Exploratory Analysis of Ramp Metering on Efficiency and Safety of Freeways Using Microsimulation

By

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Dedicated to my parents

ABSTRACT

The microscopic Verkehr In Städten – SIMulations Model (VISSIM) stochastic simulator program was used to explore the effectiveness of ramp metering on efficiency, Level of Service, and safety of freeways. Three different geometric configurations of ramp-freeway junctions were evaluated using different traffic volume conditions of the ramp and the freeway. Different signal timing scenarios were designed for the different traffic volume and geometric configuration scenarios. Calibration process was conducted for the collected traffic data that were obtained from cameras and detectors. Two-hundred-eighty models were built and run to explore the effectiveness of the performance and safety of the ramp meters on freeways. Average speed and average travel time of the vehicles passing a 3,000-ft long freeway segment were used as measures of effectiveness of the freeway efficiency evaluation. Average density in the ramp influence area was used to obtain the freeway level of service as a measure of effectiveness of the freeway capacity evaluation. Frequency, types, and severity of vehicle conflicts, which occurred on the 3,000-ft freeway segment, were used as measures of effectiveness of the freeway safety evaluation. The Surrogate Safety Assessment Model (SSAM) program, which was developed by the Federal Highway Administration (FHWA), was used to find the frequency and types of vehicle conflicts, while the severity of vehicle conflicts was separated by a designed method that was retrieved from the previous literature studies. Minitab statistical software was used for some tests such as normality test to determine the appropriate number of samples, and F-tests. A sensitivity analysis was also conducted for better understanding the effectiveness of two assumption changes on the results that were obtained from running the models. The assumptions were car following headway in the ramp influence area and traffic composition on the freeway. The findings of the study provided different results related to the different geometric configurations, signal timing designs, and traffic volumes.

Ramp metering at the Type I geometric configuration provided positive effects on the efficiency and safety of the freeway when using the two designed signal timing scenarios when the freeway traffic volume was equal to or greater than 1,250 vehicle per hour per lane (vphpl) and the ramp traffic volume was equal to or greater than 800 vphpl. Ramp metering provided negative effects on the efficiency and safety of the freeway when using it for the Type II geometric configuration. In the geometric configuration of Type III, ramp metering using the signal timing of 2 seconds green and 4 seconds red provided the best efficiency and safety increases when the freeway traffic volume was equal to or greater than 1,250 vphpl and the ramp traffic volume was equal to or greater than 800 vphpl. Conclusively, ramp metering increases efficiency and improves safety of freeways only at specific situations regarding geometric configuration of the ramp-freeway junction type, traffic volume of the freeway and the ramp, and the designed traffic signal of the ramp meters.

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CHAPTER 1: INTRODUCTION

1.1 Background

Freeways, which have high traffic volumes, are designed to provide the greatest efficiency, capacity, and safety using grade separated intersections (AASHTO, 2011). Interchanges are grade-separated intersections that make the freeways fully access controlled. According to the American Association of State Highway and Transportation Officials (AASHTO), the definition of an interchange is a system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways or highways on different levels. Interchanges have weaving, merging and/or diverging segments that can cause turbulence for the freeway's traffic stream due to the high rate of lane changes, and acceleration and/or deceleration maneuvers of the highway vehicles (Roess, et al. 2011). According to AASHTO, "the term "ramp" includes all types, arrangements, and sizes of turning roadways that connect two or more legs of an interchange" (AASHTO, 2011). Therefore, ramps are necessary elements of interchanges, which may cause problems to the safety, and delay of freeways. There are several types of ramps, which have different characteristics in shapes, and each type can be broadly classified as the basic types that are diagonal, one quadrant, loop and semi directional connection, outer connection, and direct connection as shown in Figure 1. A ramp consists of three elements: two junctions and a ramp roadway. Ramp-freeway junctions may be uncontrolled, yield-controlled, or signalized (ramp-metering) (AASHTO, 2011). On freeways, merging movements occur primarily at on-ramp junctions, which are designed to permit relatively high-speed merging maneuvers while limiting the disruption to the main traffic stream" (HCM, 2010).

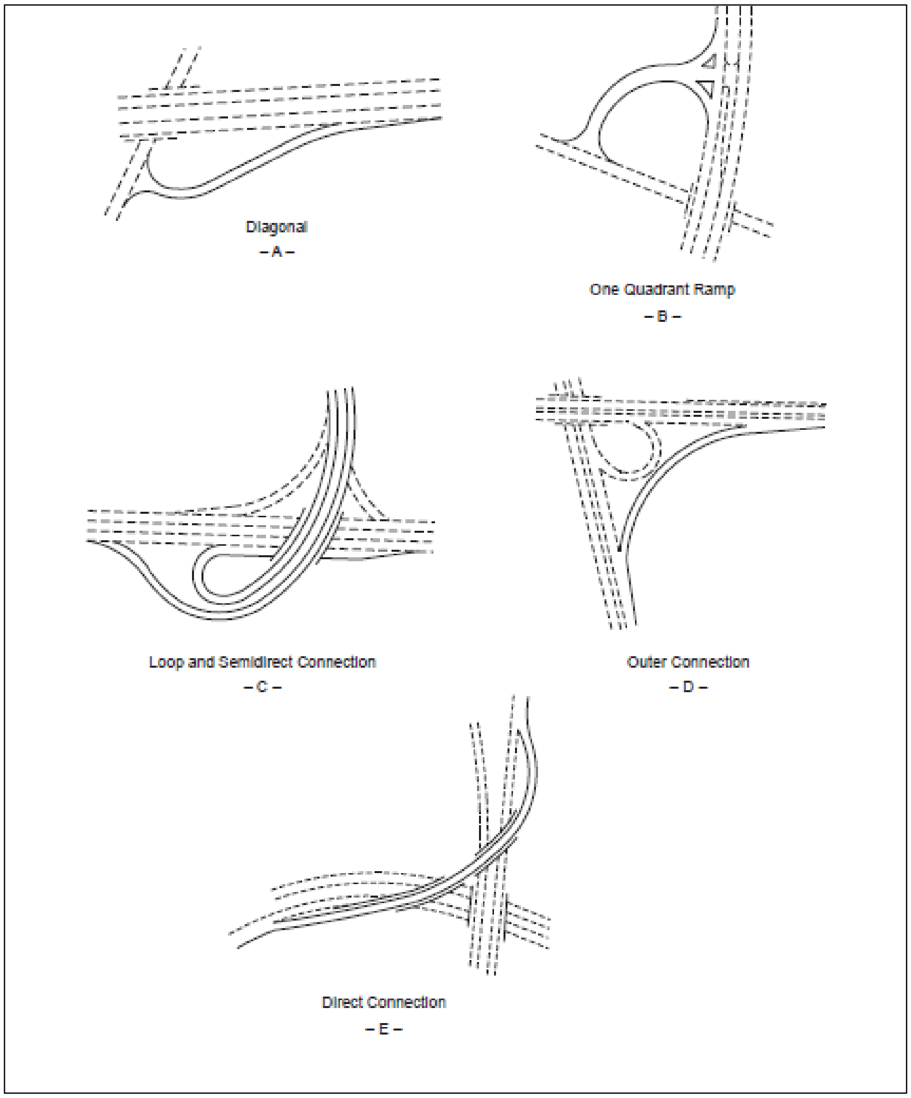


Figure 1: Ramp shapes (AASHTO, 2011)

The Highway Capacity Manual (HCM) 2010 states that there are several elements that affect merging and diverging operations such as the length and type (parallel, taper) of acceleration or deceleration lane(s), the free-flow speed of both the ramp and freeway in the vicinity of the ramp, the proximity of other ramps. Despite the turbulence that was explained, recurrent and non-recurrent congestion may occur in freeway corridors. Recurrent congestion is due to excessive peak demand, while non-recurrent congestion is primarily due to crashes that cause capacity reduction (Zhang et al. 2001).

