when the MPR% was lower than 10%. In off-peak conditions, the highest average speed occurred when the MPR% was 30%. Figure 55 provides the distribution of average speed in Case 1 for all studied MPR%.

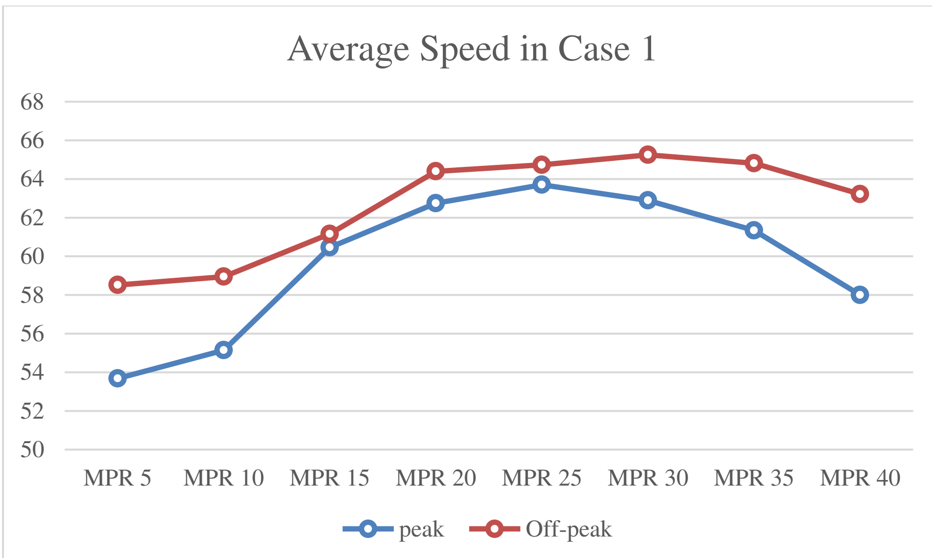



**Figure 55. Average Speed for Different MPR% in Case 1**

The results of the speed distribution in Case 2 for different MPR% set out that average speed peaked when the MPR% was 25% in peak conditions. Interestingly, in off-peak conditions, there was a clear trend of increasing the average speed with the increase of MPR% until the MPR% of 25%. Then, speeds decrease in the network with the increase of MPR%. The lowest speed occurred when the MPR% was less than 25%. Figure 56 displays the average speed distribution for peak and off-peak conditions in Case 2.

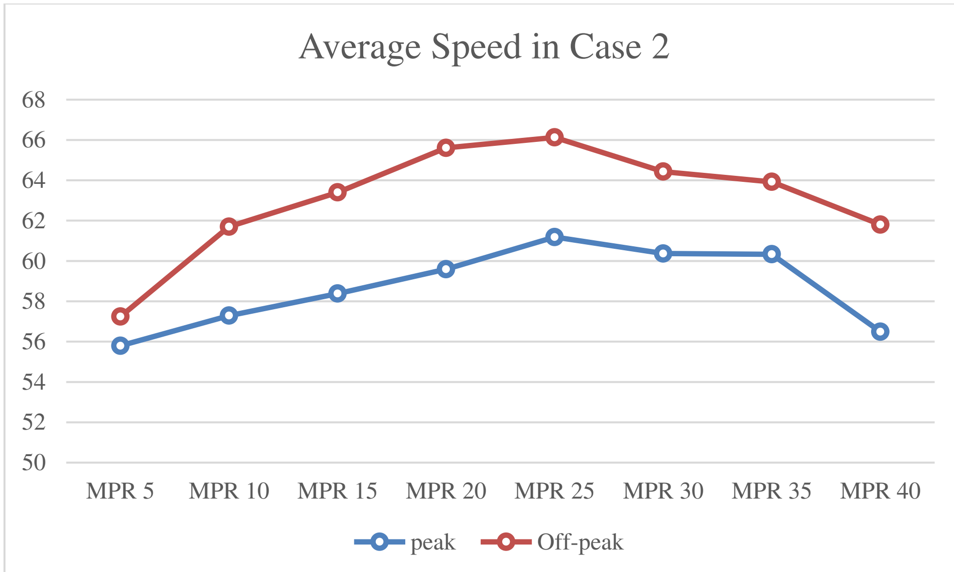



**Figure 56. Average Speed for Peak and Off-peak Conditions in Case 2**

The distribution for average speed for different MPR% in Case 3 is presented in Figure 57. What stands out in this figure is that, compared to all studied market penetration rates, the average speed peaked when the MPR% was 15% in peak conditions. In off-peak conditions, the highest average speed occurred when the MPR% was 30%. The figure also highlighted that the lowest speeds occurred when the MPR% is higher than 30%.

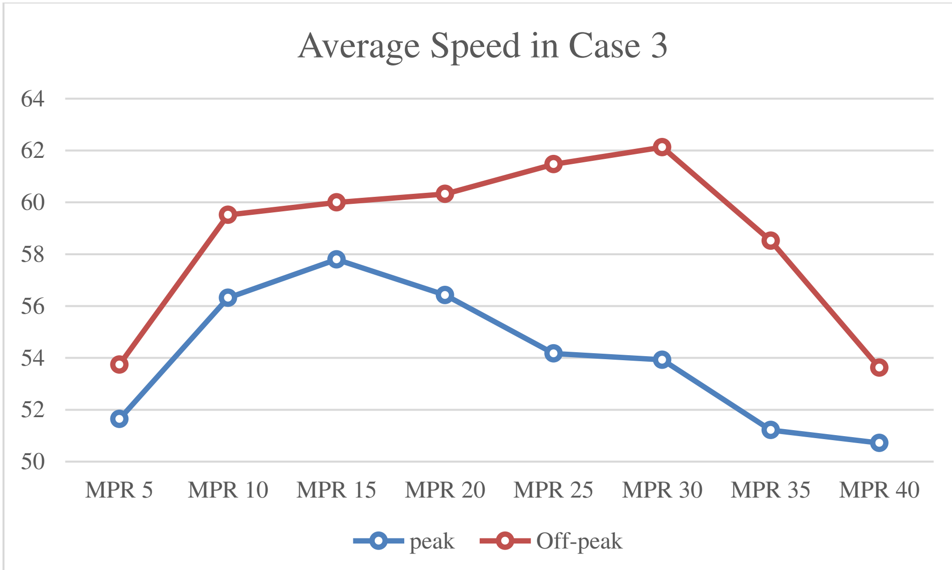



**Figure 57. Average Speed for Peak and Off-peak Conditions in Case 3**

The distribution for average speed for different MPR% in Case 4 is provided in Figure 58. What stands out in this figure is that, compared to all studied market penetration rates, the average speed peaked when the MPR% was 50% in peak conditions. The figure also highlighted that the lowest speeds occurred when the MPR% was higher than 80%. In off-peak conditions, the highest average speed occurred when the MPR% was 70%.

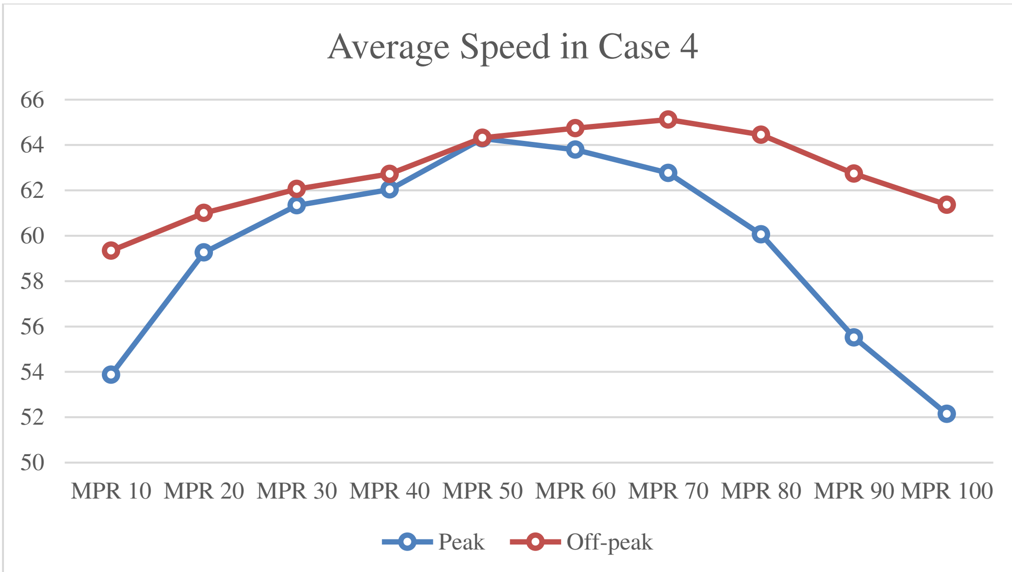



**Figure 58. Average Speed for Peak and Off-peak Conditions in Case 4**

The distribution for average speed for different MPR% in Case 5 is provided in Figure 59. What stands out in this figure is that, compared to all studied market penetration rates, the average speed peaked when the MPR% was 60% in peak conditions. The figure also highlighted that the lowest speeds occurred when the MPR% was higher than 80%. In off-peak conditions, the highest average speed occurred when the MPR% was 80%.

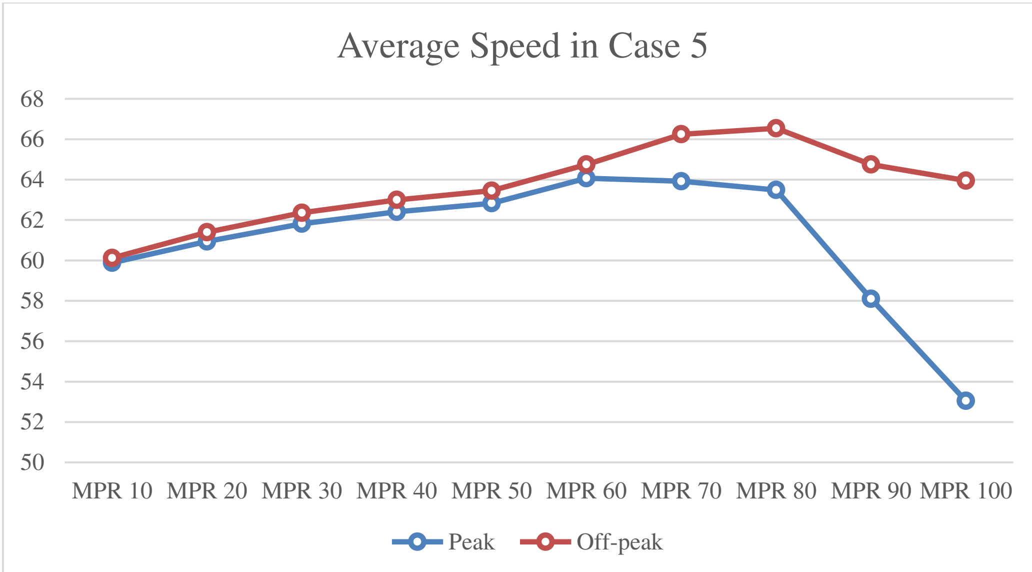



**Figure 59. Average Speed for Peak and Off-peak Conditions in Case 5**

Figure 60 provides the speeds in Case 6 for all studied MPR% in both peak and off-peak conditions. Compared to all studied market penetration rates, the average speed peaked with higher MPR% in peak conditions. The highest speeds occurred when the MPR% was 100%. Similarly, in off-peak conditions, the highest average speed occurred with higher MPR%.

