Safety and Operational Performance Evaluation of Four Types of Exit Ramps on Florida's Freeways
(Final Report)

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METRIC CONVERSION CHART

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
AREA				
in^2	squareinches	645.2	square millimeters	mm^2
ft^2	squarefeet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3

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This project mainly focuses on exit ramp performance analysis of safety and operations. In addition, issues of advance guide sign for exit ramp are also mentioned.
Safety analysis evaluates safety performances of different exit ramps used in Florida and nationally. More specific, the research objectives include the following two parts: to evaluate the impacts of different exit ramp types on safety performance for freeway diverge areas; and to identify the different factors contributing to the crashes happening on the exit ramp sections.
Operational analysis determines different ramp effects, and guidance for selecting optimal exit ramp type. Beside ramp type evaluation and selection, issues of ramp section and cross road section are also demonstrated. Minimum ramp length and minimum distance between ramp terminal and downstream/upstream intersections are calculated.
For advance guide sign, placement distance models are presented respectively for the three different installation methods: ground sign installation, overhead sign installation, and median sign installation. All possible factors, such as freeway geometric design, traffic conditions, driver behavior, exit ramp type, and posted speed limits on freeway and ramp, are taken into consideration while modeling.</td></tr>
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EXECUTIVE SUMMARY

In the State of Florida, four typical types of exit ramps are used for traffic to exit freeways. Different types of exit ramps have different design elements and factors, which can impact the integrated performance of interchanges. There are several issues and concerns about these elements and factors. Some of these concerns include the safety and operational performance for each type of exit ramp, correlations among exit ramp type, lane utilization, geometrics, land use, deceleration rate, distances for lane change, and distance for traffic to exit freeways. These issues have not been specifically studied in the past and no clear guidelines, either federal (American Association of State Highway and Transportation Officials (AASHTO) Green Book) or state, are currently available.

This report summarized a research project sponsored by Florida Department of Transportation (FDOT) to develop a method that can evaluate safety and operational performances of all four types of exit ramps, and tailored technical guidelines governing the selection of optimum exit ramp types to be used on freeways. In addition, a study on placement distance of advance guide sign was also conducted. The findings of this research project could determine the type of exit ramps that should be constructed at a given location considering the prevailing conditions applicable to traffic, roadway, and land-use developments.

Freeway exit advance guide signs play an important role in guiding drivers to exit freeways and expressways by providing them the direction information and the distance to the upcoming exit. Drivers depend highly upon advance guide signs to help them find their target exits on unfamiliar freeways. The Manual on Uniform Traffic Control Devices (MUTCD) provides the basic principles regarding how guide signs should be installed. However there is little guidance on the placement distance of advance guide signs on freeways with different geometric designs and different exit ramp types. The research team presented advance guide sign placement distance models respectively for three different installation methods: ground sign installation, overhead sign installation, and median sign installation. The factors, such as freeway geometric design, traffic conditions, driving behavior, and posted speed limits on freeway and ramp, were taken into consideration while modeling. The results showed that there were differences between the placement distances calculated using the models and those directly derived from MUTCD. These findings not intended to replace but to supplement the guidelines in the current MUTCD, provided transportation engineers and technicians more

reasonable and detailed guidelines in design.

Safety analysis evaluated safety performances of these different exit ramps used in Florida. The research team conducted such works: (1) to evaluate the impacts of different exit ramp types on safety performance for freeway diverge areas, (2) to identify the different factors contributing to the crashes happening on the exit ramp sections. To achieve the research objectives, the research team investigated crash history at 424 sites throughout Florida. The study area included two parts, the freeway diverge area and the exit ramp sections. For the freeway diverge areas, exit ramp types were defined based on the number of lanes used by vehicular traffic to exit freeways. For the exit ramp sections, four ramp configurations, including diamond, out connection, free-flow loop and parclo loop, were considered.

Cross-sectional comparisons were made to compare crash frequency, crash rate, crash severity and crash types between different exit ramp types. Crash predictive models were also built to quantify the impacts of various contributing factors. On the freeway diverge areas, the models showed that Type 1 exit ramp (single-lane with a taper) had the best safety performance in terms of the lowest crash frequency and crash rate. On the exit ramp sections, the out connection ramp appeared to have the lowest average crash rate than the other three. The results of this study could help transportation decision makers choose the optimum one by considering safety issues on freeway diverge areas and exit ramp sections.

To conduct operational analysis, comparisons of different types of exit ramp were made to present a predictive model for choosing the optimal one. Some Measures of Effectiveness (MOEs) were used to approach this objective, such as number of lane change, speed standard deviation (S.D.), control delay and etc. The research team arranged a data collection of 24 sites in Florida, and used traffic simulations by TSIS (Traffic Software Integrated System) for further analysis. Mathematical models were built up to evaluate different impacts of these ramp types. And an integrated model was developed to select optimum ramp type. Beside ramp type evaluation and selection, some design issues on ramp section and cross road section were demonstrated too. Minimum ramp length and minimum distance between ramp terminal and downstream or upstream intersections were calculated, which could be helpful for roadway design.

The final project report included detailed predictive models for safety and operational performance, method for optimum ramp type selection and supplement guideline for placement distance of advance guide sign. All these results and findings would do help for ramp design and management.

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CHAPTER 1 INTRODUCTION

1.1 Background

In the State of Florida, several types of exit ramps are used for traffic to exit freeways (i.e. Interstate and Turnpike Systems). Drivers exiting freeways need to make decisions and execute maneuvers (i.e., lane change or lane merge) prior to the exit ramp in order to access cross roads at the interchanges. If the exit ramps are not sufficiently long, drivers must complete their driving maneuvers within a short distance, resulting in potentially unsafe driving actions (i.e., fast-paced deceleration, lane changing, merging, unbalanced lane utilization, etc.), which will result in the development of shock-waves onto upstream traffic, etc. Considering these factors, there are several issues and concerns that need to be addressed in selecting the most optimum types of freeway exit ramp(s) to use at a given interchange.

Some of these concerns, include but are not limited to, the safety performance of each exit ramp type, operational performance and correlation between types of exit ramps, lane utilization, geometrics, land use along the crossroad, adequate distances for lane change, deceleration, adequate distance for traffic to transit from the exit gore to the downstream intersection which includes weaving, and advance signing on the freeways. These issues have not been studied in the past and no clear guidelines, either federal (American Association of State Highway and Transportation Officials (AASHTO) Green Book) or state, are currently available in selecting exit ramp types. Therefore, there is a need to perform a research project under Florida conditions to specifically evaluate the safety and operational performance for each exit ramp type to develop tailored guidelines that address the issues .This need is especially significant considering the rapid increase in new developments close to freeway interchanges. The Florida Department of Transportation (FDOT) in joint cooperative efforts with the local land use agencies can use the findings of this research project to determine the type of exit ramps that should be constructed at a given location considering the prevailing conditions applicable to traffic, roadway, and land-use developments.

1.2 Research Objectives

The main objective of the research project is to develop tailored technical guidelines governing the selection of optimum exit ramp types to be used on freeways, depending on safety and operational analysis. Possible exit ramp types include, but are not limited to, tangent single-lane exit ramp, single-lane exit ramp without a taper, two-lane exit ramp with an optional lane, and two-lane exit ramp without an optional lane.

The safety performance of these exit ramp types will be evaluated based on historical crash data. For historical crash data analysis, Florida crash databases (such as the Crash Analysis Reporting System (CARS)) will be used to include as many sites as possible for the research results to be statistically significant. For traffic operational performance evaluation, video cameras will be installed at selected sites to record vehicle movements so that performance data such as delay, operating speed, number of necessary or unnecessary lane changes/merge, lane utilization, vehicle queue length, level of service, capacity, etc. can be obtained for each exit ramp type. Traffic simulation methods will also be used to demonstrate operations under recommended guidelines.

Besides safety and operational analysis for different exit ramp types, a study focusing on placement distance of advance guide sign for exit ramp is conducted to present more detailed guidance by analyzing driving behavior and other factors.

1.3 Major Tasks

Major tasks of this research project are stated as follows:

Task 1: Literature Search and Review

Information databases are searched to identify whether or not there are any past similar studies that could be reviewed as references, and to search for existing methodologies and practices related to the research project.

Technical reports and papers related to the research project are searched and reviewed. Internet web sites are searched to find similar information. Other states are also contacted to find out whether similar practices have been performed or not.

Task 2: Field Operational Test Plan Development

Before field experiments and tests, a detailed test plan is developed to include test sites, test site conditions, test timing and duration, test equipment, data collection, test procedure, data analysis and storage, quality control approach, data analysis approach, and etc. The test plan is discussed to ensure the feasibility of the file experiments and tests.

Task 3: Test Site Selection

An inventory of suitable sites is prepared to cover necessary test conditions. FDOT personnel are contacted to make necessary arrangements (such as traffic control, site visiting, etc.) and their comments are taken into consideration when selecting the sites for field data collection. The appropriate number of sites and associated data points is selected so that the analysis findings are statistically significant. The Principal Investigator (PI) and the research team are responsible for contacting the District Maintenance Engineer's office to obtain permit and or discuss the proposed field operational data collection and obtaining approval prior to commencing work. This includes a plan explaining what types of equipments and methods are utilized, and how to maintain traffic on the sites during the period when the data collection is being performed.

Task 4: Field Data Collection

Once project test sites are selected, field data for traffic conflict analysis and traffic simulation should be collected. Some data need to be collected with the use of instruments such as traffic data collection systems, video cameras, vehicle speed detectors, etc. Necessary arrangements by the PI are made as described in Task 3 to ensure a viable traffic control plan is approved by the FDOT, and these field operational tests and devices are ready before Task 4. It is the responsibility of the PI to provide all necessary Maintenance of Traffic (MOT) for this project.

Task 5: Traffic Crash Data Collection

Traffic crash data related to crashes that occurred at freeway exit ramps is collected mostly from the FDOT's crash database (CARS) and Sheriff's offices. Hard copies of crash reports are obtained either from the Sheriff's offices or from a transportation agency if necessary. The collected and analyzed crash data is utilized for the safety evaluation purposes that correlate associated crash rates with ramp characteristics

Task 6: Data Reduction

Crash data is summarized in different formats. Most traffic data and conflict data is reduced from videotapes or from databases. The videotapes record traffic conflicts, speed, and volumes. The process to reduce traffic data and conflicts from videotapes and crash data from hard copy reports is supposed to be time-consuming and lab extensive. This step takes much time and energy.

Task 7: Data Analysis

Crash analysis, traffic operational analysis, and traffic simulation analysis are performed to evaluate the traffic operational and safety performances for different exit ramp scenarios. More data is collected if needed. During the task period, the research team works closely with Project Managers (PMs) to ensure the analysis results are valid and can be accepted by the PMs. The main purpose of this task is to define speed change lane with respect to traffic flow characteristics, and driver behavior in the steering control zone which involves the steering, and positioning of the vehicle along a path by steering from the controlling ramp curvature onto the speed change lane, to determinate driver's preference regarding comfortable deceleration rate for executing speed change and associated design guidelines governing lengths of deceleration lanes for safe and comfortable speed reduction from mainline flow speed to exit ramp speed, and to identify and address the needs of older drivers, point of controlling curvature on exit ramps and associated warning/advisory speed signing, to establish driver expectancy relative to required actions and factors that reduce drivers' stress at interchanges.

Task 8: Final Research Report

All research data, findings, conclusions, and recommendations are summarized into technical guidelines and presented in the final research report.

1.4 Outline of the Report

Based on the objectives of this research project, the final report is divided into three parts. Part one (Chapter 2) mainly talks about placement distance calculation of advance guide sign for exit ramps. Part two (from Chapter 3 to 7) focuses on safety analysis for different exit ramps. And part three (from Chapter 8 to 11) concentrates on operational performance analysis for different exit ramps. Figure 1.1 shows detailed research goals and assignments.

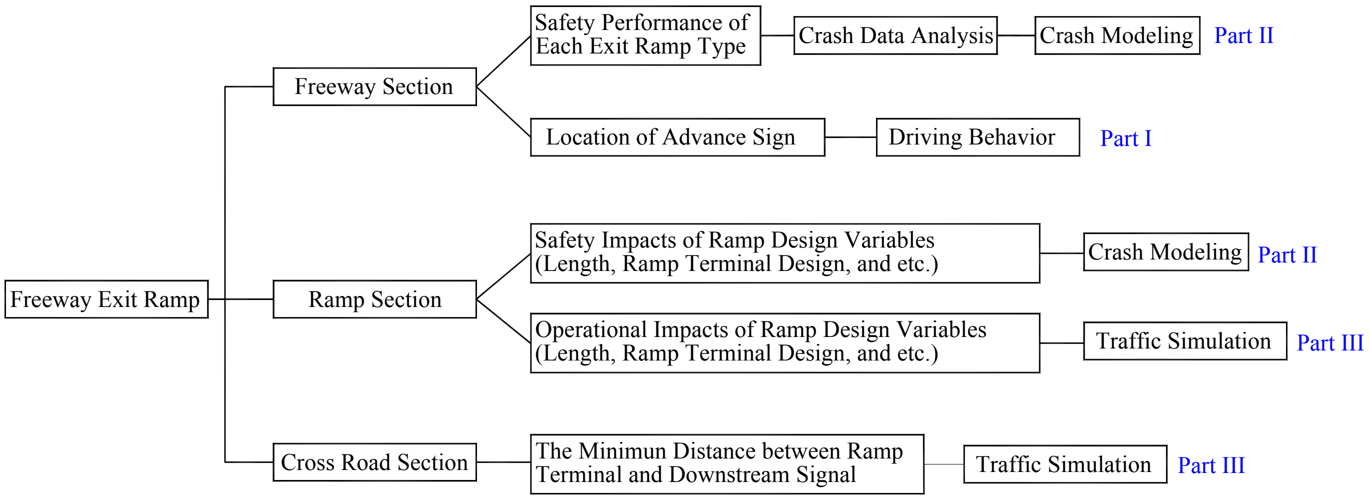


Figure 1.1 Research Tasks and Assignments

PART I - ADVANCE GUIDE SIGN

CHAPTER 2 PLACEMENT DISTANCE FOR ADVANCE GUIDE SIGN

2.1 Introduction

Advance guide signs on freeways are essential to guide drivers along freeways and expressways, to inform them of the exits, to direct them to destinations or to streets and highway routes, and to give such general information as to help them drive in the most simple and direct manner possible. The major emphasis of freeway/expressway guide signage is on destination, which is repeatedly provided in advance of the exit direction signs. Among these repeated advance signs, the one that closest to the exit is most important in directing drivers.

Proper location and installation of advance guide sign are important in helping drivers find their exact exits on unfamiliar freeways and avoid problems such as missing the exit, swerving abruptly, changing lane sharply or other erratic maneuvers. Where and how to install the advance guide signs mainly depend upon the geographical, geometrical and traffic conditions on freeway diverging areas and exit ramps. According to MUTCD 2003, two placement methods, namely ground mounted and overhead mounted, are often used to install the advance guide signs and another method, median mounted, occasionally.

As for the placement distance in MUTCD 2003, there is little detailed guidance for transportation engineers to refer to. Table 2.1 summarizes the advance guide sign placement distance guidelines in MUTCD 2003. This table demonstrates that it is too general in three aspects. Firstly, it does not provide a systematic methodology to determine the placement distance. Secondly, it does not distinguish the freeway segments and ramp segments that possess different geometric design and traffic conditions. Thirdly, classifying the interchanges into two categories, major/ intermediate interchange and minor interchange, is arbitrary and subjective.

In this research, aiming at providing a systematical advance guide sign placement distance method, the research team presents models for the three installation methods respectively. This study takes the geometric design of freeway segment, ramp type, traffic conditions and lane-changing behavior into account to make the model more reasonable. The models for the three types of advance guide sign installation have almost the same model structure, which makes the model utilization much more convenient and easier. Finally, the research team compares the safety and operational performance between the two distances, the ones being modeled and the ones from MUTCD 2003.

Table 2.1 Guide Sign Placement Distance Requirement in 2003 MUTCD

Type of Sign	Guide Sign Placement Distance Guidance	
	Major/Intermediate Interchange	Minor Interchange
Freeway/Expressway Advance Guide Sign	1st sign 1 km (0.5 mi); 2nd sign 2 km (1 mi); 3rd sign 4 km (2 mi) in advance of exit if possible	One sign 1 km ~ 2 km (0.5mi~1 mi) in advance of exit

The rest parts of this chapter are organized as follows: First to analyze the lane changing mechanism within the influence area of advance guide sign and exit ramp. Second, present the methodology used for modeling. Then, illustrate a sample case to show how to make use of the models. Finally, conclude with the comparison results between modeled distances and MUTCD 2003 distances using TSIS-CORSIM simulation.

2.2 Methods for Advance Guide Sign Installation

As shown in Figure 2.1, suppose an east-west bound freeway on a level terrain intersects a north-south bound arterial by an interchange. There is a north bound freeway exit ramp connecting the north-south bound arterial and the east-west bound freeway. Now it is needed to determine the placement distance for the advance exit guide sign that most close to the gore point.

Figure 2.1 shows one direction of east-west bound freeway with the rightmost lane directing an exit ramp to the south-north bound arterial, and an illustration exit direction guide sign indicates the current distance to the interchange along with legend “Exit Only”, which is also shown beside the exit ramp in the figure. According to 2003 MUTCD, there are two main methods to install the guide sign, overhead sign installation and ground sign installation, and in some cases the duplicate advance guide sign may be placed in the median on the opposite on roadway. The three installation methods are shown in terms of circled numbers beside the guide sign in the illustration figure:

(1) Ground sign installation

(2) Overhead sign installation

(3) Median sign installation

Method to install the advance guide sign depends on roadway conditions. For example, overhead sign installation is used at locations where some degree of lane-use control is desirable and at locations where space is not available on roadside.

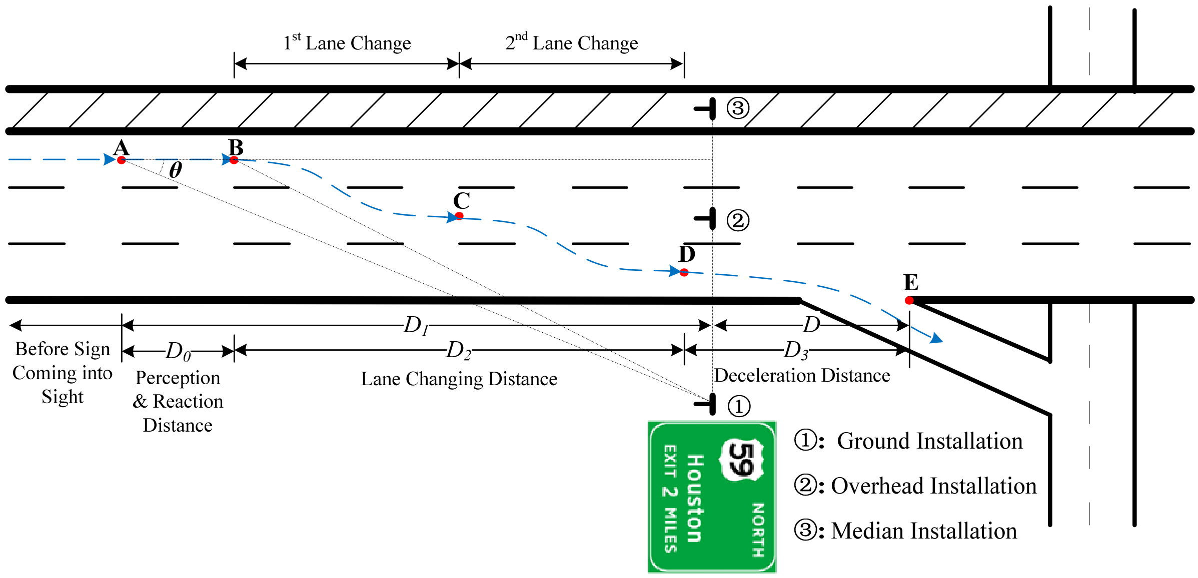


Figure 2.1 Schematic Diagram of Lane Change after Seeing the 1st Advance Guide Sign

The first Advance Guide Sign (AGS) is defined as the one that is most close to the exit ramp, the second AGS as the one that is further to the exit, and the third AGS is the furthest. In this project, main energy is focused on determining the placement distance of the first AGS. The placement distance, labeled with D in Figure 2.1, begins at the location where the AGS is installed, and ends at the physical exit gore.

From Table 2.1, it can be seen that for major and intermediate interchanges/exits, up to three advance guide signs may exist in advance of the exit ramp. If a careless driver did not notice the second and third advance guide signs while approaching his target exit, the first advance guide sign becomes the last straw to direct his exit driving maneuvers. For minor interchange/exits, there is only one advance guide sign available. If the careless driver did not see the AGS, and he/she is not on the most right lane, probably, he/she will miss the correct exit. So the first AGS plays an important role in leading drivers to the right exit ramp.

2.3 Lane Change Model Assumptions

When modeling the AGS placement distance, it has been taken the most adverse scenario into consideration to ensure safety. In this proposed model, this assumption has been made that the driver who will exit the freeway at the upcoming exit ramp is on the leftmost lane, most close to the median. This situation indicates that the driver needs to change his driving path from the leftmost lane to the rightmost lane before exiting to the exit ramp. In other words, he/she needs to make the maximum number of lane changes.

There are five critical points during the lane changing process. As shown in Figure 2.1, the five points are marked in red, labeled with A, B, C, D, and E. The blue curved line illustrates the driver's driving trajectory. Before point A, the drive doesn't see any AGS which can inform him to be ready to exit the freeway. He is driving at his normal pattern.

At point A, the AGS comes into his sight. After seeing the AGS, he perceives that he is approaching his destination exit. Meanwhile he is making a decision to change his lane. Then he becomes more concentrate on his driving, ready to make his first lane change. The time being needed for this process is called PRT, short for perception-reaction time.

After passing point B, he begins to look at his rearview mirror and right side-view mirror to search for an accepted gap to make his lane change. There are three types of driving behavior to merge into his righter lane. The driver first evaluates whether or not a lane change is possible using the existing adjacent gaps. If the gap is large enough, he will make a normal lane change, without acceleration or deceleration. If the gap is smaller than normal, lane change behavior depends not only on the traffic condition, but also on his driving behavior. If he is an aggressive driver, probably he will turn on right light and accelerate sharply to pass the all vehicles on his right lane. This calls forced or aggressive lane change. If he is a nice and patient driver, after turning on the right turn light, probably he will wait for his anticipated gap on his right lane. Some drivers on their right lane may decelerate to generate an acceptable gap for him to merge in. This case is calls courtesy lane change. The average time required for a lane change varies. As shown in Figure 2.1, a single lane change is completed during his driving from point B to point C.

If there are more than two lanes on freeway mainline, a drive needs to make lane change more than once to arrive at his target lane. It is assumed that after finishing his first lane change the driver does not need any perception reaction time to start up his second lane change. If the number of lanes on freeway mainline is N, N-1 lane changes is required to be made to move to the right most lane on the mainline. If the driving distance during a single lance change is L, (N-1)*L is the total driving distance from leftmost lane to the rightmost lane on the mainline. If there is any auxiliary lane or deceleration lane, the driver may need to make one or two more lane changes if he wants to move to the right most lane of the freeway. And thus there is more driving distance. In figure 2.1, lane change begins at Point B, and ends up at point C. And distance D_2 represents the driving distance during the whole lane changing maneuver.

After change onto the rightmost lane, the driver will slow down, adjusting his speed to coordinate with the speed on ramp. This assumption is made that initial speed is reduced from the speed on freeway mainline V_1 to the posted speed on exit ramp V_2 at the physical exit gore. The driving distance during the deceleration is D_3.

2.4 Influential Factors

In this proposed operational model, consider the middle of the leftmost lane on travel way is the critical driving path, implying that the driver needs to change his route from the leftmost lane to the rightmost lane before exiting to the exit ramp. There are some important model parameters that may greatly affect the placement distance of advance guide sign, including lateral offset of sign location, number of lanes, lane-changing time, driver’s reaction time, traffic volume, and degree of cone of vision right before lane-changing. Assumptions on these parameters are verified in the next paragraph.

The lateral offset is important in determining the placement of the exit direction guide sign in this proposed model. Proper lateral placement of guide sign can improve the visibility of the sign and reduce the probability of being hit by vehicles leaving the roadway. According to the 2003 MUTCD Section 2E.23, the standard requirement for the guide sign lateral offset is stated as, “The minimum lateral clearance outside the usable roadway shoulder for ground-mounted freeway and expressway signs or for overhead sign supports, either to the right or left side of the roadway, shall be 1.8 m (6 ft). This minimum clearance shall also apply outside of a barrier curb. If located within the clear zone, the signs shall be mounted on crashworthy supports or shielded by appropriate crashworthy barriers.” One other factor in the overhead sign installation is the vertical offset, the 2003 MUTCD Section 2E.22& Section 2A.18 point that, “All route signs, warning signs, and regulatory signs on freeways and expressways shall be at least 2.1 m (7 ft) above the level of the pavement edge." Therefore, in practice, the guide sign installation should be located at a safety distance to satisfy lateral offset and vertical offset requirements, thus to minimize possible impact forces.

From the stand point of drivers, lane-changing time is another important factor in designing the placement distance of advance guide sign. Finnegan and Green conducted a detailed research in the lane-changing behavior and found that the average visual search time for preparing a lane change is as much as 3.7 seconds without traffic and as much as 6.1 seconds with traffic (depending on the condition). Furthermore, if two standard deviations are allowed, they suggested that 6.6 seconds should be allowed for the visual search associated with a single vehicle lane change and about 1.5 seconds to execute the change. Salvucci studied the driver behavior before, during and after the lane-changing based on the driver's control and eye-moment behavior and found that drivers average 5.14 seconds per lane change.

Drivers' cone of vision is also a key parameter in the proposed model. Essentially, the proposed model assumes that sign reading and comprehension need to be completed before maximum degree of cone of fairly clear vision expires, and then lane-changing could take place. Most people have clear vision within a conical angle of 3°~ 5° and fairly clear vision within a conical angle of 10° to 12°. Here, 10°of cone of vision is assumed for convenience.

Based on this assumption, the legibility distance used by the MUTCD to determine the advance placement distance for warning sign will not be a determinant factor affecting the advance sign placement distance in the proposed model. This is certainly a great advantage over the MUTCD guideline, because if not affected by the legibility distance, the proposed sign placement distance is no longer a function of visual acuity, type and size of sign lettering, or legibility associated with the specific lettering. The complexity of the operational model is therefore greatly reduced.

2.5 Model Description

The equation notations are defined as follows:

V_1= Design speed on freeway mainline *(mph)*

V_2= Design speed on exit ramp *(mph)*

N = Number of lanes on freeway mainline, not including deceleration lane and auxiliary lane

LW = Lane width (*ft*)

LS = Lateral offset for overhead sign installation and ground sign installation (*ft*)

VS = Vertical offset for overhead sign installation (*ft*)

MW = Median width for freeway/expressway (*ft*)

θ= Cone vision angle for the driver to read the sign

T = Average lane-changing time (*seconds*)

D_1 = Distance between sign and the driver's position right after the cone vision angle is developed in reading the sign (*ft*)

D_2= Distance during lane-changing (*ft*)

D_3= Distance during deceleration from initial speed V_1 to posted advisory speed V_2 (*ft*)

Then, formula for ground sign installation:

$$D_1= ((N-0.5) * W+LS) * \cot \theta \tag{1}$$

Formula for overhead sign installation (assuming the driver's vision ability is uniformly distributed), there are two cases in determining D_1:

(*Case I*) When considering the lateral offset in satisfying the cone vision,

$$D_1^1= ((N-1.5) * W+LS) * \cot \theta \tag{2-1}$$

(*Case II*) When considering the vertical offset in satisfying the cone vision,

$$D_1^2 = VS* \cot \theta \tag{2-2}$$

If $D_1^1> D_1^2$, then the lateral offset satisfying the cone vision first, i.e., $D_1= D_1^2$; otherwise, $D_1= D_1^2$

Formula for median sign installation:

$$D_1= (0.5*MW+0.5*W)* \cot \theta \tag{3}$$

Formula for all these three installation methods:

$$D_2=1.467*V_1*T*(N-1) \quad (4)$$

$$D_3 = (V^2_1 - V^2_2)/2*g*(f+G) = (V^2_1 - V^2_2)/30*(f+G) \quad (5)$$

Where:

g = Gravitational constant (= $32.2\ ft/s^2$);

f = Coefficient of skidding friction between tires and the road pavement for different running speeds;

G = Grade, positive for upward and negative for downward.

Therefore, the advance interchange guide sign should be placed at least $D_2+D_3-D_1$ in advance of the exit ramp.

Note that if the distance between driver and sign right before 10°cone of vision expires, i.e., LD (see below) is larger than the legibility distance provided by current type and/or size of sign lettering series, upgraded sign series and/or larger lettering should be used until such distance is within the driver's legibility distance. The required sign lettering series and size can be derived as follows:

D_0= Driving distance while the drivers read the sign (*ft*)

V_A= Driver's visual Acuity (20/20 for a driver with normal vision)

L = Legibility for a given sign lettering series (*ft/in*)

M = Number of words on the sign

H = Height of the letters on the sign (*ft*)

t_{pr} = Perception-reaction time for the driver to read words from the sign (assumed *1.0+M/3, seconds)*

LD = Required legibility distance for drivers when he/she sees the sign (*ft*)

SD = Distance from the middle of the driver's driving path to the left edge of the sign for ground-mounted and overhead-mounted signs, or to the right edge of the sign for median-mounted sign (*ft*)

Then, $D_0 = 1.467*V_1*t_{pr}$;

For ground-mounted sign, $SD = (N-0.5)*W+LS$;

For overhead-mounted sign, $SD = (N-1.5)*W+LS$;

For median-mounted sign, $SD = 0.5*MW+0.5*W$.

Therefore, $LD = \sqrt{SD^2 + (D_0 + D_1)^2}$, since $LD= L*H*V_A$, the required lettering size $H= LD/(L*V_A)$, Check the sign lettering series table in MUTCD and select the closest one. Alternatively, one could upgrade the type of lettering series to provide the required legibility as computed from $L = LD/(H*V_A)$.

2.6 An Illustrative Example

In the above model, assume the east-west bound freeway has 3 lanes at the interchange approach with design speed equal to 65 *mph*. The design speed on the exit ramp is 30 *mph.* The lane width is 12 *ft* and the median width is 14 *ft* for this freeway. Assume also the lateral offset for both ground installation and overhead installation to be 6 *ft* and the vertical offset for overhead installation to be 7 *ft*, which are warranted by the *2003 MUTCD*. Without loss of generality, assume 8.1 seconds for one lane-change and drivers need to complete reading and comprehending the sign before 10º cone of vision is expired. The sign legibility distance is 175 ft for 5-in Series D lettering, which is assumed in MUTCD for advance warning sign placement. In summary, the model parameters are set up as follows:

$V_1 = 65$ *mph*, $V_2 = 30$ *mph*, $N = 3$, $W = 12$ *ft*; $LS = 6$ *ft*; $VS = 7$ *ft*; $MW = 14$ *ft*, $\theta = 10°$, $T = 8.1$ *seconds*, $G = 0.0$, $f = 0.29$ for 65 *mph*.

Finally, since the middle of the leftmost lane on travel way is considered the critical driving path, which implies that the driver needs to execute lane changes twice before existing to the exit ramp. Based on the proposed operational model and the parameters, the required advance placement distance is calculated as follows:

For ground sign installation, $D_1 = ((3-0.5)*12+6)*cot10^o = 238.2$ *ft.*

For overhead sign installation, if considering the lateral offset in satisfying the 10°cone vision, $D_1^1 = ((3\text{-}1.5)*12+6)*cot10^o=170.14\ ft$; if considering the vertical offset in satisfying the cone vision, $D_1^2= 7*cot10^o=39.7\ ft$.

Since *170.14 ft > 39.7 ft*, so the lateral offset satisfies the *10*°cone vision first, so take $D_1= D_1^1=170.14\ ft$ for overhead sign installation.

For median sign installation, $D_1= (0.5*14+0.5*12)*cot10^o=73.7\ ft$.

For all these three installation methods, $D_2= 1.467*65*8.1*2=1544.8\ ft$, $D_3= (65^2\text{-}30^2)/30*(0.29+0.0)=382.2\ ft$; $D_0=257.46\ ft$.

Therefore, the placement distance for this advance interchange guide sign should be at least,

For ground sign installation, $D_0+D_2+D_3\text{-}D_1=257.46+1544.8+382.2\text{-}238.2=1947.2\ ft$.

For overhead sign installation, $D_0+D_2+D_3\text{-}D_1=257.46+1544.8+382.2\text{-}170.14=2015.3\ ft$.

For median sign installation, $D_0+D_2+D_3\text{-}D_1=257.46+1544.8+382.2\text{-}73.7=2111.7\ ft$.

Tables show that the placement distance (rounded up to the quarter-hundred ft) for advance guide sign under different scenarios, in which V^1 ranges from 20 *mph* to 70 *mph* and V^2 ranges from 10 *mph* to 50 *mph* based on the proposed operational model. The value in each parenthesis in each cell shows the required quarter-mile based on the format used in *2003 MUTCD* guidelines for advance guide sign.

Lastly, since the required placement distance for ground-mounted signs, i.e., *238.2 ft*, is greater than 175 *ft*, the driver might not see the sign clearly under lettering series D, so the sign series should be upgraded and/or larger lettering size should be used to provide greater legibility distance. By following the above discussions on lettering series, an 11-in series D or a 9-inch series F word legend should be used.

2.7 Results

The 2003 MUTCD guideline disregards the number of lanes in designing the advance guide sign placement distance, which might underestimate the actual requirement as far as safety is concerned. In this study the required advance guide sign placement distance is determined for different number of lane configurations ranging from 2 to 5 under different guide sign installation methods. From the results based on the proposed operation model, it can be seen that the advance guide sign placement distance increases as the number of lanes increases, due to the increased travel distance during the lane-changing process as the number of lanes increases. For all three installation methods, the placement distance increases if V_1increases or V_2 decreases, due to the increased travel distance in deceleration from V_1 *to* V_2 if the speed differential becomes larger. When the number of lanes is 2, almost all of the placement distance can be rounded up to 0.25 mi. The distance requirement gradually changes from 0.25 mi to 0.5 mi if V_1 increases or V_2 decreases when the number of lanes increases. When the number of lanes is 3 and V_1 is over 60 *mph*, the placement distance is over 0.25 mi and can be rounded up to 0.5 mi. When the number of lanes increases to 4, the placement distance gradually changes from 0.25 mi to 0.5 mi and then to 0.75 mi if V_1 increases or V_2 decreases. Finally, when the number of lanes is 5 and $V_1$is over 60 *mph*, the placement distance is over 0.5 mi and can be rounded up to 0.75 mi.

For the same V_1, V_2, and number of lanes, under current parameters assumptions, the ground sign installation has the shortest placement distance requirement and the overhead sign installation has the longest placement distance requirement. However, which installation method has the shortest placement distance requirement in practice will depend on the actual roadway geometry and the sign installation location. For example, from the above derivation process, the lateral offset for the ground sign installation has great influence on the placement distance. And if the sign locates farther away from the edge of rightmost traffic lane, the placement distance decreases because the driver is required to complete sign reading and comprehension at further distance ahead (D_1 increases) before clear cone of vision expires.