According to National Highway Traffic Safety Administration (NHTSA, 2011) report, 5.338 million vehicle crashes occurred in the United States in 2011. The percent of fatal, injury, and property damage only crashes were 0.6, 28.7 and 70.8, respectively. In these crashes, 32,367 people died, 2.93 million people were injured; 52.3 percent of the people died in roadway departures that include intersections at interchange areas. There were 9.412 million vehicles involved in the crashes in which 200,000 (2.12 percent) of them occurred in entrance/ exit ramps (NHTSA, 2011).

To maintain optimum operational capacity and safety on freeways, ramp management strategies are employed. Ramp management strategies include the applications of traffic control devices such as traffic signals, signings, and gates to regulate the number of vehicles entering or leaving the freeways. Ramp metering is an application of the ramp management strategies, which is used as an attempt to reduce the impacts of on-ramps upon operational efficiency and safety on freeways (Jacobson L, 2006).

1.2 Ramp Metering

Ramp metering is the use of traffic signals deployed on freeway entrance ramps to regulate the flow of entering traffic in order to prevent or delay declined traffic performance. By discharging a measured rate of traffic from the on-ramp, ramp meters can maximize throughput, keep speeds uniform, keep demand below downstream capacity of the freeway, and reduce congestion-related crashes. Ramp meters can be used to break up platoons of vehicles that are released from an upstream traffic signal into one or two vehicles at a time, which can also promote better traffic flow at the merging area. Sideswipe and rear-end type crashes, which are associated with stop-and-go and erratic traffic flows, are reduced by alleviating turbulence in the merge zone. According to numerous states' guideline designs for ramps, the three primary considerations, which make ramps suitable for metering are: the availability of storage space, adequate acceleration distance in the merge area beyond the meter, and sight distance (Piotrowicz and Robinson, 1995). Empirical studies have shown that when ramp metering is implemented correctly and operated effectively, it provides many benefits such as increasing freeway speeds, decreasing travel times, reducing overall delay, increasing freeway throughput, improving safety, reducing congestion, reducing fuel consumption, and improving air quality by reducing gas emissions (Piotrowicz and Robinson, 1995).

1.2.1 Ramp Metering Components

A typical example of ramp metering design and its components is shown in Figure 2. The ramp metering signal may be placed on one or both sides of the ramp roadway. The ramp signals should be supplemented with traffic marking of white stop lines extending across the lanes. Regulatory signs are installed adjacent to the ramp control signals. The regulatory signs inform the drivers the number of vehicles permitted to enter during the short period of the green-time displayed on the signal; for example, a ONE VEHICLE PER GREEN sign, ONE VEHICLE PER GREEN EACH LANE sign and so on. Advance warning signage with flashing beacons indicates that the ramp metering is active. A RAMP METER AHEAD SIGN and RAMP METERED WHEN FLASHING are examples of advanced warning signs (MUTCD, 2012). Vehicle detectors are placed at upstream and downstream points of the freeways in relationship to the on-ramp. The locations of detectors are determined depending on the type of the control strategies. Some types of control strategies need both downstream and upstream detectors, while some other types need either downstream or upstream detectors. Fixed-time control strategy does not need detectors. For all types of control strategies, there are maximum and minimum metering rates, which are directly related to the timing parameters (Tian et al. 2002).

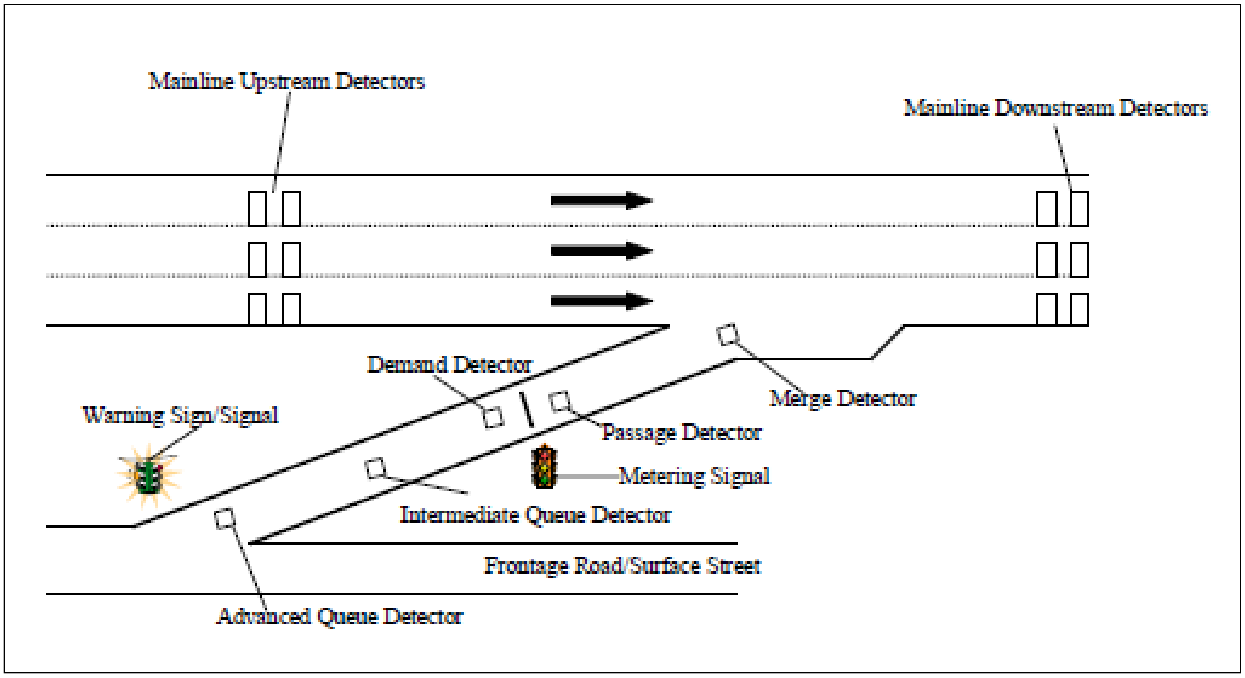


Figure 2: Typical schematic diagram of ramp metering (Tian et al. 2002)

1.2.2 Ramp Metering Traffic Flow Control Strategies

Metering strategies are defined as the approach used to control the traffic flow on the ramps. To control the flow of vehicles that enter the freeway through ramps, three metering strategies are available.

1. Single-lane one car per green

   In this strategy, the green-time period is set to allow only one car to enter the freeway in each signal cycle. A typical cycle length is the smallest possible cycle length, which is four seconds with one second green, one second yellow, and two seconds red. The metering capacity in this strategy is 900 vehicles per hour. A more reasonable cycle is 4.5 seconds, which is obtained by increasing the red-time to 2.5 seconds. The ramp meters with this increase of red-time provides a lower meter capacity 800 of vehicles per hour.

2. Single-lane multiple cars per green (known as platoon metering or bulk metering)

In this strategy, two or more vehicles are allowed to enter the freeway in each signal cycle. The most common type is allowing two cars per green, which requires 6 to 6.5 seconds cycle length and results in metering capacity of 1,100 to 1,200 vehicle per hour. This analysis illustrates that bulk metering does not double the metering capacity.

3. Dual-lane metering

In this strategy, more storage spaces for queued vehicles are provided. For each lane, the green-yellow-red cycles are displayed separately (green indications never occur simultaneously in both lanes). The green indications are timed to allow a constant headway between vehicles from both lanes, which can provide metering capacity of 1600 to 1700 vehicles per hour (Mathew, 2012).