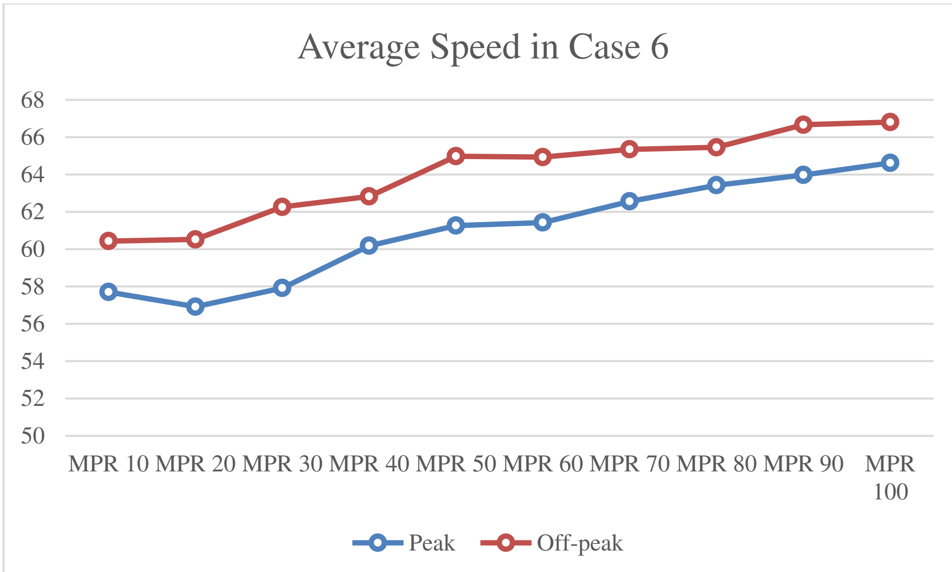



**Figure 60. Average Speed for Peak and Off-peak Conditions in Case 6**

4.4.1.1 Speed Increase

Further analysis was implemented to investigate the speed increase in different scenarios. The speed increase was calculated based on the difference between the average speeds of the different studied cases and the base case as shown in the following equation:

$$\text{Speed Increase} = \frac{\textit{Average Speed in case of CVs} - \textit{Average Speed in base case}}{\textit{Average Speed in base case}} \qquad (20)$$

For Case 1 (which allow CVs to use any of the MLs), the results of adding CVs to the MLs network revealed that the maximum speed increase (compared to the base case with no CVs) occurred at an MPR% of 25% during peak conditions. The speed increase reached 8.51% more than any other cases. Regarding off-peak conditions, the maximum speed increase was 7.87% and it happened when the MPR% was 25%. For Case 2 (which allows CVs to use either dedicated CV lanes or MLs), it was found that the maximum speed increase (7.74%) occurred when the MPR% was 25% during peak condition. On the other hand, in off-peak conditions, it was found that at an MPR% of 25%, the maximum speed increase occurred at 9.38%. For Case 3 (which allows CVs to use only dedicated CV lanes), it was found that there was no speed increase in the case of peak condition. However, in the off-peak condition, there was a speed increase of 3.54% at the optimal MPR%, which was 30%. Figures 61 and 62 show the speed increase for cases 1, 2, and 3 for peak and off-peak conditions, respectively.

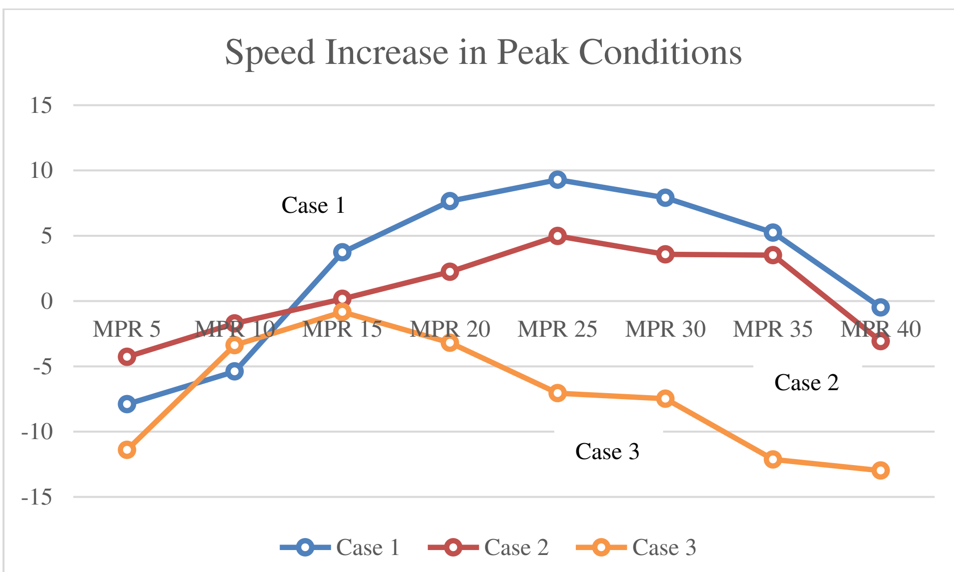



**Figure 61. Speed Increase for Peak Condition in Cases 1, 2, and 3**

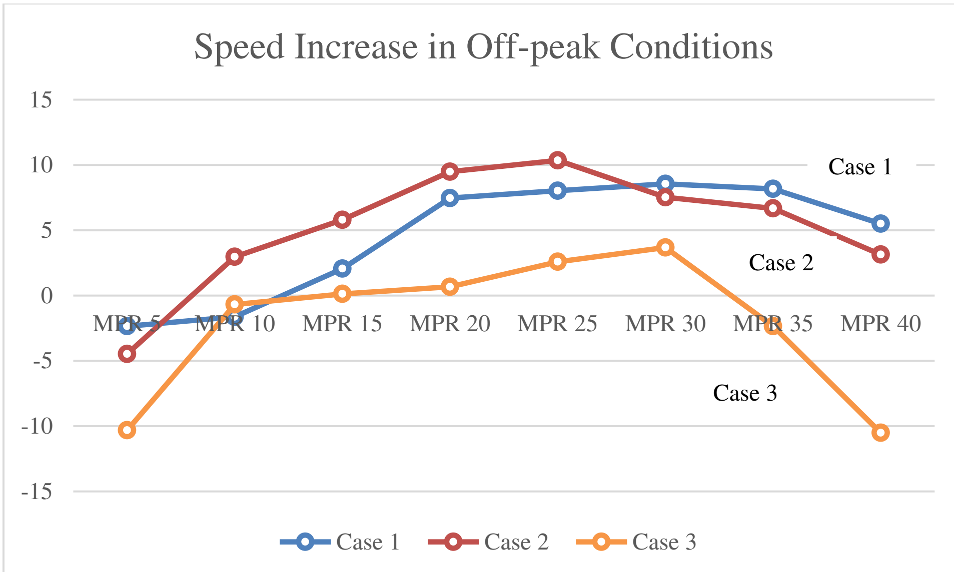



**Figure 62. Speed Increase for the Off-peak Condition in Cases 1, 2, and 3**

Figure 63 represents the speed increase for peak and off-peak conditions in Case 4. As can be seen from the figure, in Case 4, the highest speed increase occurred when the MPR% was 50% in peak conditions with a 12.45% increase compared to the base condition. The results also revealed that the speed increase deteriorated after an MPR% of 70%. In off-peak conditions, the highest speed increase occurred when the MPR% was 70% with an 11.05% speed increase.

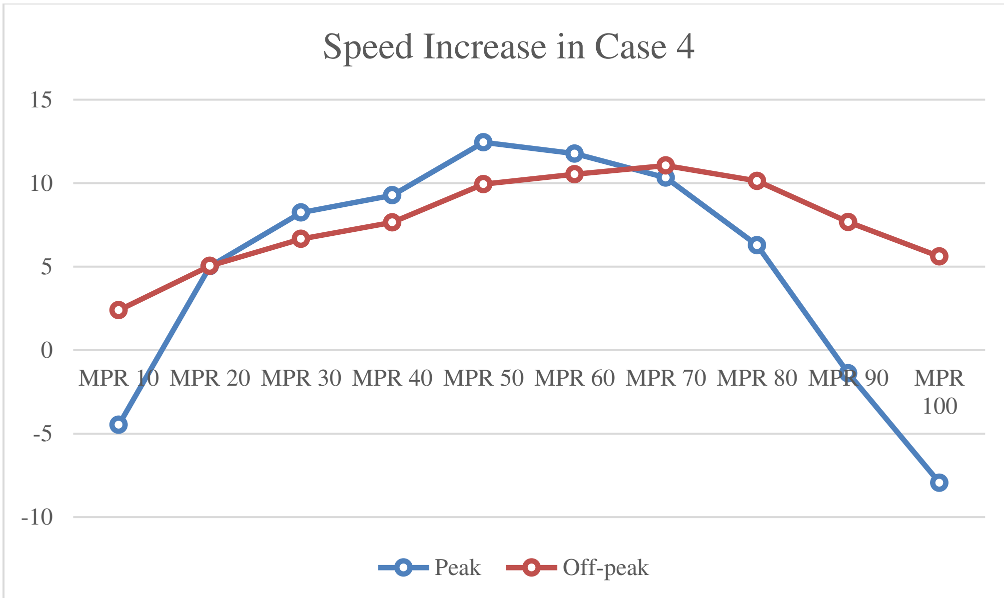



**Figure 63. Speed Increase for Peak and Off-peak Conditions in Case 4**

Figure 64 represents the speed increase for peak and off-peak conditions in Case 5. As can be seen from the figure, in Case 5, the highest speed increase occurred when the MPR% was 60% in peak conditions with a 12.15% increase compared to the base condition. The results also revealed that the speed increase deteriorated after an MPR% of 80%. In off-peak conditions, the highest speed increase occurred when the MPR% was 80% with an 12.96% speed increase.

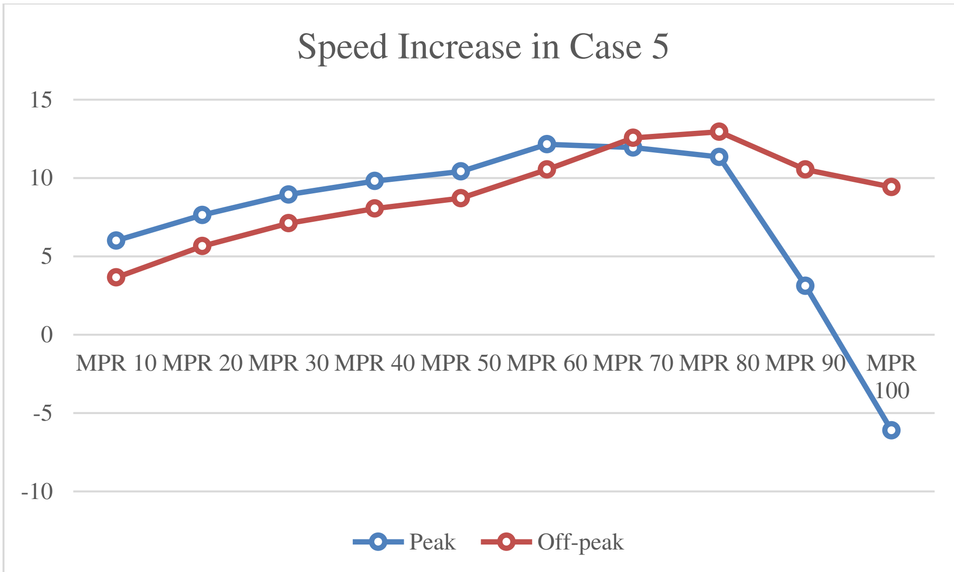



**Figure 64. Speed Increase for Peak and Off-peak Conditions in Case 5**

Figure 65 shows the speed increase (compared to the base case with no CVs) for Case 6 (which allows CVs to use any of CVLs, MLs, or GPLs) in all studied MPR%. In peak conditions, it was found that the maximum speed increase occurred at higher MPR%. There was a positive association between higher MPR% and the speed increase. The highest speed increase occurred at an MPR% of 100% with a speed increase of 12.89%. With MPR% between 70% and 90%, the speed increase could reach between 10.03% and 12.1%. It is worth noting that the off-peak conditions followed the same speed increase distribution of the peak conditions. Therefore, a higher MPR% could be recommended for improving the network safety in Case 6. The highest speed increase was reached in off-peak conditions at an MPR% of 100% with an increase of 13.29%.