Detailed placement distance standards of advance sign are shown in all following tables:

Table 2.2 Advance Sign Placement Distance (ft) Part A– Ground Type & N=2

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	170.14	594.14	184.41	850.46	103.73	769.78
55	250.12	170.14	653.55	244.92	978.45	164.24	897.77
60	255.26	170.14	712.96	311.20	1109.27	230.52	1028.59
65	257.46	170.14	772.38	383.23	1242.93	302.55	1162.24
70	256.73	170.14	831.79	461.03	1379.40	380.35	1298.72
75	253.06	170.14	891.20	544.60	1518.71	463.92	1438.03
80	246.46	170.14	950.62	633.92	1660.85	553.24	1580.17

Table 2.3 Advance Sign Placement Distance (ft) Part B– Ground Type & N=2

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	170.14	594.14	0.00	666.05	-	-
55	250.12	170.14	653.55	60.51	794.04	-	-
60	255.26	170.14	712.96	126.78	924.86	0.00	798.08
65	257.46	170.14	772.38	198.82	1058.51	72.04	931.73
70	256.73	170.14	831.79	276.62	1194.99	149.84	1068.21
75	253.06	170.14	891.20	360.18	1334.30	233.40	1207.52
80	246.46	170.14	950.62	449.51	1476.44	322.72	1349.65

Table 2.4 Advance Sign Placement Distance (ft) Part A– Ground Type & N=3

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	238.20	1188.27	184.41	1376.54	103.73	1295.86
55	250.12	238.20	1307.10	244.92	1563.94	164.24	1483.26
60	255.26	238.20	1425.92	311.20	1754.18	230.52	1673.50
65	257.46	238.20	1544.75	383.23	1947.24	302.55	1866.56
70	256.73	238.20	1663.58	461.03	2143.14	380.35	2062.46
75	253.06	238.20	1782.41	544.60	2341.86	463.92	2261.18
80	246.46	238.20	1901.23	633.92	2543.41	553.24	2462.73

Table 2.5 Advance Sign Placement Distance (ft) Part B– Ground Type & N=3

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	238.20	1188.27	0.00	1192.12	-	-
55	250.12	238.20	1307.10	60.51	1379.53	-	-
60	255.26	238.20	1425.92	126.78	1569.77	0.00	1442.98
65	257.46	238.20	1544.75	198.82	1762.83	72.04	1636.05
70	256.73	238.20	1663.58	276.62	1958.72	149.84	1831.94
75	253.06	238.20	1782.41	360.18	2157.44	233.40	2030.66
80	246.46	238.20	1901.23	449.51	2359.00	322.72	2232.21

Table 2.6 Advance Sign Placement Distance (ft) Part A– Ground Type & N=4

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	306.26	1782.41	184.41	1902.62	103.73	1821.93
Table 2.6 (Continued)							
55	250.12	306.26	1960.65	244.92	2149.44	164.24	2068.75
60	255.26	306.26	2138.89	311.20	2399.08	230.52	2318.40
65	257.46	306.26	2317.13	383.23	2651.56	302.55	2570.88
70	256.73	306.26	2495.37	461.03	2906.87	380.35	2826.19
75	253.06	306.26	2673.61	544.60	3165.00	463.92	3084.32
80	246.46	306.26	2851.85	633.92	3425.97	553.24	3345.29

Table 2.7 Advance Sign Placement Distance (ft) Part B– Ground Type & N=4

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	306.26	1782.41	0.00	1718.20	-	-
55	250.12	306.26	1960.65	60.51	1965.02	-	-
60	255.26	306.26	2138.89	126.78	2214.67	0.00	2087.89
65	257.46	306.26	2317.13	198.82	2467.15	72.04	2340.36
70	256.73	306.26	2495.37	276.62	2722.45	149.84	2595.67
75	253.06	306.26	2673.61	360.18	2980.59	233.40	2853.81
80	246.46	306.26	2851.85	449.51	3241.55	322.72	3114.77

Table 2.8 Advance Sign Placement Distance (ft) Part A– Ground Type & N=5

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	374.32	2376.54	184.41	2428.69	103.73	2348.01
55	250.12	374.32	2614.19	244.92	2734.93	164.24	2654.25
60	255.26	374.32	2851.85	311.20	3043.99	230.52	2963.31
65	257.46	374.32	3089.50	383.23	3355.88	302.55	3275.20
70	256.73	374.32	3327.16	461.03	3670.60	380.35	3589.92
75	253.06	374.32	3564.81	544.60	3988.15	463.92	3907.47
80	246.46	374.32	3802.46	633.92	4308.53	553.24	4227.85

Table 2.9 Advance Sign Placement Distance (ft) Part B– Ground Type & N=5

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	374.32	2376.54	0.00	2244.28	-	-
55	250.12	374.32	2614.19	60.51	2550.51	-	-
60	255.26	374.32	2851.85	126.78	2859.57	0.00	2732.79
65	257.46	374.32	3089.50	198.82	3171.47	72.04	3044.68
70	256.73	374.32	3327.16	276.62	3486.19	149.84	3359.40
75	253.06	374.32	3564.81	360.18	3803.73	233.40	3676.95
Table 2.9 (Continued)							
80	246.46	374.32	3802.46	449.51	4124.11	322.72	3997.33

Table 2.10 Advance Sign Placement Distance (ft) Part A– Overhead Type & N=2

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	102.09	594.14	184.41	918.52	103.73	837.84
55	250.12	102.09	653.55	244.92	1046.51	164.24	965.83
60	255.26	102.09	712.96	311.20	1177.33	230.52	1096.65
65	257.46	102.09	772.38	383.23	1310.98	302.55	1230.30
70	256.73	102.09	831.79	461.03	1447.46	380.35	1366.78
75	253.06	102.09	891.20	544.60	1586.77	463.92	1506.09
80	246.46	102.09	950.62	633.92	1728.91	553.24	1648.23

Table 2.11 Advance Sign Placement Distance (ft) Part B– Overhead Type & N=2

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	102.09	594.14	0.00	734.10	-	-
55	250.12	102.09	653.55	60.51	862.10	-	-
60	255.26	102.09	712.96	126.78	992.92	0.00	866.13
65	257.46	102.09	772.38	198.82	1126.57	72.04	999.78
70	256.73	102.09	831.79	276.62	1263.05	149.84	1136.26
75	253.06	102.09	891.20	360.18	1402.36	233.40	1275.57
80	246.46	102.09	950.62	449.51	1544.49	322.72	1417.71

Table 2.12 Advance Sign Placement Distance (ft) Part A– Overhead Type & N=3

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	170.14	1188.27	184.41	1444.60	103.73	1363.91
55	250.12	170.14	1307.10	244.92	1632.00	164.24	1551.32
60	255.26	170.14	1425.92	311.20	1822.24	230.52	1741.56
65	257.46	170.14	1544.75	383.23	2015.30	302.55	1934.62
70	256.73	170.14	1663.58	461.03	2211.19	380.35	2130.51
75	253.06	170.14	1782.41	544.60	2409.92	463.92	2329.23
80	246.46	170.14	1901.23	633.92	2611.47	553.24	2530.79

Table 2.13 Advance Sign Placement Distance (ft) Part B– Overhead Type & N=3

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	170.14	1188.27	0.00	1260.18	-	-
55	250.12	170.14	1307.10	60.51	1447.59	-	-
60	255.26	170.14	1425.92	126.78	1637.82	0.00	1511.04
65	257.46	170.14	1544.75	198.82	1830.89	72.04	1704.10
70	256.73	170.14	1663.58	276.62	2026.78	149.84	1900.00
75	253.06	170.14	1782.41	360.18	2225.50	233.40	2098.72
80	246.46	170.14	1901.23	449.51	2427.05	322.72	2300.27

Table 2.14 Advance Sign Placement Distance (ft) Part A– Overhead Type & N=4

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	238.20	1782.41	184.41	1970.67	103.73	1889.99
55	250.12	238.20	1960.65	244.92	2217.49	164.24	2136.81
60	255.26	238.20	2138.89	311.20	2467.14	230.52	2386.46
65	257.46	238.20	2317.13	383.23	2719.62	302.55	2638.94
70	256.73	238.20	2495.37	461.03	2974.93	380.35	2894.24
75	253.06	238.20	2673.61	544.60	3233.06	463.92	3152.38
80	246.46	238.20	2851.85	633.92	3494.03	553.24	3413.34

Table 2.15 Advance Sign Placement Distance (ft) Part B– Overhead Type & N=4

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	238.20	1782.41	0.00	1786.26	-	-
55	250.12	238.20	1960.65	60.51	2033.08	-	-
60	255.26	238.20	2138.89	126.78	2282.73	0.00	2155.94
65	257.46	238.20	2317.13	198.82	2535.21	72.04	2408.42
70	256.73	238.20	2495.37	276.62	2790.51	149.84	2663.73
75	253.06	238.20	2673.61	360.18	3048.65	233.40	2921.86
80	246.46	238.20	2851.85	449.51	3309.61	322.72	3182.83

Table 2.16 Advance Sign Placement Distance (ft) Part A– Overhead Type & N=5

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	306.26	2376.54	184.41	2496.75	103.73	2416.07
55	250.12	306.26	2614.19	244.92	2802.98	164.24	2722.30
60	255.26	306.26	2851.85	311.20	3112.05	230.52	3031.36
65	257.46	306.26	3089.50	383.23	3423.94	302.55	3343.26
70	256.73	306.26	3327.16	461.03	3738.66	380.35	3657.98
75	253.06	306.26	3564.81	544.60	4056.21	463.92	3975.53
80	246.46	306.26	3802.46	633.92	4376.58	553.24	4295.90

Table 2.17 Advance Sign Placement Distance (ft) Part B– Overhead Type & N=5

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	306.26	2376.54	0.00	2312.34	-	-
55	250.12	306.26	2614.19	60.51	2618.57	-	-
60	255.26	306.26	2851.85	126.78	2927.63	0.00	2800.85
65	257.46	306.26	3089.50	198.82	3239.52	72.04	3112.74
70	256.73	306.26	3327.16	276.62	3554.24	149.84	3427.46
75	253.06	306.26	3564.81	360.18	3871.79	233.40	3745.01
80	246.46	306.26	3802.46	449.51	4192.17	322.72	4065.39

Table 2.18 Advance Sign Placement Distance (ft) Part A– Median Type & N=2

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	73.73	594.14	184.41	946.87	103.73	866.19
55	250.12	73.73	653.55	244.92	1074.87	164.24	994.19
60	255.26	73.73	712.96	311.20	1205.69	230.52	1125.01
65	257.46	73.73	772.38	383.23	1339.34	302.55	1258.66
70	256.73	73.73	831.79	461.03	1475.82	380.35	1395.14
75	253.06	73.73	891.20	544.60	1615.13	463.92	1534.45
80	246.46	73.73	950.62	633.92	1757.27	553.24	1676.58

Table 2.19 Advance Sign Placement Distance (ft) Part B– Median Type & N=2

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	73.73	594.14	0.00	762.46	-	-
55	250.12	73.73	653.55	60.51	890.45	-	-
60	255.26	73.73	712.96	126.78	1021.28	0.00	894.49
65	257.46	73.73	772.38	198.82	1154.93	72.04	1028.14
70	256.73	73.73	831.79	276.62	1291.41	149.84	1164.62
75	253.06	73.73	891.20	360.18	1430.71	233.40	1303.93
80	246.46	73.73	950.62	449.51	1572.85	322.72	1446.07

Table 2.20 Advance Sign Placement Distance (ft) Part A– Median Type & N=3

posted or 85th Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	73.73	1188.27	184.41	1541.01	103.73	1460.33
55	250.12	73.73	1307.10	244.92	1728.42	164.24	1647.74
60	255.26	73.73	1425.92	311.20	1918.65	230.52	1837.97
65	257.46	73.73	1544.75	383.23	2111.72	302.55	2031.03
70	256.73	73.73	1663.58	461.03	2307.61	380.35	2226.93
75	253.06	73.73	1782.41	544.60	2506.33	463.92	2425.65
80	246.46	73.73	1901.23	633.92	2707.88	553.24	2627.20

Table 2.21 Advance Sign Placement Distance (ft) Part B– Median Type & N=3

posted or 85^{th} Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	73.73	1188.27	0.00	1356.60	-	-
55	250.12	73.73	1307.10	60.51	1544.00	-	-
60	255.26	73.73	1425.92	126.78	1734.24	0.00	1607.45
65	257.46	73.73	1544.75	198.82	1927.30	72.04	1800.52
70	256.73	73.73	1663.58	276.62	2123.19	149.84	1996.41
75	253.06	73.73	1782.41	360.18	2321.92	233.40	2195.13
80	246.46	73.73	1901.23	449.51	2523.47	322.72	2396.68

Table 2.22 Advance Sign Placement Distance (ft) Part A– Median Type & N=4

posted or 85^{th} Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	73.73	1782.41	184.41	2135.14	103.73	2054.46
55	250.12	73.73	1960.65	244.92	2381.96	164.24	2301.28
60	255.26	73.73	2138.89	311.20	2631.61	230.52	2550.93
65	257.46	73.73	2317.13	383.23	2884.09	302.55	2803.41
70	256.73	73.73	2495.37	461.03	3139.40	380.35	3058.72
75	253.06	73.73	2673.61	544.60	3397.53	463.92	3316.85
80	246.46	73.73	2851.85	633.92	3658.50	553.24	3577.82

Table 2.23 Advance Sign Placement Distance (ft) Part B– Median Type & N=4

posted or 85^{th} Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	73.73	1782.41	0.00	1950.73	-	-
55	250.12	73.73	1960.65	60.51	2197.55	-	-
60	255.26	73.73	2138.89	126.78	2447.20	0.00	2320.42
65	257.46	73.73	2317.13	198.82	2699.68	72.04	2572.89
70	256.73	73.73	2495.37	276.62	2954.98	149.84	2828.20
75	253.06	73.73	2673.61	360.18	3213.12	233.40	3086.33
80	246.46	73.73	2851.85	449.51	3474.08	322.72	3347.30

Table 2.24 Advance Sign Placement Distance (ft) Part A– Median Type & N=5

posted or 85^{th} Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				30		40	
				D3	D	D3	D
50	242.06	73.73	2376.54	184.41	2729.28	103.73	2648.60
55	250.12	73.73	2614.19	244.92	3035.51	164.24	2954.83
60	255.26	73.73	2851.85	311.20	3344.58	230.52	3263.89
65	257.46	73.73	3089.50	383.23	3656.47	302.55	3575.79
70	256.73	73.73	3327.16	461.03	3971.19	380.35	3890.51
75	253.06	73.73	3564.81	544.60	4288.74	463.92	4208.05
80	246.46	73.73	3802.46	633.92	4609.11	553.24	4528.43

Table 2.25 Advance Sign Placement Distance (ft) Part B– Median Type & N=5

posted or 85^{th} Speed(mph)	D0	D1	D2	Advisory speed on Exit Ramp(mph)			
				50		60	
				D3	D	D3	D
50	242.06	73.73	2376.54	0.00	2544.87	-	-
55	250.12	73.73	2614.19	60.51	2851.10	-	-
60	255.26	73.73	2851.85	126.78	3160.16	0.00	3033.38
65	257.46	73.73	3089.50	198.82	3472.05	72.04	3345.27
70	256.73	73.73	3327.16	276.62	3786.77	149.84	3659.99
75	253.06	73.73	3564.81	360.18	4104.32	233.40	3977.54
80	246.46	73.73	3802.46	449.51	4424.70	322.72	4297.92

2.8 Conclusions

In this study, a methodology for determining the placement distance of advance guide sign for freeway and expressway is presented. It is found that the roadway geometry in terms of number of lanes has a great impact on the length of advance placement. Important model assumptions and parameters are verified and analyzed, and the results are illustrated using an example. From the analysis, conclusions can be made as follows:

(1) When the number of lanes increases, the required advance placement distance increases.

(2) When the design speed on the main highway increases, the required advance placement distance increases.

(3) The ground sign installation has the shortest placement distance requirement and the

overhead sign installation has the longest placement distance requirement under the same condition. Selection of installation method depends on actual circumstances on roadways.

These findings are not reflected in the current *MUTCD* guideline, but could serve as an important supplement to current guidelines. The proposed operational model can also be used as a reference to assist field engineers and technicians more effectively in guide sign selection and installation.

PART II – SAFETY ANALYSIS

CHAPTER 3 INTRODUCTION AND LITERATURE REVIEW

3.1 Background

Freeways play important roles in the highway system around the country. In the United States, the interstate highway system, which composes less than 2% of the total urban highway mileage, carries more than 20% of the traffic by the end of 2006. Freeways provide the specific traffic facility which allows the traffic run smoothly in the roadway network at the highest level. They are constructed according to the highest highway design standards and regulated public movements by full controls of traffic elements such as capacity, posted speed, geometrics fundamentals, and level of service.

Exit ramps are the only control accesses used for traffic exiting freeways. They also serve as transitions from freeways to secondary crossroads which could be freeways, major or minor arterials, or local streets. The design of freeway exit ramps could significantly impact the safety and operation performances on freeways, exit ramps and crossroads. The AASHTO Green Book (A Policy on the Design of Geometric Highways and Streets) mentioned that complex design components make ramps vary from simple to comprehensive layouts so that each ramp site should be studied and planned carefully. Freeway diverge areas are the specific segments that divide the freeway traffic exiting from or continuing on the freeway mainlines. Several different diverge types (including freeways and with exit ramps) call exit ramp types in this study. These types cause different results of safety performances on the freeway diverge area by different ways. Exit ramp section is another important concern in this study. Exit ramps provide limit-accesses from freeways to other freeways, lower-speed arterials or local streets. A few factors, such as geometrics, traffics, and local conditions, have different relationships with crashes. These facts include more than deceleration distances, exit ramp lengths, design speeds, operating speeds, speed differences, exit ramp configurations, and road conditions. Better understanding the relationships among them would help improve the safety, efficiency, mobility, accessibility, and accommodation aspects for both freeway diverge areas and exit ramp sections. Ramp Management and Control Handbook, published by U.S. Department of Transportation (DOT) and Federal Highway Administration (FHWA) in 2004, aims to manage ramp policies, strategies and technologies as to improve safety on the exit ramp and the influential areas. Ramp management strategies control the flow vehicles exiting a freeway not only on the exit ramps, but also on the freeway neighboring areas. A before and after evaluation of ramp crashes in Minneapolis found that the number of peak period crashes on freeways and ramps increased 26% when there was no ramp control strategy in 2001. This case revealed the reality that resolutions to the deficiencies on the freeway diverge areas and exit ramp sections could help to improve safety.

Successful managements on the two research segments, freeway diverge areas and exit ramp sections, could obtain benefits on society, economics and cultures and gain satisfactions on safety improvements. However, the impacts of exit ramp types on the safety performance of freeway diverge areas have not been well studied or documented until recently. Few have focused on the impacts of the types of exit ramps concerning the lane balance problems such as the number of lanes used by traffic to exit freeways. The details of the relationship between the lane balance and safety are not well understood. Since the limit work that has been performed, a few tentative conclusions might to be drawn. It can assume that potential improvements will lead to fewer crashes, thus enhance safety on the freeway diverge areas. On the exit ramp sections, the various influential factors on the safety performance at entire exit ramp sections need to be revised and re-conducted since previous studies have a few limitations. For example, some predictive crash models concerned different ramp configurations and ramp length, however the control types of ramp terminals did not contain in these models. Some models combined the off ramps and on ramps. The combination might ignore the dissimilar operating factors between the two different kinds of ramps.

Several types of exit ramps are used for traffic to exit freeways on the diverge areas. The increasing vehicular crashes in freeway diverge areas lift up the need to select the best exit ramp designs to improve safety on freeway diverge areas. The problem is relatively new and highly demanded in today’s highway system. For the exit ramp sections, little focus has been put on the safety issues in the State of Florida. So this study would conduct comprehensive crash comparisons and analyses on freeway exit ramp sections for the whole state. The results of two research parts, freeway diverge areas and exit ramp sections in this study, will help transportation decision makers develop tailored technical guidelines governing the selection of the optimum exit ramp types and combinations of related factors to be used on freeway diverge areas and exit ramp sections.

3.2 Research Subject

On the freeway diverge areas, the most commonly used freeway exit ramps include two-lane exit ramps with an optional lane, two-lane exit ramps without optional lane, single-lane exit ramps with widening to two lanes on the ramp beyond the exit gore, and three basic number of through lanes changed to two through lanes with one lane reduced and designated as the exit lane. Drivers exiting a freeway must decrease vehicle speeds and weave to the deceleration lane toward the entrance of the exit ramp. Different types of exit ramps require drivers to make distinctive decisions to complete related maneuvers both for exiting and continuing with the freeway. As a result, different exit ramp design may impact the safety and operational performance of freeway diverge areas in different ways. On the exit ramp sections, different ramp configurations such as diamond, out connection, free flow, and parclo flow and other factors such as widening lanes, pavement paintings, and terminal controls might confuse drivers as well. These mixed influential features on the exit ramp cause existing problems and situations more multifaceted. This study processes to quantitatively evaluate the safety features of two issues.

3.2.1 Freeway Diverge Areas

None of the studies for the past two decades focused on the lane balance problems on the freeway diverge area which directly connects the mainline segment to exit ramps. AASHTO Green book defines the lane balance as the number of approach lanes on the highway after the exit should equal to the number of lanes on the highway beyond the exit, plus the number of lanes on the exit, minus one. The fundamental arrangement of a freeway segment is the designation of the basic number of lanes which should be consistency along the freeway. The basic number of lanes might be added or deleted where the traffic volumes increase or decreased at some degrees. On the freeway diverge area, part traffic on the freeways beyond the exits leave the freeway and so that the volumes change in this segment. The one or two outer lanes may drop to the exit lanes so that the number of lanes on the freeway mainline sections did not balance ahead of or after the exits. This would not only cause confusions for the exiting vehicles but also for the continuing vehicles on the freeways. The lane-balanced and unbalanced exit ramps require drivers take different maneuvers. Even considering the lane balanced exit ramps or the unbalanced exit ramps respectively, different numbers of exit lanes on the freeway segments have different characteristics as well. The study would focus on the lane balance issues which are innovated and original in the freeway exit ramps studies.

The exit ramp type is defined by the number of lanes used for traffic to exit freeways. They could be single-lane exit ramps or two-lane exit ramps. After reviewing the sites in the whole Florida interstate highway systems, expressways, turnpikes and parkways, four types are used frequently for the state. So four different groups based on the types of exit ramps are characterized for the study. For convenience, they were set as Type 1 exit ramps (Type 1), Type 2 exit ramps (Type 2), Type 3 exit ramps (Type 3) and Type 4 exit ramps (Type 4) respectively. The definitions of each type of exit ramp are described below and illustrated in Figure 1 through Figure 4 below.

1) Type 1 exit ramp — Parallel from a tangent single-lane exit ramp shown in Figure 3.1: It is a full width parallel from tangent that leads to either a tangent or flat exiting curve which includes a decelerating taper. The horizontal and vertical alignment of type 1 exit tamps were based on the selected design speed equal or less than the intersecting roadways. No direct drop lanes on the mainline sections beyond or after exits. The outer lane with a tangent would be a drop lane to the exits and become the though lane on the exit ramp section.

2) Type 2 exit ramp — Single-lane exit ramp without a taper shown in Figure 3.2: This type is when the outer lane becomes a drop lane at the exit gore forming a lane reduction. A paved and striped area beyond the theoretical gore were present at this type of exit ramps to provide a maneuver and recovery area. No additional lane was added when compared with Type 1.

3) Type 3 exit ramp — Two-lane exit with an optional lane shown in Figure 3.3: This type includes two exit lanes while a large percentage of traffic volume on the freeway beyond the painted nose would leave at this particular exit. An auxiliary lane to develop the full capacity of two lane exit was developed for 1500 feet. The entire operations in this type of exit ramps took place over a significant length of the freeway in most cases. The outer one of the two exit lanes directly drops to the exit ramps. But the inner lane of the two exit lanes, which is an optional lane, has two alternatives by continuing on the freeway or running off the freeways.

4) Type 4 exit ramp — Two-lane exit without an optional lane is shown in Figure 3.4: It is used where one of the through lanes, the outer lane, is reduced and another full width parallel from tangent lane developed with a taper is also forced to exit. It differs as from Type 3 exit ramps as Type 4 exit ramps do not enclose the optional lane.

From the figures, they indicate that Type 1 and Type 3 are lane balanced ones while Type 2 and Type 4 are lane unbalanced exit ramps. In practice, there is a type 5 exit ramp which is a two-lane exit ramp without optional lane and without a taper, which is not widely used in Florida and the samples being found are too small to draw defensible conclusions.

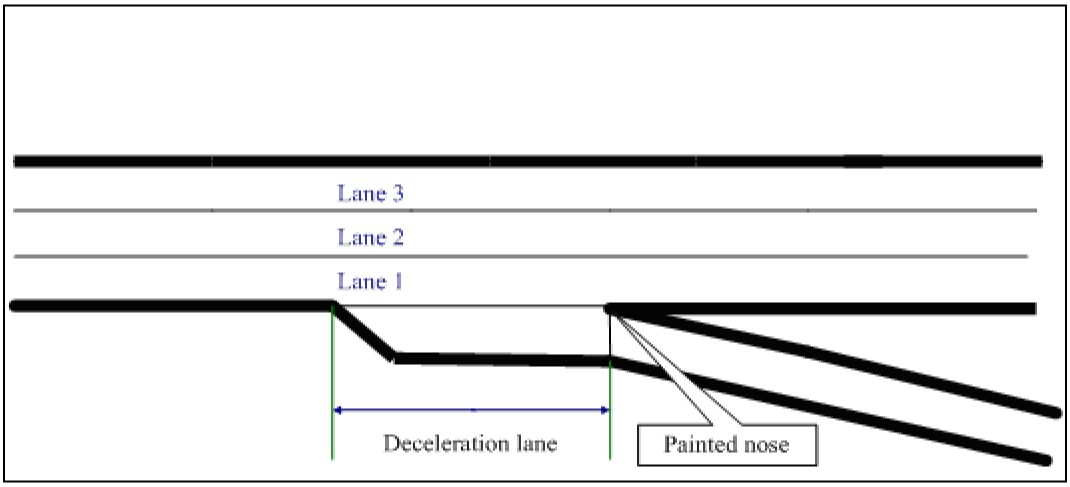


Figure 3.1 Type 1 Exit Ramp: Parallel from a Tangent Single-lane Exit Ramp

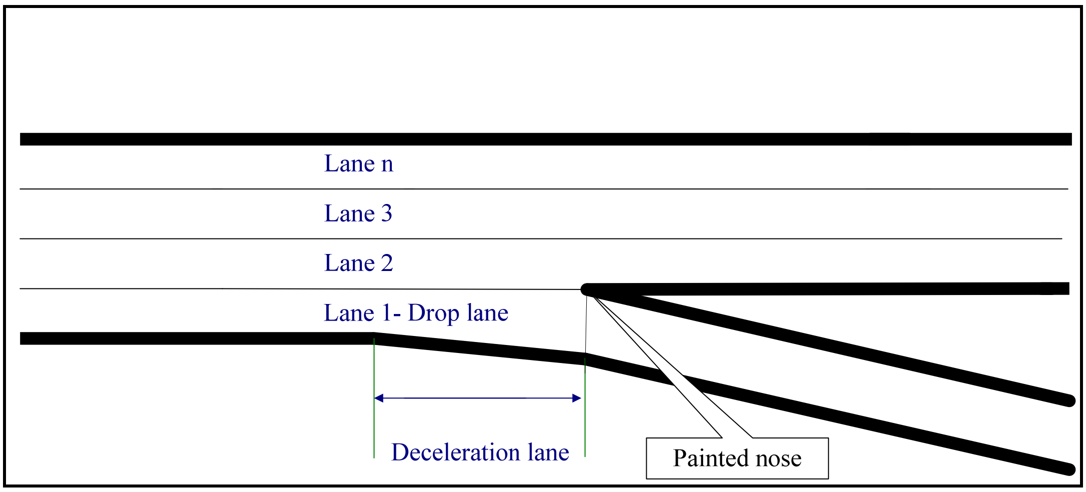


Figure 3.2 Type 2 Exit Ramp: Single-lane Exit Ramp without a Taper

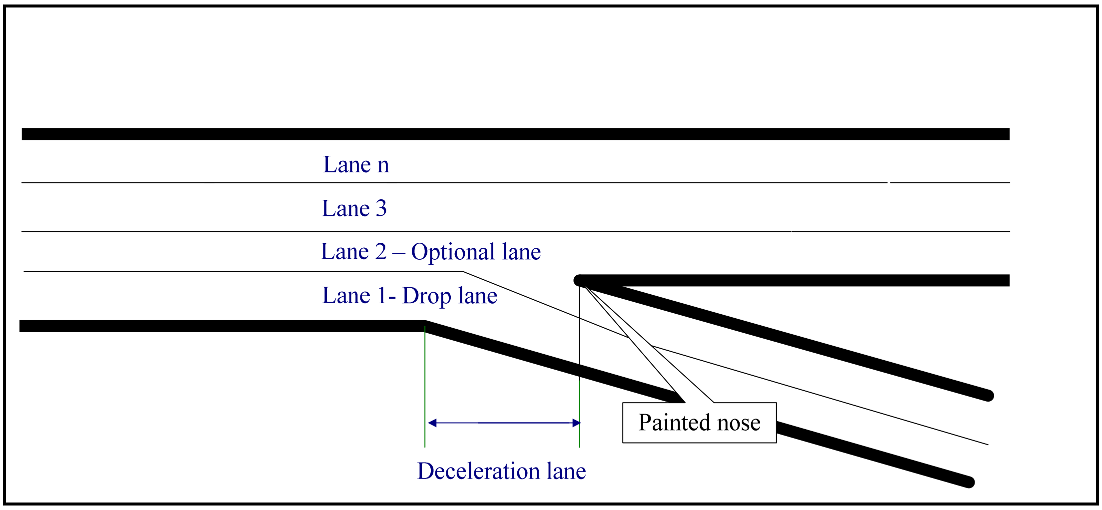


Figure 3.3 Type 3 Exit Ramp: Two-lane Exit Ramp with an Optional Lane

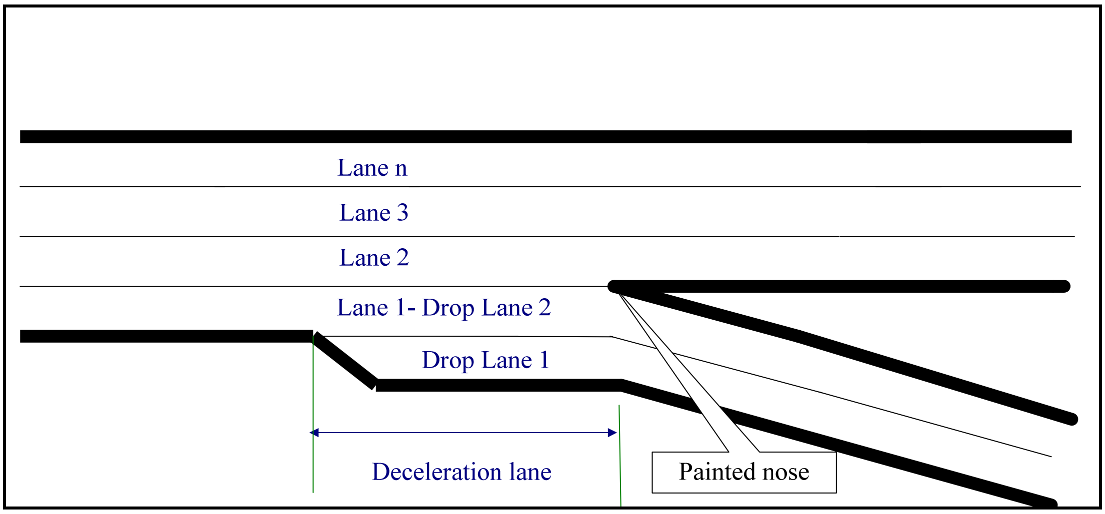


Figure 3.4 Type 4 Exit Ramp: Two-lane Exit Ramp without an Optional Lane

3.2.2 Entire Exit Ramp Sections

The entire exit ramp section from the beginning of pointed nose, which diverge the freeways and ramps, to the end of ramp terminal is another research concern. This study is to acquire an adaptable, practical, and integral transition system from the freeway to the secondary crossroad. Ramp designing contains many possible influential factors such as ramp configurations, ramp design speed, lane numbers, ramp terminal control types, ramp length, or ramp curvatures.

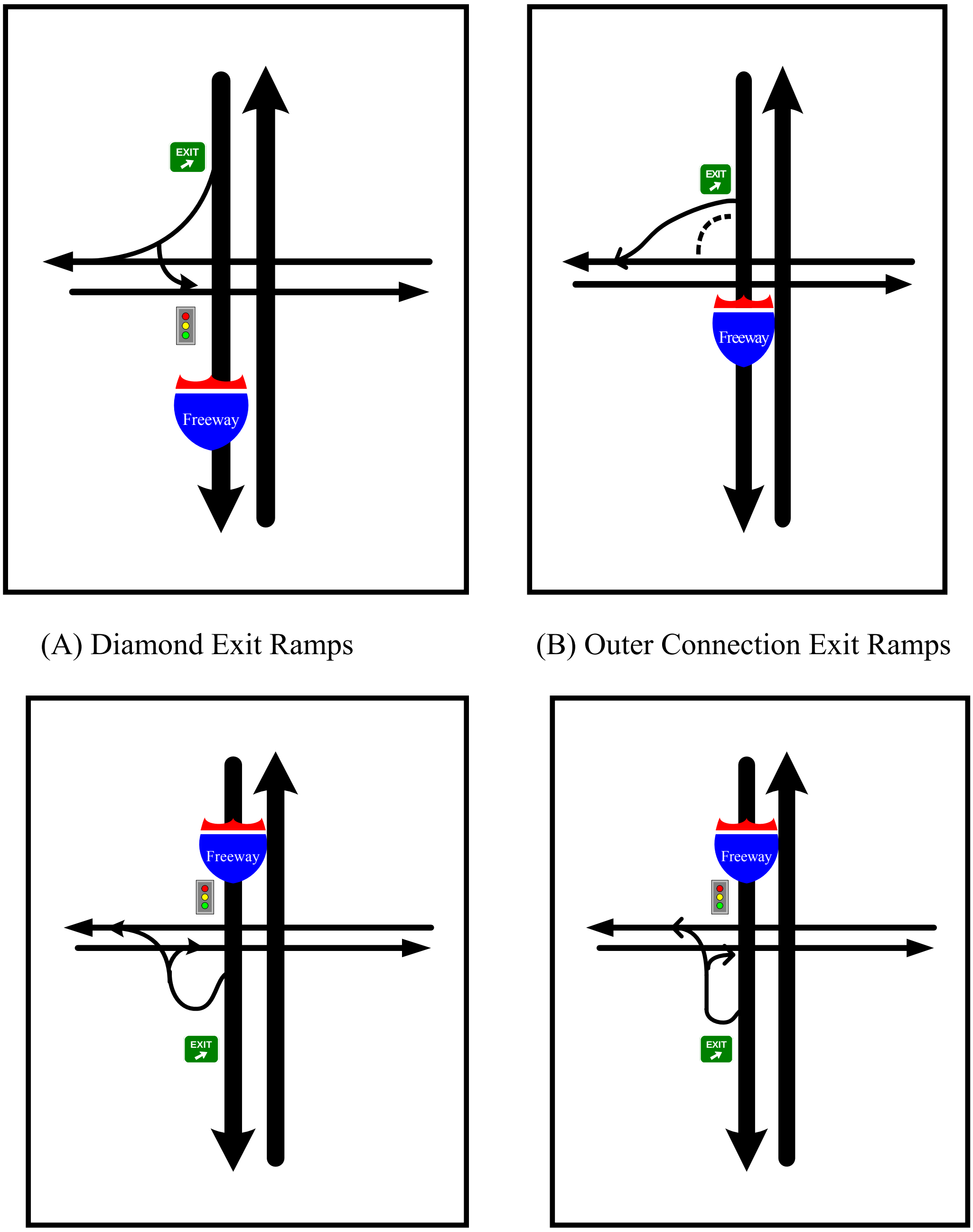
Ramp configurations are usually considered as the ramp types in the previous studies. Bauer and Harwood's analyses show that diverse ramp configuration designs have significantly dissimilar impacts on the safety performance especially for off ramps. Typically various configurations accommodate to the ramp sites by the features of site locations. In order to clearly indicate the safety performance with related parameters, the ramp configuration was considered one of them. Four widely used configurations in Florida are identified in the study. They were briefly defined as diamond exit ramps, out connection exit ramps, free-flow loop exit ramps and parclo loop exit ramps. From Figure 3.5-A to D illustrate the four ramp configurations which describe the shape of ramps in simplified modes. Figure 3.5-A is a diamond exit ramp which is a one-way road with both left and right turnings at terminals. Figure 3.5-B is an out connection exit ramp which only supplies the single turn at the ends of exit ramps.

Figure 3.5-C and 3.5-D are two classic loop ramps that make at least 270 degrees of turning movements to the secondary roads. Free-flow loop ramps are designed as full cloverleaf ramps with or without collector or distributor roads on the ramp segments. The parclo loop exit ramp is a partial cloverleaf ramp which has a preference to provide an arrangement setting the right exiting vehicles. This configuration could give either one or two turning ways at the exit terminals while the exit ramps' location meets the requirements to provide enough design radii, space, curvatures and related geometric criteria.

3.3 Research Objectives

The objective of the study is to evaluate safety performances of different exit ramps used in Florida and nationals. The research objectives can divide into two parts. The first one is to evaluate how the impacts of different exit ramp types on the safety performance of freeway diverge areas. The second one focuses on identifying the different factors contributing to the crashes happening on the exit ramp sections. This study developed quantitative evaluations and comparisons on the freeway diverge areas and exit ramp sections accordingly.

Statistical analyses among four types of exit ramps on the freeway diverge areas, parallel from a tangent- single-lane exit ramp, single-lane exit ramp without a taper, two-lane exit ramp with an optional lane and two-lane exit ramp without an optional lane, are conducted. The four different ramp configurations and other parameters on the entire exit ramp sections are examined as well to find their effects on the safety features for the entire exit ramps. Base on the result in this study, it would be a way to judge what kind of geometric, traffic, and combinations of the correlated conditions have the best safety performance on the freeway diverge sections and entire exit ramp sections. This is also a practical step to guide the methods of safety improvements on freeway diverge areas and exit ramp sections. The results could also be applied in design guidelines, handbooks or research projects.



(A) Diamond Exit Ramps (B) Outer Connection Exit Ramps

(C) Free-flow Loop Exit Ramps (D) Parclo Loop Exit Ramps

Figure 3.5 Typical Four Exit Ramp Configurations

3.4 Research Approach

Previous studies were revised and potential safety measurements for this study were selected. Crash histories at selected freeway segments were investigated and crash data were collected. Cross-sectional comparisons were conducted to compare the safety impacts the two segments of freeway diverge areas and exit ramp sections respectively. On the basis of the collected crash data for the diverge areas, statistical analyses were conducted to quantitatively evaluate the impacts of different types of exit ramps on the safety performance of freeway diverge areas and different ramp configurations on exit ramp sections. In addition, crash prediction models were developed to identify the factors that contribute to crashes at selected sites. The results of this study will help transportation decision makers develop tailored technical guidelines governing the selection of the optimum exit ramp to be used on freeways and recommend the optimal design characteristics both on the diverge areas and the entire exit ramps.

3.5 Literature Review

3.5.1 General Freeway Guidelines

Freeways provide the primary transportation networks and roadway systems by achieving the highest functional hierarchy of highway systems by design purposes. The grand reliance on the facilities requires safer and more efficient implements on existing freeways and their related infrastructure systems to improve the safety performances.

The AASHTO Green Book (A Policy on the Design of Geometric Highways and Streets) designs the key requirements on the highway facilities such as the ramps, interchanges and frontage roads. In order to accommodate high traffic demands of safety on freeways, exit ramps and secondary crossroads, designing proper handlings of freeways and ramps are essential in the highway systems. Many factors impacts safety performances on freeways and their adjacent facilities. Also, the crash is a direct index on safety evaluations. The wide variety of site geometric conditions, traffic volumes, highway types, and design layouts could eliminate or increase conflict points at some degrees while crashes related to conflict points at some levels.

During the past several decades, some design regulations mentioned the importance of safety performance of freeway diverge areas and exit ramp sections. Current state and national literature reviews include freeway and ramp management handbooks, guidelines of optimal geometric designs from Highway Capacity Manual and AASHTO, reports from National Cooperative Highway Research Program (NCHRP) and Different State Departments of Transportation, proceedings from Transportation Symposium, papers from transportation engineering journal, etc. Additionally, useful books and publications were also collected to do analysis in the project and current rules, regulations, standards, and practices in Florida were evaluated and summarized for the two research subjects in the sequent sections.

3.5.2 Freeway Diverge Areas

During the past several decades, though some studies have mentioned the freeway exit ramps, none of them focused on the impacts of the number of lanes used by traffic to exit freeways. Closely reviewed the literature, there is little direct paper or evaluation in safety performance of diverge areas which has been researched before. In previous studies, ramp types are usually defined by ramp configurations such as diamond, loop, directional, outer connector, and other instead of the lane balance issues for the diverge sections. Though many design handbooks and guidelines focused on the relationships of geometric elements and collision causes, they did not mention the influence of lane balances on the freeway diverge areas.

In 1969, Cirillo et al. did a purely innovative investigation on the traffic crash study on the interstate system for that period. They found that the relationship could be established between fatality crashes and geometric elements. The geometric factor included several types of interchanges, paved shoulders, sight distance, delineators, surface types, and other variables. After about thirty years, Garber and Fontaine developed a guideline given name as "Guidelines for Preliminary Selection of the Optimum Interchange Type for a Specific Location" to search the operational and safety characteristics for the optimal ramp design. The newest instruction is the ITE "Freeway and Interchange Geometric Design Handbook" edited by Joel in 2006. The handbook focuses on geometric and operational characteristics of freeways and interchanges. The book recognized that geometric design procedures for freeways and interchanges may vary. It also provides the evidence that is valued as an accompaniment of the AASHTO Green Book , the Highway Capacity Manual (HCM) , and Traffic engineering Handbook 5th Edition.

In 1998, Bared et al. developed a generalized regression model known as Poisson Model to estimate the crash frequency for the deceleration lanes plus the entire ramps as a function of ramp AADT, mainline freeway AADT, deceleration lane length and ramp configurations. The ramp configurations considered in that study include diamond, parclo loop, free-flow loop, and outer connecter. The model showed that the crash frequency on freeway ramps increased with the ramp and freeway AADT and decreased with the increase of the deceleration lane length. A 100 ft increase in deceleration lane length will result in a 4.8% reduction in crash frequency. The coefficients of the model also indicated that off-ramps suffered from more crashes as comparing to on-ramps. However, this study did not consider the number of lanes using for traffic leaving freeways. This problem is essential in the driving behavior because the balanced lanes and unbalanced lanes require drivers to take different operating manners.

Later, Bauer and Harwood built up several regression models to determine the relationships between traffic accidents, highway geometric design elements and traffic volumes. The statistical modeling approaches used in the research included Poisson and Negative Binomial regressions. It was found that the ramp AADT explained most of the variability in the crash data report at selected sites. Other variables found to be significant in crash prediction models contained freeway AADT, area type (rural, urban), ramp type (on, off), ramp configurations, and lengths of ramp and speed-change lane (deceleration lanes, acceleration lanes). Other models have been built to find out the functions of different variables in different kind of models. The independent variables are crash frequencies on the speed –change lanes, entire ramp sections, the selected ramp sections, and speed change sections plus the entire ramp sections. The best fit model was the one that combined crash frequency for the entire ramp, together with its adjacent speed-change lanes. The significant influential factors included area type, ramp type, ramp configurations (diamond, loop, outer connector, others), length of speed-change lanes, and length of the entire ramps. Another main finding is that models for the total crashes achieved much better than those for the only fatal and injury crashes. The models combined the on ramps and off ramps, and acceleration lanes and decelerations lanes. Off ramps usually occur more crashes than on ramps as mentioned before; the requirements for the length, curve, and design guidelines of acceleration length and deceleration lanes vary; ramp configurations could not be the ramp types on the diverge areas. Without judging these factors, models would decrease the accuracy of the conclusions, narrow the applications of the results and could not disclose the real situations. But this study provided reasonable methods such as the regression models which have been proved strappingly employed in the safety studies.

One main program is called Highway Safety Improvement Program that can help states decrease the number of crashes and provide optimal ways for arranging, applying, and estimating safety plans. From side to side of the introduction, all correlated issues to improve highway safety are recognized, measured, implemented and evaluated highway planning, designs, constructions, maintenances, and operations. Moreover, past studies emphasized the safety evaluation based on previous mentioned methods such as regression models or statistical tests that have been proved as useful methods in the safety studies. Following paragraph listed the wide applications of these methods.