In order to obtain the desired benefits from ramp metering, traffic engineers should install ramp meters with the appropriate quality of metering availability. Metering availability is defined as the percent of time the signal is displaying the green, yellow, and red sequences (Chaudhary and Messer, 2002). Each one of the three ramp metering control strategies has a specified metering availability type for a range of ramp-demand traffic volumes as shown in Figure 3. According to the figure, the metering strategy is rated as good quality if the percentage of ramp metering availability is equal or greater than 80. Single-lane ramps can be used to provide good-quality operations when the ramp demand is less than 1,200 vph, while it provides fair quality when the ramp demand is between 1,200 and 1,500 vph. Dual-lane metering provides good-quality metering for demand up to 1,650 vph (Chaudhary and Messer, 2002).

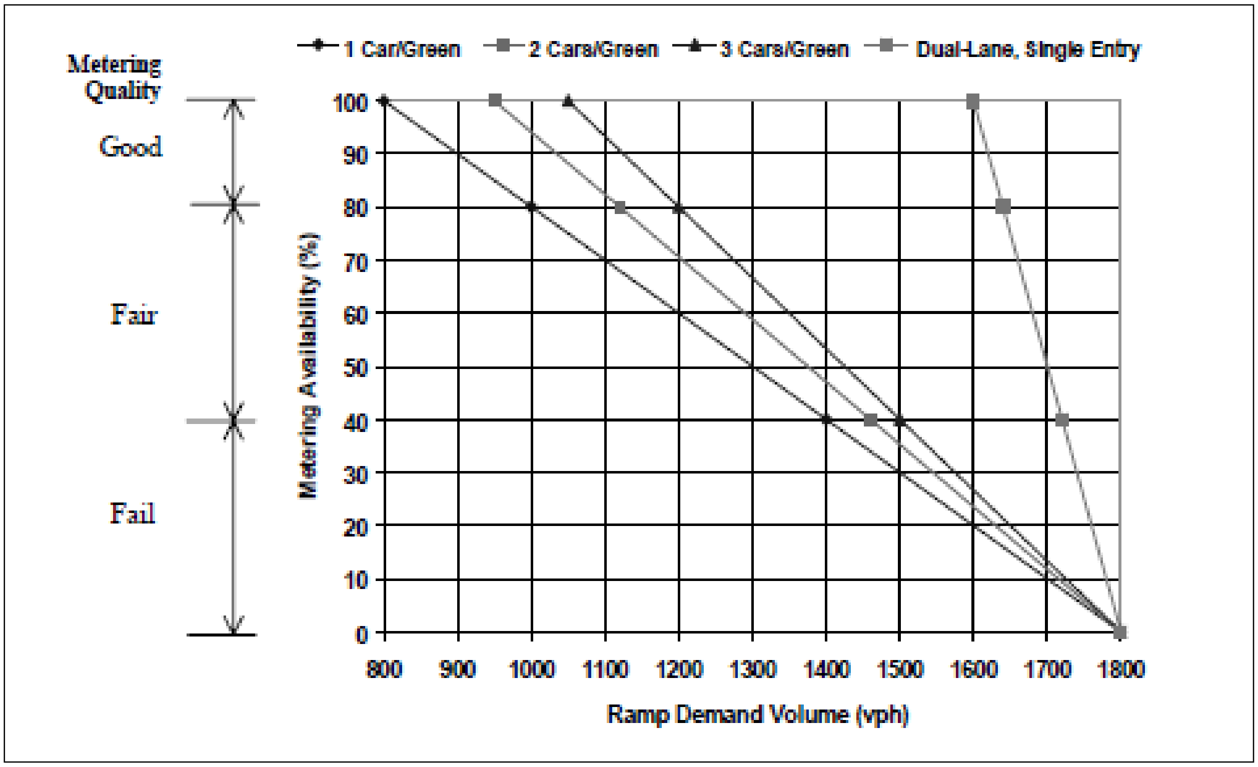


Figure 3: Quality of metering strategies (Chaudhary and Messer, 2002)

1.2.3 Timing Parameters for Different States in the United States

The usual ramp-meter signal cycle length consists of red and green intervals, and some states use a yellow interval as well. The green and yellow intervals are usually fixed, but the red interval is changed depending on the type of control strategies. The green interval ranges between 1.0 to 2.0 seconds, the yellow interval ranges between 0.7 to 1.0 second, and the red interval typically ranges between 2.0 to 15.0 seconds. Cycle lengths, which are smaller than 4.0 seconds, are not sufficient for drivers to stop and then merge into the freeway. Cycle lengths greater than 15.0 seconds cause driver frustration and high rates of violation. Different states use different timing parameters for ramp metering as shown in Table 1 (Tian et al. 2002).

Table 1: Timing parameters of ramp-metering for different states (Tian et al, 2002)

State	**Timing parameters for ramp metering, sec**		
	Green	**Yellow**	**Red**
Arizona	1.5	NA	1.5~10.0
California	2.0	2.0[1]	2.0~15.0
Colorado	2.0~2.5	NA	2.0~13.0
Georgia	1.5	NA	2.5~8.0
Illinois	1.0	NA	3.0~12.0
Michigan	1.5	NA	2.5
Minnesota	1.3	0.7	0.1~13.0
Oregon	2.0	NA	0.4~12.0
Texas	1.0 1.0~5.0	1.0 1.7	2.0~5.0[2] 2.0~4.0[3]
Wisconsin	2.0~2.5	NA	2.5~10.0[4] 1.8~8.0[5]
Utah	2.0	NA	2.0

Notes:
1- Used when cycle is greater than 6 seconds or two car per green.
2- For fixed time ramp meter
3- For traffic- responsive ramp meter
4- For single lane ramp meter
5- For multi-lane ramp meter

Further information related to the history of the ramp metering in the United States, and types of ramp metering control systems and algorithms are explained in chapter two.

1.3 Problem Statement

As mentioned in the previous section, merge and diverge ramps cause turbulence in a freeway's traffic stream. The vehicle's turbulence from the ramps affects both the safety and capacity of the freeways. Increasing crashes or conflicts, which are caused by freeway vehicles' lane change and deceleration maneuvers, are two examples of this turbulence. Changing capacity of the ramp-freeway junction, density in the ramp influence area, and speed near the ramp-freeway junction are examples of turbulence that affect the efficiency of freeways. The purpose of this study is to explore the effectiveness of ramp metering on three major subjects, the efficiency, Level of Service, and safety of the freeway. The study depends on the comparisons of efficiency, Level of Service, and safety parameters with and without ramp metering. Efficiency and Level of Service of the freeways were evaluated by comparing speed, travel time, and density of the freeway, which were obtained by using a microscopic traffic simulator program known as Verkehr In Städten – SIMulations Model (VISSIM). During running the program, on-ramp queue lengths were measured in order to avoid queue spillback on the local or arterial streets upstream of the on-ramps. Safety analyses were done by comparing conflict modification factors regarding overall conflicts, types of conflicts, and severity of conflicts. A surrogate Safety Assessment Model (SSAM) software program, which was developed by Federal Highway Administration (FHWA), was used to find numbers, types, and severity of conflicts. Both efficiency and safety analyses were done for different traffic volume scenarios at three ramp-freeway junctions with different geometric configurations. Several ramp metering rates were used and compared with the base case (no ramp metering). A sensitivity analysis was done by evaluating the changes of the effects of the ramp metering on efficiency and safety after altering two assumptions for the freeway.