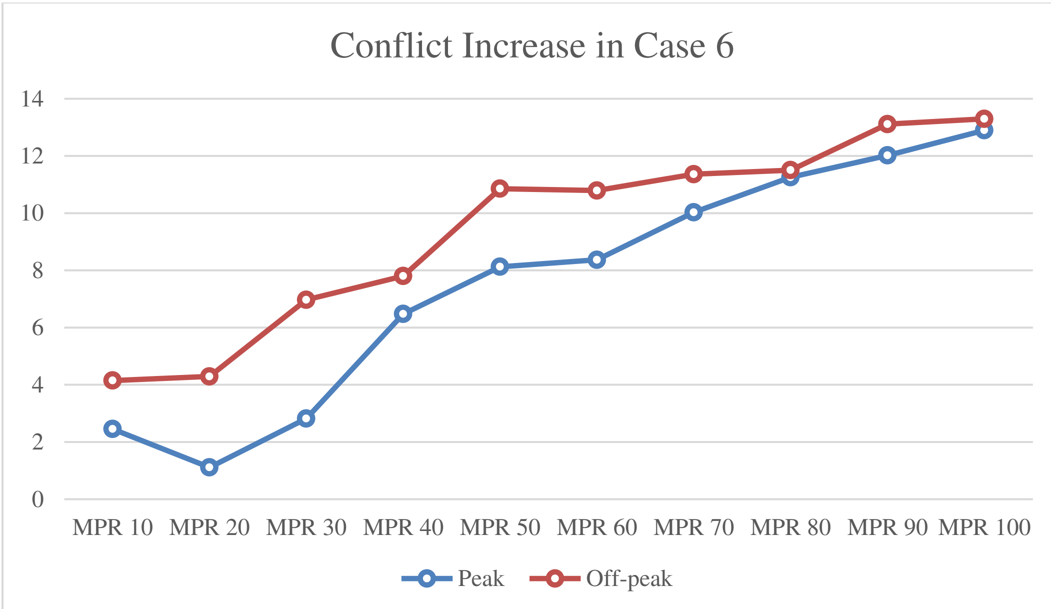



**Figure 65. Speed Increase for Peak and Off-peak Conditions in Case 6**

4.4.1.2 Statistical Modeling

Tobit models have been used in this study since it is a regression model that can model continuous dependent variable which can be censored to lower threshold, upper threshold, or both. Tobit model was developed to decide on the best scenario with an optimal MPR% among all studied scenarios. In the model, different scenario variables of various lane configuration cases and MPR% of CVs were included in the model. In addition, traffic conditions (peak, off-peak) were considered. The statistical analysis software (SAS 9.4) was used for generating the model results. Table 27 provides the model results.

## Table 27. Tobit Model for Average Speed

| Parameter | Estimate | P-value | Parameter | Estimate | P-value | Parameter | Estimate | P-value |
|---|---|---|---|---|---|---|---|---|
| **Intercept** | 61.061 | <.0001 | | | | | | |
| **Case 1 MPR 5%** | **-3.256*** | **0.003** | **Case 3 MPR 15%** | -0.456 | 0.678 | **Case 5 MPR 30%** | **2.183*** | **0.047** |
| **Case 1 MPR 10%** | -1.310 | 0.233 | **Case 3 MPR 20%** | -0.981 | 0.372 | **Case 5 MPR 40%** | **2.746*** | **0.012** |
| **Case 1 MPR 15%** | 1.455 | 0.186 | **Case 3 MPR 25%** | **-2.033*** | **0.0346** | **Case 5 MPR 50%** | **3.287*** | **0.003** |
| **Case 1 MPR 20%** | **3.716*** | **0.001** | **Case 3 MPR 30%** | **-3.127*** | **0.0012** | **Case 5 MPR 60%** | **4.406*** | **<0.0001** |
| **Case 1 MPR 25%** | **4.862*** | **<0.0001** | **Case 3 MPR 35%** | **-4.488*** | **<0.0001** | **Case 5 MPR 70%** | **4.678*** | **<0.0001** |
| **Case 1 MPR 30%** | **4.617*** | **<0.0001** | **Case 3 MPR 40%** | **-7.184*** | **<0.0001** | **Case 5 MPR 80%** | **3.760*** | **0.001** |
| **Case 1 MPR 35%** | 1.473 | 0.180 | **Case 4 MPR 10%** | **-2.746*** | **0.011** | **Case 5 MPR 90%** | 0.868 | 0.430 |
| **Case 1 MPR 40%** | 0.755 | 0.492 | **Case 4 MPR 20%** | 0.778 | 0.471 | **Case 5 MPR 100%** | -1.606 | 0.144 |
| **Case 2 MPR 5%** | **-2.835*** | **0.010** | **Case 4 MPR 30%** | 1.545 | 0.151 | **Case 6 MPR 10%** | -0.689 | 0.531 |
| **Case 2 MPR 10%** | 0.139 | 0.900 | **Case 4 MPR 40%** | **3.026*** | **0.005** | **Case 6 MPR 20%** | -1.034 | 0.346 |
| **Case 2 MPR 15%** | 1.536 | 0.162 | **Case 4 MPR 50%** | **4.697*** | **<0.0001** | **Case 6 MPR 30%** | 0.337 | 0.759 |
| **Case 2 MPR 20%** | **3.395*** | **0.002** | **Case 4 MPR 60%** | **4.762*** | **<0.0001** | **Case 6 MPR 40%** | 1.250 | 0.255 |
| **Case 2 MPR 25%** | **4.302*** | **<0.0001** | **Case 4 MPR 70%** | **4.588*** | **<0.0001** | **Case 6 MPR 50%** | 1.516 | 0.168 |
| **Case 2 MPR 30%** | **3.046*** | **0.006** | **Case 4 MPR 80%** | **2.901*** | **0.007** | **Case 6 MPR 60%** | **2.574*** | **0.019** |
| **Case 2 MPR 35%** | 1.273 | 0.247 | **Case 4 MPR 90%** | -0.229 | 0.832 | **Case 6 MPR 70%** | **3.701*** | **0.001** |
| **Case 2 MPR 40%** | -0.207 | 0.850 | **Case 4 MPR 100%** | **-2.601*** | **0.016** | **Case 6 MPR 80%** | **4.184*** | **0.0001** |
| **Case 3 MPR 5%** | **-6.663*** | **<0.0001** | **Case 5 MPR 10%** | 0.247 | 0.822 | **Case 6 MPR 90%** | **5.366*** | **<0.0001** |
| **Case 3 MPR 10%** | -1.439 | 0.190 | **Case 5 MPR 20%** | 1.414 | 0.198 | **Case 6 MPR 100%** | **5.860*** | **<0.0001** |
| **Base Condition** | | | | **Reference** | | | | |
| **Peak (v.s. off- peak)** | -3.393 | <.0001 | | | | | | |
| **α** | 1.127 | <.0001 | | | | | | |
| **R-Square** | | | | 0.443 | | | | |

The results of the Tobit model results revealed that, in Case 1 (CVs can use any of the MLs), an MPR% of 25% had significantly higher speed than the base case with no CVs in the network. Closer inspection of the results revealed that an MPR of 25% had the second highest speed among all studied MPR%, with a significantly higher speed than the base case. On the other side, an MPR% of 10% or lower was not recommended, since it had lower speed than other studied MPR%. As the results shows, an MPR% of **20%-30%** was recommended as the optimal MPR% in Case 1, since it had significantly higher speed than the base condition. Moreover, it is apparent from the table that, for Case 2 (CVs can use either MLs or CVLs), an MPR% of 25% was the best option with the highest speed among all studied rates. A range of **20% to 30%** could be recommended as the optimal MPR% in Case 2 with the highest speeds. It is also apparent from the table that an MPR% of 5% had lower speeds than all other MPR%. Furthermore, an inspection of the results in the previous table revealed that Case 3 (CVs only allowed in CVLs) was not recommended. Case 3 had lower speeds than the base case for all studied MPR%. There was a significantly lower speed, compared to the base case, when the MPR% was 25% or higher. Likewise, an MPR% of 5% showed significantly lower speed than the base case.

For Case 4 (same as Case 1 with converting one GPLs to MLs), it was found that an MPR% between 40% and 80% had significantly higher speed than the base case. Specifically, an MPR% of 50% had the highest speed with the lowest estimate among all rates. Also, it can be concluded that an MPR% between **50% and 70%** is recommended, since it generated the highest speed in the network for Case 4. Interestingly, for Case 5 (same as Case 1 with an increase in the number of MLs), it was found that an MPR% between 30% and 80% had significantly higher speed than the base case. Specifically, an MPR% of 70% had the highest speed with the lowest estimate among all rates. Also, it can be concluded that an MPR% between **60% and 80%** is recommended, since it generated the highest speed in the network for Case 5. For Case 6 (CVs can use any lane in the network), it was found that the maximum speed increase occurred at higher MPR%. There was a significantly positive association between higher MPR% and the increase of speed. Specifically, an MPR% between 60% and 100% had a significantly higher speed than the base case. An MPR% of 100% had the speed with the highest estimate among all rates. Also, it can be concluded that an MPR% between **60% and 100%** is recommended, since it generated the highest speed in the network. Furthermore, it is apparent from the traffic conditions that peak conditions had significantly lower speed than off-peak conditions.

### 4.4.2 Average Delay

The average delay of all vehicles can be measured by subtracting the theoretical travel time from the actual travel time. The theoretical travel time is the free flow travel time. The descriptive statistics of the average delay for all studied case are shown in Table 28 for both peak and off-peak periods. The results of the table indicated that Cases 4 had the lowest average delay among all cases. Case 3 showed the highest delays. An ANOVA test was carried out to compare the average delay in various CV lane design cases, MPR%, and traffic conditions. The results showed that there was a significant difference in average delay between the studied cases (F-value=47.16, p-value<0.0001). The results also showed significant differeces in average delay between different MPR% (F-value=11.87, p-value<0.0001). Additionally, the results showed that delays (F-value=178.86, p-value<0.0001) were higher in the peak conditions comparing to the off-peak conditions. A post-hoc test was conducted to test the significant difference between different cases, as shown in Table 29. Case 3 had significant higher conflicts that all other cases.