Sarhan et al. designed the approach to help achieving the optimum predictive models. The model related to the length of acceleration and deceleration lanes based on expected collision frequency. Joanne and Sayed undertook the study to quantify the relationship between the design consistencies on the roadway safety. The generalized linear regression approach is used for model development as a quantitative tool for evaluating the impact of design consistency on road safety. Garcia et al. analyzed different deceleration lengths as functions of exit trajectory types, speeds, and localization. Munoz and Daganzo predicted the queued length at a wave speed about 13 mph in congested traffic by KW model. This method is widely used in the safety evaluation of intersections as well as freeway sections. Maze et al. analyzed the TWSC expressway intersection for crash rates, crash severity rates and fatal crash rates by Poisson regression models. Keller et al. divided crashes by different types as angle, left-turn, head-on, rear-end and pedestrian/bicycle by linear regression models while speed limits were found to be significant. Bernhard et al. ranked the locations and the estimated benefits of improvement by assigning fatal, injury and PDO crashes. Hypothesis tests were conducted with normal distribution with high number of crashes and Poisson distribution with a low number of crashes. The statistical tests were usually employed to find crash-prone sites in identifying some sites as hazardous at some a particular level of confidence. In fact, the level of confidence is that 100% minus the Type I error. Type I error is the percentage that mistakes the safety sites for hazardous sites. Another Type II error is the percentage that mistakes the hazardous sites for safety sites. They concluded that the program would benefit to public traffic to make the possible efforts in order to improve the safety studies.

Other studies focus on revealing the geometric, traffic, or related influential values to the mainline sections separately. Rakha and Zhang modeled a total of 34 different weaving sections to estimate the traffic volume at weaving sections including merge and diverge areas at the appropriate boundaries on freeways. The paper demonstrated that the volume estimated by the model had a significant effect on drivers' behavior in the mainline weaving sections. Abdel-Aty et al. tested various speed limits to evaluate the safety improvement on a section of Interstate 4 in Orlando, FL. Real-time crash likelihood was calculated based on split models for predicting multi-vehicle crashes during high-speed and low-speed conditions. The improvement was proved in the case of rising medium-to-high-speed regimes on the freeway. The paper recommends that the speed limit changes upstream and downstream should be large in magnitude (15mph) and implemented within short distances (2miles) of the diverge locations. It makes obvious that speed limit have some specific effects on the collisions from the upstream to downstream of diverge areas on the freeways. Cassidy et al. noticed the problem that queuing from the segment's off-ramp spilling over and occupying its mandatory exit lane comes up frequently. The situation delayed the mainline vehicles as well and would increase weaving conflicts. Janson examined the relationship of ramp designs and truck accident rates in Washington State plus a comparison to limited data from Colorado and California. The paper grouped freeway truck accidents by ramp type, crash type, and four conflict areas of each diverge ramp. The crash data were compared for these groups on the basis of number of truck crashes per location and per truck-mile of travel. The conclusion is slight different from generally belief that a ramp with a lower accident rate per truck trip due to low truck volumes may still be a high-risk site. But these results could not represent the real conditions if applied to all the passenger cars. The higher crashes number might still be constant with high volume since truck volume is really low and have the specific feats itself.

One research study, concerning on the number of lanes used by traffic exiting freeways was conducted by Batenhors. The paper, Operational Analysis of Terminating Freeway Auxiliary Lanes with One-Lane and Two-Lane Exit Ramps A Case Study, used three simulation software packages, the Highway Capacity Software (HCS), TSIS-CORSIM and SimTraffic on the operational analysis of weaving area at twenty locations by the level-of-service. The range of traffic and geometric conditions among the twenty sites varied. The findings of the case study suggest that a one-lane exit ramp may afford the best traffic operations apart from weaving length. The experience gained from the case study is to give support to traffic engineers to design efficient freeway facilities and to help researchers understanding the operational effects of geometric design. Even though this study considered exit lane numbers on the freeway diverge areas, the better level-of- service could not necessarily stand for better safety performance, and these two might have opposite results in some cases.

Based on the studies mentioned before, the impacts of exit ramp types on the safety performance of freeway diverge areas have not been well studied or documented until recently. Several previous studies have evaluated the safety impacts of different ramp configurations such as diamond, loop, directional, outer connector, and other. However, these studies have not considered the lane balanced problems on the diverge areas to regulate the number of lanes that shall be used for traffic to exit freeways.

3.5.3 Exit Ramp Section

The entire exit ramp section is another concern in this study to provide a comprehensive evaluation of the safety performance on freeway exits. Ramps are all one-way roads with one or more legs at terminals to connecting secondary crossroads. Different involvements of design speeds, configurations, speed differences among freeway and ramp section, ramp lengths or the direct connection features determine different exit ramps which have dissimilar safety effects. Some studies have focused on exit ramp sections and prior conclusions were described below.

Lord and Bonneson calibrated predictive models for different ramp configurations at 44 selected sites. The ramp design configurations addressed in this study included diagonal ramps, non-free-flow loop ramps, free-flow loop ramps, and outer connection ramps. The non-free-flow (parclo flow loop) ramp experienced twice as many accidents as other types of ramps Bauer and Harwood as mentioned before modeled the Negative Binomial regression model on the entire ramp section as well and concluded that diamond ramp have slight less crash frequency comparing to other ramp types when other influential variables remain constant. At the same year, Khorashadi used another method known as ANOVA test to forecast the relationship among ramp configurations, geometry parameters and crash frequencies. This study found that the geometric elements had much weaker impacts than the ramp configurations. McCartt et al. examined 1,150 crashes occurring on heavily traveled urban interstate ramps in Northern Virginia. The three major common crash types, run-off-road, rear-end, and sideswipe, accounted for 95% of total crashes. The countermeasures mentioned in the study included increasing ramp design speed, increasing curve radii, installing surveillance systems such as detectors, cameras, and advanced message signs.

Abdel-Aty and Huang explored an origin-destination survey to customers on the central Florida's expressway system. The distance traveled to exit a ramp did not depend only on the spacing between ramps, but also on other factors, such as the trip purpose, vehicle occupancy, driver's income level, and E-Pass implementation when the vehicle was equipped with an electronic toll collection system. A main finding was that the guide signs beyond the expressway exits had an important impact not only on unfamiliar travelers but also on the experienced drivers. Though it was a little count-intuitive, the result shows different design features on diverge areas would have an effect on familiar drivers as well. Hunter et al. conducted field observations on speed relationships between ramps and freeways by videotaping. Notable conclusions were drawn that vehicle speeds on exit ramps were much higher than the post speed limit. Since the big difference between the ramp post speed limit and operating speed, some unfamiliar drivers might slow down the speed while some familiar drivers might enter the exit ramp at a high speed relative far above the limit speed. That might be a vital reason why rear-end crashes account a large percent of crashes in the ramp sections.

Some studies focused on the connections between different influential factors which could be the ramp volumes, configurations, crashes, curvatures, and so on. These studies included Newell's Delays Caused by A Queue at A Freeway Exit Ramp, Shaw and Mcshane's Optimal Ramp Control for Incident Response, and Hunter et al.'s Summary Report of Reevaluation of Ramp Design Speed Criteria. Newell clarified that the graphical solution is more clearly illustrating practical issues. Shaw and Mcshane attended to optimize some measurements on the crashes to minimize the crash disruption. Hunter et al.'s concluded that ramp design speed should larger than 50% of freeway speed. This conclusion accommodated to Hunter et al.'s result that operating speed on the exit ramp is higher than the post speed limit.

It is obvious that many studies defined ramp configurations as ramp types. The conclusions included that free-flow ramps have more crashes than others, increasing ramp volume might increase crashes, the post speed limit on the ramp has some impacts on both local/familiar drivers or unfamiliar drivers and the operating speed is usually much higher than the post speed. Even several useful results are made on the exit ramp sections, but few consider the following two issues in the safety effects, ramp terminal treatments and ramp lane changing named widening on the exit ramp segment. Widening in this study is defined as the number of lanes changing after the pointed nose or in the middle of the entire ramp. The definition of ramp terminal treatments in “Ramp Management and Control Handbook” is those can be implemented at ramp/arterial connections as to better manage traffic exiting the ramp facility. They normally solve the specific problems that occur at the ramps or arterials. Diverse terminal control strategies have the potentials to affect operations on the exit ramp and adjacent arterials. Ramp terminal treatments implemented at exit ramps could reduce queue spillback from the secondary roads, decrease the potential for collisions on the freeway at the back of the queue, and improve traffic flow and safety on or near ramp facilities. Typically four strategies are broadly employed, signal timing improvements, ramp channelization, geometric improvements, and signing or pavement markings improvements.

The advantages of using ramp terminal strategies are to better coordinate with ramp terminal signal timing, to offer sufficient storage space either for left turn or right turn vehicles and to accommodate consistently on both exit ramps and secondary crossroads. The method of signal timing adjustments aims to prevent queue spillback to the freeway facility beyond exit ramps. Ramp channelization can increase capacity, supply enough storage space or a separate lane adjacent to the broad-spectrum lane, and delineate separate traffic movements. Geometric improvements manage sight distances, horizontal and vertical curves, and any other geometric deficiencies. Signing and pavement marking improvements deal with guiding motorists of downward conditions and facilitating vehicle movements. Implementations of ramp terminal treatments reducing delay and queuing length, decreasing conflict points, enhancing safety and minimizing impact both on upstream and downstream highways and arterials. The functions vary by implemented treatments. Alternatively, negative impacts with different terminal treatments varied by the each site. Those might increase trip length, cause supplementary travel time, or extend queuing and signal delay. Accordingly, different terminal control designs or different combinations of terminal designs might have various powers on the safety aspects of entire ramp sections. Retting et al. endeavored to reduce urban crash rate by building potential countermeasures to the five most common crash types in fourteen cities. For the vast combinations of the crashes about (69%-81%) in each type via dissimilar cities, the author suggested that signal timing, sign visibility, sight distances would be the improvement measure to enhance safety in general solutions.

This study would consider the terminal control methods to expose the impacts of terminal control types on safety. One study conducted by Bared et al. comparing crashes between single point and tight diamond ramps related crashes on the cross road only. Single point diamond interchange is diamond ramp free-connects to the cross roads No triangle median occurs at the terminals. Tight diamond interchange differs to single point diamond interchange since there is a triangle median separation at the termination to split different traffic movements for left turns or right turns. Crash data were subtracted from 27 tight diamond sites and 13 single point sites in Washington to build a Negative Binomial model of total crashes on the exit ramp and cross-road flow. However, the safety comparison did not reveal a significant difference between the two types of interchanges for total crash. This study only compared one terminal treatment as ramp channelization; however the sites number here is not sufficient enough to do a regression model. The lanes widening is another issue as one of the strategies in the exit ramps. Several ramps from the field observations show that it will wide to two or more lanes after the pointed noses which separate the freeway mainline sections and ramp sections.

CHAPTER 4 METHODOLOGY

The principles for selecting the main methods concern on how the functions are, whether they are practical or easily applied to the data base, and what the potential results are. The research subjects included two parts defining as freeway diverge areas and entire ramp sections separately. After reviewing prior studies, guidelines, handbooks and related researches, useful methodologies and important parameters are identified for the safety analysis. The main approaches used included the cross-sectional comparison method, hypothesis tests, and generalized regression models.

4.1 Crash Frequency and Crash Rate

Crash frequencies or crash rates are two indicators that are generally used in the safety studies to compare different treatments or groups. This research project would calculate both of them for further analysis.

4.1.1 Crash Frequency

Crash frequency is the real number of crashes that have happened at a certain location or segment in a particular time or time interval. It is commonly used for several benefits. Firstly, the crash data are easy to get and simple to calculate. Next, the meaning behind is straightforward so that governmental officials, engineers, and public could understand it readily. The third virtue is that it could represent diverse selected places in one parameter and could change directly while the selected lengths or vicinity of the segments changed. The resource of the noticed crashes is only from police long form crash report which describes specific features for each crash. Florida Traffic Crash Analysis Report (CAR System) provides detailed crashes and updates the database each year.

The mathematics mean value of crash frequency is labeled as the average number of crashes. With different groups or managements, the average number of crashes was calculated based on the number of sample sites. In statistical assumption, the mean value normally is the most proficient estimator for the population groups. The following equation defines the average crash number with a specific group, C, as:

$$C = \frac{\sum_{i=1}^{n} c_i}{N} \tag{1}$$

Where,

C =average number of crashes for the sites with a particular group;

c_i = number of crashes at site i in the group; and

N = total number of sites within the group.

For the diverge areas, four exit ramp types are classified so that four groups were chosen to compare the mean values of crash frequency. Besides, three additional values stand for the accuracy and variations of the mean values. The median value is the middle rate in a series of data that have been ranked in order to scale and part the sites into two identical fractions. The maximal and minimum values are the largest and smallest crash number in a specific group. The four additional variables imply the variation of the each sample and the mean values. If the median value is much larger or smaller than mean value, the distribution curves of crash number indicate biasness in the judgment. In order to get reasonable mean value, usually the four values, mean, median, maximum, and minimal are calculated respectively to represent the distributions of the number of crashes.

4.1.2 Crash Rate

In this study, crash rate is defined as crashes per million vehicles per vehicle miles traveled for a specific section. Crash rates are used as a criterion for more truthful for segments under the same geometric and traffic conditions to narrow the impacts of these important factors. The crash rate, r, for a particular freeway segment can be calculated in the following formula:

$$r = \frac{1{,}000{,}000 \times A}{365 \times T \times V \times L} \qquad (2)$$

Where,

r = crash rate at a freeway segment (crashes per million vehicles per mile)

A= number of report crashes (crashes per year)

T= number of years

V= average daily traffic volume (vehicles per day)

L= length of the freeway segment (miles)

It is believed that the crash frequency tends to increase as the average daily traffic (ADT) goes up even through many other factors affecting the situation. In this study, the corresponding ADT for each site was obtained from annual Florida traffic information CDs. The time frame is determined for the database in continuous years when site characters have not been changed in the period. The average crash rates, which are the arithmetic means of crash rates, were calculated for the four groups in the freeway diverge areas. The statistical assumption is similar to the average number of crash as mentioned before. The average crash rate, R, is defined as:

$$R = \frac{\sum_{i=1}^{n} r_i}{N} \tag{3}$$

Where,

R =average number of crashes rates with a particular group

r_i = number of crashes rates at segment i in the group

N = total number of sites within the group

The median, maximal, and minimal values are measured as well to observe the distributions of crash rates.

4.2 Crash Type and Crash Severity

Since the objectives are to estimate the safety impacts among 4 exit ramps on diverge area and along the entire exit ramp sections, the total number of crash, crash severity, and crash types having the highest percentages to the total crashes were chosen for each group. Crash severity that is widely used in the safety analysis can be classified to two categories: PDO (Property-damage-only) and injury/fatal crashes.

4.2.1 Crash Type

In the crash database maintained by FDOT, crash type is defined by the first harmful event of at-fault vehicles. The comparison of crash types will help to identify driver behaviors that are related with the types of exit ramps. A total number of 40 crash types are concluded in the Florida's CAR system. The most three highest crash types occur on diverge areas are rear-end crash, side-swipe crash and angle crashes. Rear-end crash and side-swipe crash counted for about 60% of total crashes, 46% rear-end crashes and 16% side-swipe crashes. The target crash types on the exit ramp sections are rear-end crash, angle crash and side-swipe crash as well.

Rear-end crashes which regularly take place while the first vehicle stopped or suddenly slowed down and the following vehicle had a collision with the first vehicle in the rear piece of the vehicle. The severity of these crashes can range from minor to severe depending on the speed of the following vehicle that hits the first vehicle.

Sideswipe crash is another common crash type in this study and usually happens when changing lanes, misdirection of exiting freeway, or vehicle weaving. The severity of this type is also ranged from minor to severe.

The one vehicle crossing the passageway or changing directions in the road might conflict with another vehicle. They are frequently set as angle crashes. Angle crashes are also commonly noticed on the misdirected vehicles. The severity of the crashes usually causes severe crashes than rear-end crashes. Comparing to other types, the three types mentioned above is the most concerned types in this diverge area and exit ramp sections.

4.2.2 Crash Severity

Usually, crash severity level is recorded for each police reported crash. Three major levels of crash severity generally defined in the study can be classified to three categories:

1) Property-damage-only (PDO) crashes

2) Injury crashes

3) Fatal crashes

In a property-damage-only crash, only properties are damaged but no person is hurt; in an injury crash, at least one person is lightly hurt because of the crash; in a fatality crash, at least one person is dead within 90 days after the crash which was the most concerned problems in many other studies and this study as well.

4.3 Cross-Sectional Comparison Approach

The cross-sectional comparison analysis is satisfactory to provide adequate and reasonable consequences. It is long believed that cross-sectional approach is a logical and efficient technique of judging the safety effects. The cross-sectional method has been proved valuable and has been performed on a number of prior studies that involved median alternatives, right turns followed by u-turn to direct left turns and truck accidents at freeway ramps. In transportation fields, traffic engineers have experimental judgments as long as the most influential factors such as section length, average daily traffic (ADT), speed, ramp length are well controlled. Cross-sectional analyses to evaluate different treatments are fairly reliable for the results. Briefly, reliable conclusions could be got within this measurement. In other words, this method compares the safety of two different groups of sites with and without the treatment under investigation. It is necessary to select similar geometric conditions in order to get the reliable results in comparing site histories of different types.

In this study, cross-sectional comparison was conducted to measure freeway diverge areas with different types of exit ramps and exit ramp sections with four configurations. This approach involves comparing crash frequency, crash rate, crash type, and crash severity of a group with a treatment, to that of a group of with other treated sites. As mentioned before, the selected freeway segments were divided into four groups based on the types of the exit ramps. On the basis of the collected crash data, statistical analysis was conducted to quantitatively evaluate the safety impacts of different types of freeway exit ramps.

The major assumption behind this comparison was that all other characteristics in the sites remained the same during the study period. The significant geometric and control factors considered in this study included deceleration length, ramp length, average daily traffic(ADT), posted speed limit, number of lanes in the freeway, surface conditions, shoulder conditions and so on. By comparing crash through statistical testing, conclusions could be reached regarding the relative safe treatment among different treatments.

4.4 The Hypotheses Test

Hypothesis tests are utilized to test whether the observed differences of the selected variables such as mean values, variance values, or proportion values between two or more groups have significantly variation in a statistical term. Assumptions of observing the sample data were calculated in the hypothesis testing to measure the suppositions whether they have under similar features. If the results did not support the assumptions, then the assumed suppositions are considered doubtful. The formula of hypothesis testing includes two competing statistical hypotheses: a null hypothesis (H_0) and an alternative hypothesis (H_a). The null hypothesis is a postulation that one parameter of a population is true under sufficient statistical terms. The contrast postulation of the null hypothesis is an alternative hypothesis. It is assumed that all the other situations that did not covered by the situations under null hypothesis.

The test result is to reject or fail to reject the null hypothesis under the specific conditions based on the statistical distributions while they reply upon Z, t, F or χ^2 distribution. The decision of whether rejecting the null hypothesis is based on the statistic value range on the statistical distribution mentioned before at a statistical term named as the significant level α. Typically the level of confidence as 1- α is applied to determine the statistical confidence instead of α. The procedures of conducting a hypothesis test including four steps:

Step 1: Select Null Hypothesis- H_0, Select an Alternative Hypothesis - H_a;

Step 2: Determine the level of confidence (1- α)*100%;

Step 3: Calculate the statistical value;

Step 4: Compare the statistical value to the critical value on the distribution, and decide to reject or fail to reject the null hypothesis H_0.

The following two parts describe the detailed procedures to conduct hypothesis tests on the equality of two means and the proportionality analysis.

4.4.1 Hypotheses on the Equality of Two Means

Mean values of two different populations were tested to get reasonable conclusions whether to reject or not reject the null hypothesis. The average crash numbers and rash rates from one group to another group were examined if they are significantly different. Assumed that two populations say X_1 and X_2, where X_1 has an unknown mean μ_1 and known variance σ_1^2 and X_2 has an unknown mean μ_2 and known variance σ_2^2. The purpose is to test whether the two populations have the same mean μ_1 and μ_2. The first step is to build the null hypothesis H_0 and an alternative hypothesis H_a:

$$H_0 : \mu_1 = \mu_2 \tag{4}$$

$$H_a : \mu_1 \neq \mu_2 \tag{5}$$

The procedure is based on the fact that the difference in the sample mean, X1, X2, of two populations of interest with a sample size of n_1 and a sample size of n_2 separately, $\bar{X}_1 - \bar{X}_2$ will fit the normal distribution of:

$$\bar{X}_1 - \bar{X}_2 \sim N(\mu_1 - \mu_2, \sigma_1^2/n_1 + \sigma_2^2/n_2) \tag{6}$$

The second step is to choose the level of confidence. In this study 90% is used and α equals 10%. The third step is to calculate the statistical value Z_0 (n≥25) or t_0 (n<25):

$$Z_0 = \frac{\overline{X}_1 - \overline{X}_2}{\sqrt{\frac{\sigma_1^2}{n_1} + \frac{\sigma_2^2}{n_2}}} \tag{7}$$

$$t_0 = \frac{\overline{X}_1 - \overline{X}_2}{\sqrt{s_p^2 (\frac{1}{n_1} + \frac{1}{n_2})}} \tag{8}$$

The final step is to compare the calculated value with the critical value $Z_{\alpha/2}$ or $t_{\partial/2}$. The null hypothesis could be rejected if:

$$Z_0 > Z_{\alpha/2} \text{ or } Z_0 < Z_{\alpha/2} \tag{9}$$

$$t_0 > t_{\partial/2} \text{ or } t_0 < t_{\partial/2} \tag{10}$$

If the variance σ^2, is unknown, it can be replaced by the square of the standard deviation of the sample size n which is S^2 as following:

$$S^2 \cong \frac{\sum_1^n (X_i - \overline{X})}{n} \tag{11}$$

If the sample sizes is less or equal to 25, the populations are approximately t distribution with a pooled variance, s_p^2, based on sample variance s_1^2 and s_2^2. The formula is given by:

$$S_p^2 \equiv \frac{(n_1-1)s_1^2+(n_2-1)s_2^2}{n_1+n_2-2} \tag{12}$$

4.4.2 Hypotheses Tests on the Proportionality Analysis

On the basis of the collected crash data, statistical analysis was conducted to quantitatively evaluate the crash type and crash severity on the safety effects. The proportionality hypothesis test was utilized in this study to comparing target crash types and crash severity between different freeways diverge sections.

Proportionality test is often used to test the significance of the percentages between two populations or samples. Let p_1 and p_2 be the proportions of a particular type of crashes in two different groups. Assuming that the total crash counts in these two groups are *m* and *n* respectively, for testing the null hypothesis:

$$H_0: p_1 = p_2 \tag{13}$$

Versus

$$H_1: p_1 \neq p_2, \tag{14}$$

H_0 can be rejected if:

$$Z = \frac{|p_2 - p_1|}{\sqrt{\frac{p_2(1-p_2)}{m} + \frac{p_1(1-p_1)}{n}}} \geq Z_{\alpha/2} \tag{15}$$

4.5 Statistical Predictive Model

Crash prediction models were developed for this study at selected freeway segments and entire ramp sections respectively. The purpose to use regression predictive models is to identify the factors that contribute to the crashes and quantify the effects on crashes at selected sites. This research project would draw on the generalized linear regression models to mold crash number.

Generalized linear models have been widely used for modeling crashes at safety studies (1, 3, 11, 19, 25, 26, 27, 28, and 31) at intersections, roadways or freeways. Generalized linear models are the expansion forms of the classical linear regression models. The classical linear regression model assumes that the dependent variable is continuous and normally distributed with a constant variance. The assumption is not appropriate for crash data which are approximately Poisson distributed and are generally non-negative, random and discrete in nature. Numerous previous studies have suggested the use of Poisson models or Negative-Binomial (NB) Models for modeling crash data (1, 3). The Poisson model assumes that the dependent variable is Poisson distributed. Using a Poisson model, the probability that a particular freeway segment i or an exit ramp section experiences y_i crashes during a fixed time period is given by:

$$p(Y_i = y_i) = p(y_i) = \frac{\mu_i^{y_i} e^{-\mu}}{y_i!} \text{ , i =1, 2, 3,……, n} \tag{16}$$

Where,

μ_i = the expected number of crashes for segment i;

y_i = the probability that a particular segment i.

A logarithm link function connects μ to a linear predictor η. The link function and the linear predictor determine the functional forms of the crash prediction model. If the linear predictor is a linear function of the explanatory variables, the fitted crash prediction model takes the functional form as below:

$$\mu_i = \exp(\beta_0 + \beta_1 x_{i1} + \beta_2 x_{i2} + ... + \beta_k x_{ik}) \tag{17}$$

Where,

$\beta_0, \beta_1, \ldots, \beta_k$ = coefficients of explanatory variables;

$x_{i1}, x_{i2}, \ldots, x_{ik}$ = explanatory variables.

If the linear predictor is a linear function of the logarithm of the explanatory variables, the functional form is given below:

$$\mu_i = \beta_0 x_{i1}^{\beta_1} x_{i2}^{\beta_2} ... x_{ik}^{\beta_k} \tag{18}$$

The Poisson model assumes that the mean of the crash counts equals the variance. The assumption is usually too stringent considering the fact that the variance is often greater than the mean. In this condition, over dispersion will be observed and the estimated coefficients of the Poisson model are biased. An alternative to deal with the over dispersed data is to use the negative binomial model. The negative binomial model assumes that the crash counts are Poisson-gamma distributed. The probability density function of Poisson-gamma structure is given by:

$$p(Y_i = y_i) = \frac{\Gamma(y_i + a^{-1})}{y!\Gamma(a^{-1})}\left(\frac{a\mu_i}{1+a\mu_i}\right)^{y_i}\left(\frac{1}{1+a\mu_i}\right)^{a^{-1}}, i = 1, 2, 3\ldots n \tag{19}$$

Where,

y_i = the crash count at segment i

μ_i = the expected number of crashes for segment i

α = the dispersion parameter

The dispersion parameter determines the variance of the Poisson-gamma distribution. Usually α can be estimated either by the Moment Method or by the Maximum Likelihood Method.

Two parameters are often used for evaluating the goodness-of-fit of a generalized linear model. These two parameters are the scaled deviance (SD) and the Pearson's χ^2 statistic. For an adequate model, the two statistics should be chi-square distributed with (N-p) degrees of freedom, where N is the number of observations and p is the number of parameters in the model. The scaled deviance equals twice the difference between the log-likelihood under the maximum model and the log-likelihood under the reduced model. The scaled deviance can be calculated as:

$$SD = -2(\log(L_\beta) - \log(L_s)) \tag{20}$$

Where,

L_s = the likelihood under the maximum model

L_β = the likelihood under the reduced model

The Pearson's χ^2 statistic can be calculated as:

$$Pearson's\chi^2 = \sum_{i=1}^{n}\left(\frac{y_i - \mu_i}{\sigma_i}\right)^2 \tag{21}$$

Where,

y_i = the crash count at segment i

μ_i = the expected number of crashes for segment i

σ_i = the estimation error for segment i

It is usually assumed that the crash data are approximately normally distributed. Thus, the scaled deviance SD and Pearson's χ^2 statistic for an adequate model should be approximately chi-square distributed with (*N*-*p*) degrees of freedom, where *N* is the number of observations and *p* is the number of parameters in the model.

CHPATER 5 DATA COLLECTION

This chapter focuses on illustrating the data collection procedures that include the selected sites and relative sites information. Both freeway diverge areas and entire exit ramp sections are reviewed and the criteria for classifying the site segments and segment lengths are explained. Detailed methods of identifying road sections in FDOT's system, subtracting specific site database, and tackling with the crash data for each site were depicted in this chapter as well.

5.1 Site Selection Criteria

The study focuses on the safety effects of the freeway diverge areas and entire exit ramp sections. In order to obtain reasonable results, criteria to identify the site segments are really important in order to narrow the unstable and unrelated factors. The criteria were listed below for both freeway diverge areas and freeway exit ramp sections:

1) All the objects are on the freeway diverge areas or exit ramps;

2) Freeways defined here are the highway segments with the highest level of service and full control of accesses;

3) Only right exit ramps are considered in the sites which means all exits should be at the right hand of the directions on freeways;

4) The impacts of left exit ramps are not incorporated in this study as they have significant different features to right exits;

5) A sufficient and significant curb, bar, or other facilities in the median separates two directions;

6) The right-shoulder of freeways and exit ramps should be clear, no sight obstruction, and no dangerous facilities;

7) The grade variations are smallest so that no grade varieties are considered in both sections;

8) The freeway segments should be homogeneous segments without large horizontal or vertical curves distinctions since this research would narrow the other parameters that not compared;

9) All sites are in Florida States from District one to District seven plus an additional Florida Turnpikes generally named as District eight.

Two dissimilar sections are selected so that they both have special requirements for the segments. The following items list the special site requirements at the freeway diverge areas:

10) The minimal posted speed limit on the freeway mainline section should be larger than 50 mph;

11) The upstream and downstream distances from the deceleration lanes are long enough so that influential factors up or down from the deceleration lanes are minimal;

12) Deceleration lanes are calculated from the beginning of the taper or widening points to the painted nose;

13) Four different ramp types on the diverge areas have different number of lanes at freeways, but the research segments remain same.

The exit ramp sections that connect the diverge areas and continue until the beginning of secondary roads should meet subsequent extra criteria:

14) The exit amp lengths begin from the painted nose and end at the last part of terminals;

15) All exit ramp suggested or posted speed limits is larger is 25 than mph no matter the ramp configurations or ramp length.

Following these criteria ensures that the candidate list of field study sites could be obtained without low speed limits in the freeway diverge areas and large difference of speed limits on entire ramp sections. This would make the same characters except the concentration variables to do the statistical analysis. The lane width is an interesting parameter in this study so that the lane widths are not necessarily synchronized in the sites selection procedures. From the field studies, all the preferred segments would go for the interstate highway systems, expressways, turnpikes, and parkways in Florida.

5.2 Segment Length Definition

Two research sections are defined in this section, the freeway diverge areas and the entire exit ramp sections. The segment length of diverge areas include the deceleration areas and the adjacent vicinities that have related effects for traffic exiting or continuing on freeways. The decision is based on both previous studies and site observation experiences. The exit ramp length includes the entire ramp sections no matter the ramp configurations, ramp terminal control types or other factors. No more regions are taken into concerns as the ramp sections are continuous to the diverge areas.

5.2.1 Freeway Diverge Area Length

The freeway diverge segment in this study is a section of freeway which contains a deceleration lane and its adjacent section. The segment length for the freeway diverge area consists of two continuous sections, including (1) a 1500 ft section located in the upstream of the painted nose and (2) a 1000 ft section located in the downstream of the pained nose. Thus, the length of the freeway diverge segment in this study equals 2500 ft for each site. The definition of the freeway diverge segment for each type of exit ramp is also given in Figure 5.1 through Figure 5.4 (Compared to Fig 3.1 to 3.4 in chapter 3, Fig 5.1 to 5.4 specify area length and drop/optional line for each exit ramp type). They illustrate the whole study section that combines the declaration areas and their surrounding areas.

Using different influential distances in the upstream of painted nose could result in different safety analysis results. If the selected distance is too long, crashes reported for selected freeway segments may include some mainline crashes which are not directly related to exit ramps. If the selected distance is too short, however, the selected freeway segment is not long enough to cover the entire influential area of exit ramps. In previous studies, the selected influential distance located upstream of the painted nose ranged from 1000 ft to 2000 ft. The HCM suggests 1500 ft beyond the painted nose in the simulation software including TSIS-CORSIM and Highway Capacity Software (HCS). In addition, the length of deceleration lane at selected diverge sites varies from 26 ft to 918 ft. Field observations show that, when the distance to painted nose is greater than 1500 ft, the exit ramp type does not impact behaviors of mainline drivers in an obvious way. Due to these reasons, a 1500 ft section was selected as the influential area located upstream of pained nose and 1000 ft downstream the painted nose on the freeway mainline sections.

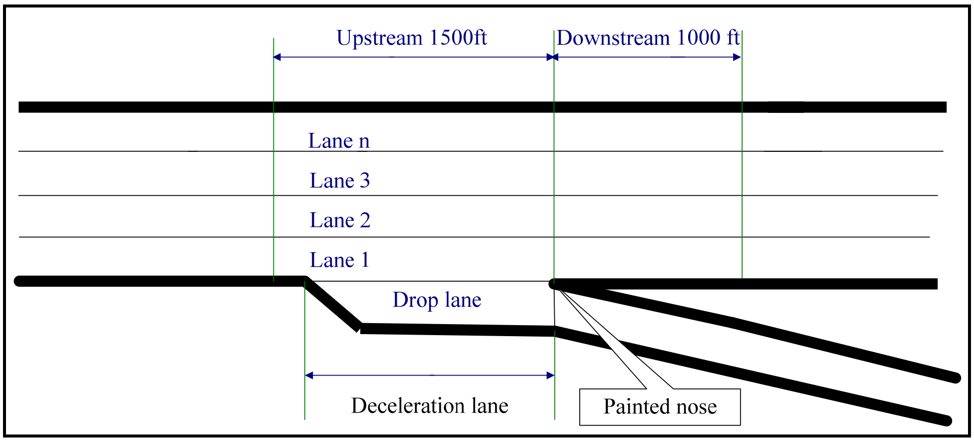


Figure 5.1 Type 1 Exit Ramp Length: Parallel from a Tangent Single-lane Exit Ramp

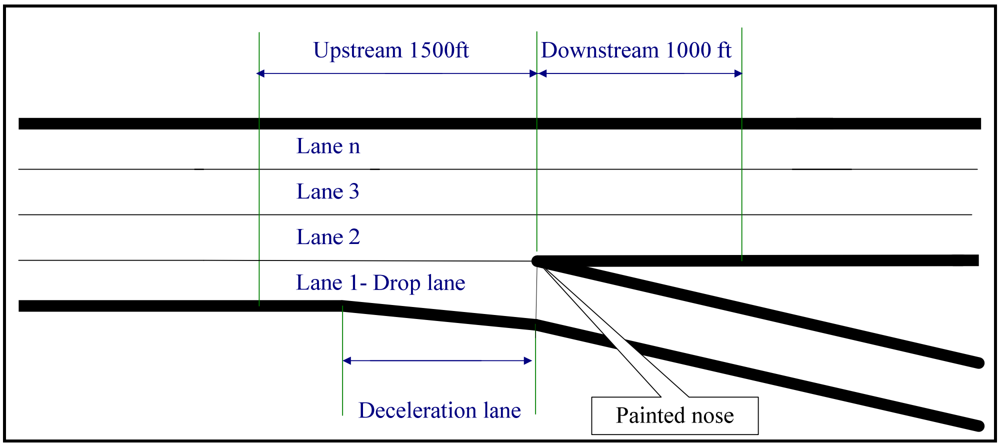


Figure 5.2 Type 2 Exit Ramp Length: Single-lane Exit Ramp without a Taper

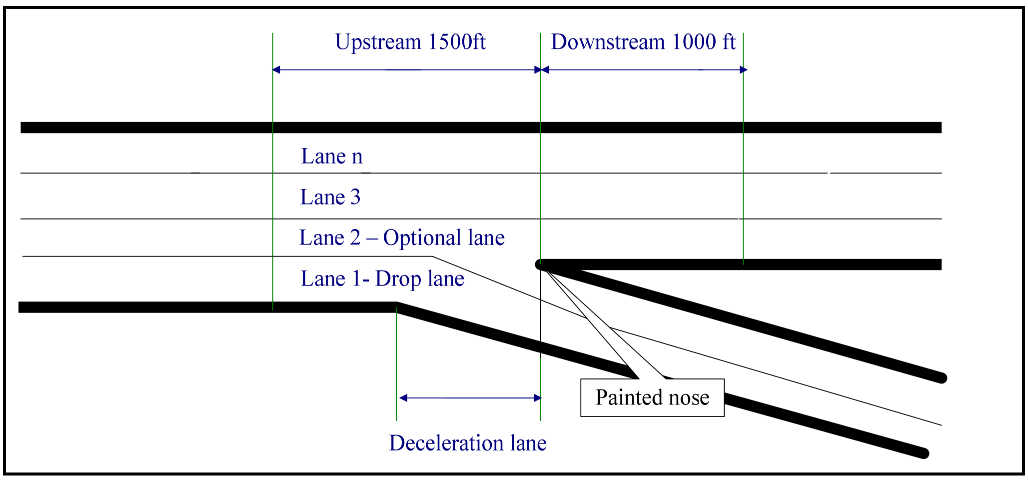


Figure 5.3 Type 3 Exit Ramp: Two-lane Exit Ramp with an Optional Lane

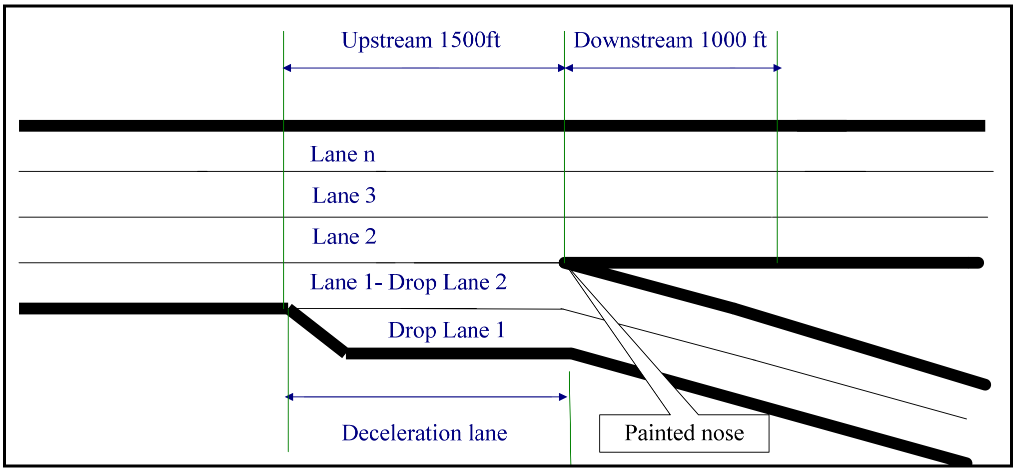


Figure 5.4 Type 4 Exit Ramp: Two-lane Exit Ramp without an Optional Lane

5.2.2 Exit Ramp Section Length

The crash frequency is related to the segment length since different distances might have different effects on the number of crashes when other situations are equal. Usually, longer distances might have more crash potentials than shorter distances. Resende and Benekohal did a comprehensive study on the influence of segment lengths and the geometric variables on crash rates. The paper proved the essences of different segment lengths.

The entire ramp section is the length of the exit ramp itself. The definition means that the painted nose is the beginning of exit ramp and the end of terminals is the closing stages for the exit ramp. It varies slightly from past studies conducted by Lord and Bonneson, Bauer and Harwood, Khorashadi, McCart et al., and Janson et al.. Some studies excluded the terminal sections from the entire exit ramps. However, different termination styles would influence the beyond sections as well as the adjacent sections. Some adjusted the exit ramp sections plus the upstream deceleration lanes. This study would separate these two continuous sections because the diverge areas and ramp sections have dissimilar crash features and prominent influential factors. The mixed of these two might get incorrect results. Even Bauer and Harwood did consider the entire ramp sections, they ruled out the all the rear-end crashes for the ramps. It might misrepresent the crash distribution and lead to misunderstand of the other factors to the rear-end crashes which are generally highly occurred in the exit ramps. As a result, the clarity of ramp length here uses the definition described before. The following Figure 5.5 from A to D present the ramp segment lengths for four ramp configurations as mentioned above.

Figure 5.5-A. Diamond Exit Ramp Segment Length

Figure 5.5-B. Out Connection Exit Ramp Length

Figure 5.5-C. Free-flow Loop Exit Ramp Segment Length

Figure 5.5-D. Parclo Loop Exit Ramp Segment Length

From the four figures, four bold lines added to each one illustrate the study field for exit ramp sections. Even they have special design patterns as they appear, the principles are unique. This is intended to obtain useful results and raise the accuracy of the analysis.

5.3 Selected Sites Information

In this study, crash data were collected at research segments in the State of Florida. After checking the available sites, the site resources are limited. In this reason, all the freeways are examined in order to get reasonable sample sites. Following the sites criteria before, a total of 12 Interstate Highways, 10 expressways, 1 turnpike and 1 parkway are overviewed and sites are collected on these freeways. These freeways provide high service level with high design standards. Figure 5.6 below lists the most important four interstate highways. Interstate Highway 75 (I-75) and Interstate Highway 95 (I-95) are both north-south directions while Interstate 4 (I-4) and Interstate Highway 10 (I-10) are east-west directions. Other highways connect intra-region or inter-regions as to provide better traffic operations at limited accesses.

Florida divided eight districts for the whole state, from District One to District Eight. District One through District Seven have their local offices to manage each district respectively. District eight is the toll roads that are built, managed and maintained by all Florida areas. FIGURE 5.7, the District Map, gives an idea about the seven districts allocation in the Florida. The figure is original from FDOT Community Traffic Safety Teams (CTST). These selected freeways are dispensed in all the eight districts and Table 5.1 lists the detailed information of each district.