1.4 Research Objectives

The objective of this research is to better understand the effects of ramp metering on the efficiency, Level of Service, and safety on the freeways on which they are used. Additionally, by exploring parameters such as volumes, geometric configuration, and ramp meter signal timings, the results of this research can be useful to guide highway agencies that may be considering installing ramp meters. Specifically, highway agencies will be able to determine the combinations of volumes, geometric configurations, and ramp signal timings that would prove to be beneficial for their specific location.

1.5 Glossary of Terms-Quick Reference Guide

- ✓ *"A weaving area* between adjacent entrance and exit ramps is essentially a combined acceleration and deceleration area, usually with a combined acceleration and deceleration lane running from one ramp to the next" (HSM, 2010).
- ✓ *An auxiliary lane* "is defined as the portion of the roadway adjoining the through lanes for speed change, turning, storage for turning, weaving, truck climbing, and other purposes that supplement through-traffic movement" (AASHTO, 2011).
- ✓ *"Crash Modification Factors* ***(****CMFs)* quantify the change in crash frequency (crash effect) at a site caused by implementing a particular treatment, also known as a countermeasure, intervention, action, or alternative. CMFs are used to estimate the potential change in crash frequency or crash severity of a particular action, or to compare among different actions. The comparison involves evaluating the crash frequency with or without a particular treatment, or estimating crash frequency with one treatment versus a different treatment" (HSM, 2010).
- ✓ *Conflict Modification Factor (cMF)* is an alternative to CMF that quantifies the potential change in conflict frequency, or conflict severity of a particular action. cMFs are calculated by using the following formula:

$$\text{Conflict Modification Factor} = \frac{\text{Conflicts using a particular action}}{\text{Conflicts without using a paricular action}}$$

- ✓ *Equivalent Property Damage Only (EPDO)* is the addition of the weighted number of injury and fatal crashes to the number of Property Damage Only (PDO) crashes number (Mulinazzi and Russell, 1994).
- ✓ *Equivalent Potential Conflicts (EPC)* is the addition of the weighted number of the slight and serious conflicts to the number of potential conflicts. The idea of EPC was taken from EPDO.
- ✓ *Influence area is* the area where the increases in local density, congestion, and reduced speeds are generally observed due to merging or diverging traffic from ramps (AASHTO, 2011).
- ✓ *Interchange spacing* is the distance from one interchange influence area to the next interchange (HSM, 2010).
- ✓ *Lane Balance*
    a) "At entrances, the number of lanes beyond the merging of two traffic streams should not be less than the sum of all traffic lanes on the merging roadways minus one, but may be equal to the sum of all traffic lanes on the merging roadways"
    b) "At exits, the number of approach lanes on the highway should be equal to the number of lanes on the highway beyond the exit, plus the number of lanes on the exit, minus one"(AASHTO, 2011).
- ✓ *Merge/diverge areas* are defined as those portions of the freeway at an interchange where vehicles entering and exiting must change lanes to continue traveling in their chosen direction (HSM, 2010).
- ✓ *Occupancy (in ramp metering algorithms):* refers to the percentage of time that there is a vehicle over the detector. Occupancy is used as a measure of traffic density. In ramp

metering, occupancy is used as a direct indication of mainline congestion (Gordon et al. 2005).

- ✓ *Post Encroachment Time (PET):* is the time lapse between the end of encroachment of a turning vehicle and the time that a through vehicle actually arrives at the potential point of collision (Gettman and Head, 2013).
- ✓ *SSAM* is a software application designed to perform statistical analysis of vehicle trajectory data output from microscopic traffic simulation models. The software was developed by Siemens and it is funded by the Federal Highway Administration (FHWA) (SSAM software manual, 2008).
- ✓ *Speed change lanes* that include acceleration and deceleration lanes at on-ramps and off-ramps, respectively, typically connects two facilities with differing speed limits. Speed change lanes include several design elements, such as lane width, shoulder width, length, and taper design (HSM, 2010). "The length of the speed change lane is measured from the point at which the ramp lane and lane one of the main facility touch to the point at which the acceleration or deceleration lane begins or ends. This definition includes the taper portion of the acceleration or deceleration lane and is the same for both parallel and tapered lanes" as shown in Figure 4 (Roess et al, 2011).
- ✓ *Time-To-Collision (TTC)* is defined as the time required for two vehicles to collide if they continue at their present speed and on the same path (Zajic, 2012).
- ✓ *Traffic Breakdown* can be defined as a transition process from an uncongested state to a congested state (stop-and-go). Conventional traffic flow theory assumes that freeway breakdown occurs when demand exceeds capacity (Lu and Hadi, 2011).

✓ *VISSIM* is a microscopic, time-step and behavior-based simulation model developed to model urban traffic and public transport operations and flows of pedestrians (VISSIM 5.30-05 User Manual, 2011).

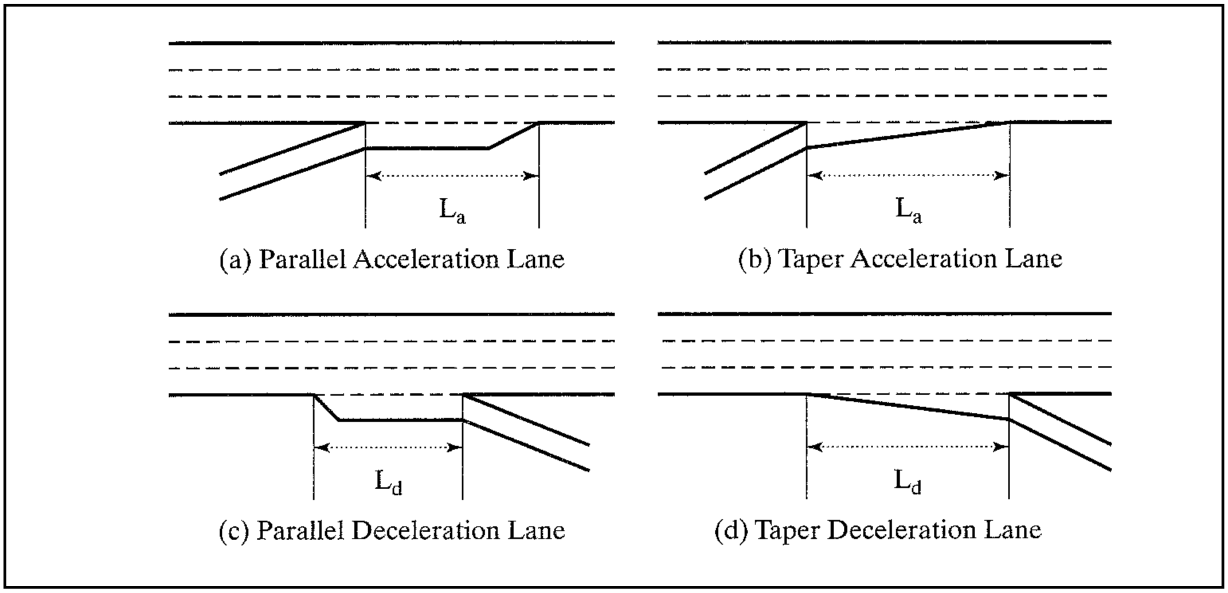


Figure 4: Measuring the length of acceleration and deceleration lanes (Roess et al. 2011)