**Table 28. Descriptive Statistics for Average Delay in All studied Cases**

| Case | Traffic Condition | Mean | Standard Deviation | Minimum | Maximum |
|---|---|---|---|---|---|
| **Base** | **Peak** | 21.555 | - | 21.555 | 21.555 |
| | **Off-peak** | 17.125 | - | 17.125 | 17.125 |
| **Case 1** | **Peak** | 22.806 | 4.421 | 18.810 | 30.304 |
| | **Off-peak** | 17.195 | 1.591 | 15.736 | 20.460 |
| **Case 2** | **Peak** | 22.447 | 2.744 | 19.265 | 27.646 |
| | **Off-peak** | 18.610 | 2.222 | 16.381 | 22.919 |
| **Case 3** | **Peak** | 30.172 | 5.725 | 22.940 | 38.210 |
| | **Off-peak** | 21.522 | 2.113 | 18.981 | 24.919 |
| **Case 4** | **Peak** | 22.748 | 4.336 | 18.005 | 30.081 |
| | **Off-peak** | 16.809 | 2.493 | 13.864 | 21.048 |
| **Case 5** | **Peak** | 18.114 | 4.672 | 13.308 | 25.742 |
| | **Off-peak** | 13.757 | 1.997 | 11.230 | 17.423 |
| **Case 6** | **Peak** | 21.608 | 3.174 | 18.687 | 28.347 |
| | **Off-peak** | 15.923 | 1.721 | 13.278 | 18.687 |

**Table 29. Post-hoc Test for Delay between Cases**

| Case | | Estimate | P-Value |
|---|---|---|---|
| **Case 1** | **Case 2** | -0.353 | 0.8147 |
| **Case 1** | **Case 4** | 3.247 | 0.0251 |
| **Case 1** | **Case 5** | 1.522 | 0.2881 |
| **Case 1** | **Case 6** | 0.213 | 0.8832 |
| **Case 4** | **Case 5** | -1.725 | 0.2022 |
| **Case 4** | **Case 6** | -3.813 | 0.0062 |
| **Case 5** | **Case 6** | -2.088 | 0.1288 |
| **Case 2** | **Case 4** | 3.601 | 0.0133 |
| **Case 2** | **Case 5** | 1.875 | 0.1914 |
| **Case 2** | **Case 6** | 0.566 | 0.6965 |
| **Case 3** | **Case 1** | 6.296 | <0.0001 |
| **Case 3** | **Case 2** | 5.944 | 0.0002 |
| **Case 3** | **Case 4** | 9.544 | <0.0001 |
| **Case 3** | **Case 5** | 7.819 | <0.0001 |
| **Case 3** | **Case 6** | 5.731 | <0.0001 |

Figure 66 shows the average delay for the average delay in Case 1 for both peak and off-peak conditions. In peak conditions, it can be noted from the figure that the lowest average delay occurred when the MPR% was 20%. Also, the figure showed that average delay increased after an MPR% of 30%. For off-peak conditions, it was noted that the lowest average delay happened when the MPR% was between 10% and 25%. Subsequently, the average delay increased after an MPR% of 30%.

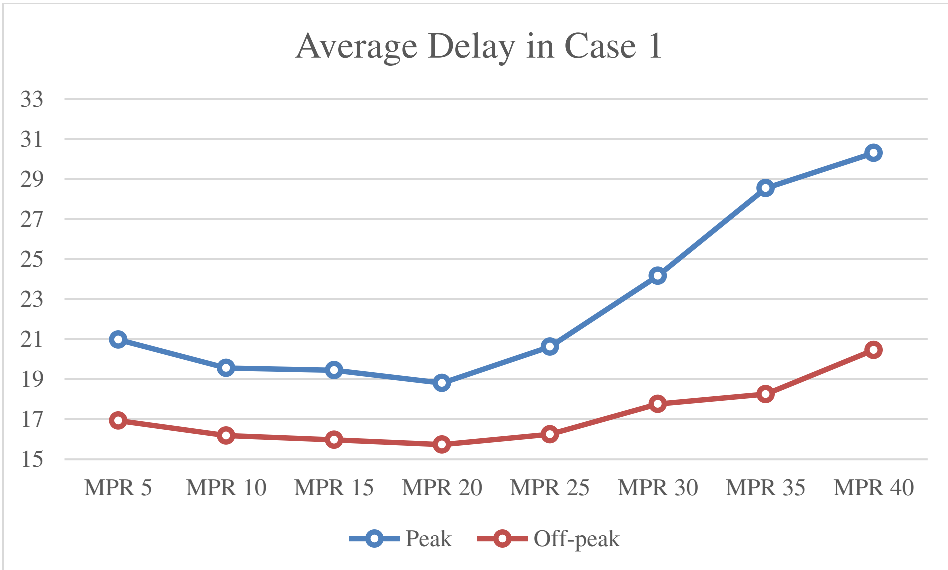



**Figure 66. Average Delay for Peak and Off-peak Conditions in Case 1**

Figure 67 shows the average delay for Case 2 for both peak and off-peak conditions. In peak conditions, it can be noted from the figure that the lowest average delay occurred when the MPR% was 30%. The average delay increased after an MPR% of 30%. For off-peak conditions, it was noted that the lowest average delay occurred when the MPR% was 25%. Subsequently, the average delay increased after an MPR% of 25%.

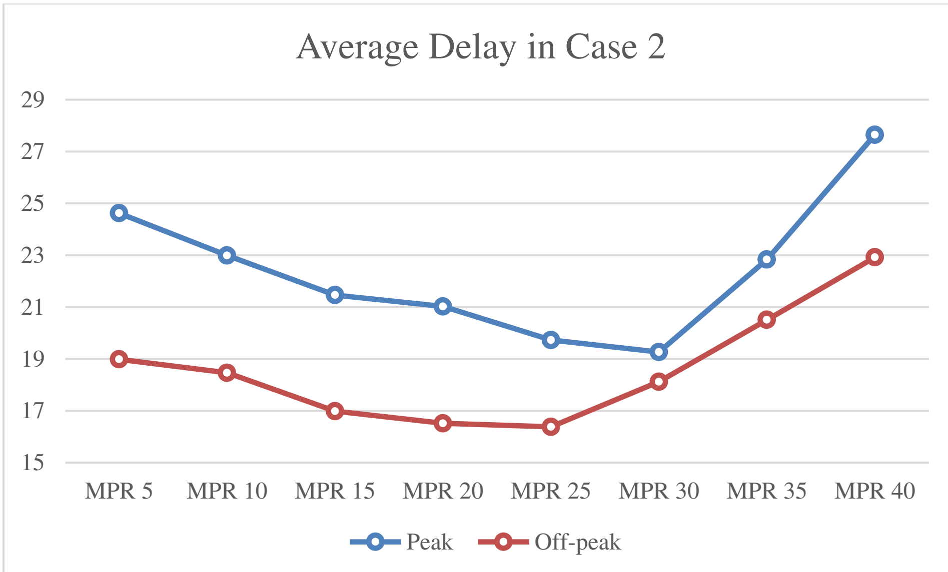



**Figure 67. Average Delay for Peak and Off-peak Conditions in Case 2**

The results of the delay in Case 3 for different MPR% set out that, in peak conditions, the lowest average delay occurred when the MPR% was 20%. Subsequently, the average delay increased after an MPR% of 25%. For off-peak conditions, it was noted that the lowest average delay happened when the MPR% was 25%. The average delay increased after an MPR% of 25%. The average delay for various MPR% in Case 3 is displayed in Figure 68 for both peak and off-peak conditions.

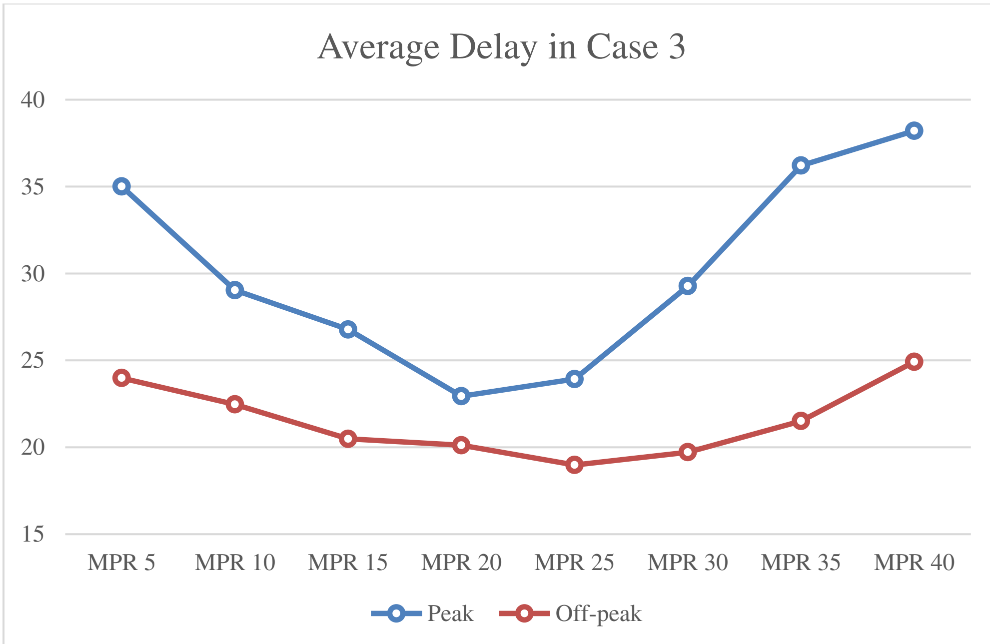



**Figure 68. Average Delay for Peak and Off-peak Conditions in Case 3**

The results of the delay in Case 4 for different MPR% set out that, in peak conditions, the lowest average delay occurred when the MPR% was 50%. Subsequently, the average delay increased after an MPR% of 70%. For off-peak conditions, it was noted that the lowest average delay happened when the MPR% was 60%. The average delay for various MPR% in Case 4 is displayed in Figure 69 for both peak and off-peak conditions.

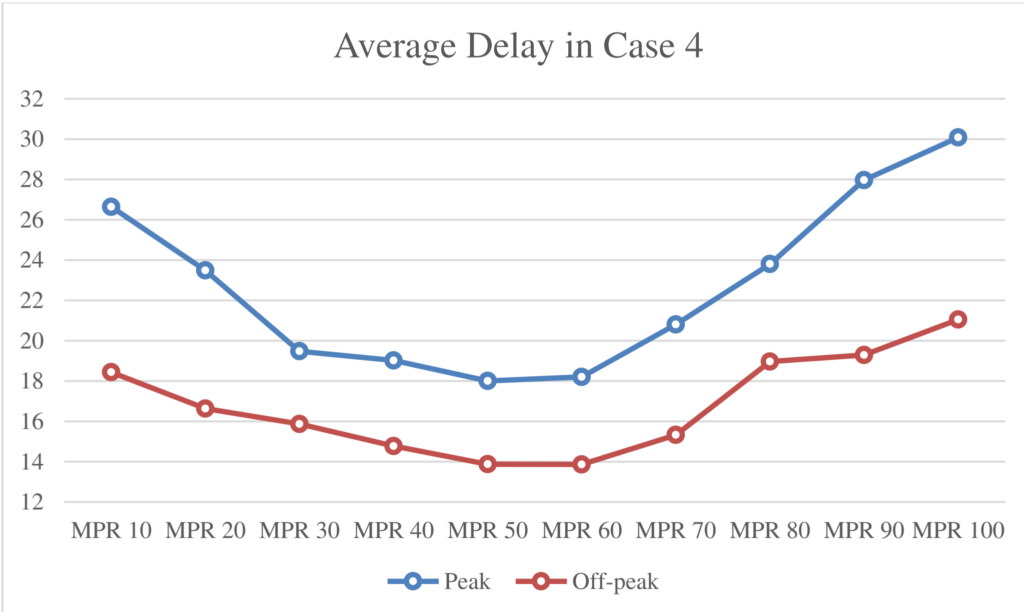



**Figure 69. Average Delay for Peak and Off-peak Conditions in Case 4**

The results of the delay in Case 5 for different MPR% set out that, in peak conditions, the lowest average delay occurred when the MPR% was 70%. Subsequently, the average delay increased after an MPR% of 80%. For off-peak conditions, it was noted that the lowest average delay happened when the MPR% was 70%. The average delay for various MPR% in Case 5 is displayed in Figure 70 for both peak and off-peak conditions.