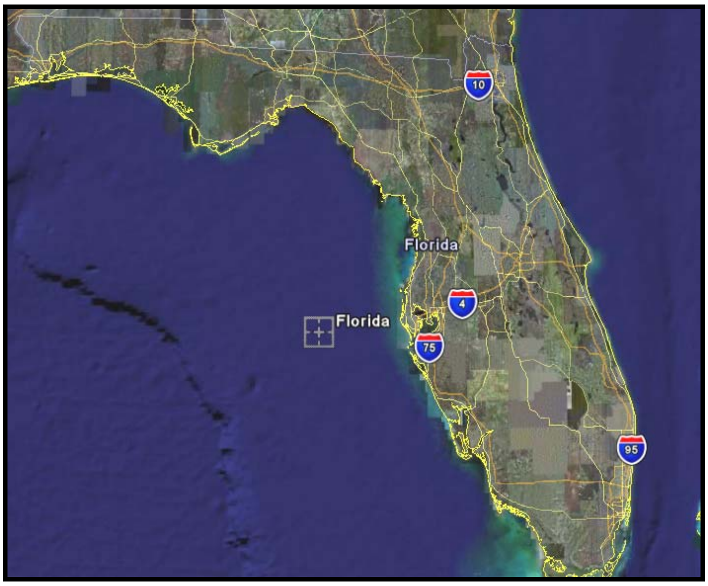


Figure 5.6 Florida Interstate Highway System

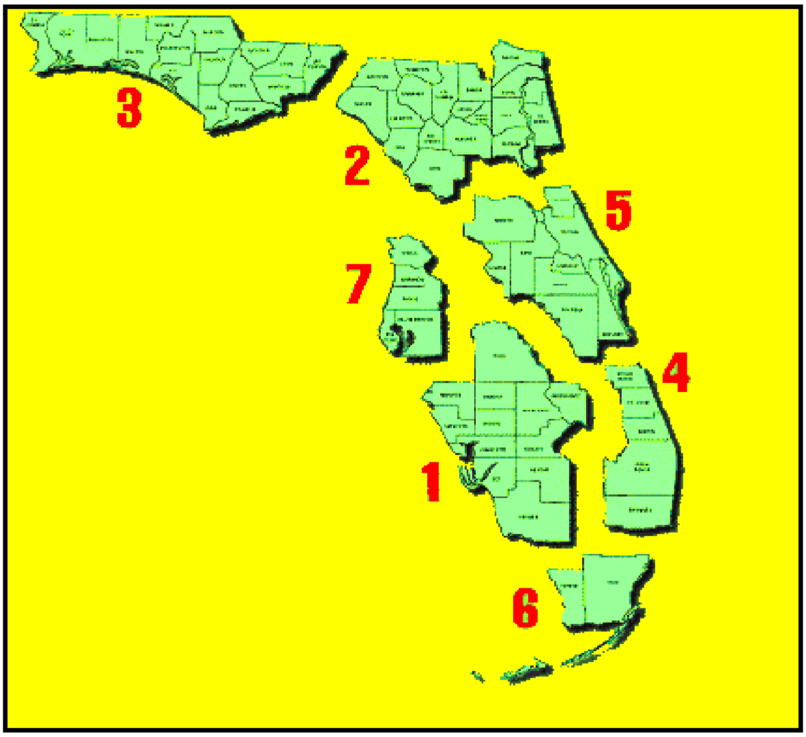


Figure 5.7 Florida District Map

Table 5.1 FDOT Districts Distributions for Selected Sample Sites

District Number	Freeways
One	I-75, I-4;
Two	I-295, I-10, I-75, I-95;
Three	I-10, I-110;
Four	I-595, I-75, I-95;
Five	I-4, I-75, I-95, Bee Line Exp, East-West Expressway, Central Florida Greenway Expressway;
Six	I-395, I-75, I-95, I-195, Dolphin Expressway, 826 State Highway, Palmetto Expressway, Florida Turnpike, Don Shula Expressway;
Seven	I-375, I-75, I-275, I-175,I-4, Veterans Expressway, S Crosstown Expressway, N Memorial Expressway;
Eight	Florida Turnpike, Polk Parkway;

5.3.1 Freeway Diverge Areas

The task of site collection is the most time-consuming and tedious work in this study. Hundreds of sites are available and each site needs to check patiently and review carefully to make sure all the related data are correct. Area photos for each site were pulled together. However, some sites are under reconstructions or have been closed for some time during the study period. Some sites did not have detailed site information such as AADT, especially at some expressways. Since some sites did not have full information, they did not meet the sites requirements as mentioned before. These sites might be large curvatures, low post speed limit as 45 mph, grade variation much higher than the expect one and so on. After reviewing the area photos for freeway diverge areas in the State of Florida. 424 sites were selected for the freeway diverge segments. Among these sites, 220 sites are Type 1 exit ramps-parallel from a tangent single-lane exit; 96 sites are Type 2 exit ramps-single lane exit ramp without a taper; 77 sites are Type 3 exit ramps-two lane exit ramp with an optional lane; and 31 sites are Type 4 exit ramps-two lane exit ramp without an optional lane. Table 5.2 lists the site resources for each type.

Table 5.2 Sites Resource Distributions for Freeway Diverge Areas

Exit Ramp Type	Total Size	Resource			
		Interstate Highways	Expressways	Turnpikes	Parkways
1	220	220	0	0	0
2	96	96	0	0	0
3	77	59	16	2	0
4	31	17	11	2	1

5.3.2 Exit Ramp Segments

The work of sites gathering on the ramp sections is labor intensive as well. Since the exit ramp sections are sequential to the diverge areas, the sample size basically equals to freeway diverge sites with available data. However several sites did not have ramp ADT because there are no detectors there. These sites are excluded from the exit ramp sites. So a total of 389 sites are determined as the sample size for the entire exit ramp segments.

5.4 Site Selection Procedures

The processes of site selection can be explained in three steps, field study, site information collection, and site review. Field study is the first step to collect raw data as geometric data, site notification data and other related factors. Based on these data, the sites ID could be obtained from Florida road identification systems: Straight-Line Diagram (SLD) and Florida Traffic Information CDs. Finally, all the selected sites are checked again to acquire available sites.

5.4.1 Site Selection Procedure 1

Step 1 - Field Study: Field study collects site location and geometric conditions which match the requirements and criteria. The photograph maps were obtained from each district traffic information CD. For each site, simple sketches with geometric information were checked to find the following information:

1) Major freeway names
2) Freeway directions
3) Ramp types
4) Deceleration lane lengths
5) Number of lanes in freeways
6) Posted Speed Limits on freeways
7) Upstream 1500 ft distances measurements from the painted nose
8) Downstream 1000 ft distances measurements from the painted nose
9) Exit ramp directions
10) Ramp lengths
11) Number of lanes in the ramp
12) Ramp suggested or post speed limit
13) Number of lanes changing on the ramp sections
14) Ramp terminal control types

15) Secondary road name

16) Distances from the first upstream intersection on the secondary road

17) Distances from the first downstream intersection on the secondary road

18) Number of lanes on the secondary roads

5.4.2 Site Selection Procedure 2

Step 2 - Extracting Road ID: SLD and Florida Traffic Information (FTI) annual CDs were obtained from corresponding FDOT district offices. The road mileposts and road identification numbers for each site were gathered from SLD and ADT each year were subtracted from traffic information CDs. These kinds of information are listed below:

19) Section and subsection number of the freeways

20) Section and subsection number of exit ramp sections

21) Milepost on the beginning and end of the segment length for diverge areas

22) Milepost on the beginning and end of the segment length for exit ramps

23) Site number for freeways

24) Site number for exit ramps

5.4.3 Site Selection Procedure 3

Step 3 - Site Review: Each site and the related information were checked again to prove that all the data are correct and confirm that no significant reconstruction had taken place at the selected study sites during the study period.

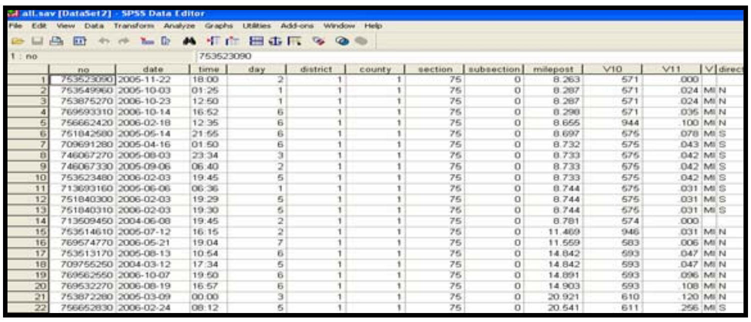
5.5 Section Number, Milepost and Site Identification Number

The section number and milepost for each selected freeway segment was obtained from the SLD provided by the Florida Department of Transportation. The purpose of using section numbers and mileposts is to consist with FDOT crash database. Each section number contains eight digital codes which were used to identify one specific road. The first two digital codes are the county number for each district. The subsequent three digital numbers are section numbers and the last three digits are the subsection numbers. While looking for a location in a site, section number is not enough. The milepost was additional information to recognize the position on the roadway segment. Mileposts are made from the beginning of a road way from south to north or from west to east. For example, I-75 in Hillsborough County (section number 10-075-000) begins at the Manatee/Hillsborough county line as milepost 0.000 and ends as milepost 36.25 at Pasco/Hillsborough County.

Site ID is another index in the annual FTI CDs which contained several essential parameters including AADT, peak hour factor, and other volume related data. Six numbers are combined. The first two are the county number and the rest four digits are the sites recognized ID. The site ID for I-75 at Bruce B. Down's exits is 10-0153. The AADT for this section could be obtained from AADT annual report through site ID.

5.6 Crash Database

Based on the range in mileposts of each segment, crash data reported was obtained from the crash database maintained by the State of Florida. In 2003, the FDOT renamed all the freeways exit ramps for the whole state. Accordingly, the crash database updated the exit ramp numbers for the entire database. Due to this reason, crash data for freeway exit ramps before 2004 include a lot of missing information and, as a result, cannot be used in this study. A three-year time frame, from 2004 through 2006, was selected to obtain crash data. Eighty-six variables are enclosed in the FDOT crash database including site identification, time of crashes, traffic conditions, geometric conditions, crash detailed information as location, direction, crash type, severity and so on. The software SPSS would be used to examine the crash data. Figure 5.8 shows the SPSS format from FDOT crash database for one site.



	no	date	time	day	district	county	section	subsection	milepost	V10	V11	V	direc
1	753523090	2005-11-22	18:00	2	1	1	75	0	8.263	571	.000		
2	753549960	2005-10-03	01:25	1	1	1	75	0	8.287	571	.024	MI	N
3	753875270	2006-10-23	12:50	1	1	1	75	0	8.287	571	.024	MI	N
4	769693310	2006-10-14	16:52	6	1	1	75	0	8.298	571	.035	MI	N
5	756662420	2006-02-18	12:35	6	1	1	75	0	8.655	944	.100	MI	N
6	751842580	2005-05-14	21:55	6	1	1	75	0	8.697	575	.078	MI	S
7	709691280	2005-04-16	01:50	6	1	1	75	0	8.732	575	.043	MI	S
8	746067270	2005-08-03	23:34	3	1	1	75	0	8.733	575	.042	MI	S
9	746067330	2005-09-06	06:40	2	1	1	75	0	8.733	575	.042	MI	S
10	753523480	2006-02-03	19:45	5	1	1	75	0	8.733	575	.042	MI	S
11	713693160	2005-06-06	06:36	1	1	1	75	0	8.744	575	.031	MI	S
12	751840300	2006-02-03	19:29	5	1	1	75	0	8.744	575	.031	MI	S
13	751840310	2006-02-03	19:30	5	1	1	75	0	8.744	575	.031	MI	S
14	713509450	2004-06-08	19:45	2	1	1	75	0	8.781	574	.000		
15	753514610	2005-07-12	16:15	2	1	1	75	0	11.469	946	.031	MI	N
16	769574770	2006-05-21	19:04	7	1	1	75	0	11.559	583	.006	MI	N
17	753513170	2005-08-13	10:54	6	1	1	75	0	14.842	593	.047	MI	N
18	709756250	2004-03-12	17:34	5	1	1	75	0	14.842	593	.047	MI	N
19	769562550	2006-10-07	19:50	6	1	1	75	0	14.891	593	.096	MI	N
20	769532270	2006-08-19	16:57	6	1	1	75	0	14.903	593	.108	MI	N
21	753872280	2005-03-09	00:00	3	1	1	75	0	20.921	610	.120	MI	N
22	756652830	2006-02-24	08:12	5	1	1	75	0	20.541	611	.256	MI	S

Figure 5.8 SPSS Example format from FDOT crash database

5.7 Combination of Crash Data with Site Information

Each site has a specific database consisted of geometric variables, traffic data and relative crash information. The Excel file will be used to arrange the format of each location for useful variables. The following Figure 14 shows part data from the combining database for some sites.

D264 Outer Connection

	A	B	C	D	E	F	G	H	I	J	K
1	Numb	Exit name	Ramp type	Ramp Configurations	Mainlane	Dist	SECTION NUMBER	Secondary name	Direction	paint nos	sub-section
2	1	I-75 EXIT 158-1	1	Diamond	I-75	1	01-075-000	Trucks Grade	S---N	8.259	301
3	2	I-75 EXIT 158-2	1	Diamond	I-75	1	01-075-000	Trucks Grade	N---S	8.775	304
4	3	I-75 EXIT 161-1	1	Diamond	I-75	1	01-075-000	Jones Loop Rd	SE---NW	11.544	307
5	4	I-75 EXIT 161-2	1	Diamond	I-75	1	01-075-000	Jones Loop Rd	NW---SE	12.073	310
6	5	I-75 EXIT 164-1	1	Diamond	I-75	1	01-075-000	Federal 17	S---N	14.788	311
7	6	I-75 EXIT 167-2	1	Diamond	I-75	1	01-075-000	Harbor View Rd	NW---SE	18.168	318
8	7	I-75 EXIT 170-1	1	Diamond	I-75	1	01-075-000	Kings Hwy	S---N	20.797	319
9	8	I-75 EXIT 170-2	1	Diamond	I-75	1	01-075-000	Kings Hwy	N---S	21.342	322
10	9	I-75 EXIT 101-1	1	Diamond	I-75	1	03-175-000	Collier Blvd	E---W	50.107	1
11	10	I-75 EXIT 123-1	1	Diamond	I-75	1	12-075-000	Corkscrew Rd	S---N	8.095	5
12	11	I-75 EXIT 123-2	1	Diamond	I-75	1	12-075-000	Corkscrew Rd	N---S	8.681	8
13	12	I-75 EXIT 128-1	1	Outer Connection	I-75	1	12-075-000	Alico Rd	S---N	12.23	9
14	13	I-75 EXIT 138-1	1	Diamond	I-75	1	12-075-000	Dr Martin Luther King	S---N	22.352	23
15	14	I-75 EXIT 138-2	1	Diamond	I-75	1	12-075-000	Dr Martin Luther King	N---S	22.907	26
16	15	I-75 EXIT 139-1	1	Diamond	I-75	1	12-075-000	Luckett Rd	S---N	23.894	27
17	16	I-75 EXIT 139-2	1	Diamond	I-75	1	12-075-000	Luckett Rd	N---S	24.39	30
18	17	I-75 EXIT 141-1	1	Diamond	I-75	1	12-075-000	Palm Beach Blvd(80)	S---N	25.809	31
19	18	I-75 EXIT 141-2	1	Diamond	I-75	1	12-075-000	Palm Beach Blvd(80)	N---S	26.307	34
20	19	I-75 EXIT 143-2	1	Diamond	I-75	1	12-075-000	Bayshore Rd	N---S	28.658	38
21	20	I-75 EXIT 217-1	1	Outer Connection	I-75	1	13-075-000	State 70	S---N	3.409	3
22	21	I-75 EXIT 217-2	1	Outer Connection	I-75	1	13-075-000	State 70	N---S	3.997	8
23	22	I-75 EXIT 220-1	1	Parclo Loop	I-75	1	13-075-000	State 64	S---N	6.993	9
24	23	I-75 EXIT 220-2	1	Outer Connection	I-75	1	13-075-000	State 64	N---S	7.599	14
25	24	I-75 EXIT 224-2	1	Outer Connection	I-75	1	13-075-000	Federal 301	NW---SE	11.287	18
26	25	I-75 EXIT 229-1	1	Diamond	I-75	1	13-075-000	Moccasin Wallow Rd	SW---NE	15.85	19
27	26	I-75 EXIT 229-2	1	Diamond	I-75	1	13-075-000	Moccasin Wallow Rd	NE---SW	16.455	22
28	27	I-4 EXIT 31-1	1	Diamond	I-4	1	16-320-000	Kathleen Road	SW---NE	4.86	60
29	28	I-4 EXIT 31-2	1	Diamond	I-4	1	16-320-000	Kathleen Road	NE---SW	5.373	62
30	29	I-4 EXIT 33-1	1	Diamond	I-4	1	16-320-000	Lakeland Hills Blvd (3	SW---NE	8.468	82
31	30	I-4 EXIT 38-1	1	Diamond	I-4	1	16-320-000	33 State Road	SW----NE	12.132	90

Figure 5.9 Example of Combining Database

CHAPTER 6 DATA ANALYSIS

Detailed procedures and results of crash data analyses were performed in this chapter. As mentioned before, freeway diverge areas and entire exit ramp sections are two separate research subjects in the study. Quantitative investigations were conducted to find out crash characteristics and the contributing factors in order to evaluate safety performances both on the freeway diverge areas and exit ramp sections.

6.1 Outline of Data Analysis

Crash data for freeway diverge areas and exit ramps are analyzed independently as to evaluate the safety performances on the two research sections in this study. As mentioned previously, the cross-sectional comparisons were conducted to compare the effects of the four exit ramp types on the safety performance of freeway diverge areas and effects of ramp configurations on the safety performance of the exit ramp sections respectively. On the freeway diverge areas, a total of 424 sample sites were collected. The sample size was divided into four groups according to the four different exit ramp types as mentioned before. Group 1 has 220 sites for Type 1 exit ramps, Group 2 has 96 sites for Type 2 exit ramps, Group 3 has 77 sites for Type 3 exit ramps and Group 4 has 31 sites for Type 4 exit ramps. On the exit ramp sections, a total of 389 sites with 247 sites for diamond ramps, 93 sites for out collection ramps, 26 sites for free-flow loop ramps, and 23 sites for parco loop ramps were categorized. Two crash predictive models were developed for the two research subjects to find the contributing factors to the crashes occurring at diverge areas and exit ramp sections.

First, average crash frequency and crash rate for each group on the two research subjects were calculated. Statistical tests were conducted to compare each section at a 90% confidence level one by one. Second, each group had the sample sites classified by target crash types that have three most crash frequencies among all the crash types. Then the average crash number and crash rates by target crash types were calculated by using crash data from 2004 to 2006 and the corresponding statistical tests were performed. Third, crash severity categories such as PDO (property-damage-only), injury and fatality were compared with corresponding average crash number and crash rate by each ramp configuration. The comparisons were followed by statistical significance tests at 90% confidence level which is believable and commonly used in crash analysis. Finally, two predict models were built to find the predictive crash number under some definite conditions according to the independent variables.

6.2 Freeway Diverge Areas

6.2.1 Comparison of Average Crash Frequency and Crash Rate

A total of 13968 crashes were reported at selected freeway diverge segments for three years from 2004 to 2006. The crash frequency at selected sites varies from 0 to 60 with a mean of 11.01 crashes per year. Summary statistical analyses of crash frequency and crash rate for four exit ramp groups were illustrated in previous table. The average crash frequency and crash rate for different exit ramp groups were compared in Figure 6.1 and 6.2. Average crash frequency is the mean value of all the crashes in one group each year. In this study, crash rate is defined in the methodology chapter, set as crashes per million vehicles per mile. The average daily traffic for each site was collected and the segment length was identified equally for each site. For example, if site I has 10 crashes for the three years from 2004 to 2006, segment length is 0.47 miles (2500 ft), and the ADT is 10,000 vehicles per day, the crash rate for this site I could be calculated as following:

$$\text{Crash Rate for the Site I} = \frac{1{,}000{,}000 \times 10\,crashes}{365\,days \times 3\,years \times 10{,}000\,vpd \times 0.47\,miles} = 1.94$$

The average crash rate for a particular group is calculated by the mean value of crash rates for all sites. As shown in Figure 6.1 and 6.2, the type 1 exit ramp group has the best safety performance in terms of the lowest average crash frequency and crash rate comparing to other exit ramp types. The figures also show that the type 2 exit ramp group has the highest average crash frequency and crash rate. The trends of average crash frequency and crash rate among the four types showing in the figures are sequent. Type 1 and Type 2 have the lowest and highest average crash frequency and crash rate among the 4 groups while the average crash frequency and crash rate for Type 3 and Type 4 is a little higher than Type 1 and a little lower than Type 2. Table 6.1 listed the detailed analysis such as mean, median, max and min values for each group. On average, the sites in type 2 exit ramps group report the most average crash frequency as 15.4 crashes per year in freeway diverge segments. As compared those in Type 1 exit ramp group, sites in Type 2 exit ramp group reports 75% more crashes per year for one lane exit ramp. The average crash rate at sites with Type 2 is also 35.6% higher when comparing those with Type 1 per year. For two lane exits, Type 3 appears 20% and 14% less average crash frequency and crash rate than Type 4.

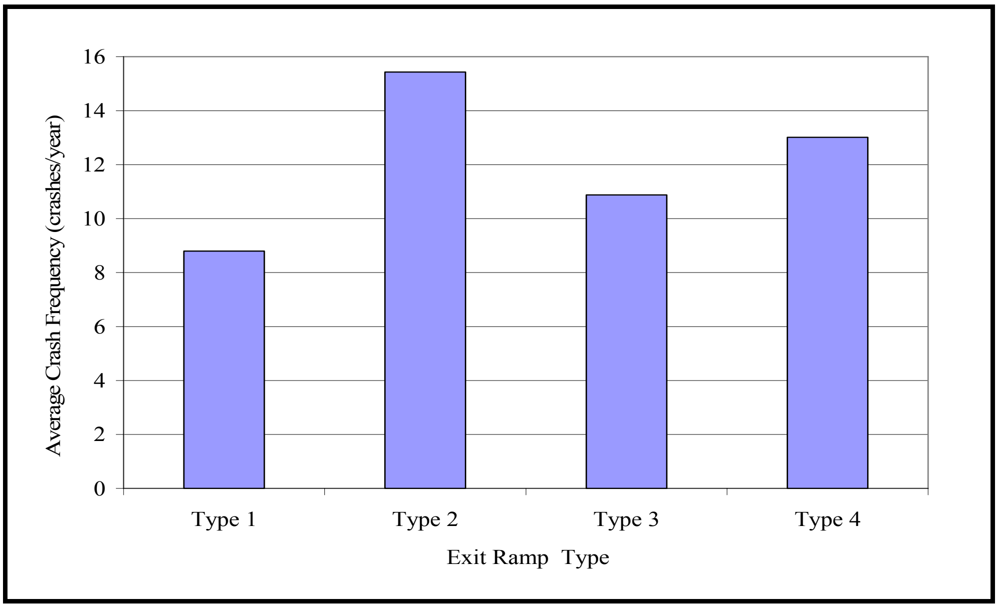


Figure 6.1 Comparison of Average Crash Frequency among Four Exit Ramp Types

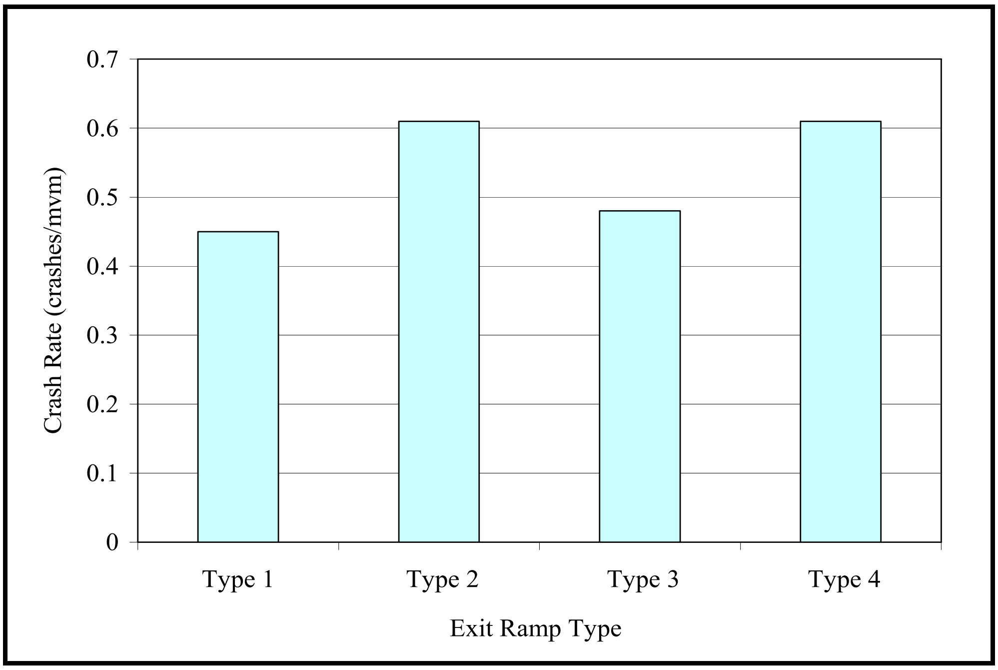


Figure 6.2 Comparison of average Crash Rate among Four Exit Ramp Types

Table 6.1 Summary of Average Crash Frequency and Crash Rate for Four Exit Ramp Types

	Crash Frequency (No. of crashes per year)				Crash Rate (No. of crashes per million vehicles per mile)			
Type	1	2	3	4	1	2	3	4
No. of Sites	220	96	77	31	220	96	77	31
Total No. of Crashes per year	1934	1481	824	417	1934	1481	824	417
Average No. of Crashes	8.8	15.4	10.7	13.45	0.45	0.61	0.48	0.61
St. Deviation	6.23	13.8	8.14	11.3	0.36	0.45	0.32	0.37
Median	4.7	13.2	8.67	12.3	0.36	0.48	0.42	0.55
Max	54	30	31	60	1.36	1.98	1.18	1.24
Min	0	0	1	0	0	0	0.061	0

The site with the highest crash frequency is located on Interstate Highway 95 (I-95) in District 4 along the southbound. Figure 6.3 below showed the site picture. During the three-year time period, 179 crashes were reported at selected freeway segments. 101 are injury plus fatal crashes and the others are PDO crashes. Field observation was made to the particular site to identify the undesirable driving behaviors contributing to the high crash frequency. The segment is located on a five-lane freeway with a posted speed limit of 55 mph. The exit ramp is found to be a type 4 exit ramp which is a two-lane exit ramp without an optional lane. The annual daily traffic volume (ADT) on the freeway is 224,000 vehicles per day. The reasons that had most crashes might be the traffic volume was higher than usual, and the exit ramp type in the site caused more weaving maneuvers in diverge areas. Drivers who mistakenly entered the exit lane need to merge back into through lanes to continue on the freeway; while vehicles exiting freeways may need to change up to four lanes to weave to the outer exit lane. Some severe weaving conflicts have been observed at the site that indicates a high potential crash prone area.

Figure 6.3 Site Picture for I-95 Southbound Exit 74

In order to compare whether the average crash frequencies and crash rates for the four exit ramp types have significant different from each other, hypothesis tests were applied to evaluate the samples. For example, the statistical Z test to compare the average crash frequency for Type 1 exit ramp and Type 2 exit ramp was performed as following:

1) The mean values for two populations Type 1 exit ramp and Type 2 exit ramp are μ1 and μ2;

2) Mean value and standard deviation of the two samples for Type 1 exit ramp are 8.8 and 6.73 respectively, while those for Type 2 exit ramp are 15.4 and 13.8;

3) The sample numbers for Type 1 exit ramp and Type 2 exit ramp are 220 and 96 accordingly;

4) The null hypothesis is H0 : μ1 = μ2, the alternative hypothesis is Ha: μ1 ≠ μ2;

5) Assuming the difference in the sample means of the two population fit the normal distribution and 90% confidence level was chosen for this study;

6) $Z_0 = \dfrac{|\ 8.8 - 15.4|}{\sqrt{\dfrac{6.23^2}{220} + \dfrac{13.8^2}{96}}} = 3.06$;

7) The critical value for $Z_{\alpha/2}$ is 1.645 which is smaller than 3.06 so that the null hypothesis is rejected;

8) The conclusion could be get as the average crash number for Type 1 and Type 2 exit ramp is significant different at a 90% confidence level.

The average crash frequency and crash rate for each population were tested at a 90% confidence level. Table 6.2 listed all the results for the hypothesis tests. The comparison of the average number of crashes for Type 1 and Type 2 exit ramp showing “1:2” is significantly different at a 90% confidence level meaning “YES” in the table. For average crash frequency, Type 1 shows significant different from the other three types while Type 2 has significantly different average crash frequency with Type 3 but not with Type 4 exit ramps. The results were consistent for average crash frequency and crash rate except comparing Type 1 and Type 3 exit ramps. This might be the cause that crash rate has limited the traffic volume impacts. For one lane exit ramp, Type 1 exit ramp is much safer than Type 2 exit ramp. For two-lane exit ramp, Type 3 group did appear significant difference with Type 4 exit ramp on average crash rate.

Table6.2 Summary Hypotheses Tests of Average Crash Frequency and Crash Rate for Four Exit Ramp Types

Crash	Statistics Results for Two Mean Tests: 90%					
	1:2	1:3	1:4	2:3	2:4	3:4
Frequency	YES	YES	YES	YES	NO	NO
Rate	YES	NO	YES	YES	NO	YES

6.2.2 Comparison of Target Crash Type

Three target crash types as mentioned before, rear-end crashes, angle crashes and sideswipe crashes, were compared for each exit ramp type to find the crash characteristics among the four ramp types. Table 6.3 lists the total numbers of crashes, percentages of total crashes, average crash numbers, standard deviations and median values for the four ramp types by three target crash types. The average crash numbers for rear-end crashes and sideswipe crashes among the four types have larger differences among each other while those for angle crashes have minor distinctions among the four ramp types. In Table 6.4, the average crash rate for Type 1 and Type 3 equal of 0.21 crashes per million vehicles per mile per year for rear-end crashes. But Type 2 and Type 4 have 30% and 34.4% more crashes than these two types.

Figure 6.4 illustrates that the percentage of rear-end crashes for 4 types are 45.97%, 48.41%, 41.26%, and 44.60%. Type 3 group counts less percentage than the other three groups. It is reasonable that two-lane exit ramp with an operational lane will provide more spaces for vehicles acceding or decreasing speed in the diverge area than single-lane exit ramp. With the optional lane, some unfamiliar drivers or these drivers on the wrong lanes would have an opportunity to either continue or leave the freeway mainline segments. The sideswipe crashes is the crash type that have the second largest crash number. Table 6.3 shows the percentage of each group for sideswipe crashes is 15.82%, 15.67%, 15.05% and 16.31%. That might be a result of the additional weaving maneuvers for Type 4 comparing to Type 3. As Type 4 exit ramp group, some drivers are willing to continue on the freeways when they may misunderstand the inner lane of two exits as a through lane. When they found it was an exit lane, they might take some dangerous maneuvers such as quickly reducing speed, immediately changing lanes, or even completely stopping which often cause more sideswipe crashes happening to continue driving on freeways. Type 3 appears less rear end and sideswipe crashes than other three exit ramp types.

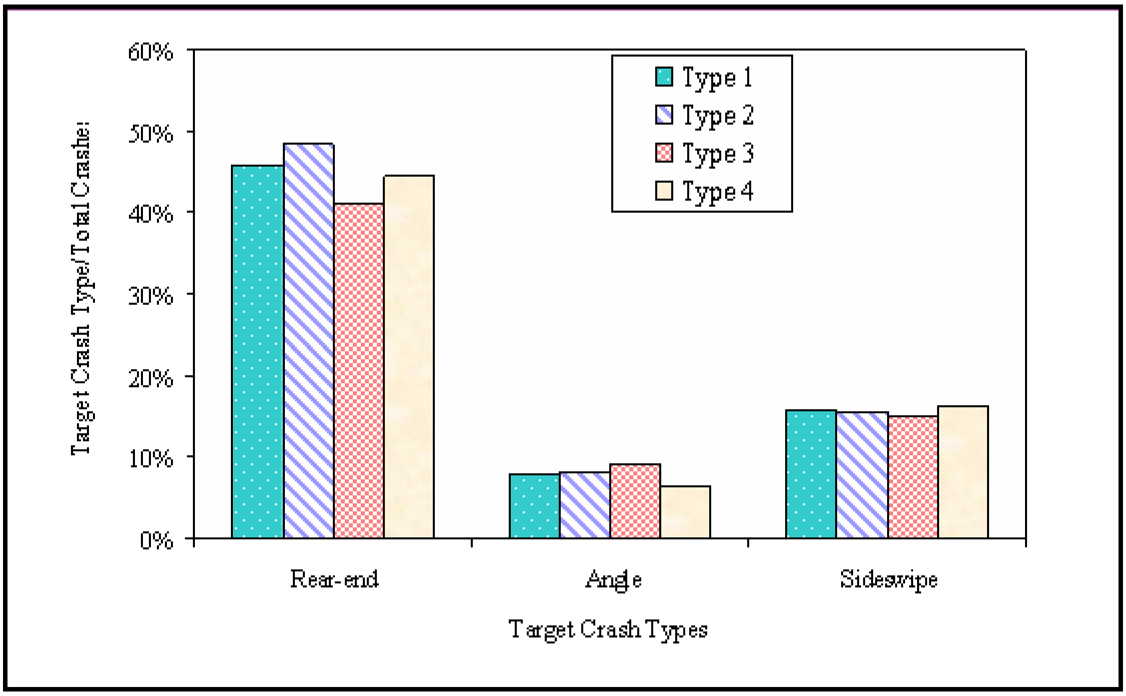


Figure 6.4 Comparisons of Percentages by Target Crash Types for Four Exit Ramp Types

Table 6.3 Summary of Average Crashes Numbers by Target Crash Types for Four Exit Ramp

Target Crash	Statistics	Type 1	Type 2	Type 3	Type 4
Rear-end Crashes	No. of Crashes per year (% of Total)	899 (45.97%)	717 (48.41%)	340 (41.26%)	186 (44.60%)
	Average No. of Crashes	4.09	8.06	4.42	6.00
	Standard Deviation	7.50	8.75	4.40	7.05
	Median	2	6	3	6
Angle Crashes	No. of Crashes per year (% of Total)	152 (7.88%)	121 (8.19%)	76 (9.22%)	27 (6.47%)
	Average No. of Crashes	0.69	1.26	0.99	0.87
	Standard Deviation	0.91	1.16	0.79	0.89
	Median	0.67	1.33	1	1
Sideswipe Crashes	No. of Crashes per year	306 (15.82%)	232 (15.67%)	124 (15.05%)	68 (16.31%)
	Average No. of Crashes	1.39	2.42	1.61	2.19
	Standard Deviation	3.52	2.10	1.43	1.97
	Median	1	2.33	1.33	2.67

Table 6.4 Summary of Average Crashes Rates by Target Crash Types for Four Exit Ramp

Target	Statistics	Type 1	Type 2	Type 3	Type 4
Rear-end Crashes	Average No. of Crashes	0.210	0.300	0.210	0.320
	Standard Deviation	0.250	0.291	0.225	0. 350
	Median	0.120	0.170	0.260	0.130
Angle Crashes	Average No. of Crashes	0.054	0.054	0.050	0.053
	Standard Deviation	0.100	0.028	0.029	0.032
	Median	0.040	0.050	0.055	0.050
Sideswipe Crashes	Average No. of Crashes	0.091	0.115	0.118	0.098
	Standard Deviation	0.118	0.111	0.067	0.054
	Median	0.060	0.090	0.120	0.060

Proportionality tests were then conducted to compare the percentages of difference among the four groups. The procedures of proportionality test are similar to Z tests mentioned before. For example, the portions of rear-ends crashes to total crashes for Type 1 exit ramps and Type 2 exit ramps were tested as following:

1) The two populations, Type 1 exit ramp and Type 2 exit ramp, have the percentages of rear-end crashes to the total crashes as p_1 and p_2;

2) The percentages of rear-end crashes to the total crashes for the two samples of Type 1 exit ramp and Type 2 exit ramp are $\hat{p}_1$, 45.97%, and $\hat{p}_2$, 48.41%;

3) Type 1 exit ramp has 220 sites and Type 2 exit ramp has 96 sites;

4) The null hypothesis is H0 : $p_1 - p_2 = 0$, the alternative hypothesis is Ha: $p_1 - p_2 \neq 0$;

5) Assuming the difference of proportions for rear-end crashes in the two samples fits the normal distribution and 90% confidence level was chosen for this study;

6) $$Z^* = \frac{|\,45.97 - 48.41|}{\sqrt{\frac{45.97(100-45.97)}{220} + \frac{48.41(100-48.41)}{96}}} = 0.40\,;$$

7) The critical value for $Z_{\alpha/2}$ is 1.645 which is much larger than Z* so that the null hypothesis can not be rejected;

8) The conclusion is that the proportions of rear-end crashes for Type 1 and Type 2 exit ramp is not significantly different at 90% confidence level.

All the results are given in Table 6.5. The results of the proportionality tests show that the percentages of both rear-end and angle/right-turn crashes among the four exit ramp groups on the freeway diverge areas did not have statistically significant differences with 90% level of confidence. This conclusion indicated that the three crash types having the highest crashes did not differ a lot for the four types.

Table 6.5 Z Statistics for Proportionality Tests by Target Crash Types for Four Exit Ramp Types

Crash Type	Proportionality Tests:90%					
	1:2	1:3	1:4	2:3	2:4	3:4
Rear-end	NO	NO	NO	NO	NO	NO
Angle	NO	NO	NO	NO	NO	NO
Sideswipe	NO	NO	NO	NO	NO	NO

6.2.3 Comparison of Crash Severity

Among the total crashes reported for selected freeway segments, 7518 property damage only (PDO) crashes, 6333 injury crashes and 117 fatal crashes were included. In this study, crash severity was compared among different exit ramp groups by comparing percentages of PDO crashes and injury plus fatal crashes. Summary statistics for crash severity for different exit ramp groups are given in Table 6.6 and 6.7 and compared in Figure 6.5. For one lane exit ramp, Type 1 exit ramp has less average crash frequency and crash rate for both PDO crashes and injury plus fatality crashes than the type 2 exit ramp group. Also, Type 3 exit ramp appears less average crash frequency and average crash rate for the two crash severity categories for two-lane exit ramps. As compared in Figure 6.5, the percentage of injury plus fatality crashes does not significantly differ from each other among different exit ramp groups. Type 2 exit ramp has slightly higher percentage of injury plus fatality crashes comparing to Type 1 exit ramp for one lane exit ramp and Type 4 exit ramp is a bit higher than Type 3 exit ramp for that as well.