1.6 Study Organization

The flowchart of this dissertation is illustrated in Figure 5. Accordingly, the dissertation consists of six chapters that they subdivided into several sections. Chapter 1 is an introduction that provides an explanation of freeway components and merging maneuver turbulence of vehicles, detailed explanation related to components, strategies, and timing parameters of ramp metering, problem statement, research objectives, and selected definitions related to the study and the literature review. Chapter 2 is a comprehensive literature review, which consists of history of ramp metering, and many studies that have been done by researchers. This chapter starts by explaining the history and types of ramp metering control systems and algorithms. Also in this chapter, previous studies are reviewed by concentrating on the effectiveness of ramp metering: on on-ramp and freeway operational capacity and safety, work zones, air pollution, driver behavior, benefit-cost ratio, metering types and ramp-metering algorithms. Both simulation and field study assessments are analyzed. Chapter 2 also contains some other studies about geometric design, driver behavior, safety, capacity, bottlenecks of ramp influence areas without using ramp metering. Chapter 3 explains the research methodology relating to simulation models, calibration process, and efficiency and safety evaluation criteria that were used in the study. Research methodology is continued in Chapter 4, which includes the detailed procedures about site selection of the interchanges, traffic data collection, the calibration process, designing of both ramp metering signal rates and geometrics of the freeway and on-ramps, building VISSIM models and assumptions, running SSAM programs, and detailed steps taken for the operational and safety analyses. It also includes the sensitivity analysis of several assumed factors. Chapter 5, which shows the study results and discusses the findings in detail, consists of five parts: the effectiveness of ramp metering on efficiency, Level of Service, and safety of the freeway, queue length on the on-ramp, and the sensitivity analysis. Chapter 6 contains conclusions, recommendations, and thoughts about future studies. Appendices show the tables and charts of the detailed calculations. In addition, the appendices include the results of the outputs, which were obtained by using the VISSIM and the SSAM programs.

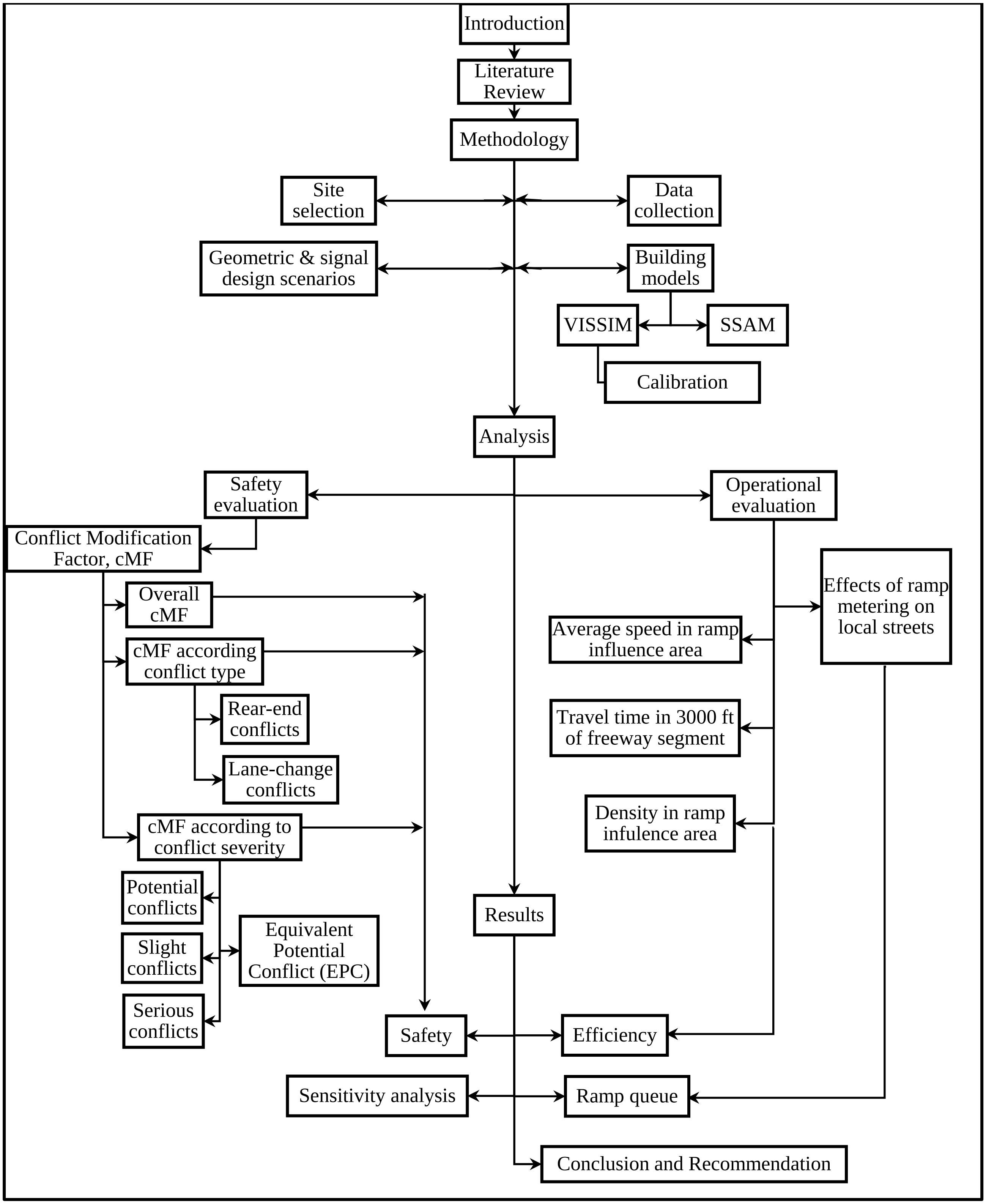


Figure 5: Study flowchart

CHAPTER 2: LITERATURE REVIEW

Many studies have been conducted to know the effectiveness of merging maneuvers on ramp-freeway junctions and/or ramp metering systems on freeway efficiency and safety. This chapter starts by explaining the history and types of ramp-metering control systems and algorithms that were used in several previous studies. The chapter also contains a comprehensive literature review of some of the traffic, economic, social, and environmental factors related to merging maneuvers and ramp metering systems. The merging maneuver studies cover the effectiveness of many traffic flow, driver, and road design parameters on the efficiency, capacity, and safety of freeways. Also, the studies that have been done about the effectiveness of geometric design configurations, ramps and freeway traffic flow, driver behavior at merging and diverging ramps, and traffic compositions on the merging maneuver, are reviewed. The mentioned ramp-metering studies cover the impacts of ramp-metering systems on efficiency and safety, including fixed-time, algorithm control systems, evaluation by using field data, evaluation by using traffic simulation, temporary ramp metering at work zones, violations, driver behavior, traffic control management, benefit-cost ratio analysis, society feedback, and environmental concerns.

2.1 History and Types of Ramp Metering Systems and Algorithms

This section contains detailed information related to the history and types of ramp metering control systems and algorithms that were used in many previous studies.

2.1.1 History of Ramp Metering in the United States

In 1963, the first ramp metering system was implemented on Chicago’s Eisenhower Expressway, which was manually controlled in the field by a traffic enforcement officer. In 1970, the first two fixed-time ramp meters were installed on I-35E north of downtown St. Paul, Minnesota. In 1972, Minnesota DOT upgraded the ramp metering system to operate on a traffic responsive basis. By the end of 2005, it was estimated that 2,370 ramp meters had been deployed in the United States (Gordon et al. 2005). In early March 2010, The Kansas Department of Transportation (KDOT) and the Missouri Department of Transportation (MoDOT) deployed ramp metering systems on seven interchanges on I-435 between Metcalf Avenue and the Three Trails Memorial Crossing in the Kansas City metropolitan area (KDOT & MoDOT, 2011). At the time of this dissertation, ramp metering has become an effective ramp management strategy, which has been deployed in several states.