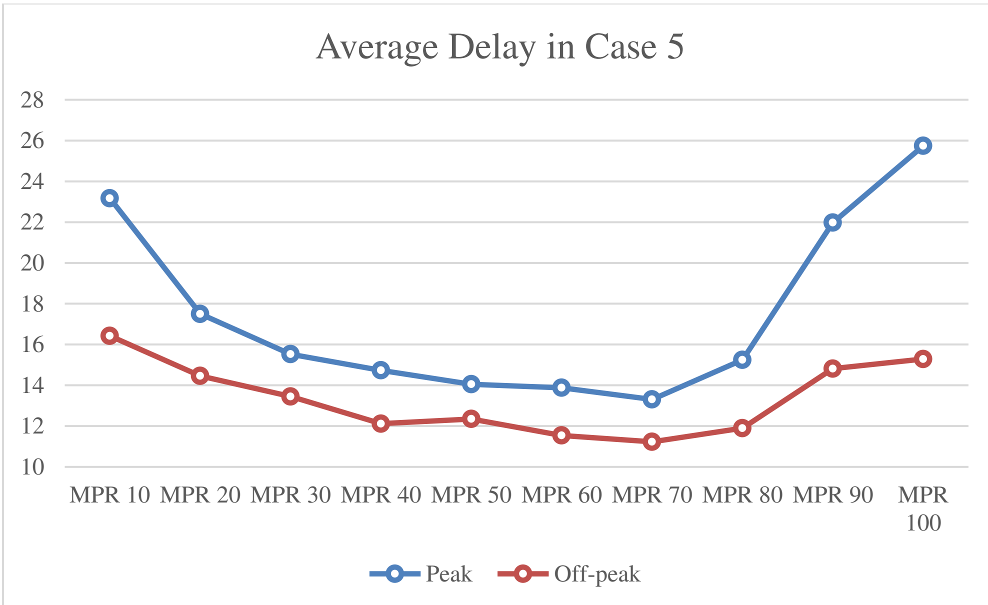



**Figure 70. Average Delay for Peak and Off-peak Conditions in Case 5**

Figure 71 shows the average delay for Case 6 for both peak and off-peak conditions. In peak conditions, it can be noted from the figure that the lowest average delay occurred at higher values of MPR%. The lowest delay occurred when the MPR% was 100%. It can also be seen in the figure that lower MPR% (e.g., 10%, 20%) had higher delay. For off-peak conditions, it was noted that the lowest average delay occurred when the MPR% was 100%.

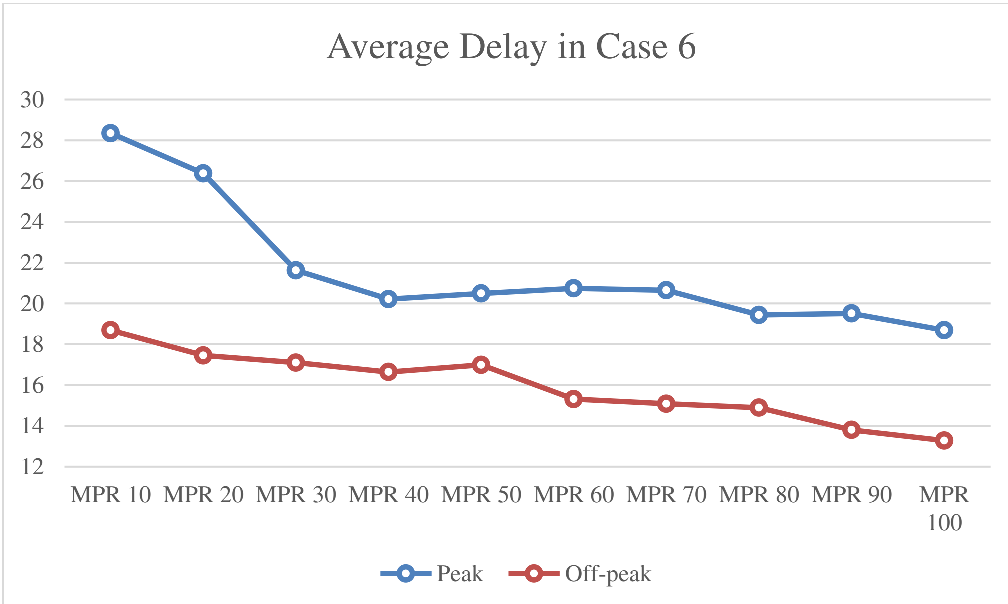



**Figure 71. Average Delay for Peak and Off-peak Conditions in Case 6**

4.4.2.1 Delay Reduction

Delay reduction was calculated based on the delay in the base case and the delay in the studied cases. The delay reduction was calculated as follows:

$$\text{Delay Reduction} = \frac{\textit{Average Delay in base case} - \textit{Average Delay in cases of CVs}}{\textit{Average Delay in base case}} \quad (21)$$

For Case 1 (which allow CVs to use any of the MLs), the results of adding CVs to the MLs network revealed that the maximum delay reduction (compared to the case of no CVs) occurred at an MPR% of 20% during peak conditions. The delay reduction reached 16.61% more than any other cases. Regarding off-peak conditions, the maximum delay reduction was 13.18% and it occurred when the MPR% was 20%. For Case 2 (which allows CVs to use either dedicated CV lanes or MLs), it was found that the maximum delay reduction (15.47%) occurred when the MPR% was 30% during peak condition. On the other hand, in off-peak conditions, it was found that for an MPR% of 25%, the maximum delay reduction occurred at 9.62%. For Case 3 (which allows CVs to use only dedicated CV lanes), it was found that there was no delay reduction in the case of peak condition. The optimal MPR was 20%, which had an increase of delay by 1.71%. Similarly, in the off-peak condition, there was a delay increase of 10.24% at the optimal MPR% of 25%. Figures 72 and 73 show the delay reduction (value more than zero) and delay increase (value less than zero) for cases 1, 2, and 3 for peak and off-peak conditions, respectively.

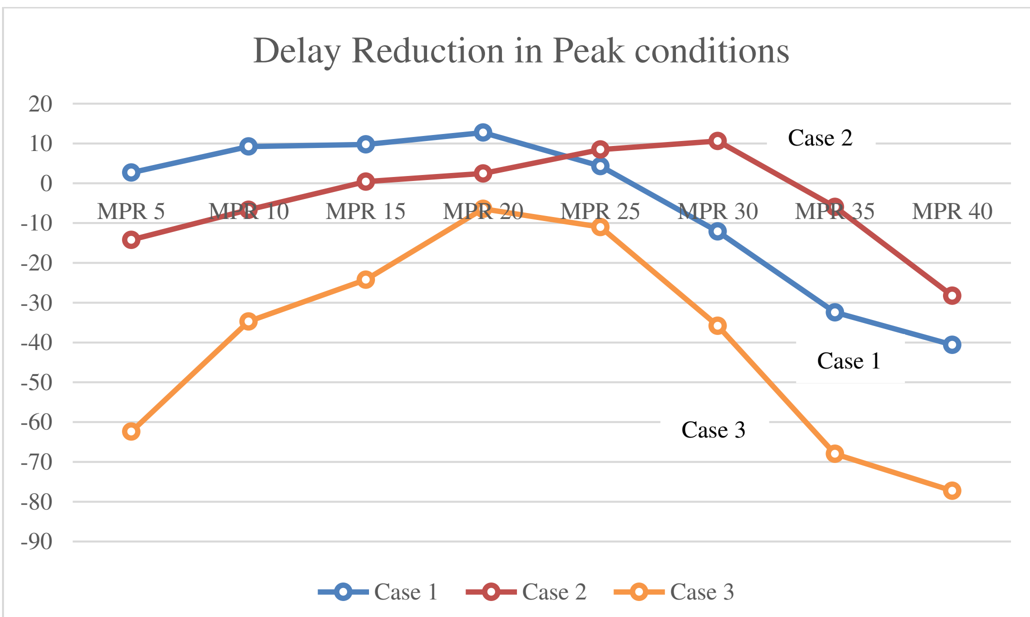



**Figure 72. Average Delay Reduction in Peak Conditions**

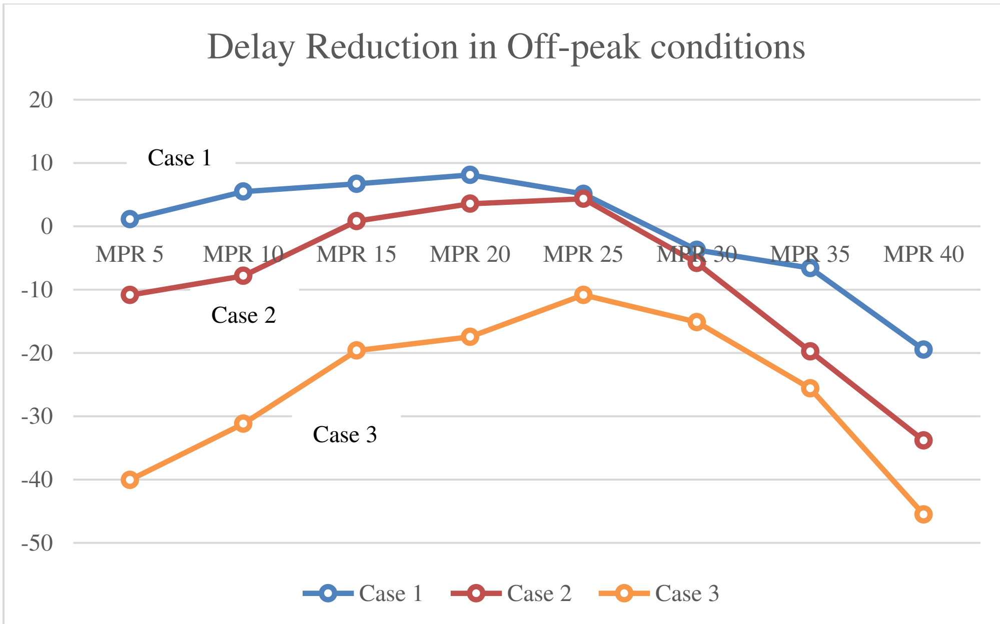



**Figure 73. Delay Reduction in Off-Peak Conditions**

Figure 74 represents the delay reduction for peak and off-peak conditions in Case 4. As can be seen from the figure, the highest delay reduction occurred when the MPR% was 50% in peak conditions with a 16.5% increase compared to the base condition. The results also revealed that the delay reduction deteriorated after an MPR% of 80%. In off-peak conditions, the highest delay reduction occurred when the MPR% was 60% with an 19% delay reduction.