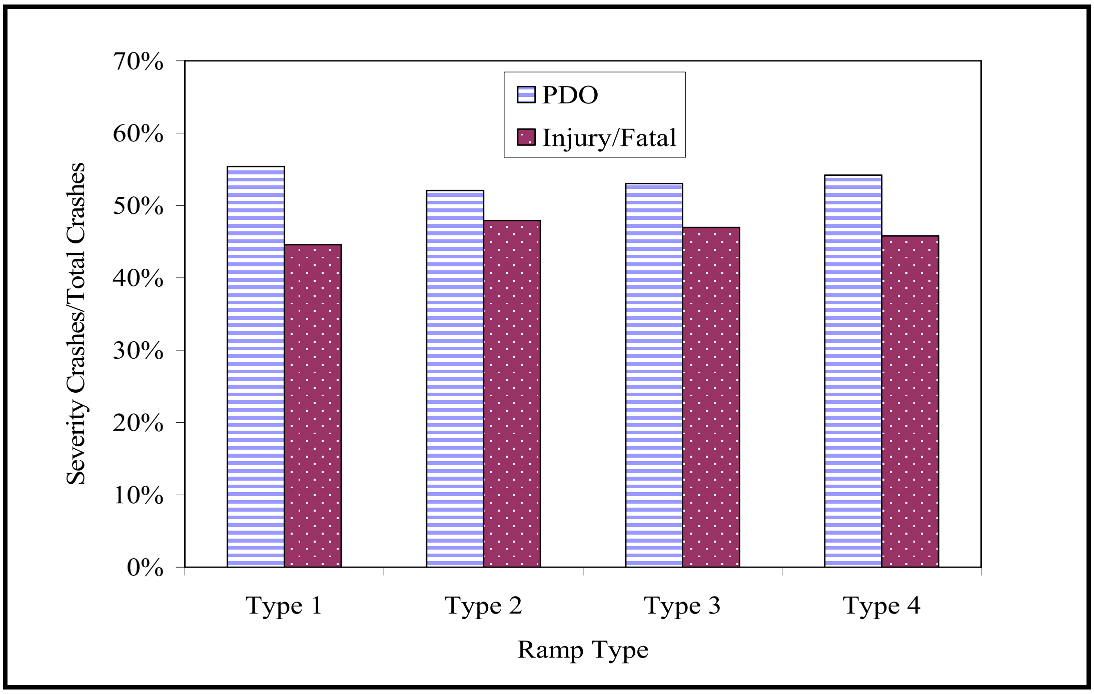


Figure 6.5 Comparisons of Percentages by Crash Severity for Four Exit Ramp Types

Table 6.6 Summary of Average Crash Number by Crash Severity for Four Exit Ramp Types

Crash Severity	Statistics	Type 1	Type 2	Type 3	Type 4
PDO	No. of Crashes (% of Total)	1072 (55.43%)	771 (52.06%)	444 (53.88%)	219 (52.52%)
	Average No. Of Crashes	4.87	8.03	5.77	5.23
	Standard Deviation	6.92	7.64	4.82	6.57
	Median	3.67	13.80	4.67	9.00
Injury/ Fatality Crashes	No. of Crashes (% of Total)	862 (44.57%)	710 (47.94%)	380 (46.12%)	198 (47.48%)
	Average No. Of Crashes	3.92	7.40	4.94	6.39
	Standard Deviation	5.38	7.18	4.16	5.18
	Median	2.33	6	3.33	4.67

Table 6.7 Summary of Average Crash Rates by Crash Severity for Four Exit Ramp Types

Crash Severity	Statistics	Type 1	Type 2	Type 3	Type 4
PDO	Average No. of Crashes	0.325	0.342	0.276	0.356
	Standard Deviation	0.292	0.314	0.204	0.231
	Median	0.205	0.245	0.24	0.32
Injury/ Fatality Crashes	Average No. of Crashes	0.204	0.287	0.238	0.278
	Standard Deviation	0.155	0.2	0.167	0.174
	Median	0.17	0.23	0.18	0.28

Proportionality tests were also conducted for testing the differences in crash severity among four exit ramp groups. The crash database includes 6420 injury plus fatality crashes for three years time frame. The null hypothesis of the proportionality test is that the percentages of injury plus fatal crashes in different exit ramp groups are equal. The conclusions of Z statistics for the proportionality tests are listed in Table 6.8. The calculating procedures are same as target crash types mentioned above. Based on the Z statistic tests, there is no evidence to reject the null hypothesis with 90% level of confidence. The results suggest that, even the exit ramp types significantly impacts the average crash frequency and average crash rate, the differences of their impacts on crash severity are not statistically significant.

Table 6.8 Z Statistics of Proportionality Tests by Crash Severity for Four Exit Ramp Types

Crash Severity	Z Statistics for Proportionality Tests					
	1:2	1:3	1:4	2:3	2:4	3:4
PDO	NO	NO	NO	NO	NO	NO
Injury/Fatal	NO	NO	NO	NO	NO	NO

6.2.4 Crash Predictive Model

In this study, a crash prediction model was developed to identify the factors that contribute to the crashes reported at selected freeway segments and to quantify the safety impacts of different types of freeway exit ramps. Considering the available data source, a total of 404 observation sites were used in the model. Since some sites did not have ramp ADT and ramp design speeds. The variables were believed significantly important to have potential crashes. The dependent variable of the model is the average crash frequency per year reported at selected freeway diverge areas. Seventeen independent variables were initially considered when building the crash prediction model. The four exit ramp types were defined as three indicator variables. The initially selected independent variables are described in Table 6.9. The value of each variable are also listed in the table.

Table 6.9 Description of Initially Considered Independent Variables on Freeway Diverge Areas

Independent Variable	Value	Frequency
Type 2 exit ramp	1 Type 2 exit ramp 0 Otherwise	92
Type 3 exit ramp	1 Type 3 exit ramp 0 Otherwise	75
Type 4 exit ramp	1 Type 4 exit ramp 0 Otherwise	22
Number of lanes on mainline	1 One lane on mainline 2 Two lanes on mainline 3 Three lanes on mainline ……. n N lanes on mainline	404
Number of lanes on exit ramps	1 One lane on mainline 2 Two lanes on mainline 3 Three lanes on mainline ……. n N lanes on mainline	404
Length of deceleration lanes	Distance of the deceleration lanes (mi)	404
Length of entire exit ramps	Distance for the entire ramp from the painted nose to the end of ramp (mi)	404
ADT per year in thousand on freeway sections	Average ADT in thousands for three years 2004~2006	404
ADT per year in thousand on exit ramp sections	Average ADT in thousands for three years 2004~2006	404
Speed difference between mainline and exit ramp	Maximal speed limit difference (mi/h)	404
Road surface condition	0 Dry 1 Wet	404
Land type	0 Primarily business 1 Primarily residential	404
Road surface type	0 Blacktop 1 Concrete	404
Right shoulder type	0 Paved 1 Unpaved	404
Right shoulder width	Width for the right shoulder (ft)	404
Post speed on mainline	Maximal speed limit (mi/h)	404
Post (suggested) speed on ramp	Maximal speed limit (mi/h)	404

The crash modeling starts from a Poisson model. For an adequate model, the scaled deviance and Pearson's χ^2 divided by the degrees of freedom shall be close to one. These two values are used to detect over dispersion or under dispersion in the Poisson regression model. Values greater than 1 indicate over dispersion, while values smaller than 1 indicate under dispersion. In this study, the Pearson's χ^2 divided by the degrees of freedom was found to be 10.50, indicating the fact that the crash data are over dispersed and NB models shall be used. Stepwise regression method was used to select independent variables in the model. Seven variables were not found to be statistically significant. As a result, these variables were not included into the model. The best model contains ten independent variables. The regression results of the best model are given in Table 6.10. As shown in the table 6.10, the scaled deviance and Pearson's χ^2 divided by the degrees of freedom are 1.12 and 1.27 which are reasonably close to one, indicating the fact that the model is adequately fitted. The final equation of the model is given as follows:

$$Y = \exp(3.1523 + 0.1416X_1 + 0.1354X_2 + 0.2244X_3 + 0.1302X_4 + 1.3470X_5 - 0.9385X_6 + 0.0679X_7 + 0.0223X_8 + 0.0614X_9 - 0.0301X_{10})$$

(22)

Where,

Y = expected average crash frequency in a freeway diverge area (crashes/year);

X_1 = 1 if the site has a Type 2 exit ramp, 0 others;

X_2 = 1 if the site has a Type 3 exit ramp, 0 others;

X_3 = 1 if the site has Type 4 exit ramp, 0 others;

X_4 = Number of lanes on the mainline sections;

X_5 = Length of the deceleration lanes (mile);

X_6 = Length of the entire exit ramp (mile);

X_7 = ADT per year in thousands on mainline sections;

X_8 = ADT per year in thousands on exit ramp sections;

X_9 = Speed difference between the post speed limit on mainline and exit ramp section (mph);

X_{10} = Post speed limit on mainline sections (mph).

Table6.10 Regression Results for Crash Prediction Model for Diverge Areas

Criteria for Goodness of Fit			
Criteria	DF	Value	Value/DF
Deviance	393	441.5189	1.12
Scaled Deviance	393	441.5189	1.12
Pearson Chi-Square	393	501.1979	1.27
Scaled Pearson	393	501.1979	1.27
Log Likelihood	38746.0924		

Analysis of Parameter				
Parameter	Coefficient	Standard error	χ^2	Pr > χ^2
Intercept	3.1523	0.4205	132.12	<0.0001
Type 2 exit ramp	0.1416	0.1066	0.19	0.0610
Type 3 exit ramp	0.1345	0.1239	0.38	0.0536
Type 4 exit ramp	0.2240	0.1033	0.80	0.0543
Number of lanes on mainline	0.1302	0.0512	4.41	0.1002
Length of deceleration lanes	1.3470	1.2667	1.02	<0.0001
Length of entire ramp	-0.9385	0.1616	35.46	<0.0001
ADT in thousands on mainline	0.0679	0.0079	73.66	<0.0001
ADT in thousands on ramp	0.0223	0.0049	21.00	<0.0001
Speed difference	0.0614	0.0023	69.68	<0.0001
Post speed limit on mainline	-0.0301	0.0188	12.56	0.0129
Dispersion	0.4365	0.0339		

All selected independent variables were statistically significant with 90% confidence level. The coefficients of the model show that the crash counts at freeway diverge areas increase with the mainline lane number, the deceleration lane length, mainline ADT, ramp ADT and post speed limit difference between mainline sections and ramp sections, however decrease with the entire ramp length, and post speed limit on mainline. With the more numbers of lanes on the freeway segments, the potential conflict points will increase so that the chances occurring crashes increase. ADT both on freeway mainline areas and exit ramp sections would increase the opportunities occurring crashes. It is consistent with previous studies (1, 3). Another two positive variables are the deceleration lengths on diverge areas and the post speed limit differences. It was long believed that crash number would decrease if longer deceleration lengths were applied. However, recently a study presented in last International Symposium on Highway Geometric Design indicated the hypothesis is not correct. The study also proved that longer deceleration length might increase the number of weaving maneuvers and cause more potential crashes than short distances. Speed differences between mainline sections and exit ramp sections have positive influences on the crashes as well. It is intuitive as the larger variations on posted speed, more difficult for vehicles to control operating speeds. Some vehicles might lose controls as hard driving maneuvers.

From the model, it points out fewer crashes with longer exit ramp length. It make sense that longer ramp length would diminish the impacts of exit ramps on the freeway diverge areas. The coefficient for the posted speed limit is negative, implying that crash counts increase with the decrease of the posted speed limit of the freeway. This result is a little bit counter-intuitive. A possible explanation is that the variable posted speed limit is correlated with other variables which were not included into the model. For example, it is very possible that a freeway with higher posted speed limit is also designed according to higher standards. Thus, higher posted speeds may also imply wider lane width, better lighting conditions, better signing or pavement marking; and these missing variables could reduce crash freeway at freeway diverge areas.

The coefficients for the three indicator variables are all positive, indicating the fact that the site with the type 1 exit ramp has the least numbers of crashes. This conclusion is consistent with the result of the cross-sectional comparison. The coefficients of the model can be used to quantify the safety impacts of different types of freeway exit ramps. Based on the model, replacing a type 1 exit ramp with a type 2 exit ramp will increase crash counts at freeway diverge areas by exp (0.1416-0)-1=15.57%. Replacing a type 3 ramp with a type 4 ramp will increase crash counts at freeway diverge areas by exp (0.2244-0.1354)-1=10.80%.

6.3 Exit Ramp Section

6.3.1 Crash Characteristics

Four different exit ramp configurations were grouped for each category to evaluate the impacts on the safety performance. A total of 2520 crashes were stated for the entire segments for three years from 2004 to 2006. The sites were grouped for four configurations simply named as D (Diamond), O (Out-connector), F (Free-flow Loop) and P (Parclo Loop). The group D has 247 sites, the group O has 93 sites, the group F has 26 sites and the group P has 23 sites. The average crash frequencies for the four groups are 2.20, 2.32, 2.21 and 1.00 crashes per site per year. Summary statistics for average crash frequency and average crash rate by four exit ramp configuration groups were given in Table 6.11.

Average crash frequency is the mean value of all the crash frequencies in one group for each year. Crash rate is defined in the methodology chapter as crashes per million vehicles per mile. The volume for each site was collected and segment length was set as the whole ramp length for the site. The procedures of calculating each exit ramp site were similar to the diverge areas. For example, if site II has 5 crashes for the three years from 2004 to 2006, the entire ramp length is 0.25 miles (1320 ft), and the ADT is 5,000, the crash rate for this site II could be calculated as following:

$$\text{Crash Rate for the Site II} = \frac{1{,}000{,}000 \times 5 crashes}{365 days \times 3 years \times 5{,}000 vpd \times 0.25 miles} = 3.65$$

The average crash rate for a ramp configuration group is calculated by the mean value of crash rate for all sites. In Table 6.11, the average crash frequencies indicate the parclo loop group has the less average crash frequency, however the average crash rates point out that the out connection group has the best safety performance while considering the ramp volume and ramp length. The average crash rate is more reliable as it eliminates the impacts of different ramp volumes and ramp distances. The free-flow loop group has more potential crashes in terms of the maximum average crash rate comparing to the other three exit ramp types. The average crash rate for the free-flow loop group is almost 162%, and 69% more than the out connection group and the diamond group. This result shows different ramp configurations might influence the exit ramps in different ways and the free-flow ramp would have more chances to occur crashes. The conclusion is consistent with previous studies (1, 3, and 5). In the past researches (1, 3), diamond ramps had the best safety performances comparing to other ramp configurations. But the out connection ramps have less average crash rate than the diamond ramps. This might be the reason that the out connection ramps in Florida are widely used as the freeway interchanges that have high design standards than normal exits. These improved standards might be better sign locations before and after the entrances of exit ramps, better road conditions, or less variations along the exit ramps. Table 6.11 also listed the detailed statistical analysis results such as the total crashes per year, mean value, median value, and max and min values for each group in the exit ramp sections. For the loop exits, parclo loop ramps reported 16.7% less average crash rate than free-flow loop exit ramps.

Table 6.11 Summary of Average Crash Frequency and Crash Rate for Four Exit Ramp Configurations

	Crash Frequency (No. of crashes per year)				Crash Rate (No. of crashes per million vehicles per mile)			
Type	D	O	F	P	D	O	F	P
No. of Sites	247	93	26	23	247	93	26	23
Total No. of Crashes	544	216	57	23	544	216	57	23
Average No. of Crashes	2.20	2.32	2.21	1.00	3.47	2.24	5.86	4.88
Standard Deviation	2.46	3.44	2.20	1.09	6.35	3.89	8.33	8.9
Median	1.33	1.33	2	0.67	1.86	0.85	2.16	2.20
Max	11	22	8	4	77.11	22.25	37.28	41.51
Min	0	0	0	0	0	0	0	0

In order to compare whether the average crash frequencies and crash rates for the four exit ramp configurations have significant differences from each one, hypothesis tests were used to evaluate two populations. For example, the statistical Z or t test of average crash rates for the diamond ramp group and the out connection ramp group was performed as following:

1) The mean values for two populations the diamond exit ramp and the out connection exit ramp are $\mu 1$ and $\mu 2$;

2) Mean value and standard deviation for the diamond exit ramp configurations are 3.47 and6.35, while those for the out-connector exit ramp are 2.24 and 3.89;

3) 247 sites are diamond exit ramps and 93 sites are out connection sites;

4) The null hypothesis is H0 : $\mu 1 = \mu 2$, the alternative hypothesis is Ha:$\mu 1 \neq \mu 2$;

5) Assuming the difference in the sample means of the two population fit the normal distribution and 90% confidence level was chosen for this study;

6) $Z_0 = \dfrac{|\ 3.47 - 2.24\ |}{\sqrt{\dfrac{6.35^2}{247} + \dfrac{3.89^2}{93}}} = 4.97$;

7) The critical value for $Z_{\alpha/2}$ is 1.645 which is smaller than 4.97 so that the null hypothesis is rejected.

8) The conclusion could be got as the average crash rate for the diamond exit ramps and the out-connector exit ramps is significant different at 90% confidence level.

The average crash frequency and crash rate for each population were tested at a 90% confidence level. Considering the sample size for parclo loop group is less than 25, t tests were chosen to use for this particular group as mentioned in the methodology parts. The basic procedures are same instead of the functional form which has been described in the methodology part. Table 6.12 listed all the results for the hypothesis tests. The comparison of the average number of crashes for the diamond exit ramps and the out connection exit ramps showing “D:O” is significantly different at 90% confidence level meaning “YES” in the table. For average crash rate, the out connection exit ramps have significant difference to the other three configurations. The out connection ramps have the least average crash rate so that it has the best safety performance among the four exit ramp configurations at 90% confidence level. The free-flow ramps have the highest average crash rate and the hypothesis tests documented this ramp configuration appears more dangerous than the diamond ramps and out connection ramps. However, the difference between the free-flow ramps and parclo ramps is not significant at 90% confidence level.

Table 6.12 Statistical Hypotheses Tests of Average Crash Frequency and

Crash Rate for Four Exit Ramp Configurations

Crash Type	Statistics for Two Mean Tests: 90%					
	D:O	D:F	D:P	O:F	O:P	F:P
Frequency	NO	NO	YES	NO	YES	YES
Rate	YES	YES	NO	YES	YES	NO

6.3.2 Target Crash Types

Three target crash types that have the three highest crash numbers, rear-end crashes, angle crashes and sideswipe crashes, were compared for each ramp configuration among the four exit ramp configurations types. Table 6.13 lists the total numbers of target crashes, percentages of target crashes to total crashes, average crash numbers, standard deviations and median values for the four configurations by three target crash types.

The average crash numbers for rear-end crashes and angle crashes among the four configurations have larger differences between each other while the sideswipe crashes have minor distinction among the four configurations. In Table 6.14, the average crash rates for diamond ramps have highest per million vehicles per mile per year for rear-end crashes. Free-flow ramps have a little higher average crash rate than the other three configurations for angle crashes and sideswipe crashes. This is because diamond interchanges did not include large curves and most of crashes happened by the operating speed differences between vehicles. But the loop ramps such as free-flow loops have a 360 degree changing on the ramp sections alliance. Usually post or suggested speed limits on these ramps are smaller than diamond ramps, the causation of crashes are more related to the large variations of the alignments on the ramp itself. This geometric design feature lead to more angle and sideswipe crashes on the free-flow ramps.

Table 6.13 Summary of Average Crashes Numbers by Target Crash Types

For Four Exit Ramp Configurations

Crash Severity	Statistics	D	O	F	P
Rear-end Crashes	No. of Crashes (% of Total)	274 (50.37%)	80 (37.04%)	14 (24.56%)	8 (34.78%)
	Average No. of Crashes	1.11	0.96	0.54	0.35
	Standard Deviation	1.71	1.78	2.48	1.58
	Median	0.4	0.33	0	0
Angle Crashes	No. of Crashes (% of Total)	44 (8.81%)	19 (8.80%)	13 (22.81%)	1 (4.35%)
	Average No. of Crashes	0.18	0.20	0.50	0.04
	Standard Deviation	0.24	0.17	0.36	0.11
	Median	0.18	0	0	0
Sideswipe Crashes	No. of Crashes (% of Total)	30 (5.50%)	10 (4.63%)	11 (19.30%)	2 (8.70%)
	Average No. of Crashes	0.15	0.11	0.42	0.09
	Standard Deviation	0.32	0.3	0.22	0.16
	Median	0	0	0	0

Table 6.14 Summary of Average Crash Rates by Target Crash Types for

Four Exit Ramp Configurations

Crash Severity	Statistics	D	O	F	P
Rear-end Crashes	Average No. of Crashes	1.52	0.61	0.59	0.67
	Standard Deviation	2.78	1.31	1.23	1.01
	Median	0.43	0	0	0
Angle Crashes	Average No. of Crashes	0.29	0.19	0.90	0.06
	Standard Deviation	0.26	0.66	0.76	0.21
	Median	0	0	0	0
Sideswipe Crashes	Average No. Of Crashes	0.28	0.05	0.76	0.11
	Standard Deviation	0.45	0.16	0.98	0.32
	Median	0	0	0	0

Proportionality tests were then conducted to compare the percentages of difference among the ramp configuration groups. The procedures of proportionality tests are mentioned before in the diverge areas. For example, the portions in rear-ends crashes for the diamond exit ramps and the out connection exit ramps were tested as follows:

1) The two populations of the diamond exit ramps and the out-connector exit ramps have the percentages of rear-end crashes to the total crashes p_1 and p_2;

2) The percentages of rear-end crashes to the total crashes for the two samples of Type 1 exit ramp and Type 2 exit ramp are $\hat{p}_1$ and $\hat{p}_2$;

3) 247 sites are diamond exit ramps and 93 sites are out connection sites;

4) The null hypothesis is H0 : $p_1 - p_2 = 0$, the alternative hypothesis is Ha: $p_1 - p_2 \neq 0$;

5) Assuming the difference of proportions for rear-end crashes in the sample fit the normal distribution and 90% confidence level was chosen for this study;

6) $Z^* = \dfrac{|\ 50.37 - 37.04|}{\sqrt{\dfrac{50.37(100-50.37)}{247} + \dfrac{37.04(100-37.04)}{93}}} = 2.25$;

7) The critical value for $Z_{\alpha/2}$ is 1.645 which is much larger than Z* so that the null hypothesis can be rejected;

8) The conclusion is the proportions of rear-end crashes for the diamond exit ramps and the out-connector exit ramps is significantly different at a 90% confidence level.

Table 6.15 exhibited all the statistical tests results for target crash types of exit ramp configurations. The diamond exit ramps have significant higher average rear-end crash rate than the other three types at 90% confidence level; while free-flow loop exit ramps have higher the average crash rates for angle and sideswipe crashes than the diamond exit ramps and out connection exit ramps. But the free-flow loop exit ramps and parclo loop exit ramps did not have significant difference on average sideswipe crash rate. This conclusion is consistent with the reason mentioned above as loop exit ramps have more opportunities occurring sideswipe crashes due to the continuous changeable on the ramp.

Table 6.15 Z Statistics for Proportionality Tests by Target Crash

Types for Four Exit Ramp Configurations

Crash Type	Z Statistics for Proportionality Tests					
	D: O	D:F	D:P	O:F	O:P	F:P
Rear-end	YES	YES	YES	YES	NO	NO
Angle	NO	YES	NO	YES	NO	YES
Sideswipe	NO	YES	NO	YES	NO	NO

6.3.3 Crash Severity

Summary statistics for crash severity for different exit ramp configuration groups are given in Table 6.16 and 17. Even free-flow loop and parclo loop exit ramps have less average crash frequency for crash severity than the other two configurations. They both have higher average crash rates on crash severity and percentages in injury/fatality crashes to total number of crashes.

Table 6.16 Summary of Average Crash Numbers by Crash Severity for

Four Exit Ramp Configurations

Crash Severity	Statistics	D	O	F	P
PDO	No. of Crashes (% of Total)	305 (56.07%)	119 (55.09%)	20 (35.09%)	8 (34.78%)
	Average No. of Crashes	1.23	1.28	0.77	0.35
	Standard Deviation	1.44	1.61	1.12	1.69
	Median	0.7	0.67	0.24	0.60
Injury/ Fatality Crashes	No. of Crashes (% of Total)	239 (43.93%)	97 (44.91%)	37 (64.63%)	15 (65.22%)
	Average No. of Crashes	0.97	1.04	1.42	0.65
	Standard Deviation	1.21	1.15	1.30	0.69
	Median	0.30	0.67	1	0.40

Table 6.17 Summary of Average Crash Rates by Crash Severity

for Four Exit Ramp Configurations

Crash Severity	Statistics	D	O	F	P
PDO	Average No. of Crashes	1.91	1.12	3.16	2.06
	Standard Deviation	3.96	2.17	4.39	5.14
	Median	0.93	0.30	1.65	0
Injury/ Fatality Crashes	Average No. of Crashes	1.56	0.99	2.70	2.82
	Standard Deviation	2.69	2.04	4.27	4.79
	Median	0.74	0.32	0.79	0.94

Proportionality tests were also conducted to test the differences in crash severity among different configuration groups. The null hypothesis of the proportionality test is that the percentages of PDO or injury plus fatality crashes in different groups are equal. The results of

Z statistics for the proportionality tests are listed in Table 6.18. The calculating procedures are as same as target crash type mentioned above. Based on the Z statistic tests, there is no evidence to reject the null hypothesis with 90% level of confidence. The results suggest that the impacts of different exit ramp configurations on crash severity are statistically significant especially for those loop exit ramps and non-loop exit ramps. Free-flow loop exit ramps and parclo loop exit ramps have higher percentage of injury plus fatality crashes but less percentage of PDO crashes comparing to diamond exit ramps and out connection exit ramps at 90% confidence level. Loop exit ramps seem to have more chances occurring high severity crashes. This is reasonable as angle and sideswipe crashes usually cause higher crash severity than rear-end crashes.

Table 6.18 Z Statistics for Proportionality Tests by Crash Severity for

Four Exit Ramp Configurations

Crash Type	Z Statistics for Proportionality Tests					
	D:O	D:F	D:P	O:F	O:P	F:P
Table 6.18 (Continued)						
PDO	NO	YES	YES	YES	YES	NO
Injury/fatal	NO	YES	YES	YES	YES	NO

6.3.4 Crash Predictive Models

Another crash prediction model was developed to identify the factors that contribute to the crashes reported at selected exit ramp segments. Considering the available data source, a total of 388 observation sites were included in the model. One site did not have ramp design speeds which were believed significantly important to crashes. The dependent variable of the model is the average crash frequency per year reported at selected exit ramp sections. Nineteen independent variables were initially considered when building the crash prediction model. The initially selected independent variables are described in Table 6.19. The value of each variable are also listed in the table. The four exit ramp configurations were defined as three indicator variables.

The crash modeling starts from a Poisson model. For an adequate model, the scaled deviance and Pearson's χ^2 divided by the degrees of freedom shall be close to one. These two values are used to detect over dispersion or under dispersion in the Poisson regression model. Values greater than 1 indicate over dispersion, while values smaller than 1 indicate under dispersion.

In this study, the Pearson's χ^2 divided by the degrees of freedom was found to be 5.84, indicating the fact that the crash data are over dispersed and NB models shall be used. Stepwise regression method was used to select independent variables in the model. Eight variables were not found to be statistically significant. As a result, these variables were not included into the model. The best model contains eleven independent variables. The regression results of the best model are given in Table 6.20. As shown in the table 6.20, the scaled deviance and Pearson's χ^2 divided by the degrees of freedom are 1.18 and 1.06 which are reasonably close to one, indicating the fact that the model is adequately fitted. The final equation of the model is given as follows:

$$\begin{aligned} Y = \exp(&-1.0721 - 0.2253X_1 + 0.4392X_2 + 0.2973X_3 - 0.2608X_4 - 0.0062X_5 + \\ &0.6861X_6 + 0.3679X_7 + 0.2470X_8 - 0.0978X_9 + 0.0129X_{10} + 0.0580X_{11}) \end{aligned} \quad (23)$$

Where,

Y = expected average crash frequency in an exit ramp section (crashes/year);

X_1 = 1 if the site has an out connection exit ramp, 0 others;

X_2 = 1 if the site has a free-flow loop exit ramp, 0 others;

X_3 = 1 if the site has parclo loop exit ramp, 0 others;

X_4 = Length of the entire exit ramp (mile);

X_5 = Number of lanes on the ramp sections;

X_6 = 1 if the number of lanes widening after the entrance of exit ramps, 0 no;

X_7 =Upstream distances between exit ramp terminal and first intersection (mile);

X_8 = ADT per year in thousands on exit ramp sections;

X_9 = Ramp shoulder width (mile)**;**

X_{10} =Post speed limit on mainline (mph);

X_{11} = Post or suggested speed limit on exit ramp sections (mph);

Table 6.19 Description of Initially Considered Independent Variables

On Exit Ramp Sections

Independent Variable	Value	Frequency
Out-connector exit ramp	1 out-connector exit ramp 0 Otherwise	93
Free-flow loop exit ramp	1 free-flow loop exit ramp 0 Otherwise	26
Parclo loop exit ramp	1 parclo loop exit ramp 0 Otherwise	23
Number of lanes on mainline	1 One lane on mainline 2 Two lanes on mainline 3 Three lanes on mainline ……. n N lanes on mainline	388
Length of entire ramp	Distance for the entire ramp from the painted nose to end	388
Number of lanes on exit ramps	1 One lane on mainline 2 Two lanes on mainline 3 Three lanes on mainline ……. n N lanes on mainline	388
Widening	0 No widening on the ramp 1 Exit ramp widening on the exit ramp Section	388
Signal	0 No signal control 1 Signal control Ramp terminal	388
Channalization	0 No channalization 1 Ramp terminal is channalization	388
Secondary upstream intersection	Distance between ramp terminal and the first upstream intersection	388
Secondary downstream intersection	Distance between ramp terminal and the first downstream intersection	388
ADT per year in thousand on exit ramp sections	Average ADT in thousands for three years 2004~2006	388
Road surface condition	0 Dry 1 Wet	388
Land type	0 Primarily business 1 Primarily residential	388

Table 6.19 Continued

	1 Concrete	388
Right shoulder type	0 Paved 1 Unpaved	388
Right shoulder width	Width for the right shoulder (ft)	388
Post speed on mainline	Maximal speed limit (mi/h)	388
Post or suggested speed on ramp	Maximal speed limit (mi/h)	388

Table 6.20 Regression Results for Crash Prediction Model for Exit Ramp Sections

Criteria for Goodness of Fit			
Criteria	DF	Value	Value/DF
Deviance	375	441.8539	1.1783
Scaled Deviance	375	441.8359	1.1783
Pearson Chi-Square	375	397.9857	1.0613
Scaled Pearson	375	397.9857	1.0613
Log Likelihood	3221.6867		

Analysis of Parameter				
Parameter	Coefficient	Standard error	χ^2	Pr > χ^2
Intercept	-1.0721	0.8577	0.6089	0.1113
Out-connect exit ramp	-0.2253	0.1577	0.0837	0.0530
Free-flow loop exit ramp	0.4392	0.2428	0.9150	0.0704
Parclo loop exit ramp	0.2973	0.2897	0.2704	0.0946
Length of entire ramp	-0.2608	0.3117	0.3502	0.0428
Number of lanes on exit ramp	-0.0062	0.1477	0.2833	0.0335
Widening	0.6861	0.1466	0.9732	<0.0001
Secondary Upstream	0.3679	0.1689	0.6990	0.0294
ADT in thousands on ramp	0.2470	0.0860	0.4155	0.0041
Should width	-0.0978	0.0775	0.0540	0.0266
Post speed limit on mainline	0.0129	0.0093	0.0311	<0.0001
Post speed limit on the ramp section	0.0580	0.0133	0.840	<0.0001
Dispersion	1.1143	0.0993		

All selected independent variables were statistically significant with a 90% confidence level. The coefficients of the model show that the crash counts at exit ramp sections increase with the mainline lane number, ramp ADT, post speed limit both on mainline sections and ramp sections, distances from ramp terminals to the first upstream intersection (First upstream intersection refers to opposing direction ramp terminal, and this distance is based on field data, without considering limited access right of way along the cross street.), and widening, but decrease with the ramp length, the exit ramp lane number, and ramp shoulder type. With the increase of number of lanes on the exit ramp sections, the situation is different from diverge areas. Since more number of lanes on the ramp sections might diminish vehicle distributions on the ramp sections which are particular transition from freeway sections to the secondary roads. The desperation of vehicles would diminish conflict points on the ramp section. With long ramp length, the impacts of freeway diverge areas and secondary cross roads would be minimal, so fewer crashes would occur comparing these short distance ramps that both freeways and cross roads have influences on the ramp itself. With larger should width, drivers have more flexible spaces while dangerous situations happened especially for loop exit ramps that need more space to avoid angle and sideswipe crashes.

ADT exit ramp sections would increase the opportunities occurring crashes. It is consistent with previous studies. Posted speed limits on both mainline and ramp sections have positive influences on the crashes. Since ramp speed is much lower than freeway segments, such as 25-40 mph, drivers would continually maintain high speed on the ramp section while the post speed limit is low. However, usually ramp sections do not have a high design standard comparing to freeways. This would mislead drivers, and chances of having potential crashes would rise. Another two positive variables are the widening conditions and distance from ramp terminals to first upstream intersection. It is obvious that widening would cause more merging or diverging maneuvers which are generally the main reasons of crashes. The coefficient of distance from ramp terminals to first upstream intersection is 0.3679 which has a significant increase in crash frequency while increasing the distances. It means if the intersection is far away the ramp terminals, it would raise the chances of crashes. If the intersection is close to the ramp terminals, more attentions should be paid at those intersection areas as most drivers are more sensitive to intersections than the normal driveways or roadways.

The coefficients for the three indicator variables have different signs, indicating the fact that the site with the out connection exit ramp has the least numbers of crashes. This conclusion is consistent with the result of the cross-sectional comparison. The coefficients of the model can be used to quantify the safety impacts of different exit ramp configurations. Based on the model, the sign of out connection exit ramp is negative. It can concluded that replacing a diamond exit ramp with an out connection exit ramp, will reduce crashes in the sections by exp (0.2253)-1=26.90%. However, replacing a diamond exit ramp with a free-flow loop ramp and a parclo loop ramp will increase crash counts at exit ramp by exp (0.4392)-1=56.86%, and exp (0.2973)-1= 35.62%. Thus, it can be calculated the increasing percentages for replacing an out connection exit ramp with 68.47% and 48.72%. While only concerning on the loop exit ramp, replacing a parclo loop exit ramp with a free-flow loop exit ramp would increase crash counts by exp (0.4392-0.2973)-1=15.66%.

CHAPTER 7 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 Summary

The objective of this study is to evaluate the impacts of different exit ramp types on the safety performance. Two research subjects, freeway diverge areas and exit ramp sections, were selected. Impacts of different exit ramp types on the diverge areas and different ramp configurations on the exit ramps were analyzed respectively. This study developed quantitative evaluations and comparisons on the freeway diverge areas and exit ramp sections correspondingly. The results of this study will help transportation decision makers develop tailored technical guidelines governing the selection of the optimum exit ramp types to be used on freeways and exit ramps.

For the freeway diverge areas, in order to find the impacts of exit ramp types on the safety performance of freeway diverge areas, lane balance issues were considered to determine the exit ramp types on the freeway diver areas. The exit ramp types were defined by the number of lanes used by traffic to exit freeways. Four different types of exit ramps were considered in this study. For convenience, they are defined as Type 1, Type 2, Type 3, and Type 4 exit ramps. Among these exit ramp types, Type 1 and Type 2 are one-lane exit ramps, while Type 3 and Type 4 are two-lane exit ramps. Type 1 is a parallel from a tangent single-lane exit ramp. Type 2 is a single-lane exit ramp without a tangent. Type 3 is a two-lane exit with an optional lane and Type 4 is a two-lane exit without an optional lane. A total of 424 freeway segments were collected in the State of Florida, 220 sites for Type 1 exit ramps, 96 sites for Type 2 exit ramps, 77 sites for Type 3 exit ramps and 31 sites for Type 4 exit ramps. The selected sites were divided into four groups based on the types of exit ramps. Crash data were selected for three years, from 2004 to 2006 for each site. Cross-sectional comparison was conducted for comparing the crash frequency, crash rate and crash severity between different exit ramp groups. Three target crash types that have the three most crashes were chosen from all the crash types. They are rear end crashes, sideswipe crashes and angle crashes. The average crash number and crash rate was calculated by each exit ramp type on each freeway diverge site. The hypothesis tests were conducted for four exit ramp types to compare whether significant differences for average crash frequency and crash rate are present between the four exit ramp types at 90% confidence level. Crash severity was grouped by two categories, property-damage-only crashes and injury/fatality crashes for four exit ramp types. The average crash frequency and crash rate for each target crash type and crash severity were calculated by four exit ramp types on the freeway diverge areas as well. Proportionality tests were performed for the target crash types and two crash severity categories by four exit ramp types. A crash prediction model containing 404 sites was developed to identify the factors that contribute to the crashes reported at selected freeway segments and to quantify the safety impacts of different freeway exit ramps.

On the exit ramp sections, the exit ramp configurations were grouped by four regular categories, which are diamond exit ramps, out connection exit ramps, free-flow loop exit ramps and parclo loop exit ramps. A total of 389 exit ramp sites were collected in the State of Florida, 247 sites for the diamond exit ramps, 93 sites for the out connection exit ramps, 26 sites for the free-flow loop exit ramps and 23 sites for the parclo loop exit ramps. Crash data were selected for the same years in the diverge areas, from 2004 to 2006 for each site. Cross-sectional comparison was also conducted for comparing crash frequency, crash rate and crash severity between different exit ramp configuration groups. Rear-end crashes, sideswipe crashes and angle crashes are the target crash types that have the three most crashes among all the crash types. Crash severity was grouped by two categories, property-damage-only crashes and injury/fatality crashes. The hypothesis tests were completed respectively at 90% confidence level. A negative binomial crash prediction model including 388 sites was developed to identify the factors that contribute to the crashes reported at selected exit ramp segments.

7.2 Conclusions

In this thesis, two research parts, freeway diverge areas and exit ramp sections are analyzed separately. The conclusions would describe separately for the two parts.

7.2.1 Freeway Diverge Areas

Based on the research analysis, the conclusions on freeway diverge areas can be obtained as following:

1) Type 1 exit ramp has the best safety performance in terms of the lowest crash frequency and crash rate on freeway diverge areas. However, statistical tests show that crash severity and crash types did not have significant differences among the four exit ramp types on the freeway diverge areas at 90% confidence level.

2) The predictive model was built. The coefficients of the model show that the crash counts at freeway diverge areas increase with the mainline lane number, the deceleration lane length, mainline ADT, ramp ADT and post speed limit difference between mainline sections and ramp sections, however decrease with the entire ramp length, post speed limit on mainline sections and surface type.

3) The model also quantifies the impacts of different exit ramp types. For one-lane freeway exit ramp, replacing a type 1 exit ramp with a type 2 exit ramp will increase crash counts at freeway diverge area by 15.57%. For two-lane exit ramps, replacing a type 3 ramp with a type 4 ramp will increase crash counts at freeway areas by 10.80%.

7.2.2 Freeway Exit Ramp Sections

Summary of safety evaluation on exit ramp sections were given in following conclusions:

1) The results of average crash rates on four ramp configurations show that the out connection group has the best safety performance. The free-flow loop group has more dangerous in terms of the greatest average crash rate comparing to the other three exit ramp types.

2) Statistical tests suggest that the loop exit ramps have significant higher crash severity level than non-loop exit ramps at 90% confidence level. Three target crash types, which have the three highest crash numbers, are rear-end crash, angle crash and sideswipe crash. Diamond exit ramps have significant higher average rear-end crash than the other three types; while free-flow loop exit ramps have higher average crash rates for angle and sideswipe crashes than the non loop exit ramps.

3) The coefficients of the model show that the crash counts at exit ramp sections increase with the mainline lane number, ramp ADT, post speed limit both on mainline sections and ramp sections, distances from ramp terminals to the first upstream intersection, and widening, but decrease with the ramp length, the exit ramp lane number and ramp shoulder type.

4) The coefficients for ramp configurations indicate the fact that the site with the out connection exit ramp has the least numbers of crashes. Based on the model, replacing an out connection exit ramp with a diamond exit ramp, a free-flow loop ramp and a parclo

loop ramp will increase crash counts at exit ramp sections by 26.90%, 68.47%, and 48.72%. For the loop exit ramp, replacing a parclo loop exit ramp with a free-flow loop exit ramp would increase crash counts by 15.6%.

7.3 Applications and Recommendations

7.3.1 Applications

This study conducted statistical methods and tests to evaluate safety performances of freeway exit ramps on two parts, freeway diverge areas and exit ramp sections. On the freeway diverge areas, four typical exit ramp types used in Florida were compared and it was found that a parallel from a tangent single-lane exit ramp has the best safety performances among the four exit ramp types. On the exit ramp sections, four widely used exit ramp configurations were selected and compared in the State of Florida. The study provided technical specifications for transportation agencies to develop tailored guidelines or practical design instructions. Transportation engineers, researchers and investigators would benefit from the study as well. The contributing factors to crashes and their impacts were identified and concluded. The results of this study would help transportation decision makers select the optimal exit ramp types and design combinations on freeway mainline segments under different site situations.

7.3.2 Recommendation

Four types of freeway exit ramps were considered on the freeway diverge areas, the crash data analysis results between one lane exit ramps (Type 1 and Type 2 exit ramps) and two-lane exit ramps (Type 3 and Type 4 exit ramps) confirm the general assumption that lane balanced exit ramps would be safer than those not lane balanced exit ramps on the freeway diverge areas (12). In practice, however, there is also a type 5 exit ramp which is a two-lane exit ramp without optional lane and without a taper. This exit ramp is not widely used in Florida and the samples being found are too small to draw defensible conclusions.