2.1.2 Types of Ramp Metering Control System

Selecting the type of ramp metering system depends on many factors such as the desired improvement, existing traffic conditions, costs of installation, and operating and maintaining the system effectively. Ramp metering is divided into two classes according to its response to real-time traffic conditions: fixed and actuated times. Fixed-time operation is the simplest type of ramp metering; it breaks up platoons into single vehicle entries and limits the flow rates that enter the freeway (Piotrowicz and Robinson, 1995). Fixed-time can be effective in eliminating recurrent congestion and reduce the likelihood of severe incidents or sudden changes in demand. Historical traffic data determine the rate of metering in fixed-time systems (Zhang et al. 2001). Actuated-time metering can be used by installing presence and passage detectors that terminate the metering cycles and is based on average traffic conditions at a particular ramp. As an initial operation system, pre-timed control can be established until the information becomes available from the individual ramps. Traffic responsive ramp metering is the next level of control that is based on actual freeway conditions. This type utilizes detectors and a microprocessor to determine the freeway flow and ramp demand. Based on total freeway conditions, system-wide control can be established by centralized computer controlled systems at numerous ramps (Piotrowicz and Robinson, 1995). Ramp metering can be classified according to operational level, geometry, location, and operations rules as shown in Figure 6.

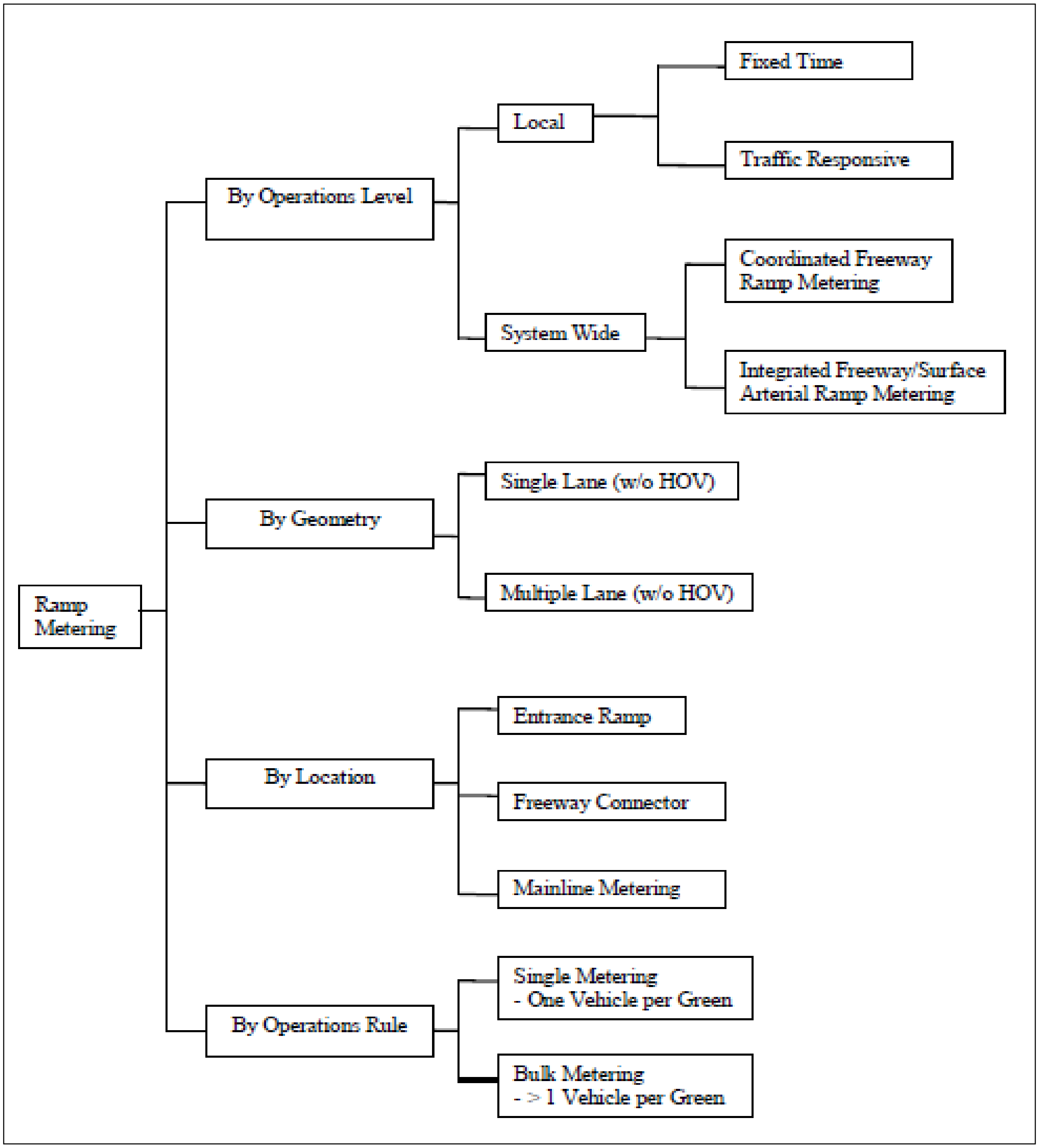


Figure 6: Ramp metering classification (Tian et al. 2002)

2.1.3 Ramp Metering Algorithms

Pre-timed ramp metering systems have been largely replaced by more sophisticated traffic responsive ramp metering algorithms that were developed to cope with daily fluctuations and non-recurrent freeway conditions. Ramp metering algorithms are designed for variable metering rates depending on real-time freeway conditions that are measured in terms of flow, density or occupancy, queue presence and speed from loop detectors on the freeways and on-ramps. Different ramp metering algorithm systems have been deploying in different states. As examples, the Zone algorithm in Minneapolis/St. Paul, Minnesota; the Fuzzy and Bottleneck algorithms in Seattle, Washington; the HELPER algorithm in Denver, Colorado; the SDRMS in San Diego, California; the MILOS algorithm in Phoenix, Arizona; the RAMBO II algorithm in Houston, Texas; the SPERRY algorithm in Arlington, Virginia; and the SWARM algorithm in Orange County, California (Tian et al. 2002). Figure 7 shows the Zhang et al. classification tree for the existing traffic-responsive ramp metering algorithms regarding freeway and measured metering rates conditions.

The ramp metering algorithms are divided into two groups: isolated (local) and coordinated. In isolated ramp metering algorithms, the metering rates are determined based solely on local traffic conditions around the ramp. Coordinated ramp metering algorithms, in which the metering rates are determined based on both local and system-wide freeway conditions, are subdivided into three types: cooperative, competitive and integral algorithms. In cooperative ramp metering algorithms, metering rates are computed based on local traffic information, and then adjusted according to system-wide information about the traffic situation on the whole highway segment. With competitive ramp metering, in which two ramp metering rates are computed for each ramp, one is based on local traffic condition and the other is based on system-wide traffic conditions, and then choosing the more restrictive one. In integral ramp metering algorithms, optimal ramp metering rates are computed by incorporating both local and system-wide traffic conditions (Zhang et al. 2001). In the next section, short summaries of six well-known algorithms are explained.