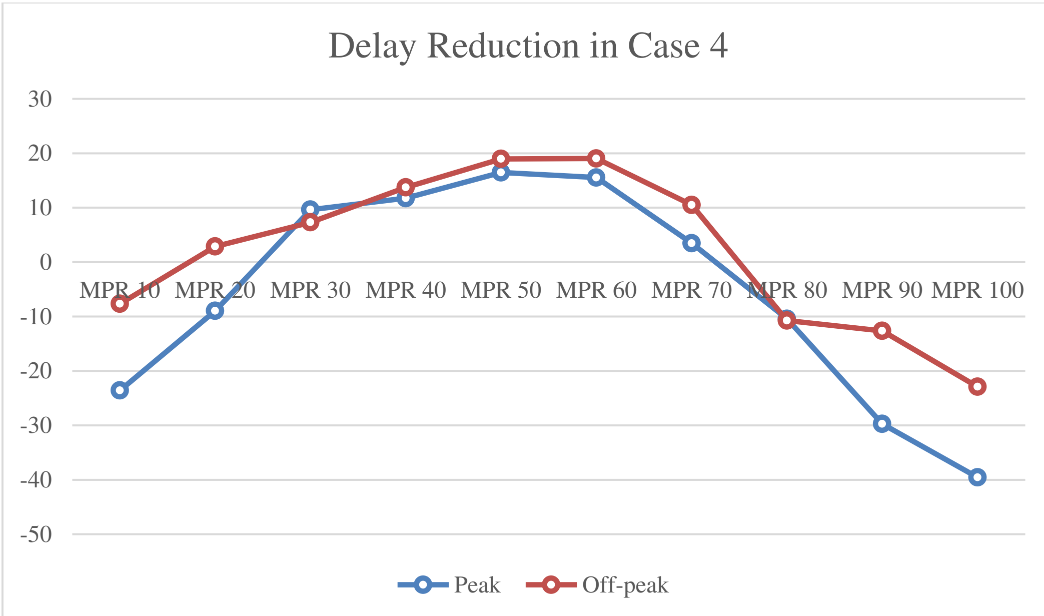



**Figure 74. Delay Reduction in Case 4**

Figure 75 represents the delay reduction for peak and off-peak conditions in Case 5. As can be seen from the figure, the highest delay reduction occurred when the MPR% was 70% in peak conditions with a 27.26% increase compared to the base condition. The results also revealed that the delay reduction deteriorated after an MPR% of 80%. In off-peak conditions, the highest delay reduction occurred when the MPR% was 70% with an 24.53% delay reduction.

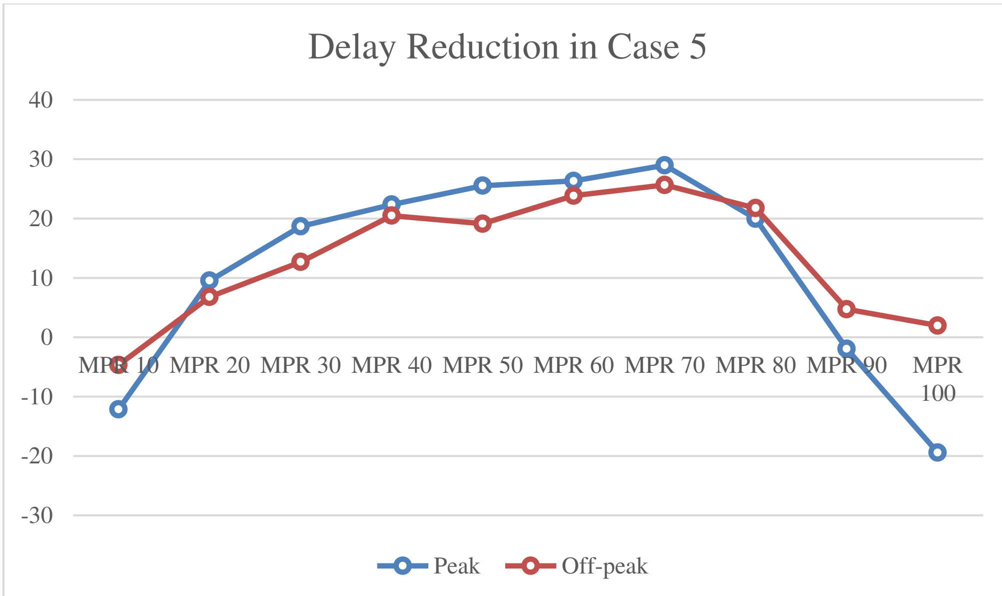



**Figure 75. Delay Reduction in Case 5**

Figure 76 shows the delay reduction (compared to the base case with no CVs) for Case 6 (which allows CVs to use any of CVLs, MLs, or GPLs) in all studied MPR%. In peak conditions, it was found that the maximum delay reduction occurred at higher MPR%. There was a positive association between higher MPR% and the delay reduction. The highest delay reduction occurred at an MPR% of 100% with a delay reduction of 21.65%. With MPR% between 80% and 100%, the delay reduction could reach between 9.8% and 13.3%. It was also noted that at an MPR% of 30% or lower, there was no delay reduction in the network. It is worth noting that the off-peak conditions followed the same delay reduction distribution of the peak conditions. Therefore, higher MPR% could be recommended for improving the network safety in Case 6. The highest delay reduction was reached at an MPR% of 100% with a reduction of 23.64%.

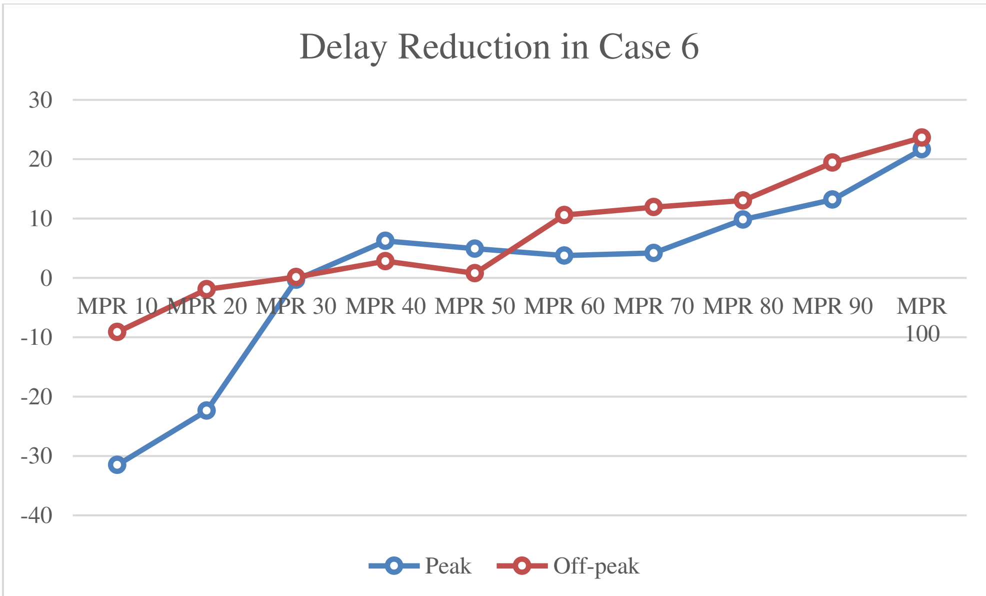



**Figure 76. Delay Reduction in Case 6**

<u>4.4.2.2 Statistical Modeling</u>

Similar to the average speed analysis, a Tobit model was developed to determine the best scenario with the optimal MPR% among all studied scenarios. The model formulation is similar to the model in the conflict frequency section. The results of the Tobit model are shown in Table 30.

## Table 30 Tobit Model for Delay

| Parameter | Estimate | P-value | Parameter | Estimate | P-value | Parameter | Estimate | P-value |
|---|---|---|---|---|---|---|---|---|
| **Intercept** | 16.921 | <0.0001 | | | | | | |
| **Case 1 MPR 5%** | -0.384 | 0.805 | **Case 3 MPR 15%** | 2.141 | 0.164 | **Case 5 MPR 30%** | **-4.851*** | **0.0016** |
| **Case 1 MPR 10%** | -1.466 | 0.346 | **Case 3 MPR 20%** | 2.188 | 0.159 | **Case 5 MPR 40%** | **-5.915*** | **0.0001** |
| **Case 1 MPR 15%** | -1.625 | 0.296 | **Case 3 MPR 25%** | 2.112 | 0.169 | **Case 5 MPR 50%** | **-6.141*** | **<0.0001** |
| **Case 1 MPR 20%** | -2.667** | 0.086 | **Case 3 MPR 30%** | **5.152*** | **0.0008** | **Case 5 MPR 60%** | **-6.631*** | **<0.0001** |
| **Case 1 MPR 25%** | -2.952** | 0.058 | **Case 3 MPR 35%** | **9.519*** | **<0.0001** | **Case 5 MPR 70%** | **-6.772*** | **<0.0001** |
| **Case 1 MPR 30%** | -0.024 | 0.987 | **Case 3 MPR 40%** | **12.224*** | **<0.0001** | **Case 5 MPR 80%** | **-3.765*** | **0.014** |
| **Case 1 MPR 35%** | **3.061*** | **0.049** | **Case 4 MPR 10%** | **3.198*** | **0.029** | **Case 5 MPR 90%** | 0.055 | 0.970 |
| **Case 1 MPR 40%** | **5.042*** | **0.001** | **Case 4 MPR 20%** | 0.719 | 0.624 | **Case 5 MPR 100%** | 1.174 | 0.450 |
| **Case 2 MPR 5%** | 1.963 | 0.207 | **Case 4 MPR 30%** | -1.665 | 0.256 | **Case 6 MPR 10%** | 3.177 | 0.041 |
| **Case 2 MPR 10%** | 0.888 | 0.568 | **Case 4 MPR 40%** | -2.441* | 0.096 | **Case 6 MPR 20%** | 1.574 | 0.311 |
| **Case 2 MPR 15%** | -0.616 | 0.692 | **Case 4 MPR 50%** | **-3.399*** | **0.020** | **Case 6 MPR 30%** | -0.978 | 0.529 |
| **Case 2 MPR 20%** | -2.221 | 0.156 | **Case 4 MPR 60%** | **-3.305*** | **0.024** | **Case 6 MPR 40%** | -1.613 | 0.299 |
| **Case 2 MPR 25%** | -2.786** | 0.073 | **Case 4 MPR 70%** | -1.274 | 0.385 | **Case 6 MPR 50%** | -2.148 | 0.167 |
| **Case 2 MPR 30%** | -2.698** | 0.081 | **Case 4 MPR 80%** | 2.043 | 0.163 | **Case 6 MPR 60%** | -2.512 | 0.109 |
| **Case 2 MPR 35%** | 1.832 | 0.233 | **Case 4 MPR 90%** | **4.287*** | **0.003** | **Case 6 MPR 70%** | -2.774** | 0.074 |
| **Case 2 MPR 40%** | **5.442*** | **0.001** | **Case 4 MPR 100%** | **6.225*** | **<0.0001** | **Case 6 MPR 80%** | **-3.176*** | **0.041** |
| **Case 3 MPR 5%** | **10.154*** | **<0.0001** | **Case 5 MPR 10%** | 0.956 | 0.533 | **Case 6 MPR 90%** | **-3.685*** | **0.018** |
| **Case 3 MPR 10%** | **6.410*** | **<0.0001** | **Case 5 MPR 20%** | -1.859 | 0.226 | **Case 6 MPR 100%** | **-4.358*** | **0.005** |
| **Base Condition** | | | | **Reference** | | | | |
| **Peak (v.s. off- peak)** | 4.838 | <0.0001 | | | | | | |
| **α** | 1.573 | <0.0001 | | | | | | |
| **R-Square** | | | | 0.394 | | | | |

The results of the Tobit model results revealed that, in Case 1, an MPR% of 20% is considered the optimal option for case 1 (CVs can use any of the MLs), with the lowest delay compared to all other MPR%'s. Closer inspection of the results revealed that an MPR of 25% had the second lowest delay compared to the other studied MPR%. On the other hand, an MPR% of 35% or higher was not recommended, since it had a significantly higher delay than the base case. What emerged from the results reported here was that an MPR% of **20%-25%** is the most optimal MPR% for Case 1. Furthermore, it is be inferred from the results that for Case 2 (CVs can use either MLs or CVLs), an MPR% of 25% was the optimal option with the lowest delay among all studied rates. A range of **25% to 30%** can be recommended as the optimal MPR% in Case 2 with the lowest delay. It is also apparent from the table that an MPR% of 40% and higher had a significantly higher delay than the baseline. For Case 3 (CVs only allowed in CVLs), an inspection of the results revealed that an MPR% of **25%** had the least delay among all other rates. The results also revealed that an MPR% of 30% or higher had a significantly higher delay than the base condition. Likewise, a significant higher delay occurred when the MPR% was 10% or lower. As mentioned before, limiting CVs to use only CVLs is not recommended since it generated higher delay than other cases.