To select the optimal exit ramp type, the safety performance of freeway ramp section, further study might focus on the secondary crossroads at ramp terminals which are also critical segments during highway safety improvement. The authors recommend that future studies could be made on these specific segments.

Another important consideration is the conflict studies on these sites to further refine the methodology. In addition, operational analysis and simulation analysis need to be applied. Operational impact and safety impacts should look closely to determine the practical design for both freeway diverge areas and exit ramp sections.

PART III – OPERATIONAL ANALYSIS

CHAPTER 8 INTRODUCTION AND LITERATURE REVIEW

8.1 Background

In the State of Florida, four typical types of exit ramps are used for traffic to exit the freeways (see definitions in chapter 3 and chapter 5). Drivers exiting freeways need to make decisions and execute maneuvers (i.e., lane change or lane merge) prior to the exit ramp in order to access cross roads at the interchanges. If the exit ramps are not sufficiently long, drivers must complete their driving maneuvers within a short distance, resulting in potentially unsafe driving actions (i.e., fast-paced deceleration, lane changing, merging, unbalanced lane utilization, etc.), which will result in the development of shock-waves onto upstream traffic, etc. Considering these factors, there are several issues and concerns that need to be addressed in selecting the most optimum types of freeway exit ramp(s) to use at a given interchange.

Some of these concerns, include but are not limited to, the operational performance and correlation between types of exit ramps, lane utilization, geometrics, land use along the crossroad, adequate distances for lane change, deceleration, adequate distance for traffic to transit from the exit gore to the downstream intersection which includes weaving. These issues have not been studied in the past and no clear guidelines, either federal (AASHTO Green Book) or state, are currently available in selecting exit ramp types. Therefore, there is a need to perform a research under Florida conditions to specifically evaluate the operational performance for each exit ramp type to develop tailored guidelines that address the issues.

This need is especially significant considering the rapid increasing in new developments close to freeway interchanges. The Florida Department of Transportation in joint cooperative efforts with the local land use agencies can use the findings of this research project to determine the type of exit ramps that should be constructed at a given location considering the prevailing conditions applicable to traffic, roadway, and land-use developments.

8.2 Research Objectives

The main goal of the research is to evaluate operational performance of different exit ramps and to develop technical guidelines governing the selection of optimum exit ramp type to be used on Florida freeways. Typical exit ramp types include, but are not limited to, single lane exit ramp with an taper, single lane exit ramp without an taper, two lane exit ramp with an optional lane, and two lane exit ramp without an optional lane (see Fig 5.1 to Fig 5.4).

In addition, operational analysis is also trying to present some design guidelines, such as ramp length design, ramp curve design, super elevation design, minimal distance design on cross road, and etc.

All the analysis would base on traffic operational performance evaluation. Video cameras will be installed at selected sites to record vehicle movements so that performance data such as delay, operating speed, number of necessary or unnecessary lane changes/merge, lane utilization, vehicle queue length, level of service, capacity, etc. can be obtained for each exit ramp type. After capture the existing data of exit ramps, simulation software TSIS-CORSIM (Version 6) will be used to change possible variables to simulate different traffic, geometric, and control conditions. Simulation calibration and validation is conducted to meet certain level of accuracy.

8.3 Sections of Exit Ramp

The whole analysis of exit ramp included three main sections, freeway section, ramp section, and cross road section, see Figure 8.1.

Freeway section refers to the upstream section of exit ramp on freeway, whose length is 1500 ft, which is generally considered the impact distance of exit ramp.

Exit ramp section is from the start point of ramp, the painted nose, to the end of ramp, ramp terminal. If there is a left or right taper at ramp terminal, the end of ramp is the point where the taper intersects cross road.

Cross road section is started from the downstream intersection of ramp terminal to the upstream intersection of terminal. All data of these two intersections are included in this area.

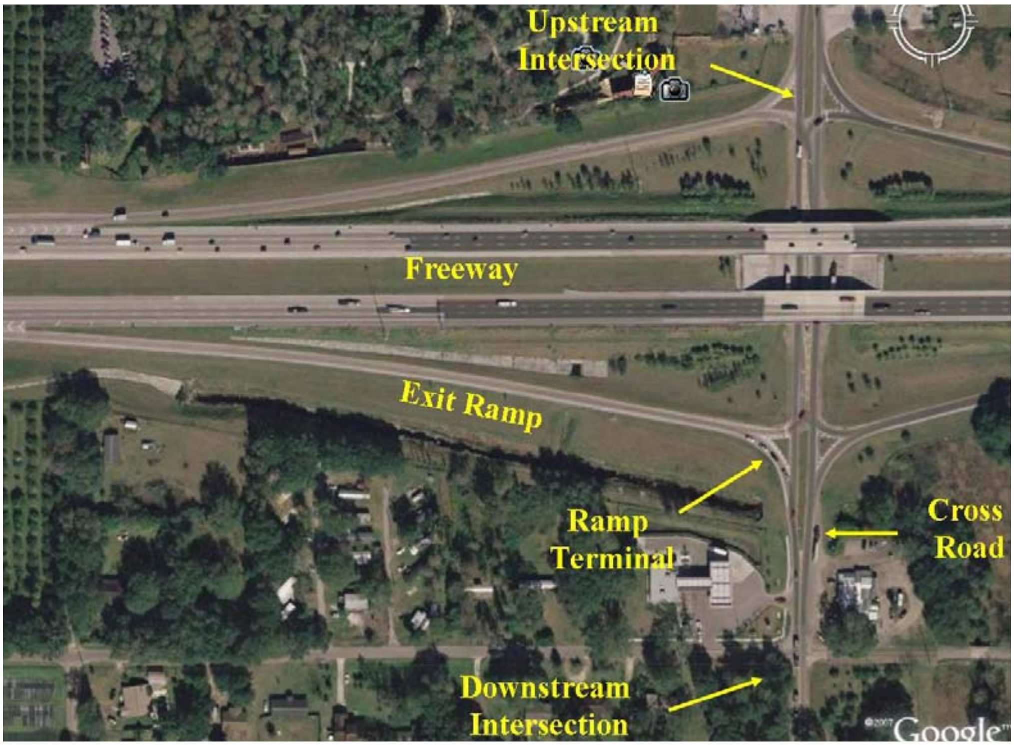


Figure 8.1 Main Sections for Analysis

8.4 Literature Review

Previous studies and findings of the operational performances on the freeway diverge areas, exit ramp and cross road sections are reviewed and summarize in this chapter. The freeway is one of the primal components of the transportation network and is categorized as the highest functional hierarchy at the highway system. The grand reliance on this facility promoted the essence of applying a much reliable, efficient and sustainable infrastructure system, thus the operational performance is obviously an important consideration in the freeway exit ramp design. Many factors related to operations on freeways and their adjacent facilities. The wide variety of site geometric conditions, traffic volumes, ramp types, and design layouts could increase or decrease the operation levels.

In 1978, Al-Kaisy, A used a simulation approach for examining capacity and operational performance at freeway diverge areas. Freeway diverge areas, and particularly those in the proximity of exit ramps, are often viewed as potential bottlenecks in freeway operations. The existing diverge procedures within the 1994 and 1997 Highway Capacity Manual updates are limited in that they do not provide a direct estimate of freeway capacity nor do they model performance at oversaturated traffic conditions. Moreover, a parallel investigation on these procedures revealed some inconsistencies in predicting measures of performance at those critical areas. This paper describes the use of computer traffic simulation to explore the patterns of capacity and operational performance behavior at these areas under the impact of some key geometric and traffic variables. For this purpose, the microscopic traffic simulation model INTEGRATION was selected to conduct an extensive experimental work on a typical ramp-freeway diverge section. Five control variables were investigated, namely, total upstream demand, off-ramp demand, length of deceleration lane, off-ramp free-flow speed, and number of lanes at mainline. The impact of upstream or downstream ramps is considered beyond the scope of this research. Except for off-ramp free-flow speed, the impact of other control variables on capacity and operational performance was shown to be significant. Also, the simulated trends of traffic behavior showed considerable agreement with logic and expectations in light of the current state of knowledge on freeway operations.

In 2000, Michael J Cassidy did a research of freeway traffic near exit ramp. He assumed freeway section near exit ramp is a bottleneck. And a bottleneck with a diminished capacity is shown to have arisen on a freeway segment whenever queues from the segment's off-ramp spilled-over and occupied its mandatory exit lane. Although the ramp' s queues were confined to the right-most exit lane, non-exiting drivers reduced their speeds upon seeing these queues and this diminished flows in all lanes. It is also shown that the lengths of these exit queues were negatively correlated with the discharge flows in the freeway segment's adjacent lanes; i.e., longer exit queues from the over-saturated off-ramp were accompanied by lower discharge rates for the non-exiting vehicles. Whenever the off-ramp queues were prevented from spilling-over to the exit lane (by changing the logic of a nearby traffic signal), much higher flows were sustained on the freeway segment and a bottleneck did not arise there. These observations underscore the value of control strategies that enable diverging vehicles to exit a freeway unimpeded.

In 1998, G.F. Newell studied the delays caused by a queue at a freeway exit ramp. It happens to traffic on a freeway when a queue from an exit ramp backs onto the freeway causing a partial blockage of the right lane. Exiting vehicles are confined to the right lane but through vehicles can travel in any lane. The two vehicle types interact but their queues must be treated separately. This illustrates a special case of a model of ``freeways with special lanes'' formulated by Daganzo and Munoz (2000). Whereas Daganzo and Munoz presented a numerical scheme of calculating flows, the emphasis here is on graphical evaluation of the complete evolution of the queues. The graphical solution more clearly illustrates the practical issues.

A Synthesis of Highway Practice Report, in 1976, focused on the design and control of freeway off-ramp terminals. A more successful design and operating practices used at freeway exit-ramp terminals and concludes that design of exit ramps should be related to both the freeway and the crossroad. Grades should be as flat as possible and, where possible, the entire ramp should be visible from the freeway exit. The ramp should have a relatively flat platform at the intersection with the crossroad. Adequate stopping sight distance must be provided throughout the length of the ramp, and enough sight distance is needed at the intersection to allow for safe turns.

In 2007, Xiao, Zhongbin, he studied the minimum-length-requirement model for expressway off-ramp joint. To augment the capacity of off-ramp joint, a method to calculate its length is needed. With the definition and basic hypothesis of off-ramp joint, the characteristic of its structure and traffic flow are analyzed. From a systematic viewpoint, kinematics, gap-acceptance theory and probability theory are employed to establish the minimum-length-requirement model for expressway off-ramp joint. While modeling, the more difficult traffic maneuver of running off the off-ramp road, finishing its interweaving and running onto the left-turn lane of downstream intersection are taken into consideration comprehensively. For a newly constructed road, the required minimum length can be computed using the model. For an existing road, based on the comparison of the measured value and calculated value, the model is helpful for finding out the reasons of congestion on the off-ramp joint, and taking corresponding improvement measures. Finally, the model is verified to be feasible through comparison with the simulation results of TSIS-CORSIM (corridor simulation model).

In 2007, Li, Hong-Ping, did research about factors influencing free flow speed on expressway. In order to research the pattern of the free flow speed (FFS) on the expressway, the measured FFS, the theoretical FFS and the 85 percentile speed and their correlation were analyzed statistically using the traffic data acquired by the loop vehicle detectors buried in the expressway in Shanghai. The attention was focused on the measure FFS, and the regression models between it and the radius of the horizontal curve, between it and the distance to the inlet or from the exit ramp, and between it and the traffic saturation degree. On this basis, a model was presented to estimate the FFS on the expressway without the need of the field data, providing a base for evaluating the service level of the expressway operation system and estimating its traffic flow capacity.

In 2003, Bunker, Jonathan, predicted minor stream delays at a limited priority freeway merge.

He talked about the development and application of a limited priority gap acceptance model to freeway merging. In the limited priority model, drivers in the major stream at a merge area may incur delay in restoring small headways to a larger, sustainable minimum headway between them and the vehicle in front. This allows minor stream drivers to accept smaller gaps. The headway distributions are assumed to be distributed according to Cowan's M3 model, whose terms were calibrated for this system. Minor stream minimum follow-on time was calibrated, and a realistic range of the critical gap identified. An equation was developed for minimum average minor stream delay.

A function was identified to model the relationship between minor stream average delay and degree of saturation. The shape parameter of this function was calibrated using simulated traffic flow data, under three different minor stream arrival pattern regimes. The model provides a useful means of comparing performance, through average minor stream delay, for varying minor and major stream flow rates and minor stream critical gap, under arrival patterns that differ due to traffic control upstream of the on-ramp. Minor stream delay is a particularly useful measure of effectiveness for uncongested freeway merging as it relates directly to the distance required to merge. Observations from the model developed provide physical evidence that minor stream drivers incur lesser delay, or have a better chance of merging quickly, when they arrive at constant intervals as is the case under constant departure ramp metering, than when they arrive in bunches downstream of a signalized intersection, or even a semi-bunched state downstream of an unsignalized intersection.

In 2008, Zhou, Huaguo, developed a methodology to evaluate the effects of access control near freeway interchange areas. Access connections and signalized intersections within the functional area of an interchange can adversely impact safety and operations at the interchange crossroad and on the freeway, and can cause the interchange to fail prematurely. Standard practice is to acquire a minimum of 90 m (300 ft) of limited access right-of-way beyond the end of the acceleration/deceleration lanes for rural interchanges and 30 m (100 ft) in urban areas.

Although the safety and operational benefits of managing access in freeway interchange influence areas are clear, the cost effectiveness of purchasing access rights at the time of interchange construction has not been established through national or state-level research. The primary objective of this study was to assess the relative costs and benefits of purchasing additional limited access right-of-way at the time of construction in lieu of retrofitting interchange areas after functional failure.

The study methodology included the following basic steps: (1) traffic operations analysis of the study interchange with varying configurations of signalized access spacing using TSIS-CORSIM; (2) safety analysis of a sample of Florida interchanges with varied access spacing; and (3) cost/benefit analysis of acquiring varying amounts of limited access right-of-way. This study indicates that the long-term safety, operation, and fiscal benefits of purchasing additional limited access right-of-way at interchange areas greatly exceed the initial costs. The findings suggest that state transportation agencies and the traveling public may benefit greatly by an increase in the amount of limited access right-of-way at interchange areas to a minimum of 180 m (600 ft) and a desirable 400 m (1,320 ft).

CHAPTER 9 DATA COLLECTION

This chapter mainly describes information about field data collection, including sites selection, data collection equipments, data collection procedures, and data reduction.

9.1 Sites Selection

There are thirteen sites total being selected for data collection in the State of Florida. The selection criteria for all these sites were based on the discussions of FDOT project officials and USF researchers, which should meet some requirements as following:

(1) All these sites are freeway interchanges in central Florida;

(2) All these sites are representative and typical ones in central Florida;

(3) All these sites should cover the four different types of exit ramps; and

(4) All these sites serve high traffic volume at peak hour.

Table 9.1 shows the locations and area of these thirteen sites, all sites are located at Tampa Bay area and Orlando area, in central Florida. And Figure 9.1 tells the exact scatter grams of observing sites on the map of Florida. Generally, each completed interchange contains two exit ramps on the two opposite sides, and only two interchanges have one exit ramp. Thus, there are 24 exit ramps totally for the 13 sites. Table 9.2 shows the 24 exit ramps with detailed classifications of ramp types.

Table 9.1 Lists of 13 Observing Sites in Florida

No.	Location	Area
1	I-75 at State Road 56	Tampa
2	I-4 at County Road 579	Tampa
3	I-275 at Hillsborough Avenue	Tampa
4	I-75 at I-4	Tampa
5	I-275 at Ulmerton Road	Saint Petersburg
6	I-275 at 4th Street	Saint Petersburg
7	I-75 at Fowler Avenue	Tampa
8	I-4 at Universal Boulevard	Orlando
9	I-4 at Conroy Road	Orlando
10	I-4 at Lee Road	Orlando
11	I-4 at Altamonte Drive	Orlando
12	I-4 at State Road 434	Orlando
13	I-75 at County Road 581 (Bruce B. Downs Boulevard)	Tampa

Table 9.2 List of 24 Exit Ramps with Classification of Ramp Type

Ramp No.	Ramp Type	Ramp Location	Ramp Direction	Number of Through Lanes on Freeway	Number of Lanes on Ramp
1	I	I-75 at SR 56	SB	2	1
2	I	I-4 at CR 579	WB	3	1
3	I	I-275 at Hillsborough Ave	NB	3	1
4	I	I-275 at Hillsborough Ave	SB	3	1
5	I	I-75 at I-4	SB	3	1
6	I	I-275 at 4th St	SB	4	1
7	I	I-4 at Universal Blvd	SB	3	1
8	I	I-75 at CR 581 (BBD)	SB	2	1
9	II	I-75 at Fowler Ave	SB	3	1
10	II	I-4 at Lee Rd	NB	4	1
11	II	I-4 at Lee Rd	SB	4	1
12	II	I-4 at SR 434	SB	4	1
13	III	I-75 at SR 56	NB	4	2
14	III	I-4 at CR 579	EB	4	2
15	III	I-4 at Universal Blvd	NB	4	2
16	III	I-4 at Conroy Rd	NB	5	2
17	III	I-4 at Conroy Rd	SB	5	2
18	III	I-4 at Altamonte Dr	NB	4	2
19	III	I-4 at SR 434	NB	4	2
20	III	I-4 at Altamonte Dr	SB	4	2
21	III	I-75 at CR 581 (BBD)	NB	3	2
22	IV	I-75 at I-4	NB	4	2
23	IV	I-275 at Ulmerton Rd	SB	4	2
24	IV	I-75 at Fowler Ave	NB	3	2

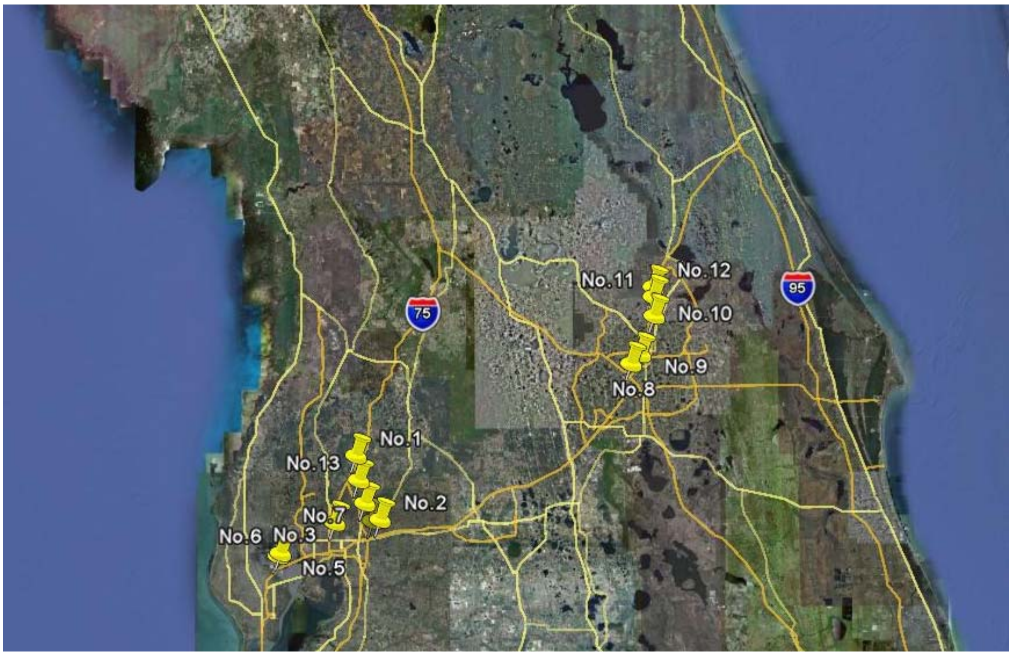


Figure 9.1 Scatter Grams of 13 Observing Sites in Florida

9.2 Data Collection Equipments

Several equipments were used for field data collection, including video camera, traffic counter, radar gun, stop watch, traffic cones and etc. Detailed information is shown as follows:

Video Camera – to capture traffic volume and number of vehicles in queue;

Traffic Counter – to assist video camera;

Radar Gun – to detect operating speed on roadway;

Stop Watch – to obtain timing plan for intersections;

Traffic Cones – to set a safety zone at roadside for all observers and equipments;

Rough Measurer – to measure geometry dimension; and

Flash Coat – to protect observers by reminding other drivers.

(a) Video Camera with Stand

(b) Use of Video Camera in Data Collection

(c) Traffic Counter

(d) Use of Traffic Counter in Data Collection

(e) Radar Gun

(f) Use of Radar Gun in Data Collection

(g) Stop Watch

(h) Traffic Cone

(i) Rough Measurer

(j) Flash Coat

Figure 9.2 Data Collection Equipments

9.3 Data Collection Procedures

Data collection was divided into three sections, freeway section, exit ramp section, and cross road section. Several kinds of data were collected for these three sections, such as traffic volume, heavy vehicles (%), operation speed, signal timing plan, number of lane change, number of lanes, turn lane assignment, and etc. All the data were collected at peak hour, in order to capture the high volume situation of operation. The peak hour time extended to two hours for both morning and afternoon peak (7:00 – 9:00 am, and 4:00 – 6:00 pm), because of the long time of observation. And, based on some data already gained, the range of peak hour time is proper due to the relatively constant traffic.

For freeway section, hourly traffic volume of each lane was collected by video camera with ratio of heavy vehicles, and operation speed was collected by radar gun. And number of lane change was also captured by video camera, in the 1500 ft upstream section of exit ramp.

For ramp section, besides hourly traffic volume of each lane, timing plan for ramp terminal, and queuing length for each lane at each approach was also captured.

For cross road section, data collection was mainly focused on upstream and downstream intersection. All traffic data (volume, assignment and etc.) and timing data were collected, as most intersections were signalized. Radar gun was used to detect operational speed of all approaches on cross road.

Google Earth was used to collect geometric data, including number of lanes, turn bays at intersections, lane width, curvature, median, channelized island, and etc. Table 9.3 and Figure 9.3 demonstrate the comprehensive method of data collection.

Some signalized intersections were actuating control, whose timing plan were affected by traffic volume and might vary at each cycle. And it is hard to get the actuating timing plan from observation, because it depends on some values, such as minimal initial time, minimal crossing time, and etc., which are difficult to know. An assumption was made to simplify the observation and give a reasonable result, set pre-timed signalized control for these intersections by using the average timing plan from actuating signal. This method had been testified due to some field data. The split time for each phase was pretty close because of the relevantly constant traffic at peak hour.

Table 9.3 Time Period and Method for All Data Collection

Observing Time	Parameters	Methods
7:00 to 8:00 am or 5:00 to 6:00 pm	Hourly volume of each lane and total HV ratio in freeway	Counted by observer
	Number of lane change in freeway in front of painted nose of exit ramp	Counted by observer
	Hourly volume of each lane and total HV ratio in ramp terminal	By video camera
	Queuing length of each lane in ramp terminal	By video camera
	Signal timing and phasing in ramp terminal	Read by observer using timer
	Speed in freeway and ramp	By radar gun
8:00 to 9:00 am or 6:00 to 7:00 pm	Hourly volume of each lane and total HV ratio of each approach (downstream intersection)	By video camera
	Queuing length of each lane in each approach (downstream intersection)	By video camera
	Signal timing and phasing in downstream intersection	Read by observer using timer
	Hourly volume of each lane and total HV ratio of each approach (upstream intersection)	By video camera
	Queuing length of each lane in each approach (upstream intersection)	By video camera
	Signal timing and phasing in upstream intersection	Read by observer using timer
	Speed in downstream and upstream intersection	By radar gun

Figure 9.3 Location of Device for Data Collection

9.4 Data Reduction

After data collection is finished, data reduction is conducted. All video camera data are read and transferred to computer, timing data are calculated, and data recorded on paper are input to computer too. Following tables show general data for each observing site, which includes two directions (NB and SB or EB and WB).

Table 9.4 I-4 at Conroy Road (NB)

Freeway				
Basic number of lane	4	Volume of ramp	1038	
Volume of each lane (from left to right)	1137、1296、954、486	Exit ramp type	III	
Number of lane on ramp	2	Number of lane change on freeway	72	
Ramp Terminal				
Traffic Volume				
EB: 1083	WB: 801	SB: 0	NB: 1038	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB	16	3	1
2	EB and WB	36	3	1
Downstream Intersection				
Traffic Volume				
EB: 393	WB: 498	SB: 978	NB: 708	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB and WB left	14	3	1
2	EB and WB thru	14	3	1
3	SB	26	3	1
4	SB and NB thru	24	3	1
5	NB	44	3	1
Upstream Intersection				
Traffic Volume				
EB: 974	WB: 1043	SB: 603	NB: 845	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB and WB left	10	3	1
2	EB and WB thru	12	3	1
3	SB	15	3	1
4	SB and NB thru	26	3	1
5	NB	26	3	1

Table 9.5 I-4 at Conroy Road (SB)

Freeway				
Basic number of lane	5	Volume of ramp	1052	
Volume of each lane (from left to right)	1892,1594,1078,696,740	Exit ramp type	III	
Number of lane on ramp	2	Number of lane change on freeway	216	
Ramp Terminal				
Traffic Volume				
EB: 2118	WB: 2367	SB: 1052	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB	12	3	1
2	EB and WB thru	40	3	1
Downstream Intersection				
Traffic Volume				
EB: 1083	WB: 1287	SB: 705	NB: 1116	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB and WB left	11	3	1
2	EB and WB thru	16	3	1
3	SB	17	3	1
4	SB and NB thru	34	3	1
5	NB	20	3	1
Upstream Intersection				
Traffic Volume				
EB: 1947	WB: 1578	SB: 0	NB: 1874	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB	34	3	1
2	EB left and thru	31	3	1
3	EB and WB thru	42	3	1

Table 9.6 I-4 at Altamont Drive (SB)

Freeway				
Basic number of lane	3	Volume of ramp	645	
Volume of each lane (from left to right)	1284,1338,1257	Exit ramp type	III	
Number of lane on ramp	2	Number of lane change on freeway	36	
Ramp Terminal				
Traffic Volume				
EB: 1629	WB: 2271	SB: 396	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB	35	3	1
2	WB thru and left	36	3	1
3	EB and WB thru	60	3	1
Downstream Intersection				
Traffic Volume				
EB: 1578	WB: 1908	SB: 441	NB: 528	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	WB	17	3	1
2	EB and WB thru	36	3	1
3	EB	19	3	1
4	SB and NB left	17	3	1
5	SB and NB thru	21	3	1
Upstream Intersection				
Traffic Volume				
EB: 1776	WB: 1668	SB: 0	NB: 1008	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB	32	3	1
2	EB and WB thru	71	3	1
3	EB thru and left	28	3	1

Table 9.7 I-4 at Altamont Drive (NB)

Freeway				
Basic number of lane	3	Volume of ramp	1098	
Volume of each lane (from left to right)	1710,1716,882	Exit ramp type	III	
Number of lane on ramp	2	Number of lane change on freeway	78	
Ramp Terminal				
Traffic Volume				
EB: 2655	WB: 3669	SB: 0	NB: 1728	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB	42	3	1
2	EB and WB thru	76	3	1
3	EB thru and left	33	3	1
Downstream Intersection				
Traffic Volume				
EB: 2258	WB: 1923	SB: 372	NB: 477	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB thru and left	27	3	1
2	WB thru and left	43	3	1
3	EB and WB left	20	3	1
4	SB	20	3	1
5	NB	20	3	1
Upstream Intersection				
Traffic Volume				
EB: 1878	WB: 2133	SB: 714	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB	24	3	1
2	WB thru and left	34	3	1
3	EB and WB thru	81	3	1

Table 9.8 I-275 at 4th Street (SB)

Freeway			
Basic number of lane	4	Volume of ramp	332
Volume of each lane (from left to right)	837,868,1185,965	Exit ramp type	II
Number of lane on ramp	1	Number of lane change on freeway	126
Ramp Terminal			
Traffic Volume			
EB: 47	WB: 43	SB: 298	NB: 552
Timing and Phasing (This intersection is yield controlled, SB and NB approaches belong to main road, and EB and WB approaches belong to minor road.)			
Downstream Intersection			
Traffic Volume			
EB: 38	WB: 27	SB: 261	NB: 487
Timing and Phasing (This intersection is yield controlled, SB and NB approaches belong to main road, and EB and WB approaches belong to minor road.)			

Table 9.9 I-75 at SR 56 (SB)

Freeway				
Basic number of lane	2	Volume of ramp	731	
Volume of each lane (from left to right)	873,767	Exit ramp type	II	
Number of lane on ramp	1	Number of lane change on freeway	38	
Ramp Terminal				
Traffic Volume				
EB: 674	WB: 1097	SB: 719	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB	24	3	1
2	WB left	16	3	1
3	EB and WB thru	30	3	1
Downstream Intersection				
Traffic Volume				
EB: 1095	WB: 993	SB: 734	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB left	29	3	1
2	EB left	17	3	1
3	EB and WB thru	31	3	1
Upstream Intersection				
Traffic Volume				
EB: 737	WB: 972	SB: 0	NB: 530	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB	21	3	1
2	EB left	17	3	1
3	EB and WB thru	27	3	1

Table 9.10 I-75 at SR 56 (NB)

Freeway				
Basic number of lane	4	Volume of ramp	1056	
Volume of each lane (from left to right)	1001,876,831,1113	Exit ramp type	III	
Number of lane on ramp	2	Number of lane change on freeway	103	
Ramp Terminal				
Traffic Volume				
EB: 978	WB: 1421	SB: 0	NB: 996	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB	26	3	1
2	EB left	15	3	1
3	EB and WB thru	39	3	1
Downstream Intersection				
Traffic Volume				
EB: 871	WB: 1341	SB: 16	NB: 23	
Timing and Phasing (This intersection is yield controlled, EB and WB approaches belong to main road, and SB and NB approaches belong to minor road.)				
Upstream Intersection				
Traffic Volume				
EB: 1021	WB: 1209	SB: 767	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB	24	3	1
2	WB left	21	3	1
3	EB and WB thru	36	3	1

Table 9.11 I-4 at CR 579 (WB)

Freeway				
Basic number of lane	3	Volume of Ramp	983	
Volume of each lane (from left to right)	330,687,240	Exit ramp type	II	
Number of lane on ramp	1	Number of lane change on freeway	46	
Ramp Terminal				
Traffic Volume				
EB: 0	WB: 945	SB: 1250	NB: 1876	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	WB	20	3	1
2	NB and SB thru	33	3	1
3	NB thru and left	18	3	1
Downstream Intersection				
Traffic Volume				
EB: 331	WB: 64	SB: 1654	NB: 634	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB and SB left	12	3	1
2	NB and SB thru	24	3	1
3	EB and WB	16	3	1
Upstream Intersection				
Traffic Volume				
EB: 1184	WB: 0	SB: 1342	NB: 1653	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB	23	3	1
2	NB and SB thru	38	3	1
3	NB thru and left	14	3	1

Table 9.12 I-4 at CR 579 (EB)

Freeway				
Basic number of lane	4	Volume of Ramp	1140	
Volume of each lane (from left to right)	870,934,656,1240	Exit ramp type	III	
Number of lane on ramp	2	Number of lane change on freeway	87	
Ramp Terminal				
Traffic Volume				
EB: 1089	WB: 0	SB: 1243	NB: 1709	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB	28	3	1
2	NB and SB thru	42	3	1
3	NB thru and left	21	3	1
Downstream Intersection				
Traffic Volume				
EB: 351	WB: 478	SB: 1457	NB: 960	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB and WB	26	3	1
2	NB and SB left	19	3	1
3	NB and SB thru	37	3	1
Upstream Intersection				
Traffic Volume				
EB: 0	WB: 670	SB: 813	NB: 1534	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	WB	23	3	1
2	NB and SB thru	35	3	1
3	NB thru and left	27	3	1

Table 9.13 I-275 at Ulmerton Road (SB)

Freeway			
Basic number of lane	3	Volume of each ramp	1784
Volume of each lane (from left to right)	654,886,704	Exit ramp type	II
Number of lane on ramp	2	Number of lane change on freeway	57
Ramp Terminal[1]			
Traffic Volume			
EB: 1194	WB: 1542	SB: 0	NB: 15
Timing and Phasing (This intersection is yield controlled; EB and WB approaches belong to main road, and NB approach belongs to minor road.)			
Downstream Intersection[1]			
Traffic Volume			
EB: 1023	WB: 1439	SB: 16	NB: 23
Timing and Phasing (This intersection is yield controlled, EB and WB approaches belong to main road, and SB and NB approaches belong to minor road.)			
Ramp Terminal[2]			
Traffic Volume			
EB: 354	WB: 0	SB: 363	NB: 225
Timing and Phasing (This intersection is yield controlled, NB and SB approaches belong to main road, and EB approach belongs to minor road.)			
Downstream Intersection[2]			
Traffic Volume			
EB: 379	WB: 0	SB: 371	NB: 209
Timing and Phasing (This intersection is yield controlled, NB and SB approaches belong to main road, and EB approach belongs to minor road.)			

Table 9.14 I-4 at SR 434 (SB)

Freeway				
Basic number of lane	3	Volume of ramp	1103	
Volume of each lane (from left to right)	1764, 1572, 769	Exit ramp type	I	
Number of lane on ramp	1	Number of lane change on freeway	76	
Ramp Terminal				
Traffic Volume				
EB: 1789	WB: 1702	SB: 1021	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB left	28	3	1
2	WB thru & left	46	3	1
3	EB and WB thru	147	3	1
Downstream Intersection				
Traffic Volume				
EB: 1346	WB: 1156	SB: 346	NB: 451	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB and WB thru	24	3	1
2	EB thru & left	42	3	1
3	WB thru & left	19	3	1
4	SB and NB left	39	3	1
5	SB and NB thru	21	3	1
Upstream Intersection				
Traffic Volume				
EB: 1453	WB: 1134	SB: 0	NB: 987	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB left	22	3	1
2	EB and WB thru	46	3	1
3	NB left	19	3	1
4	EB and WB left	37	3	1

Table 9.15 I-4 at SR 434 (NB)

<table>
<tr><td colspan="5">Freeway</td></tr>
<tr><td>Basic number of lane</td><td>3</td><td>Volume of ramp</td><td colspan="2">1011</td></tr>
<tr><td>Volume of each lane (from left to right)</td><td>2184, 1752, 735</td><td>Exit ramp type</td><td colspan="2">III</td></tr>
<tr><td>Number of lane on ramp</td><td>2</td><td>Number of lane change on freeway</td><td colspan="2">96</td></tr>
<tr><td colspan="5">Ramp Terminal</td></tr>
<tr><td colspan="5">Traffic Volume</td></tr>
<tr><td>EB: 1944</td><td>WB: 1647</td><td>SB: 0</td><td colspan="2">NB: 1164</td></tr>
<tr><td colspan="5">Timing and Phasing</td></tr>
<tr><td>Phase</td><td>Maneuver</td><td>Green (s)</td><td>Yellow (s)</td><td>All Red (s)</td></tr>
<tr><td>1</td><td>NB left</td><td>24</td><td>3</td><td>1</td></tr>
<tr><td>2</td><td>EB and WB thru</td><td>50</td><td>3</td><td>1</td></tr>
<tr><td>3</td><td>NB left</td><td>21</td><td>3</td><td>1</td></tr>
<tr><td>4</td><td>EB and WB left</td><td>41</td><td>3</td><td>1</td></tr>
<tr><td colspan="5">Downstream Intersection</td></tr>
<tr><td colspan="5">Traffic Volume</td></tr>
<tr><td>EB: 1767</td><td>WB: 1575</td><td>SB: 198</td><td colspan="2">NB: 798</td></tr>
<tr><td colspan="5">Timing and Phasing</td></tr>
<tr><td>1</td><td>SB left</td><td>24</td><td>3</td><td>1</td></tr>
<tr><td>2</td><td>WB thru & left</td><td>47</td><td>3</td><td>1</td></tr>
<tr><td>3</td><td>SB left</td><td>22</td><td>3</td><td>1</td></tr>
<tr><td>4</td><td>EB and WB thru</td><td>146</td><td>3</td><td>1</td></tr>
<tr><td colspan="5">Upstream Intersection</td></tr>
<tr><td colspan="5">Traffic Volume</td></tr>
<tr><td>EB: 1797</td><td>WB: 1692</td><td>SB: 879</td><td colspan="2">NB: 0</td></tr>
<tr><td colspan="5">Timing and Phasing</td></tr>
<tr><td>Phase</td><td>Maneuver</td><td>Green (s)</td><td>Yellow (s)</td><td>All Red (s)</td></tr>
<tr><td>1</td><td>EB and WB thru</td><td>85</td><td>3</td><td>1</td></tr>
<tr><td>2</td><td>NB thru & left</td><td>32</td><td>3</td><td>1</td></tr>
<tr><td>3</td><td>EB and WB left</td><td>17</td><td>3</td><td>1</td></tr>
<tr><td>4</td><td>SB thru & left</td><td>17</td><td>3</td><td>1</td></tr>
<tr><td>5</td><td>EB thru & left</td><td>15</td><td>3</td><td>1</td></tr>
</table>

Table 9.16 I-75 at Fowler (SB)

Freeway				
Basic number of lane	2	Volume of Ramp	1057	
Volume of each lane (from left to right)	1765, 1457	Exit ramp type	IV	
Number of lane on ramp	2	Number of lane change on freeway	87	
Ramp Terminal				
Traffic Volume				
EB: 1579	WB: 1764	SB: 667	NB: 0	
Timing and Phasing				
Ramp terminal is yield control.				
Downstream Intersection				
Traffic Volume				
EB: 1701	WB: 1879	SB: 430	NB: 391	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB and WB left	29	3	1
2	EB and WB thru	120	3	1
3	NB and SB left	20	3	1
4	NB and SB thru	33	3	1
Upstream Intersection				
Traffic Volume				
EB: 1684	WB: 1760	SB: 0	NB: 572	
Timing and Phasing				
Upstream intersection is yield control.				

Table 9.17 I-75 at Fowler (NB)

Freeway			
Basic number of lane	2	Volume of Ramp	998
Volume of each lane (from left to right)	1543, 1321	Exit ramp type	IV
Number of lane on ramp	2	Number of lane change on freeway	75
Ramp Terminal			
Traffic Volume			
EB: 1589	WB: 1549	SB: 0	NB: 754
Timing and Phasing			
Ramp terminal is yield control.			
Downstream Intersection			
Traffic Volume			
EB: 1356	WB: 1305	SB: 75	NB: 0
Timing and Phasing			
Downstream intersection is yield control.			
Upstream Intersection			
Traffic Volume			
EB: 1621	WB: 1678	SB: 574	NB: 0
Timing and Phasing			
Upstream intersection is yield control.			

Table 9.18 I-4 at I-75 (SB)

Freeway			
Basic number of lane	2	Volume of ramp	773
Volume of each lane (from left to right)	1543, 1059	Exit ramp type	I
Number of lane on ramp	1	Number of lane change on freeway	56
Ramp Terminal			
Traffic Volume			
EB: 1734	WB: 1521	SB: 773	NB: 0
Timing and Phasing			
Ramp terminal is yield control.			
Downstream Intersection			
Traffic Volume			
EB: 1712	WB: 1671	SB: 346	NB: 0
Timing and Phasing			
Downstream intersection is yield control.			
Upstream Intersection			
Traffic Volume			
EB: 1653	WB: 1534	SB: 0	NB: 549
Timing and Phasing			
Upstream intersection is yield control.			

Table 9.19 I-4 at I-75 (NB)

Freeway			
Basic number of lane	3	Volume of ramp	1214
Volume of each lane (from left to right)	1987, 1552, 741	Exit ramp type	IV
Number of lane on ramp	2	Number of lane change on freeway	121
Ramp Terminal			
Traffic Volume			
EB: 1744	WB: 1529	SB: 0	NB: 621
Timing and Phasing			
Ramp terminal is yield control.			
Downstream Intersection			
Downstream intersection is another ramp of freeway, not the cross street. Furthermore, the distance is about 4750 feet, which exceeds ramp influence distance of 1500 feet. Therefore, ignore existence of downstream intersection.			
Upstream Intersection			
Traffic Volume			
EB: 1697	WB: 1492	SB: 679	NB: 0
Timing and Phasing			
Upstream intersection is yield control.			