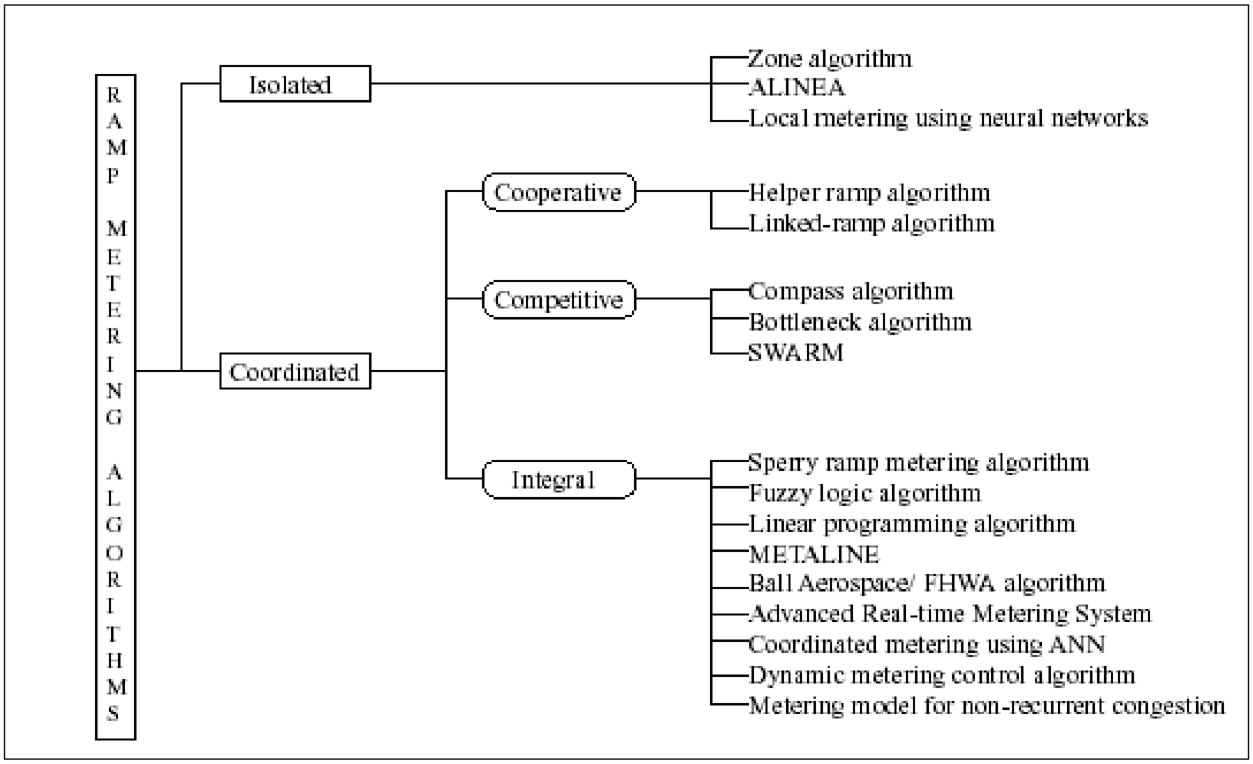


Figure 7: Categories of ramp metering algorithm (Zhang et al. 2001)

2.1.3.1 ALINEA Algorithm

The Asservissement Linéaire d'Entrée Autoroutière (ALINEA) algorithm was the first local feedback ramp-metering strategy, which was proposed by Papageorigou et al. 1997. The ALINEA algorithm has been applied in several European countries (Lee et al. 2006). In the ALINEA algorithm, a straightforward application of classical local feedback control theory was used in an attempt to maximize the mainline throughput by maintaining a desired level of occupancy on the downstream mainline freeway. Two detector measurement stations were required to implement the ALINEA algorithm, one on the entrance of the ramp (station 1 in Figure 8) and the other on the downstream of the freeway mainline (station 2 in the Figure 8) (Papageorigou et al. 1997).

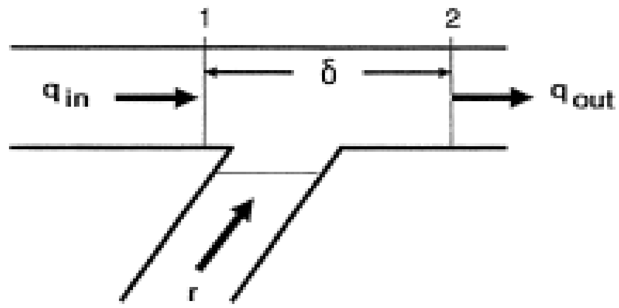


Figure 8: Local ramp metering variables (Papageorigou et al. 1997)

The ALINEA algorithm uses the following equation for deriving ramp metering rates for each period k = 1, 2 … (e.g., every minute).

$$r(k) = r(k-1) + K_R[\hat{o} - o_{out}(k)]$$

Where:

- $\hat{o}$ is the desired occupancy rate downstream of the ramp,
- $o_{out}(k)$ is the measured occupancy rate downstream of the ramp,
- $r(k-1)$ is the measured on-ramp volume for time interval k-1, and
- K_R is a regulator parameter which is greater than zero (K_R = 70 vph was found as excellent results at many different sites) (Papageorigou et al. 1997).

The inability to resolve upstream congestion of the particular ramp was the main disadvantage of the ALINEA algorithm (Greguric et al. 2013).

2.1.3.2 Zone Algorithm

In 1970, MnDOT in the Minneapolis-St. Paul area implemented the Zone algorithm. In the first trial period, they operated the system based on pre-timed metering rates and then they converted to a local traffic responsive system (Lau R, 1997). The mainline freeway corridor was divided into multiple zones of three to six miles in length based on the location of critical bottlenecks in the corridor. The divided zones contained several metered or non-metered on-ramps and off-ramps. Typically, the upstream end of a zone was a free-flow area, whereas the downstream end of a zone was a critical bottleneck. Metering rates were calculated based on traffic volume control in each zone. Making a balance between inflows (traffic volume entering the zone) and outflows (traffic volume leaving the zone) was the basic concept of the Zone algorithm. The Zone algorithm calculates metering rates for each zone by the following equation (Chu et al. 2002):

$$M + F = X + B + S - (A + U)$$

Where:

- M = total metered on-ramp volumes,
- F = total metered freeway-to-freeway volumes,
- X = total measured off-ramp volumes,
- B = downstream bottleneck capacity,
- S = space available within the zone which can be calculated using measured freeway occupancy,
- A = Total upstream freeway volume, and
- U = total measured non- metered ramps volume.

2.1.3.3 Helper Algorithm

In 1981, the Colorado Department of Highways first implemented a real-time local traffic responsive ramp-metering application on five on-ramps on northbound I-25 freeway in Denver, Colorado (Lipp et al. 1991). The implementation of the ramp meters showed beneficial results by a 58 percent increase in freeway speed and 37 percent reduction in vehicle-hours of travel; therefore, in 1984 and in subsequent years, they expanded a centralized coordinated ramp-metering system as well as additional meters on I-25, I-225, US-6, and I-270.

The Helper algorithm (also known as Denver Ramp Metering Control Software) consists of a local traffic-responsive algorithm with the added feature of central override control. The system was divided into six groups, with one to seven ramps per group. Based on local traffic condition, each ramp meter selected one of six available metering rates. Main-line primary and secondary detectors were used to determine traffic parameters in each lane. Metering rates were increased when the queue in the ramp extended back to the cross street. The system coordination plan was considered effective and the ramp was defined as "critical" if both the ramp and the freeway were congested. The plan reduced green time rates in the next upstream ramp after calculating travel time between ramps. In the case of continuing the "critical" conditions, the plan reduced the green time rates of the next two upstream ramps. Adding upstream ramps to the coordination system was continued until the ramps returned to a noncritical condition.