According to the model results, for Case 4 (same as Case 1 with converting one GPLs to MLs), it was found that an MPR% between 40% and 60% had a significantly lower delay than the base case. Specifically, an MPR% of 50% had the lowest delay with the lowest estimate among all rates. Also, it can be concluded that an MPR% between **40% and 60%** is recommended, since it generated the lowest delay in the network in Case 4. For Case 5 (same as Case 1 with an increase in the number of MLs), it was found that an MPR% between 30% and 80% had a significantly lower delay than the base case. Specifically, an MPR% of 70% had the lowest delay with the lowest estimate among all rates. Also, it can be concluded that an MPR% between **40% and 70%** is recommended, since it generated the lowest delay in the network in Case 5. For Case 6 (CVs can use any lane in the network), it was found that the maximum delay reduction occurred at higher MPR%. There was a significantly positive association between higher MPR% and the reduction of delay. Specifically, an MPR% between 70% and 100% had significantly lower delay than the base case. An MPR% of 100% had the delay with the lowest estimate among all rates. Also, it can be concluded that an MPR% between 7**0% and 100%** is recommended, since it generated the lowest delay in the network in Case 6. Furthermore, it is apparent from the traffic conditions that peak conditions had significantly higher delays than off-peak conditions. Therefore, more attention should be paid to peak conditions.

## 4.5 Summary and Conclusion

This part of the dissertation was undertaken for investigating the safety and operational effect of adding connected vehicles (CVs) and CV lanes to the managed lanes network with the intention of maximizing system-wide efficiency. Microscopic traffic simulation techniques were developed and applied, including 9 mi corridor of MLs segment on Interstate (I-95) in South Florida. Several tasks were determined to achieve the goal of chapter 4. The networks of the managed lanes with CVs and CV lanes for different Cases were built. In all networks, CVs followed the normal driving logic provided by PTV VISSIM 11. In normal logics, vehicles have the capability of measuring speeds and gaps with the surrounding vehicles with its sensors. The parameters for car following and lane changing models in VISSIM 11 were calibrated and validated using real-world CVs data in a project named CoEXist conducted by PTV. The base case (Case 0) represented the current design of the managed lanes network with one access zone in the middle of the network (one entrance and one exit). The first case (Case 1) included adding CVs to the managed lanes. In this case, CVs were not allowed to use GPLs except for the CVs which exited the managed lanes to use the off-ramps. The second case (Case 2) allowed CVs to use either MLs or CVLs. In this case, CVLs were one lane at the left side of the network. In Case 3, CVs were only allowed in the dedicated CV lanes. Case 4 included allowing CVs on any of the MLs, by increasing the capacity of MLs by converting one lane of the GPLs to a managed lane. In this case, CVs were only allowed in the managed lanes and have the choice to use any of the managed lanes. Case 5 included allowing CVs on any of the MLs, by increasing the capacity of MLs with more MLs. In this case, CVs were only allowed in the managed lanes and have the choice to use any of the managed lanes. Nevertheless, in Case 6, CVs could use any of the lanes in the network. For each case, several market penetration rates were applied and investigated to determine the optimal MPR% for different designs. For each scenario, ten random runs with different random seeds were applied. The comparison between the different cases of MLs designs with the presence of CVs and CVLs with different market penetration rates were generated for different traffic conditions, including peak and off-peak conditions.

The safety and operational analysis of the CVs and CVLs configurations in MLs were successfully demonstrated. Regarding the market penetration rate, Table 31 shows the optimal

MPR% for each case based on three measures of performance including: conflict reduction, speed increase, and delay reduction compared to the base case with no CVs. The best scenarios in Case 1 occurred when the market penetration rate was between **20% and 25%** for peak conditions with a conflict reduction of 65%. Similarly, for off-peak conditions, the best scenarios happened when the market penetration rate was between **20% and 30%** with a conflict reduction of 53%. For Case 2, the maximum conflict reduction, speed increase, and delay reduction happened when the MPR% was between **25% and 30%.** For off-peak conditions, the best scenarios occurred when the market penetration rate was between **25% and 30%.** Moreover, Case 3 (CVs can only use CVLs) was not recommended since it showed lower conflict reduction than other studied cases.

For Case 4 (which is similar to Case 1 with converting one lane of GPLs to MLs), it was found that the maximum conflict reduction occurred when the MPR% was between 4**0% and 60%** for the peak condition. The maximum conflict reduction occurred at an MPR% of 50% with a reduction of 70%. For off-peak conditions, it is worth mentioning that the lowest conflict reduction occurred when the MPR% was between 5**0% to 70%.** The maximum reduction occurred when the MPR was 60% with 61.02%. For Case 5 (which is similar to Case 1 with an increase in the number of MLs), it was found that the maximum conflict reduction occurred when the MPR% was between **50% and 70%** for the peak condition. The maximum conflict reduction occurred at an MPR% of 60% with a reduction of 74.19%. For off-peak conditions, it is worth mentioning that the lowest conflict reduction occurred when the MPR% was between **60% to 80%.** The maximum reduction occurred when the MPR was 70% with 64.21%.

For Case 6 (which allows CVs to use any of CVLs, MLs, or GPLs), it was found that the maximum conflict reduction occurred at a higher MPR%. There was a positive association between higher MPR% and the conflict reduction. With MPR% between **60% and 100%**, the conflict reduction could reach between 50% and 70%. Also, the conflict reduction could reach 10% to 20% when the MPR% was 20% to 40%. It was also noted that at an MPR% of 10%, there was no conflict reduction in the network. It is worth noting that the off-peak conditions followed the same conflict reduction distribution as the peak conditions. Hence, a higher MPR% could be recommended for improving the network safety in Case 6. The highest conflict reduction was reached at an MPR% of 100% with a reduction of 71.14% and 62.75% for peak and off-peak conditions, respectively.

**Table 31. Optimal Market Penetration Rates (MPR%) for Different Cases**

| Traffic Condition | Case | Conflict Reduction | | Speed Increase | | Delay Reduction | |
|---|---|---|---|---|---|---|---|
| | | Optimal MPR% | Reduction% | Optimal MPR% | Increase% | Optimal MPR% | Reduction% |
| Peak | Case 1 | 20% | 66.87% | 25% | 8.51% | 20% | 12.73% |
| | Case 2 | 25% | 57.53% | 25% | 4.74% | 30% | 10.62% |
| | Case 3 | 15% | No Reduction | 15% | No Increase | 20% | No Reduction |
| | Case 4 | 50% | 70.67% | 50% | 11.75% | 50% | 16.47% |
| | Case 5 | 60% | 74.19% | 60% | 12.15% | 70% | 27.26% |
| | Case 6 | 100% | 71.14% | 100% | 12.89% | 100% | 21.65% |
| Off-peak | Case 1 | 30% | 53.23% | 30% | 7.87% | 20% | 8.11% |
| | Case 2 | 30% | 51.03% | 25% | 9.38% | 25% | 4.34% |
| | Case 3 | 20% | 9.46% | 30% | 3.54% | 25% | No Reduction |
| | Case 4 | 60% | 61.02% | 60% | 11.05% | 70% | 19.04% |
| | Case 5 | 70% | 64.21% | 80% | 12.96% | 70% | 24.53% |
| | Case 6 | 100% | 62.75% | 100% | 13.29% | 100% | 23.64% |

Furthermore, based on the Tobit and Negative Binomial models, Case 6 (allowing CVs in MLs and GPLs) proved to be the superior case, in regards to the safety and operations of the lane configuration in CVs environment. In this case, the recommended MPR% was shown to be between **70% and 100%,** based on the modeling results of conflict frequency, speed, and delay. If CVs were only allowed in the MLs, Case 1 (CVs can use any on MLs only) would be the best case. In this case, the optimal MPR% was determined to be between **15% and 25%**. It is worth noting that case 2 (CVs can use either MLs or CVLs) could also be considered, since there was no significant difference between Case 1 and Case 2. In this case, the recommended MPR% was between **20% and 25%.** Moreover, it is worth mentioning that an MPR% higher than 40% and lower than 10% is not recommended for Cases 1, 2, and 3 since it might result in a significantly high number of conflicts along the network. Case 3 (CVs can only use CVLs) was not recommended since it showed significantly higher conflict frequency, higher delays, and lower speeds than other studied cases.

One of the most prominent findings from this study was that, the safety and operation of the network improved by converting one GPLs to MLs (Case 4). In this case, it was found that an MPR% between **40% and 60%** had a significantly lower conflict frequency, higher speeds, and lower delays than the base case. Specifically, an MPR% of 60% had the lowest conflict frequency, lowest delays, and higher speed among all studied rates. Lastly, it was found that the off-peak periods had better safety and operational performance (e.g., lower conflict frequency, less delay, higher speed) in comparison to the peak periods. It was also found that the safety and operation of the network improved increasing the number of lanes in managed lanes (Case 5). In this case, it was concluded from the statistical models that an MPR% between **30% and 70%** had a significantly lower conflict frequency, higher speeds, and lower delays than the base case. Specifically, an MPR% of 70% had the lowest conflict frequency, lowest delays, and higher speed among all studied rates. Lastly, it was found that the off-peak periods had better safety and operational performance (e.g., lower conflict frequency, less delay, higher speed) in comparison to the peak periods. For future studies, more attention should be allotted to the peak conditions.

It is expected that the outcomes from this study could be used as guidance to establish effective safety and operational plans for managed lanes in connected vehicles environment. The findings of this study have several important implications for future practice or policy. It is recommended that both lane configuration in CVs environment and market penetration rate should be taken into account when designing the managed lanes in CVs environment. The study gives recommendations to the transportation agencies for improving the mobility and the efficiency of the MLs.

Taken together, the findings of this study have important practical implications for future practice. Table 32 shows the suggestions of the CV lane design for different MPR%. The results highlighted that an MPR% of 10% and lower had no significant improvement than the base case with no CVs. Therefore, an MPR% lower than 10% was not recommended in managed lanes network. The findings suggested that an MPR% between 10% and 30% was recommended when the CVs were only allowed in MLs (Case 1 or Case 2). By converting one lane of the GPLs to a lane of MLs (Case 4), the MPR% could be increased to reach 60%. When increasing the number of managed lanes (Case 5), the MPR% could be improved to reach 70%. Lastly, the findings suggested that MPR% of 100% could be achieved by allowing the CVs to use all the lanes in the network (Case 6). In this case, the conflict reduction could reach 72% for an MPR% of 100% and could achieve 59% for an MPR% between 70% and 90%.

**Table 32. CV Lane Design Recommendations for Different MPR%**

| MPR% | CV Lane Design Recommendations |
|---|---|
| 0-10% | Not recommended |
| 10%-30% | Case 1: CVs can use any lane of the MLs or Case 2: CVs can use MLs or CVLs |
| 40%-60% | Case 4: Converting one GPLs to MLs |
| 30%-70% | Case 5: Increasing the number of MLs |
| 70%-100% | Case 6: CVs can use any lane in the network (GPLs, MLs, CVLs) |

# CHAPTER 5: CONCLUSIONS

On freeways, Managed Lanes (MLs) have emerged as an effective dynamic traffic management strategy. MLs have been successfully implemented as an important facility in improving traffic mobility and in generating revenue for transportation agencies. This study had two main objectives. First, the optimal managed lanes access design including accessibility level and weaving distance for an at-grade access design (I-95, South Florida) were determined. Second, the effect of applying connected vehicles (CVs) on the safety and the operation of the network was explored.