Table 9.20 at Hillsborough Avenue (SB)

Freeway				
Basic number of lane	4	Volume of Ramp	831	
Volume of each lane (from left to right)	1721, 1636, 1201, 698	Exit ramp type	II	
Number of lane on ramp	1	Number of lane change on freeway	97	
Ramp Terminal				
Traffic Volume				
EB: 1235	WB: 1198	SB: 827	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB left	22	3	1
2	WB thru & left	16	3	1
3	EB and WB thru	32	3	1
Downstream Intersection				
Traffic Volume				
EB: 1301	WB: 1279	SB: 730	NB: 491	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB and WB left	17	3	1
2	EB and WB thru	49	3	1
3	NB and SB left	13	3	1
4	NB and SB thru	16	3	1
Upstream Intersection				
Traffic Volume				
EB: 1284	WB: 1260	SB: 0	NB: 452	
Timing and Phasing				
Upstream intersection is yield control.				

Table 9.21 I-275 at Hillsborough Avenue (NB)

Freeway				
Basic number of lane	3	Volume of Ramp	547	
Volume of each lane (from left to right)	1641, 1410, 882	Exit ramp type	II	
Number of lane on ramp	1	Number of lane change on freeway	54	
Ramp Terminal				
Traffic Volume				
EB: 1389	WB: 1349	SB: 0	NB: 554	
Timing and Phasing				
Ramp terminal is yield control.				
Downstream Intersection				
Traffic Volume				
EB: 1456	WB: 1405	SB: 75	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB and WB left	21	3	1
2	EB and WB thru	65	3	1
3	SB and NB left	19	3	1
4	SB and NB thru	26	3	1
Upstream Intersection				
Traffic Volume				
EB: 1221	WB: 1378	SB: 674	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB left	23	3	1
2	WB thru & left	17	3	1
3	EB and WB thru	41	3	1

Table 9.22 I-4 at Universal Blvd. (NB)

Freeway				
Basic number of lane	4	Volume of Ramp	164 (HV 4%), 340 (HV 4%)	
Volume of each lane (from left to right)	1644 (HV 2%), 1584 (HV 3%), 1196 (HV 3%), 252 (HV 4%)	Exit ramp type	III	
Number of lane on ramp	2	Number of lane change on freeway	164	
Ramp Terminal				
Traffic Volume				
EB: 1296	WB: 0	SB: 776	NB: 694	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB left and right	39	3	1
2	SB thru and left	30	3	1
3	NB thru and right	29	3	1
Downstream Intersection				
Traffic Volume				
EB: 996	WB: 1080	SB: 780	NB: 642	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB & WB left	15	3	1
2	EB thru and left	24	3	1
3	EB & WB thru	47	3	1
4	NB & SB left	13	3	1
5	NB & SB thru	27	3	1
Upstream Intersection				
Traffic Volume				
EB: 1296	WB: 0	SB: 1476	NB: 834	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	EB	22	3	1
2	NB & SB left	19	3	1
3	NB & SB through	42	3	1

Table 9.23 I-4 at Universal Blvd. (SB)

Freeway				
Basic number of lane	3	Volume of Ramp	564 (HV 0%)	
Volume of each lane (from left to right)	1398 (HV 2%), 1404 (HV 3%), 1089 (HV 3%)	Exit ramp type	II	
Number of lane on ramp	1	Number of lane change on freeway	108	
Ramp Terminal				
Traffic Volume				
EB: 498	WB: 765	SB: 0	NB: 396	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB left and right	20	3	1
2	WB thru and left	16	3	1
3	EB & WB thru	22	3	1
Downstream Intersection				
Traffic Volume				
EB: 0	WB: 741	SB: 1050	NB: 27	
Timing and Phasing				
It is yield control. EB and WB are the major approaches, and NB is the minor approach.				
Upstream Intersection				
Traffic Volume				
EB: 648	WB: 0	SB: 738	NB: 888	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	WB thru & left	39	3	1
2	NB & SB left	6	3	1
3	SB thru & left	21	3	1
4	NB & SB thu	75	3	1

Table 9.24 I-4 at Lee Road (SB)

Freeway				
Basic number of lane	4	Volume of Ramp	894 (HV 0%)	
Volume of each lane (from left to right)	2262 (HV 0%), 1929 (HV 1%), 1626 (HV 0.5%), 966 (HV 0%)	Exit ramp type	II	
Number of lane on ramp	1	Number of lane change on freeway	54	
Ramp Terminal				
Traffic Volume				
EB: 1131	WB: 909	SB: 894	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB left	33	3	1
2	EB & WB thru	62	3	1
3	WB thru and left	35	3	1
Downstream Intersection				
Traffic Volume				
EB: 1128	WB: 1357	SB: 311	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB thru & left	16	3	1
2	WB thru and left	5	3	1
3	EB & WB thru	90	3	1
4	EB thru & left	15	3	1
Upstream Intersection				
Traffic Volume				
EB: 1485	WB: 1461	SB: 0	NB: 618	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB left	36	3	1
2	EB thru & left	51	3	1
3	EB & WB thru	41	3	1

Table 9.25 I-4 at Lee Road (NB)

Freeway				
Basic number of lane	4		Volume of Ramp	718 (HV 1%)
Volume of each lane (from left to right)	1712 (HV 1%), 1612 (HV 1%), 1872 (HV 1%), 718 (HV 1%)		Exit ramp type	II
Number of lane on ramp	1		Number of lane change on freeway	214
Ramp Terminal				
Traffic Volume				
EB: 2000	WB: 1480	SB: 0	NB: 652	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	NB left and right	15	3	1
2	EB thru and left	68	3	1
3	EB & WB thru	45	3	1
Downstream Intersection				
Traffic Volume				
EB: 1282	WB: 976	SB: 344	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB & NB left	8	3	1
2	SB & NB thru	13	3	1
3	EB thru & left	31	3	1
4	EB & WB thru	58	3	1
5	WB thru & left	13	3	1
Upstream Intersection				
Traffic Volume				
EB: 958	WB: 1024	SB: 494	NB: 0	
Timing and Phasing				
Phase	Maneuver	Green (s)	Yellow (s)	All Red (s)
1	SB left	32	3	1
2	EB & WB thru	62	3	1
3	WB thru & left	36	3	1

CHAPTER 10 SIMULATIONS AND OPERATIONAL ANALYSIS

10.1 Introduction to Simulation

All operational analysis is based on traffic simulation software TSIS-CORSIM (or just TSIS). TSIS can satisfy all requirements of this project. After data validation and calibration, variables can be changed in TSIS to simulate different traffic situations, which save much energy and time. All collected data are input to TSIS for simulation, and output data can provide analysis results for further calculation and comparison.

The Federal Highway Administration's (FHWA) Traffic Software Integrated System (TSIS) is an integrated development environment that enables users to conduct traffic operations analysis. Built using component architecture, TSIS is a toolbox that contains tools that allow the user to define and manage traffic analysis projects, define traffic networks and create inputs for traffic simulation analysis, execute traffic simulation models, and interpret the results of those models (Figure 10.1).

Figure 10.1 TSIS Interface

TSIS is microscopic traffic software with long history (see following), which guarantees reliability and practicability.

(1) Mid-1970's – UTCS-1 (Urban Traffic Control System)

(2) Mid-1980's – NETSIM

(3) Late-1980's – TRAF-NETSIM

(4) 1990 – TSIS/NETSIM

(5) 1994 – TSIS/FRESIM

(6) 1995 – TSIS/CORSIM (DOS version)

(7) 1997 – TSIS/CORSIM (Windows version)

TSIS is a complete software package, different individual tools are included. Each tool has its exclusive function. Here are 10 main components in TSIS version 6 and their use, which can help better understand how TSIS works.

TShell: TShell is the graphical user interface for the TSIS integrated development environment. It provides a Project view that enables you to manage your TSIS projects. It is also the container for the pre-configured tools and any tools that you add to the suite. See the TShell User's Guide for additional details.

TSIS Next: TSIS Next contains the same type of functionality that can be seen in the TShell, TRAFED, and TextEditor component programs. TSIS Next is a "quicker-and-easier" version of TSIS that contains specific advantages and disadvantages. Certain advanced TSIS-CORSIM applications will continue to require TShell and TRAFED. By having access to both TSIS and TSIS Next on the same computer, you can choose whichever functionality you prefer.

CORSIM: The CORSIM simulation consists of an integrated set of two microscopic simulation models (NETSIM and FRESIM) that represent the entire traffic environment as a function of time. NETSIM represents surface-street traffic and FRESIM represents freeway traffic. Microscopic simulations model the movements of individual vehicles, which include the influences of driver behavior. Thus, the effects of very detailed strategies, such as relocating bus stations or changing parking restrictions, can be studied with such models. CORSIM provides its own interface in TSIS 6 that enables you to control the simulation and the accumulation of traffic measures of effectiveness. See the CORSIM User's Guide for additional details.

TRAFED: TRAFED is a graphical user interface-based editor that allows you to easily create and edit traffic networks and simulation input for the CORSIM model. See the TRAFED User's Guide for additional details.

TRAFVU: TRAFVU (TRAF Visualization Utility) is a graphics post-processor for FHWA's CORSIM microscopic traffic simulation system. TRAFVU displays traffic networks, animates simulated traffic flow operations, animates and displays simulation output measures of effectiveness, and displays user-specified input parameters for simulated network objects. See the TRAFVU User's Guide for additional details.

TSIS Text Editor: This editor is a standard text editor that has the additional capability of "understanding" the CORSIM TRF file format. When editing a TRF file with this editor, the TShell output window displays text describing the entry field and record type at the current cursor position. Clicking a specific field description in the output window highlights the corresponding entry field in the displayed TRF file. This makes manual editing of the text file much easier than with previous text editors. See the TSIS Text Editor User's Guide for additional details.

TSIS Script Tool: The TSIS Script Tool is a combined script editor and tool for executing Visual Basic Scripts. Using the built-in TSIS interfaces, the Script Tool is a powerful mechanism for extending the functionality of the other TSIS components. Also, two scripts with this release are included. One is a multi-run script that repeatedly runs CORSIM on a test case, applying different random number seeds to each run. The other script runs CORSIM on many different test cases. See the Script Tool User's Guide for additional details.

TSIS Translator: The TSIS Translator converts TRF files for use by TRAFED. This translator also performs the reverse operation of translating the TRAFED native format (TNO) files into TRF files for use by CORSIM and other tools. See the Translator User's Guide for additional details.

TSIS Output Processor: The TSIS Output Processor enables the user to automatically compute selected statistics and summary data during multiple runs of CORSIM. The collected data is written to an Excel workbook, a comma-separated file, an XMLtagged file, or a tab-separated text file. The Output Processor can also compute 95th percentile confidence intervals, and can recommend sample sizes (i.e., the number of simulation runs that should be performed with varying random number seeds) for achieving desired accuracy.

The Output Processor has been redesigned for TSIS 6 to efficiently summarize any model result generated by CORSIM. Cumulative MOEs may be obtained from the start of simulation, or just for the current time interval, or just for the current time period, or any combination of those three.

CORSIM Runtime Extension (RTE): Although it comes pre-configured with a set of tools, TSIS provides a mechanism by which an external application can interface directly with CORSIM simulation. This type of application has become known as a CORSIM run-time extension (RTE). Run-time extensions can be built to replace existing logic in CORSIM, or to supplement the logic. The original run-time extensions were tailored for signal timing studies. However, the concept has been expanded to support freeway monitoring, incident detection and ramp metering run-time extension packages.

TSIS-CORSIM has a very strong capability of many applications, some of these are mentioned here which are related to this project: Freeway and surface street interchanges, Signal timing and signal coordination, Freeway weaving sections, lane adds and lane drops, Ramp metering and HOV lanes, Queuing studies involving turn pockets and queue blockage, and etc.

TSIS-CORSIM combines two of the most widely used traffic simulation models, NETSIM for surface streets, and FRESIM for freeways. FRESIM is mainly for freeway system, and NETSIM is for roadways other than freeway.

Thus, in this project, the NETSIM can be used to build up cross road and part of exit ramp, and FRESIM can be used to build up freeway and part of exit ramp. Also, CORSIM can put them together in one network. Figure 10.2 shows a network example combined NETSIM and FRESIM in TSIS.

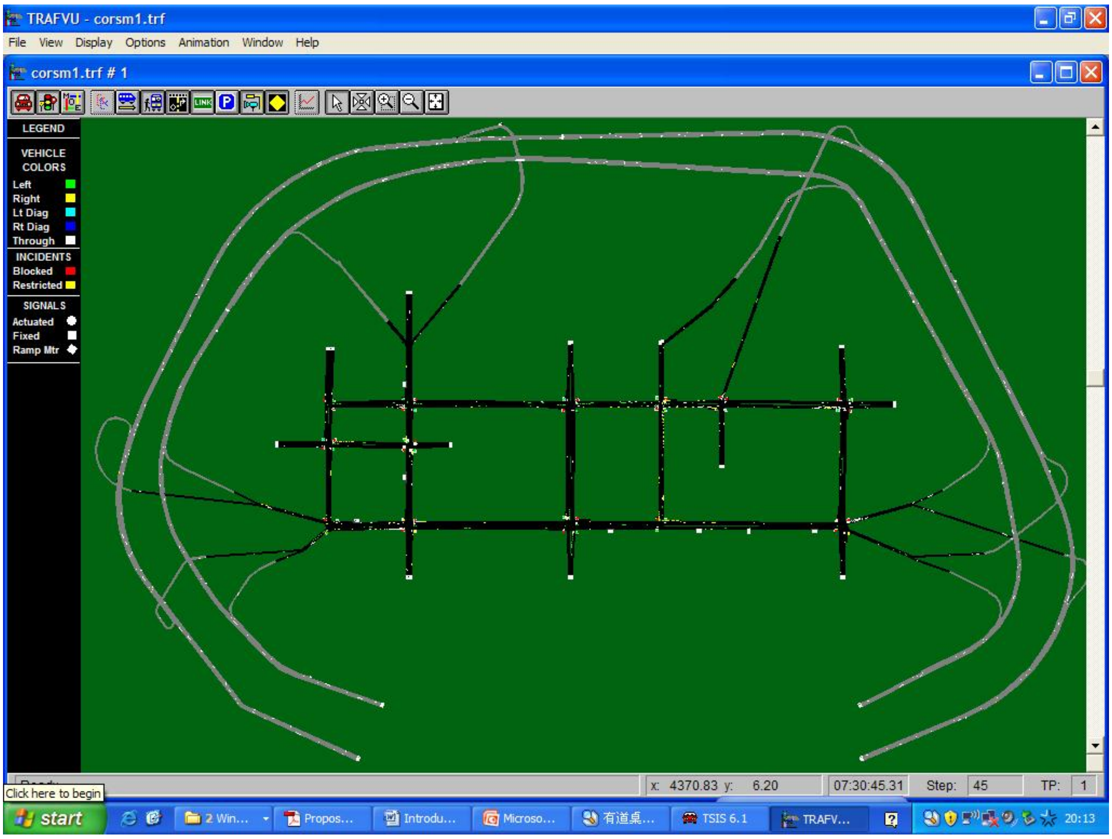


Figure 10.2 NETSIM and FRESIM in TSIS

10.2 Procedures of Simulation

There are several typical steps for a complete TSIS simulation application:

Step 1: Geometry data input. This step includes nodes, links, frameworks, property of node and link. Detailed factors are lane assignment, length, width, grade, curve, median, sign, mark and etc.

Step 2: Traffic data input. This step mainly inputs traffic volume and related data, such as hourly volume, heavy vehicle, bicycle, pedestrian, bus, bus station, and etc. not only total volume needs to be input, but also volume for each turning direction should be indicated.

Step 3: Traffic control data input. This step tells TSIS the type of traffic control. Normally, signalized control is used for intersections at ramp terminal, downstream intersection or upstream intersection. Even some intersections are actuating control, they are considered as pre-time control intersections. Timing and phasing data are observed during peak hours, which keep them stay constant.

Step 4: Simulation running. After all data accomplished, TSIS will start running, during this step, all warnings and errors can be stated, which indicate necessary correction.

Step 5: Data output: TSIS can produce a report of all MOEs, tables and charts. Useful data are selected for further analysis.

Step 6: Calibration: Some MOEs will be selected for calibration, such as queuing length at intersection approach, acceleration/deceleration rate, start-up lost time, car following sensitivity factor, and etc. TSIS output data and field data are compared to make sure the errors are under control. A 15% difference is set as standard. This step assures accuracy of whole simulation.

Step 7: Modeling: After data calibration is passed, useful data are chosen for mathematical modeling, presenting relationships among variables.

10.3 Methods for Operational Analysis

Whole network of each observing site in TSIS is divided into three sections: freeway section, exit ramp section and cross road section. These sections will be separated for further analysis. And different MOEs will be presented to evaluate performance for each section.

10.3.1 Freeway Section

In the freeway section, the main task is to find out whether the impacts of different exit ramps are significantly different bases on operational analysis. If the impacts are different, there is a need to select an optimal one under certain conditions.

Based on previous study and some data collection, number of lane change, standard deviation (S.D.) of speed, and control delay are considered the measure of effectiveness for operational performance evaluation.

(1) Number of Lane Change

Number of lane change is the total number that vehicles changing lane in the freeway upstream section (1500 ft before exit ramp) within one hour, see Figure 10.3.

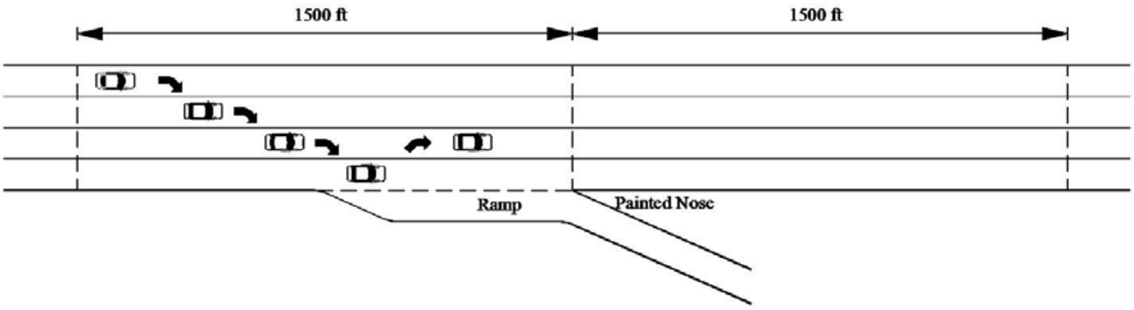


Figure 10.3 Number of Lane Change

Number of lane change is a significant factor that impacts operational performance on freeway section adjacent to exit ramp. And this change is mainly caused by exiting traffic to ramp. The larger number of lane change, the worse operational performance on freeway.

One kind of change is the exiting vehicles change lane from left side through lane to right side ramp, which is called mandatory lane change. The other kind of lane change happens between through lanes, just to find the better driving environment, which is optional lane change. The last kind of number of lane change is the through traffic changing lane from right side lane to the left.

Several independent variables may affect number of lane change, for instance, ramp type, traffic volume, and number of through and etc. A mathematical model is presented to demonstrate variable of number of lane change.

$$Y = \exp(a_0 + a_1 X_1 + a_2 X_2 + a_3 X_3 + a_4 X_4 + a_5 X_5 / 1000 + a_6 X_6 / 1000)$$

Where,

Y —— Number of Lane Change,

X1 —— 1 for ramp type II, 0 for others,

X2 —— 1 for ramp type III, 0 for others,

X3 —— 1 for ramp type IV, 0 for others,

X4 —— Number of through lane on freeway,

X5 —— Freeway volume (vph), and

X6 —— Ramp volume (vph).

(2) Speed S.D.

Besides number of lane change, speed S.D. on freeway section is another factor which can be used to estimate the impacts of exit ramp. The larger speed S.D., the worse performance, the larger probability of crash.

Variable of ramp type, traffic volume, and number of through lanes are contributing to speed S.D. And a prediction model to estimate speed S.D. is as follow:

$$Y = \exp(a_0 + a_1X_1 + a_2X_2 + a_3X_3 + a_4X_4 + a_5X_5/1000 + a_6X_6/1000)$$

Where,

Y —— Speed S.D.,

X1 —— 1 for ramp type II, 0 for others,

X2 —— 1 for ramp type III, 0 for others,

X3 —— 1 for ramp type IV, 0 for others,

X4 —— Number of through lane on freeway,

X5 —— Freeway volume (vph), and

X6 —— Ramp volume (vph).

By using of TSIS simulation, many scenarios are specified, such as different level of traffic, different through lanes, different ramp types. All the extended examples can help find the correlation ships.

(3) Control Delay

Control delay per vehicle in freeway section indicates the impacts the existence of exit ramp. And ramp type is an important factor contributes to it. Field data show that exit ramp types can affect control delay on freeway near ramp area. A predict model is presented to estimate control delay per vehicle, and the format is the same as model for number of lane change and speed S.D.

$$Y = \exp(a_0 + a_1 X_1 + a_2 X_2 + a_3 X_3 + a_4 X_4 + a_5 X_5 / 1000 + a_6 X_6 / 1000)$$

Where,

Y —— Control Delay,

X1 —— 1 for ramp type II, 0 for others,

X2 —— 1 for ramp type III, 0 for others,

X3 —— 1 for ramp type IV, 0 for others,

X4 —— Number of through lane on freeway,

X5 —— Freeway volume (vph), and

X6 —— Ramp volume (vph).

When different impacts of different ramp types are found under same traffic and geometric conditions, there are evidences to choose optimal exit ramp for certain situation. Besides all three MOEs mentioned above, safety might be another aspect for the selection. Results from safety analysis part will also be used.

(4) Length Design for Deceleration Lane of Ramp Type I and IV

Besides number of lane change and speed S.D., length design for deceleration lane is another important issue. For ramp type I and IV, the length of deceleration lane can be verified. And the change of length might impact the performance.

In simulation, the length is changed from 100 ft to 1500 ft in TSIS, to see the distribution of MOE speed S.D., certainly, under different level of volume. Actually, AASHTO green book has already presented the proper length for freeway exit lane, but the standards are mainly based on stop distance. New suggested distances are based on operational analysis.

10.3.2 Ramp Section

There are two issues in the ramp section, determine the minimal length for ramp, and discuss the ramp configuration.

(1) Ramp Length Design

Ramp length design is based on this assumption, that the minimum length of ramp shall meet requirements of holding exiting traffic, including queuing length, deceleration length, and perception-reaction distance. The exiting traffic spilling back to freeway must be avoided. The deceleration distance can be calculated by initial speed and deceleration rate, and the perception-reaction distance depends on speed and time. Queuing length needs simulations.

Several factors will affect the minimum ramp length, such as volume level, control type, number of lanes on ramp, ramp terminal and etc. Thus, all these independent variables will be changed to simulate respectively, in order to find the minimal queuing length under each scenario.

(2) Ramp configuration

ASSHTO green book presets three kinds of exit ramp configuration, type A, B and C, see Figure 10.4.

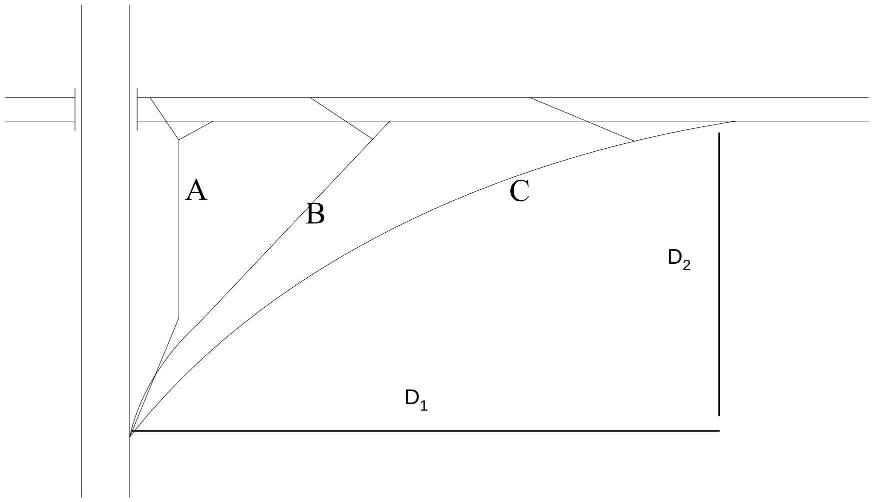


Figure 10.4 Exit Ramp Configurations

In type A, ramp terminal is close to freeway, exit ramp is almost parallel to freeway. This type is usually caused by limited land use. In type B, ramp terminal to a little far away from freeway and length of exit ramp is longer. But ramp is close to straight or curve is sharp. In type C, ramp terminal is far enough from freeway, and ramp curve can be made smooth and slightly.

To factors are changed to see changes of operational performance, D_1 and D_2 in Figure 10.4. Distance change is to find out how speed S.D. changes. Larger speed S.D. value under certain ramp configuration can cause potential problems.

10.3.3 Cross Road Section

The main task on cross road is to find out minimal distance between ramp terminal and upstream/downstream intersection.

Take the distance between ramp terminal and downstream intersection as an example. It is calculated bases on this assumption, that the queuing length of vehicles on crossroad does not block the traffic coming out from exit ramp. For distance between downstream intersection and ramp terminal, the weaving distance is also considered, besides queuing length. Figure 10.5 shows general method to calculate minimum distance.

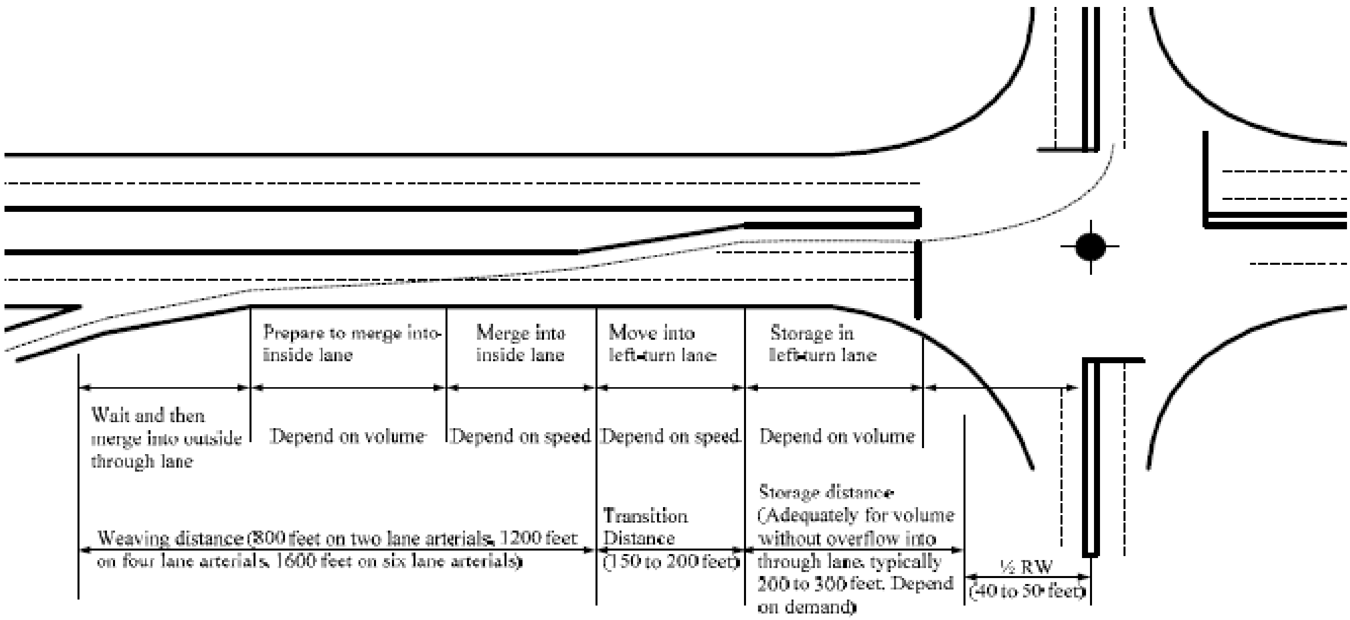


Figure 10.5 Distances between Ramp Terminal and Downstream Intersection

(Source: NCHRP 420-Summary Impacts of Access Management Techniques (1999))

There is not a specified minimal distance requirement between ramp terminal and upstream intersection, which is also the distance between two exit ramps at a diamond interchange. Several factors will impact this distance, such as geometric configuration and land use.

A minimal distance is presented here mainly based on queuing length simulation. This method assures queuing length will not spill back from segments between to exit ramps, which will worsen through traffic conditions on cross road.

Figure 10.6 shows the simulation network in TSIS to test different distance, traffic volume, signal timing plan, and the geometry conditions are changed respectively. And minimum distance can be found under heavy traffic conditions.

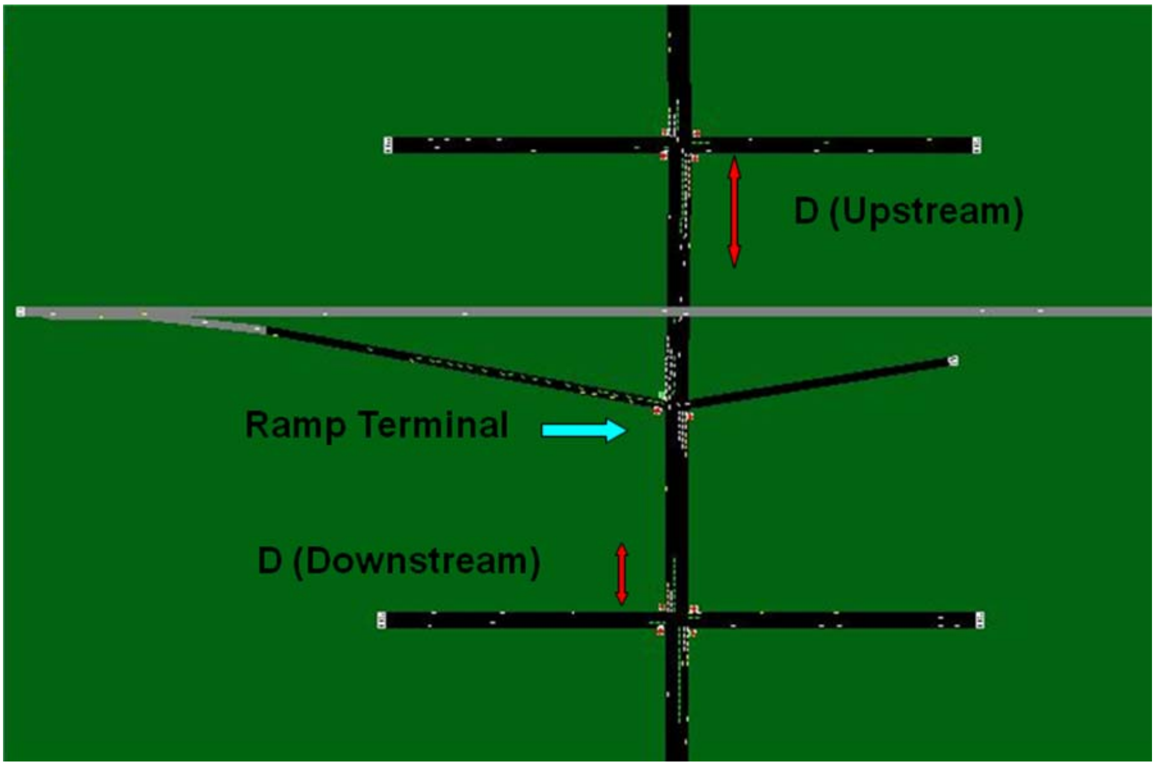


Figure 10.6 Distances between Upstream/Downstream Intersection and Ramp Terminal

CHAPTER 11 RESULTS AND CONCLUSIONS

Following simulations and methodologies of Chapter 10, all results are classified in three sections, freeway section, exit ramp section, and cross road section.

11.1 Freeway Section

11.1.1 Number of Lane Change

Comparisons of number of lane change among four exit ramp types are shown from Figure 11.1 to Figure 11.3. In Figure 11.1, the freeway volume is 3600 vph, number of through lanes on freeway is 3, and number of lane change increases obviously with ramp volume increasing. In Figure 11.2, the ramp volume is 800 vph, number of through lanes on freeway is 3, and number of lane change increases lightly with freeway volume increasing. In Figure 11.3, the freeway volume is 3600 vph, the ramp volume is 800 vph, and number of lane change increases with number of through lanes increasing. Thus, all the three independent variables have positive impacts to number of lane change. And under same condition, exit ramp type IV has the largest number of lane change, type I has the smallest number of lane change.

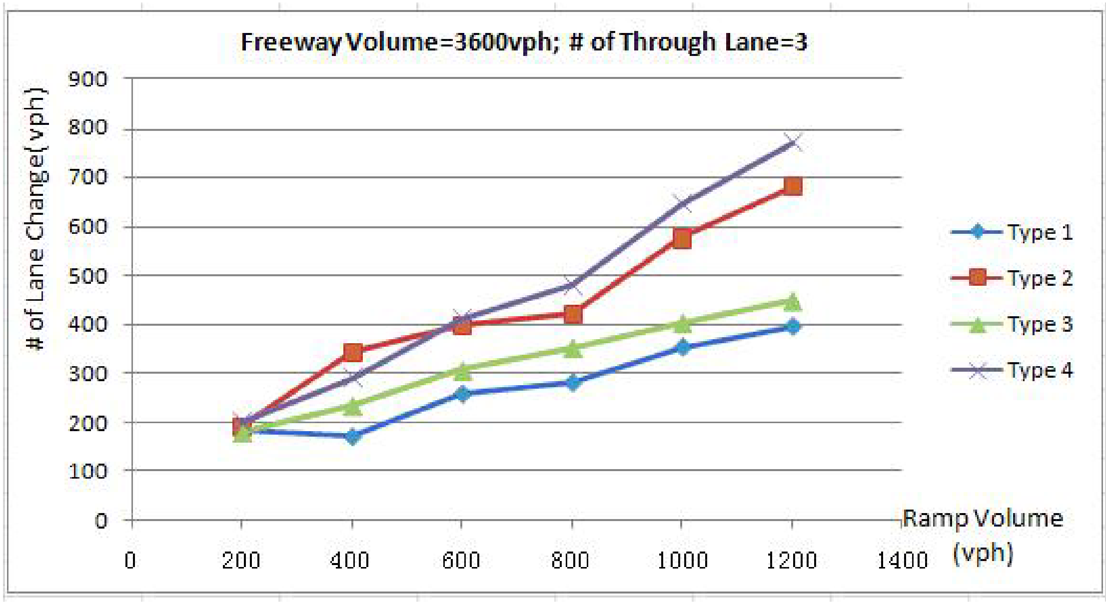


Figure 11.1 Number of Lane Change VS Ramp Volume

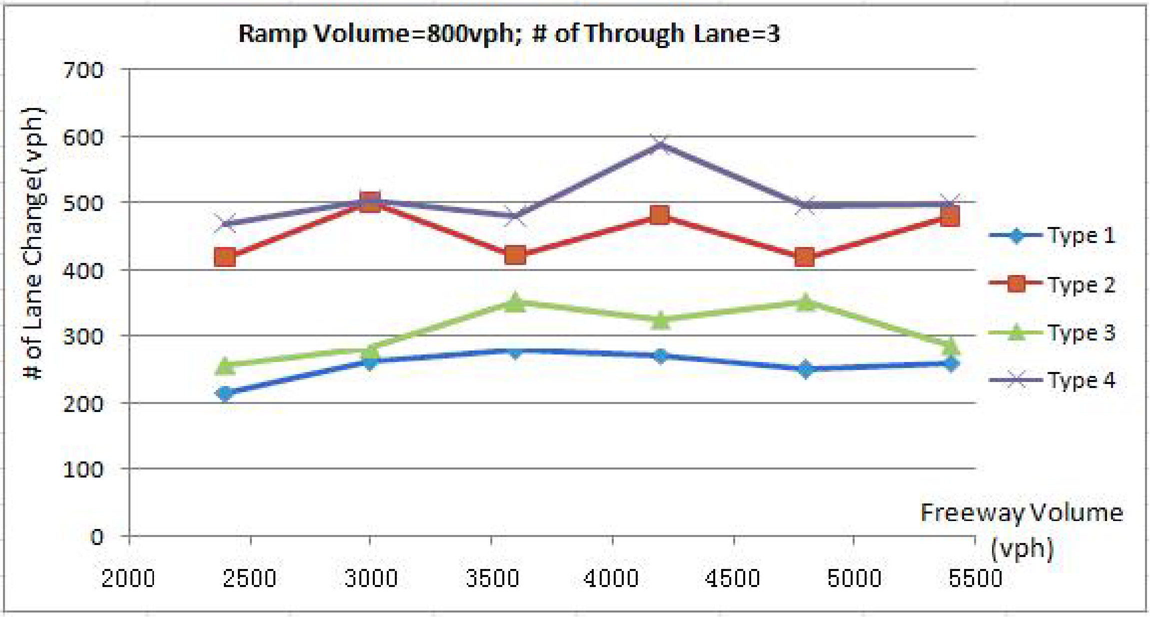


Figure 11.2 Number of Lane Change VS Freeway Volume

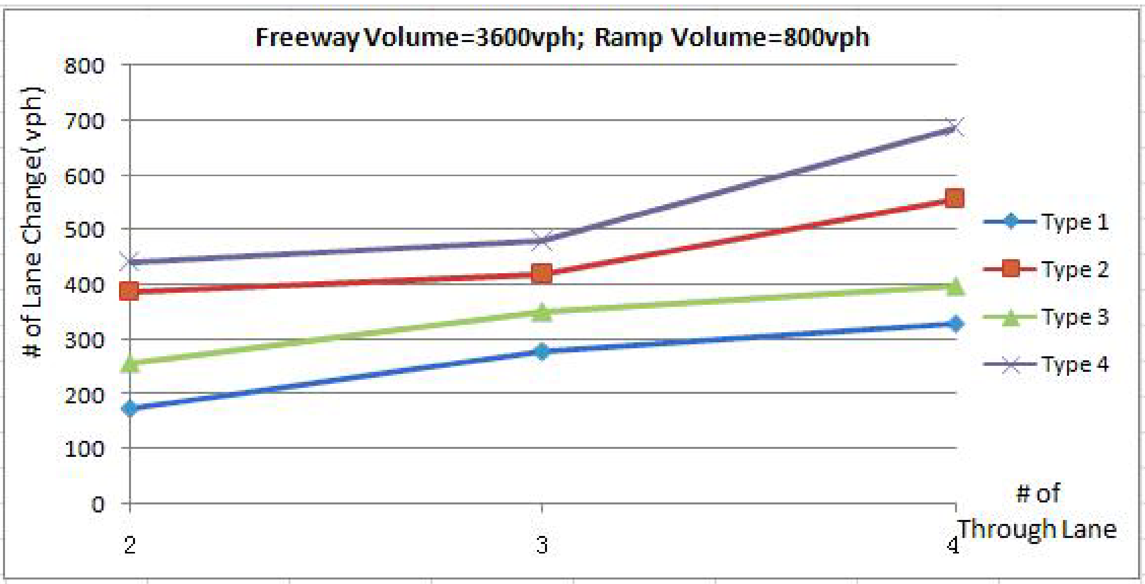


Figure 11.3 Number of Lane Change VS Number of Through Lane

All simulation conditions are used to calculate coefficients in the predict model. Results see Table 11.1. Column B is the coefficients for all independent variables.

$$Y = \exp(a_0 + a_1X_1 + a_2X_2 + a_3X_3 + a_4X_4 + a_5X_5/1000 + a_6X_6/1000)$$

Where,

Y —— Number of Lane Change,

X1 —— 1 for ramp type II, 0 for others,

X2 —— 1 for ramp type III, 0 for others,

X3 —— 1 for ramp type IV, 0 for others,

X4 —— Number of through lane on freeway,

X5 —— Freeway volume (vph), and

X6 —— Ramp volume (vph).

Table 11.1 Coefficient Values

Model	Un-standardized Coefficients		Standardized Coefficients	t
	B	Std. Error	Beta	
Constant	3.209	0.049		66.121
X1	0.555	0.030	0.341	18.720
X2	0.181	0.030	0.111	6.091
X3	0.637	0.030	0.392	21.496
X4	0.229	0.016	0.266	14.137
X5	0.101	0.008	0.243	12.346
X6	1.445	0.034	0.676	42.352

11.1.2 Speed S.D.

Comparisons of speed S.D. among four exit ramp types are shown from Figure 11.4 to Figure 11.6. In Figure 11.4, the freeway volume is 3600 vph, number of through lanes on freeway is 3, and speed S.D. increases with ramp volume increasing. In Figure 11.5, the ramp volume is 800 vph, number of through lanes on freeway is 3, and speed S.D. increases with freeway volume increasing. In Figure 11.6, the freeway volume is 800 vph, the ramp volume is 800 vph, and number of lane change decreases with number of through lanes increasing. Thus, two independent variables have positive impacts to number of lane change, while number of through lanes has negative impacts. And under same condition, exit ramp type I has the largest speed S.D., type IV has the smallest speed S.D.

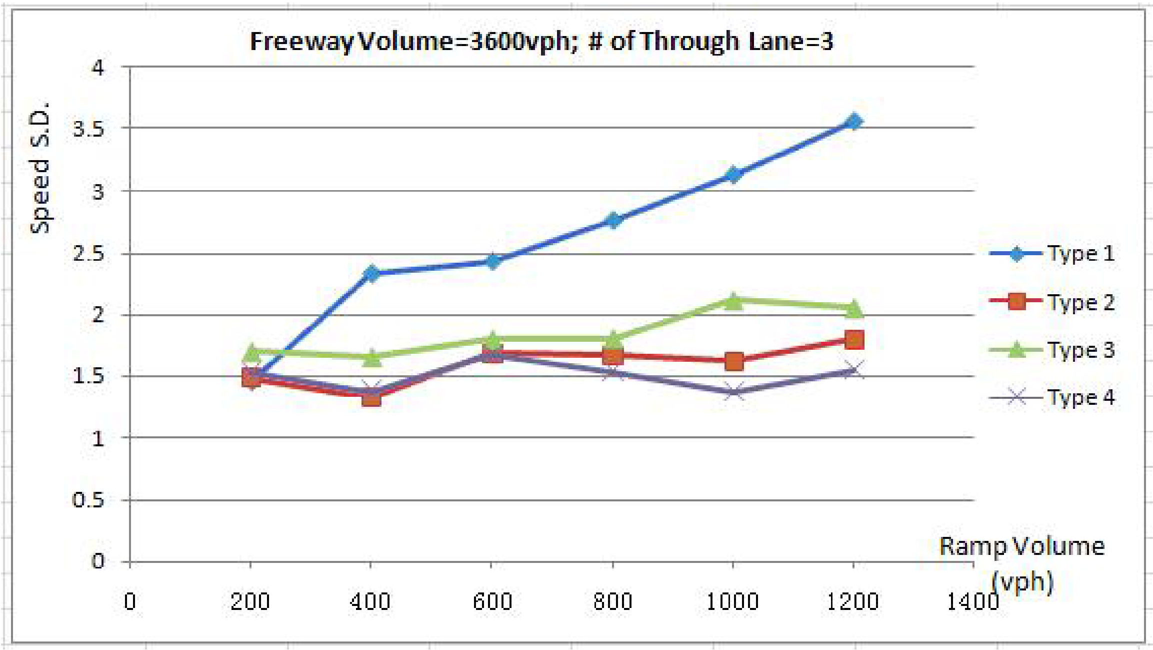


Figure 11.4 Speed S.D. VS Ramp Volume

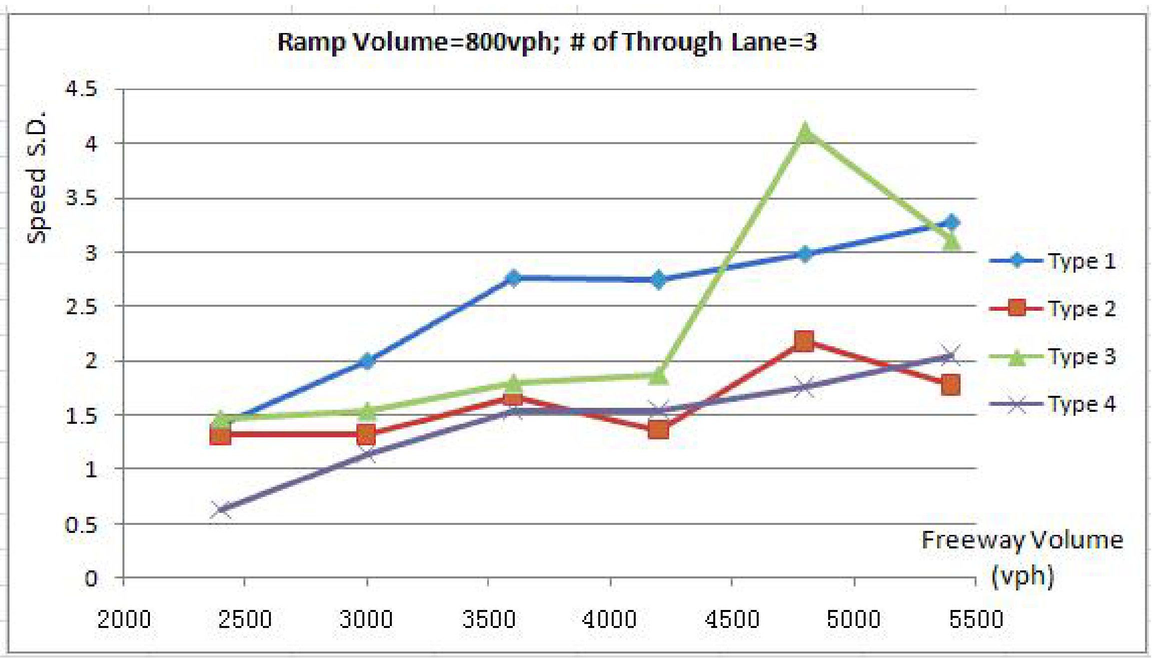


Figure 11.5 Speed S.D. VS Freeway Volume

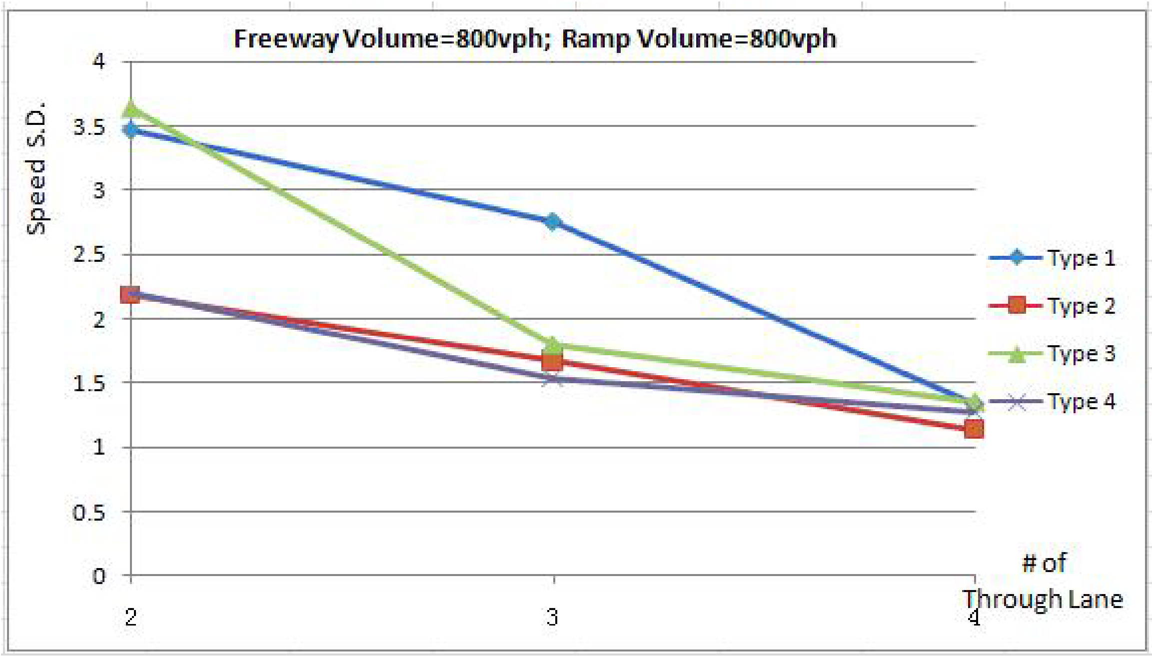


Figure 11.6 Speed S.D. VS Number of Through Lane

All simulation conditions are used to calculate coefficients in the predict model. Results see Table 11.2. Column B is the coefficients for all independent variables.

$$Y = \exp(a_0 + a_1X_1 + a_2X_2 + a_3X_3 + a_4X_4 + a_5X_5 / 1000 + a_6X_6 / 1000)$$

Where,

Y —— Speed S.D.,

X1 —— 1 for ramp type II, 0 for others,

X2 —— 1 for ramp type III, 0 for others,

X3 —— 1 for ramp type IV, 0 for others,

X4 —— Number of through lane on freeway,

X5 —— Freeway volume (vph), and

X6 —— Ramp volume (vph).

Table 11.2 Coefficient Values

Model	Un-standardized Coefficients		Standardized Coefficients	t
	B	Std. Error	Beta	
Constant	0.081	0.088		.918
X1	-0.394	0.054	-0.231	-7.309
X2	-0.058	0.054	-0.034	-1.074
X3	-0.499	0.054	-0.293	-9.261
X4	-0.360	0.029	-0.398	-12.217
X5	0.359	0.015	0.820	24.088
X6	0.506	0.062	0.226	8.173

11.1.3 Control Delay

Model of control delay is similar to number of lane change and speed S.D. All simulation conditions are used to calculate coefficients in the predict model. Results see Table 11.3. Column B is the coefficients for all independent variables. A very important statement here is about the factor V/C ratio (V refers to volume, C refers to capacity), which affects control delay. During the simulation and modeling, V/C ratio is set the same for different ramp types, in order to eliminate impacts of V/C ratio to control delay.

$$Y = \exp(a_0 + a_1 X_1 + a_2 X_2 + a_3 X_3 + a_4 X_4 + a_5 X_5 / 1000 + a_6 X_6 / 1000)$$

Where,

Y —– Control Delay,

X1 —– 1 for ramp type II, 0 for others,

X2 —– 1 for ramp type III, 0 for others,

X3 —– 1 for ramp type IV, 0 for others,

X4 —– Number of through lane on freeway,

X5 —– Freeway volume (vph), and

X6 —– Ramp volume (vph).

Table 11.3 Coefficient Values

Model	Un-standardized Coefficients		Standardized Coefficients	t
	B	Std. Error	Beta	
Constant	1.237	0.076		2.314
X1	-0.346	0.043	-0.274	-6.801
X2	-0.153	0.043	-0.101	-2.399
X3	-0.297	0.043	-0.198	-5.428
X4	-0.360	0.017	-0.413	-9.237
X5	1.459	0.09	1.433	21.546
X6	0.786	0.052	0.387	7.688

11.1.4 Length Design for Deceleration Lane of Ramp Type I and IV

For the length design of deceleration lane, ramp type I, the speed S.D. decreases quickly when length increases, especially when the volume is high. Figure 11.7 to Figure 11.12 show the speed S.D. VS length under different volume level.

But for ramp type IV, this kind of change is not obvious. The speed S.D. decreases slowly when length increases. Which means the deceleration lane doesn't have to be very long to lower speed S.D. Anyway, it is also suggested that the length should be long if possible.

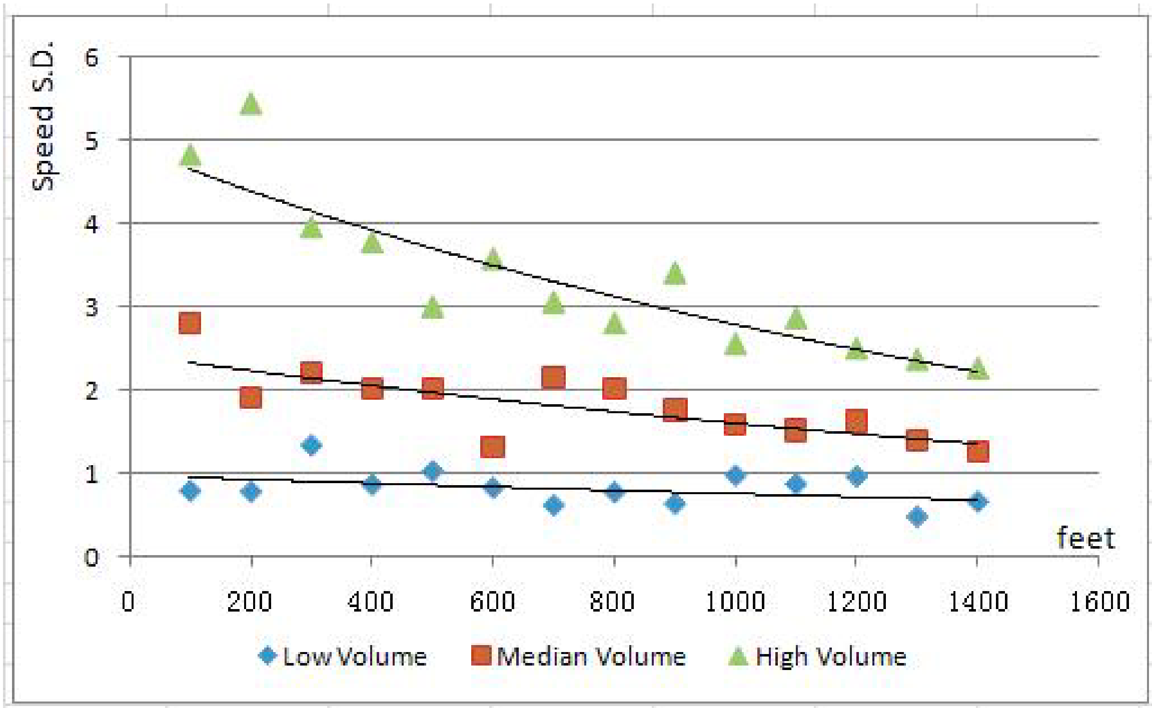


Figure 11.7 Speed S.D.VS Length (Type I, 2 thru lane)

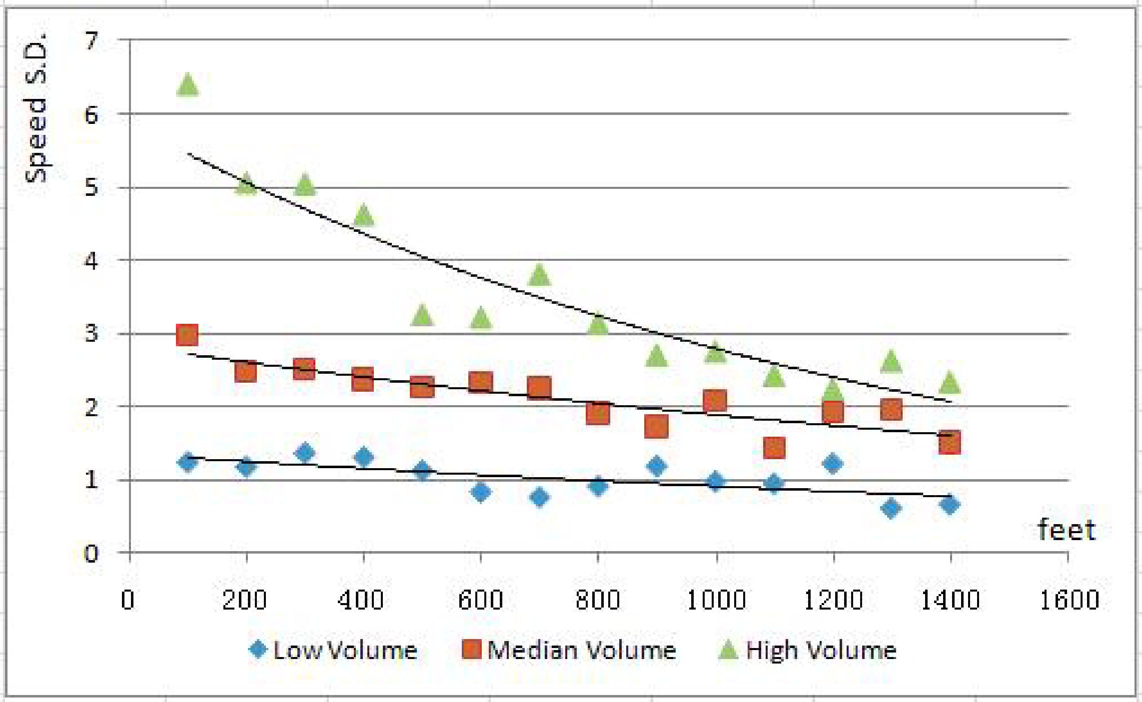


Figure 11.8 Speed S.D.VS Length (Type I, 3 thru lane)

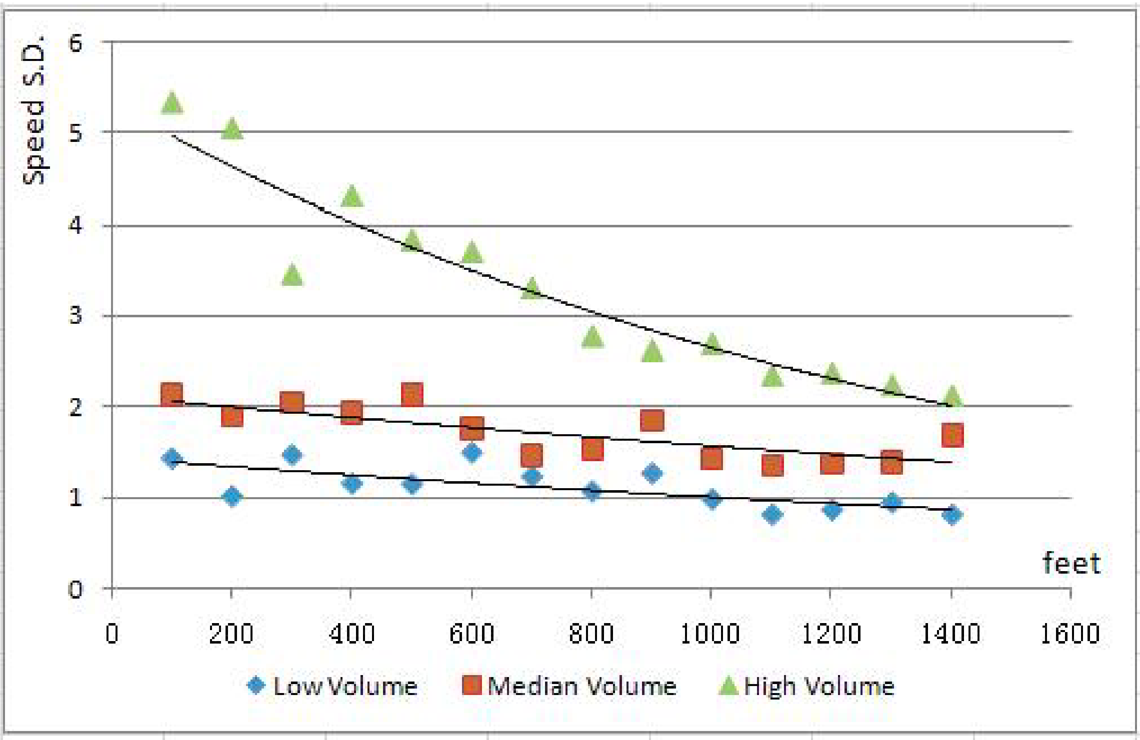


Figure 11.9 Speed S.D.VS Length (Type I, 4 thru lane)

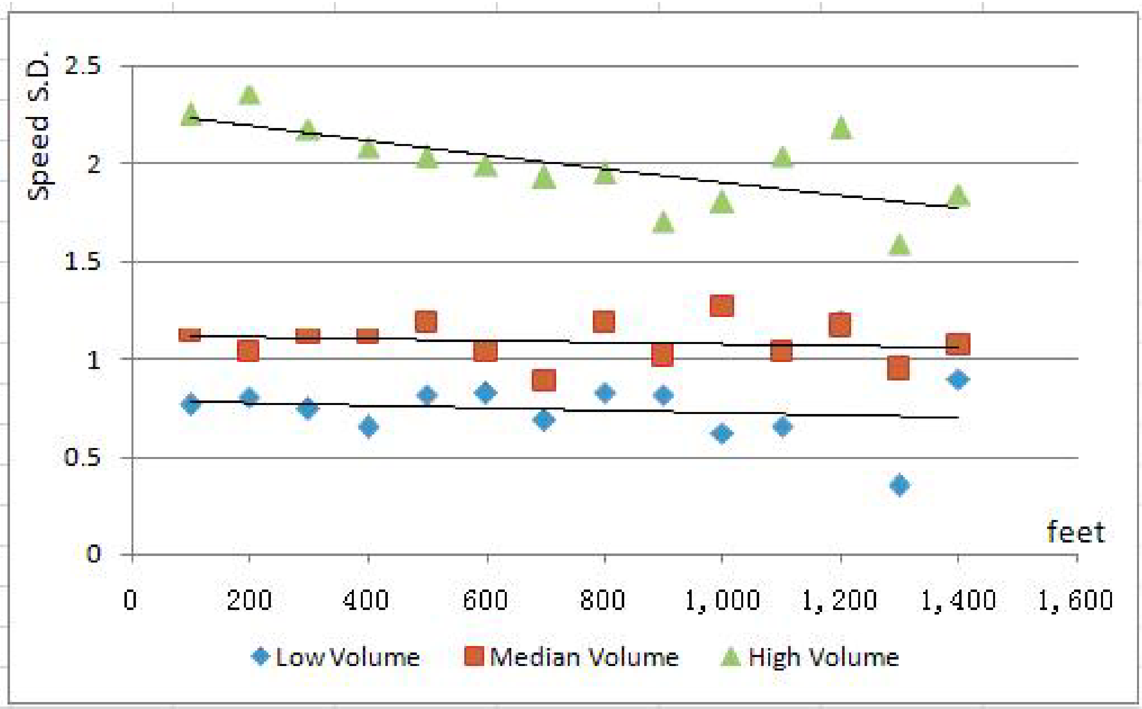


Figure 11.10Speed S.D.VS Length (Type IV, 2 thru lane)

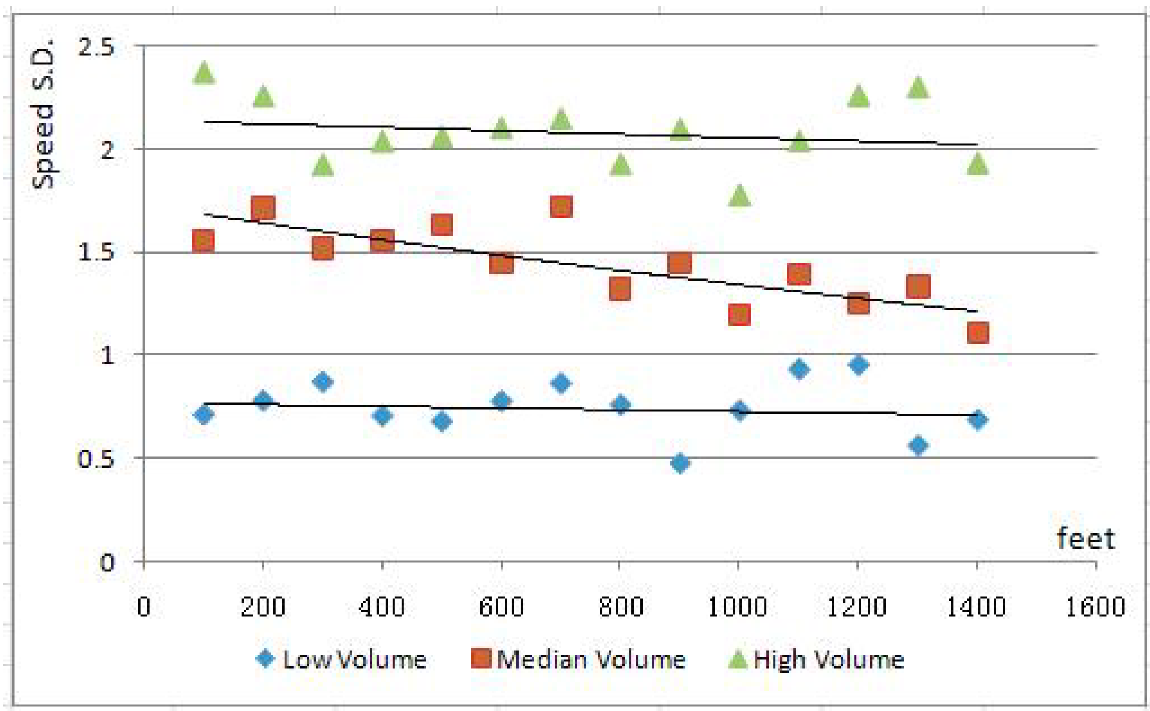


Figure 11.11 Speed S.D.VS Length (Type IV, 3 thru lane)

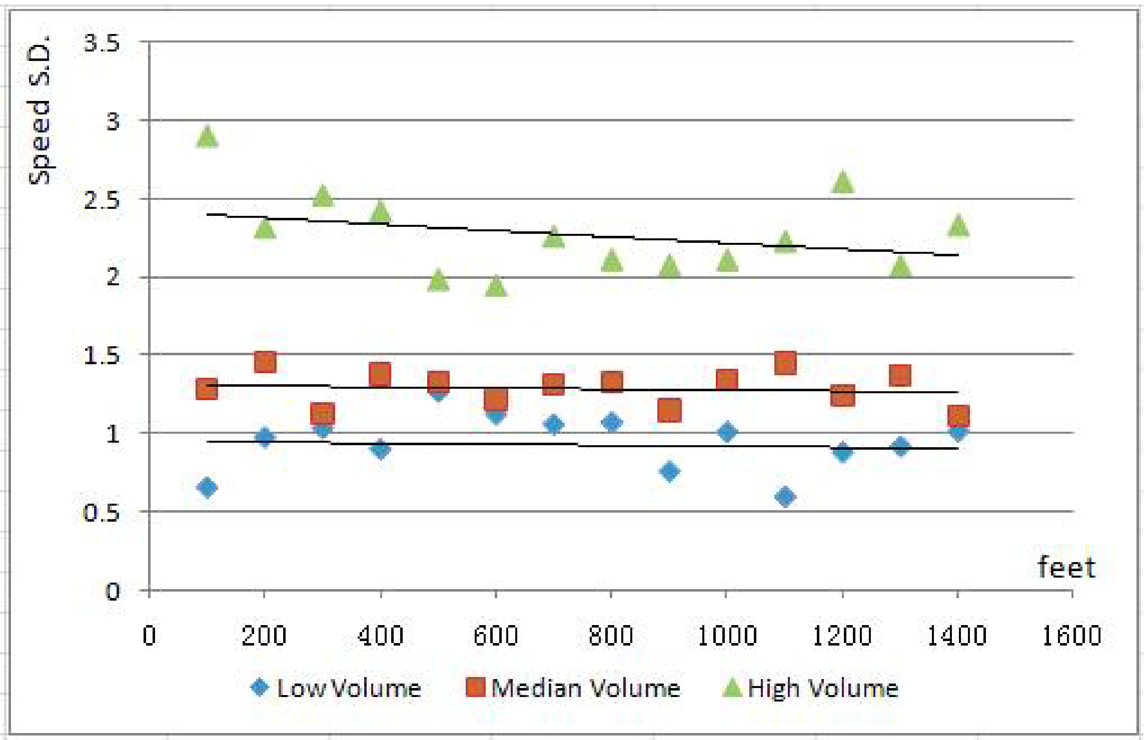


Figure 11.12 Speed S.D.VS Length (Type IV, 4 thru lane)

The speed S.D. should be controlled under certain level to research good operational performance. Simulation results are shown in Table 11.4, and simulation results are larger than AASHTO standard.

Table 11.4 Minimum Deceleration Lane

Operating speed (mph)	AASHTO Standard (ft)	Simulation Type I (ft)	Simulation Type IV (ft)
55	480	750	550
60	530	800	600
65	570	850	650
70	615	875	700
75	660	900	725

11.1.5 Selection for Optimal Exit Ramp Type

The exponential models show different impacts of four types of ramp on number of lane change, speed SD, and control delay. Larger value of coefficient ai means more contribution of independent variable to dependent variable. Based on Table 11.1, 2, and 3, a comparison table (Table 11.5) is made to tell the difference. It is clear that ramp type I has the least number of lane change out of four types, type IV has the largest. For speed SD, the situation is opposite, ramp type IV the best, type I the worst. And for control delay, ramp type II is the best, type I the worst.

Table 11.5 Comparisons of Exit Ramp Types

MOE	Best → Worst
Number of Lane Change	Type I → Type III → Type II → Type IV
Standard Deviation of Speed	Type IV → Type II → Type III → Type I
Control Delay per Vehicle	Type II → Type IV → Type III → Type I

Because the priority ranking of ramp type for each parameter is totally different, it is hard to say which the optimal type of exit ramp is. But the importance of three parameters is different under different conditions. For example, if expected exiting traffic for a ramp is very high, then number of lane change should be paid more attention, in order to reduce crashes cause by decreasing lane change. Or, another case, operational performance is required to strengthen, and then control delay is to be the first consideration. Thus, different weights can be added to the three parameters due to different design situations or requirements.

Take ramp type I as the reference, coefficients of all other types can be compared based on the exponential model, shown in Table 11.6. Take the second line of assigned weights (0.5 for lane change, 0.3 for speed SD, and 0.2 for control delay) as an example, the total value is 1 for ramp type I, 1.214 for type II, 1.057 for type III, and 1.276 for type IV. Therefore, ramp type I has the smallest value, and it is the optimal one under this condition.

The comprehensive evaluation model includes three MOEs (number of lane change, standard deviation of speed, and control delay per vehicle). And different weights for each MOE are assigned for different design conditions and considerations. Finally, the optimal one can be found. It is flexible for any necessary changes. Different MOEs can be added or deleted if available. Also, weight for each MOE can be changed.

Table 11.6 Selection of Optimal Exit Ramp

Ramp Type	MOE	Relative a_i	Assigned Weights						
	Lane Change	1	0.33	0.5	0.5	0.3	0.3	0.2	0.2
I	Speed SD	1	0.33	0.3	0.2	0.5	0.2	0.5	0.3
	Control Delay	1	0.33	0.2	0.3	0.2	0.5	0.3	0.5
Total Value for I			1	1	1	1	1	1	1
	Lane Change	1.7419	0.33	0.5	0.5	0.3	0.3	0.2	0.2
II	Speed SD	0.6743	0.33	0.3	0.2	0.5	0.2	0.5	0.3
	Control Delay	0.7075	0.33	0.2	0.3	0.2	0.5	0.3	0.5
Total Value for II			1.030	1.214	1.218	1.001	1.011	0.897	0.904
	Lane Change	1.2056	0.33	0.5	0.5	0.3	0.3	0.2	0.2
III	Speed SD	0.9435	0.33	0.3	0.2	0.5	0.2	0.5	0.3
	Control Delay	0.858	0.33	0.2	0.3	0.2	0.5	0.3	0.5
Total Value for III			0.992	1.057	1.048	1.005	0.979	0.970	0.953
	Lane Change	1.8908	0.33	0.5	0.5	0.3	0.3	0.2	0.2
IV	Speed SD	0.6071	0.33	0.3	0.2	0.5	0.2	0.5	0.3
	Control Delay	0.7431	0.33	0.2	0.3	0.2	0.5	0.3	0.5
Total Value for IV			1.069	1.276	1.289	1.019	1.060	0.904	0.931
Optimal Type			III	I	I	I	III	II	II

11.2 Exit Ramp Section

11.2.1 Ramp Length Design

Simulations for different conditions suggest different minimum ramp length, see Table 11.7. Table 11.8 compares field data to standard, and red number shows field data shorter than standard. This table indicates short ramp length is an important problem in practical situations.

Please Note that queuing length is based on simulation for observing sites during peak hour. Deceleration length is based on average speed of 40 mph, the distance is 200 ft for 50 mph, and 225 ft for 60 mph.

Table 11.7 Minimum Ramp Length

No. of lanes on ramp	No. of lanes on cross road	No. of Left turn bay	Queuing Length (ft)	Deceleration Length (ft)	Perception Reaction Length (ft)	Volume After queue (ft)	Total Length (ft)
1	2	0	600	175	600	330	1705
1	4	0	850	175	600	415	2040
1	6	0	950	175	600	445	2170
1	2	1	550	175	600	315	1640
1	4	1	750	175	600	380	1905
1	6	1	900	175	600	430	2105
2	4	0	700	175	600	365	1840
2	6	0	875	175	600	420	2070
2	4	1	600	175	600	330	1705
2	6	1	800	175	600	400	1975

Table 11.8 Observing Ramp Length

No.	Exit Ramp	Number of Through Lanes on Cross Road	Ramp Length (ft)
1	I-75 at State Road 56- SB	4	2575
2	I-4 at County Road 579- WB	2	1500
3	I-275 at Hillsborough Ave- NB	6	910
4	I-275 at Hillsborough Ave- SB	6	1100
5	I-75 at I-4- SB	6	4300
6	I-275 at 4th St- SB	4	3950
7	I-4 at Universal Blvd- SB	6	2665
8	I-75 at CR 581 (BBD)- SB	6	2530
9	I-75 at Fowler Avenue- SB	6	1750
10	I-4 at Lee Road-NB	6	1770
11	I-4 at Lee Road- SB	6	1840
12	I-4 at SR 434- SB	6	1000
13	I-75 at State Road 56- NB	6	2400
14	I-4 at County Road 579- EB	4	1630
15	I-4 at Universal Blvd- NB	4	1630
16	I-4 at Conroy Road- NB	6	3800
17	I-4 at Conroy Road- SB	6	2415
18	I-4 at Altamonte Dr- NB	8	1050
19	I-4 at SR 434- NB	4	1170
20	I-4 at Altamonte Dr- SB	8	800
21	I-75 at CR 581 (BBD)- NB	6	2600
22	I-75 at I-4- NB	6	3900
23	I-275 at Ulmerton Rd- SB	4	3800
24	I-75 at Fowler Avenue- NB	6	3800

11.2.2 Ramp Configuration

Speed S.D. is selected for evaluating ramp configuration. D1 and D2 are changed in a range to see changes of speed S.D. Take speed S.D. as reference 1, at D1 less than 400ft, and D2 at level of 1600 ft. All other values are compared with 1. Based on this table, the longer distance of D1 and D2, the smaller value of speed S.D., the better performance it is.

Table 11.9 Relative Speed S.D.

D_2 \ D_1	Type A: <=400	Type B: 600	Type B: 800	Type C: >=1000
1600	1	0.954	0.910	0.865
1800	0.987	0.941	0.904	0.853
2000	0.975	0.939	0.879	0.821

11.3 Cross Road Section

All simulation scenarios show results of minimum distance. The design minimum distance is tested under heavy traffic conditions, see Table 11.10 and 11.11.

11.3.1 Distance between Ramp Terminal and Downstream Intersection

Table 11.10 Minimum Distance between Ramp Terminal and Downstream Intersection

Distance (ft)	Number of Lanes on Cross Road					
	2		4		6	
Weaving-moving across through lanes	800		1200		1600	
Transition-moving into lanes	150 U	200 R	150 U	200 R	150 U	200 R
Perception-reaction distance	100 U	150 R	100 U	150 R	100 U	150 R
Storage	550 (200-300)		700 (200-300)		750 (200-300)	
Distance to centerline of intersection	40(50)		50(50)		60(50)	
Total Distance	1640	1740	2200	2300	2660	2760

Note: U signifies Urban Area, R signifies Rural Area.

11.3.2 Distance between Ramp Terminal and Upstream Intersection

The distance between ramp terminal and downstream or upstream intersection is based on simulations which include the whole interchange (two ramps). Thus, two ramps are both considered to present these distance guidelines.

Table 11.11 Minimum Distance between Ramp Terminal and Upstream Intersection

Distance (ft)	Number of Lanes on Cross Road					
	2		4		6	
Transition-moving into lanes	150 U	200 R	150 U	200 R	150 U	200 R
Perception-reaction distance	100 U	150 R	100 U	150 R	100 U	150 R
Storage	650 (200-300)		750 (200-300)		850 (200-300)	
Distance to centerline of intersection	40(50)		50(50)		60(50)	
Total Distance	940	1040	1050	1150	1160	1260

Note: U signifies Urban Area, R signifies Rural Area.

For example, here it is a typical diamond interchange, including two exit ramps: No.1 and No.2 (see Fig 11.13). The number of lanes on cross road is four, and this site is located at rural area. For exit ramp No.1, assume distance AB is the minimum distance between ramp terminal B and downstream intersection, distance BC is the minimum distance between ramp terminal B and upstream intersection. For exit ramp No.2, assume distance CD is the minimum distance between ramp terminal C and downstream intersection, distance BC is the minimum distance between ramp terminal C and upstream intersection. Table 11.10 and 11.11 suggests that AB = CD = 2300 ft, BC = 1150 ft.

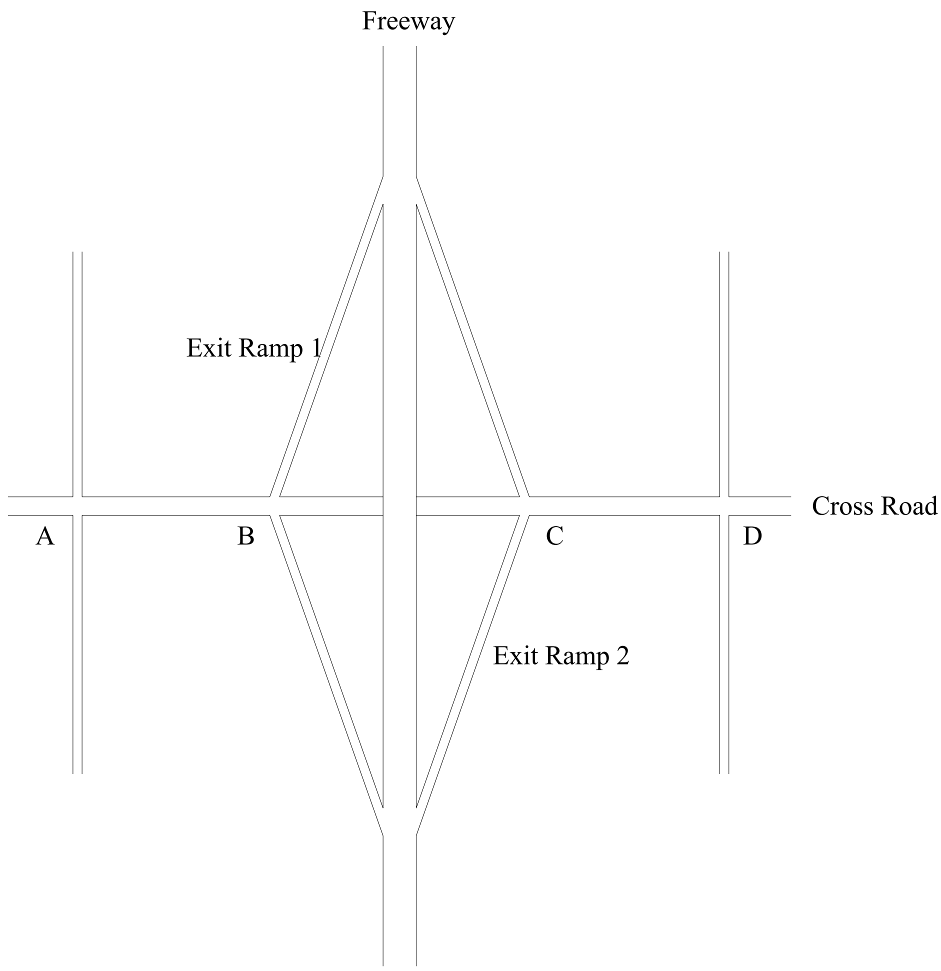


Figure 11.13 Examples for Minimum Distance

11.4 Conclusions

This chapter represents simulation results and mathematical models to evaluate operational performance of exit ramps. Comparisons are made to find out optimal one. Ramp length and minimum distance on cross road are also presented. Detailed conclusions see the following contents:

(1) Numerical evaluations are provided for different ramp types on number of lane change, speed S.D., control delay. Three predict models are presented.

(2) Minimum ramp length standard is presented based on analysis of speed S.D. by simulations. This standard is longer than traditional one.

(3) A method for selecting optimal exit ramp type is indicated. Different weights can be

added due to different purposes. Optimal one is not constant, but type III and IV are suggested when traffic volume is heavy.

(4) Minimum exit ramp length is presented which includes queuing length, movement distance and etc. This distance helps regulate future design.

(5) Simulation for ramp configuration shows that the longer distance between freeway and ramp terminal, and the longer distance between cross road and exit ramp nose, the smaller of speed S.D. And the better of ramp operational performance.

(6) Minimum distance between ramp terminal and downstream/upstream intersections are calculated. This distance standard lowers speed variance and conflict, and assures traffic mobility.

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