The first goal focuses on studying the effect of access design on the safety and the operation of the MLs. The primary research task of this objective is to use microsimulation to maximize the system-wide efficiency, by determining the optimal accessibility level in conjunction with sufficient length and locations of weaving segments near access zones. 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 segment in its entirety without considering accessibility levels and weaving distance of the access design. In the following study, several scenarios were tested using microscopic traffic simulation to determine the optimal access design while also taking into consideration accessibility levels and weaving lengths. The studied accessibility levels varied from one to three along the studied network. In order to achieve the study objective, a 9-mile corridor of general purpose lanes (GPLs) and MLs in Miami-Dade County, Florida were replicated in a microsimulation environment in terms of traffic data, geometric design, and driving behavior (i.e., car following, lane changing). Several safety measures of effectiveness (i.e., speed standard deviation, time-to-collision, and conflict rate) and operational performance measures (i.e., level of service, average speed, average delay) were analyzed using statistical models.

The safety analysis of the access design in MLs was successfully demonstrated. The findings of this study have several important implications for future practice and policy. It is recommended that both access control level and weaving configuration should be taken into account when designing the access openings of MLs for expressways. The study gives recommendations to the transportation agencies for improving the mobility and the efficiency of the MLs. One of the most prominent findings from this study was that the conflict rate on MLs were 48% and 11% lower than that of GPLs in the peak and the off-peak periods, respectively. After comparing the surrogate safety measures between MLs and GPLs, it was found that MLs were safer than GPLs since it had higher time-to-collision, higher post-encroachment-time, and lower maximum deceleration. A log-linear model was developed for investigating the safest access zone design that would minimize traffic conflicts. Analysis of conflicts proposed that one accessibility level is the safest option in a 9-mile corridor. Additionally, it was found that a length of 1,000 ft per lane change is indeed the optimal length for the weaving segments. Furthermore, from the findings of this study, a weaving length of 600 ft per lane change is not recommended near the access zones of the MLs. Additionally, Tobit models were developed for investigating the factors that affect safety measures. ANOVA and level of service (LOS) calculations were also used to evaluate traffic operation. Tobit models were able to be successfully developed to investigate the optimal MLs access zone design. Analysis of safety measures (i.e., conflict rate, speed SD, and TTC) proposed that a weaving length between 1,000 feet and 1,400 feet per lane change should be considered. Moreover, the operational measurements were investigated, which included the LOS, average speed, and average delay. The results of the operational measures confirmed several findings from the safety results. One access zone was found as the optimal level, with better LOS, higher speed, and less delay. The results of the average speed, average delay, and LOS proposed a weaving length between 1,000 feet and 2,000 feet per lane change for a more efficiently operated network. Lastly, it was found that the off-peak periods had better safety and operational performance (e.g., lower conflict rate, less delay) compared to the peak periods. For future studies, more attention should be allotted to peak conditions.

The second goal of this study focused on investigating the effect of applying CVs and connected vehicle lanes on the safety and the operation of the network. Also, this objective sought to determine the optimal market penetration rate (MPR%) of CVs by investigating various lane configurations in MLs network with the presence of CVs and CVLs. A comparison between the different cases of MLs designs with the presence of CVs with different market penetrations was generated for different traffic conditions. Similar to the first objective, a 9-mile corridor located on Interstate 95 (I-95) in South Florida was used in the second objective. VISSIM microsimulation was used for investigating various scenarios of CV lane design and MPR% in the managed lanes network. In all networks, CVs followed the normal driving logic provided by PTV VISSIM 11. In normal logics, vehicles have the capability of measuring speeds and gaps with the surrounding vehicles with its sensors. The parameters of car following and lane changing models in VISSIM

11 were calibrated and validated using CoEXist real-world data conducted by PTV (Groves, 2018; PTV, 2018; Sukennik, 2018). Five main cases were considered. The base condition (Case 0) included the I-95 corridor with one access zones (one ingress and one egress) in the middle of the corridor. In this case, three types of vehicles were considered: passenger cars (PCs), heavy goods vehicles (HGVs), and carpools. It is worth mentioning that CVs were not considered in the base case. A total of 110 scenarios were studied with different lane configuration cases, market penetration rates, and traffic conditions. Six different cases of CV lane configuration in the MLs network were studied. In case 1, CVs were only allowed in the managed lanes and had the choice to use any of the managed lanes. In Case 2, CVs could use either the dedicated CV lanes or the managed lanes. In Case 3, CVs were only allowed to use the dedicated CV lanes. Case 4 was similar to Case 1 with converting one lane of GPLs to a lane of MLs in order to increase the capacity of the managed lanes. In this case, CVs were only allowed in the managed lanes and had the choice to use any of the managed lanes. Case 5 was similar to Case 1 with adding one lane to the MLs in order to increase the capacity of the network. In Case 6, CVs could use any of the lanes in the network.

The safety and operational analysis of the CVs and CVLs configurations in MLs were successfully represented. Various market penetration rates were studied and compared using three performance measures including: conflict reduction, speed increase, and delay reduction compared to the base case with no CVs. The best scenarios in Case 1 occurred when the MPR% was between 20% and 25% for peak conditions. Similarly, for off-peak conditions, the best scenarios happened when the MPR% was between 20% and 30%. For Case 2, the maximum conflict reduction, speed increase, and delay reduction happened when the MPR% was between 25% and 30%. For off-peak conditions, the best scenarios occurred when the market penetration rate was between 25% and 30%. Moreover, Case 3 (CVs can only use CVLs) was not recommended since it showed no conflict reduction for peak conditions, and lower conflict reduction than other studied cases for off-peak condition.

For Case 4, it was revealed that the maximum conflict reduction occurred when the MPR% was between 40% and 60% for the peak condition. For off-peak conditions, it is worth mentioning that the lowest conflict reduction occurred when the MPR% was between 60% to 80%. For Case 5, it was found that the maximum conflict reduction occurred when the MPR% was between 50% and 70% for the peak condition. For off-peak conditions, it is worth mentioning that the lowest conflict reduction occurred when the MPR% was between 60% to 80%. For Case 6, it was found that the maximum conflict reduction occurred at a higher an MPR% between 60% and 100%. There was a positive association between higher MPR% and the conflict reduction. It is worth noting that the off-peak conditions followed the same conflict reduction distribution as the peak conditions. Therefore, a higher MPR% could be recommended for improving the network safety in Case 6. The highest conflict reduction was reached at an MPR% of 100%. Figure 77 shows the highest conflict reduction and the corresponding MPR% for all studied cases for peak and off-peak conditions.

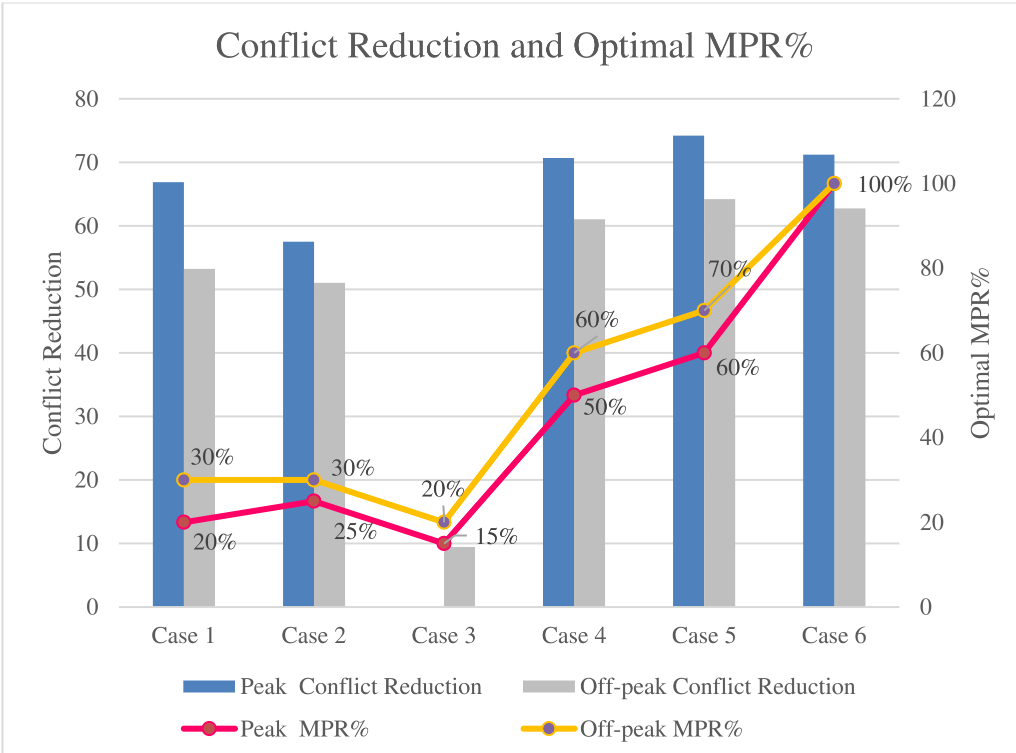



**Figure 77. Highest Conflict Reduction and Safest MPR% for All Studied Cases**

The findings of this study have several important implications for future practice or policy. It is recommended that both CVLs configuration and market penetration rate should be taken into account when designing the managed lanes in CVs environment. The study gives recommendations to the transportation agencies for improving the mobility and the efficiency of the MLs and CVLs. Based on the Tobit and Negative Binomial models, the results highlighted that an MPR% of 10% and lower had no significant improvement than the base case with no CVs. Therefore, an MPR% lower than 10% was not recommended in managed lanes network. The findings suggested that an MPR% between 10% and 30% was recommended when the CVs were only allowed in MLs (Case 1 or Case 2).

The MPR% could be increased to reach 60% by converting one lane of the GPLs to a lane of MLs (Case 4). When increasing the number of managed lanes (Case 5), the MPR% could be improved to reach up to 70%. Lastly, the findings suggested that an MPR% of 100% could be achieved by allowing the CVs to use all of the lanes in the network (Case 6). In this case, the conflict reduction could reach 70% for an MPR% of 100% and could achieve 60% for an MPR% between 70% and 90%. Lastly, it was found that the off-peak periods had better safety and operational performance (e.g., lower conflict frequency, less delay, higher speed) in comparison to the peak periods. For future studies, more attention should be allotted to the peak conditions.

Several potential applications beyond the study scope are worth investigation in future studies. The findings introduce a step towards enhancing the overall safety and operational performance of the road (Chen et al., 2018; Gong, Abdel-Aty, Cai, et al., 2019; Gong, Abdel-Aty, & Park, 2019; Jian Yuan et al., 2019; Yue et al., 2018). Further investigations need to be carried out in order to study the impact that restrictive access zones have on corridor travel times, travel time reliability and usage of the managed lanes. Studies also need to be implemented to investigate the new designs of MLs access zones using simulation techniques. For example, the direct and slip ramps have been used to connect the ramps to MLs directly without generating weaving segments. These could be a better design but with much higher construction cost. Additionally, new technologies and transportation strategies are being proposed for maximizing the traffic performance in MLs. The active traffic management (ATM) techniques (i.e., variable speed limit, ramp metering, and dynamic shoulder lanes) should be tested with MLs using a simulation technology for safety improvement. Wang et al. (2017) proved that ATM strategies could improve the safety of the weaving segments by generating lower conflict frequency (M. Abdel-Aty & Wang, 2017; Ling  Wang et al., 2017; Wang et al., 2019; Wang et al., 2018). Moreover, ramp metering and variable speed limit (VSL) can be developed and tested in MLs environment using microscopic traffic simulation for improving traffic operational and safety performance. These new strategies may be excellent approaches for improving traffic safety and operation at MLs since it is responsive to real-time traffic.

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