2.1.3.4 Bottleneck Algorithm

In 1981, the Washington Department of Transportation (WSDOT) initiated the Bottleneck ramp metering algorithm, in response to growing congestion problems in the Seattle area (Jacobsen et al, 1989). The Bottleneck algorithm was a competitive, traffic-responsive ramp metering system in which system-level metering rate is calculated based on dividing the freeway segment into several sections. In the Bottleneck algorithm, both local-level and system-level metering rates could be generated. Local-level metering rates were based on local conditions of occupancy levels upstream of the given metered ramp, while the system-level or Bottleneck metering rate was based on system capacity constraints. The more restrictive metering rate of the local-level and system-level was selected and then it was subject to adjustment based on ramp queues adjustment, minimum metering rates, and potentially other conditions. The Bottleneck algorithm was activated when the following two conditions were met (Jacobsen et al, 1989):

1) Capacity Condition

$$P_{it} \geq P_{THRESH_i}$$

Where:

- P_{it} is the average occupancy across the downstream detectors of section *i* over the previous (1-min) period, and
- P_{THRESH_i} is the occupancy threshold for the downstream detector station that defines when section *i* is operating near capacity.

2) Vehicle storage condition

$$q_{IN_{it}} + q_{ON_{it}} \geq q_{OUT_{it}} + q_{OFF_{it}}$$

Where:

- $q_{IN_{it}}$ is the volume entering section *i* across the upstream detector station during the past minute,
- $q_{ON_{it}}$ is the volume entering section *i* during the past minute from the entrance ramp,
- $q_{OUT_{it}}$ is the volume exiting section *i* across the downstream detector station during the past minute, and
- $q_{OFF_{it}}$ is the volume exiting section *i* during the past minute on the exit ramp.

In the Bottleneck algorithm, the metering rate was calculated by the following equation:

$$BMR_{ji(t+1)} = q_{ON_{jt}} - MAX_{i=1}^{n}\left(U_{i(t+1)} * \frac{WF_j}{\sum_j^n \left(WF_j\right)_i}\right)$$

Where:

- $BMR_{ji(t+1)}$ is the bottleneck metering rate of ramp *j*,
- $q_{ON_{jt}}$ is the entrance volume on ramp *j* during the past minute,
- $U_{i(t+1)}$ is upstream ramp volume reduction for section *i* to be acted on in the next metering interval (t+1),
- WF_j is weighting factor for ramp *j*, and $\sum_j^n \left(WF_j\right)_i$ is the summation of weighting factors for all ramps within the area of influence for section *i*,
- $MAX_{i=1}^{n}$ is the operator of selecting the maximum volume reduction if a ramp is inside of multiple areas of influence, and
- $U_{i(t+1)}$ can be calculated in the following equation:

$$U_{i(t+1)} = \left(q_{IN_{it}} + q_{ON_{it}}\right) - \left(q_{OUT_{it}} + q_{OFF_{it}}\right)$$

2.1.3.5 System-Wide Area Ramp Metering (SWARM) Algorithm

SWARM (Paesani G. et al. 1997) was a competitive, traffic responsive ramp metering algorithm, which was developed by the California Department of Transportation's (Caltrans) Freeway Transportation Management System in the Los Angeles area. The algorithm was first implemented at District 12 in Orange County, and later in Los Angles, California. The SWARM algorithm included the use of two approaches: SWARM 1 was a centrally controlled system wide algorithm based on predicted densities at the system's bottleneck location and, SWARM 2 was composed of two separate algorithms. SWARM 2a was a local traffic responsive ramp-metering algorithm, which was based on headway theory. SWARM 2b was based on the number of vehicles stored in the determined section of freeway. In SWARM 1 mode, densities around the bottleneck were used as control parameters to determine and apportion metering rates across the entire freeway network. A mathematical technique (Kalman Filter) accounted for the “noise” in the data to provide a non-linear forecast of the density. The high-level SWARM system implemented the most restrictive potential metering rates by using SWARM 1 and SWARM 2. Figure 9 shows the forecasting theory of SWARM global mode. A tunable parameter, T_{crit} in Figure 9, is the forecasting time span into the future and the excess density is the difference between the forecast density and predetermined threshold density that represents the saturation level at the bottleneck. To avoid congestion, the excess density was converted to the required density as shown in the equation below:

$$Required\ density = current\ density - \left(\frac{excess\ density}{T_{crit}}\right)$$

The corresponding volume reduction at each detector station is computed as

Volume reduction = (local density – required density) * (no. of lanes) * (distance to next station)

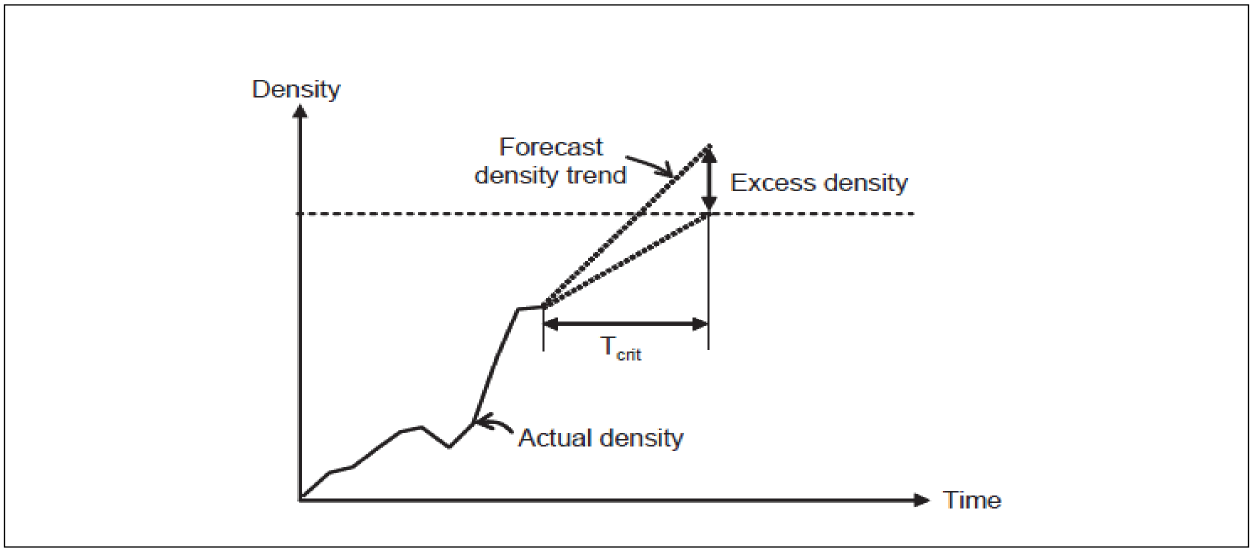


Figure 9: SWARM 1 forecasting theory, (Ahn S. et al. 2007)

The volume reduction was distributed to upstream on-ramps based on weighting factors which regards demand, queue storage, etc. of each on-ramp. One of the advantages of the SWARM algorithm was the capability of cleaning the defective data in case of loop detector failure. Despite this, SWARM was accurate in detecting and avoiding congestion in advance, but its benefits were limited in the case of poor prediction models or inaccurate loop detector data.

2.1.3.6 Fuzzy Logic Algorithm

Fuzzy logic algorithm (Tian et al. 2002) was developed by the University of Washington and implemented in the Seattle metropolitan area. It was designed to overcome the limitations of conventional ramp metering strategies. Seven detector inputs were used with this algorithm, which were: downstream occupancy, downstream speed, upstream occupancy, occupancy at merge, speed at merge, queue occupancy, and advance queue occupancy. The algorithm stressed qualitative information over quantitative information that consisted of three key components. The first key component of fuzzy logic ramp-metering was the defuzzification process in which the detector measurements were converted into one of five different textual classes: very small, small, medium, big, and very big (VS, S, M, B, and VB). The next key component was running the "fuzzified" inputs into an IF-THEN rule presented below:

$$[IF\ very\ small\ AND\ queue\ THEN\ high\ metering\ rate]$$

The final key component was the "defuzzification" process in which the metering rates were determined depending on the rule-base in the second step. The fuzzy logic algorithm had several advantages such as it did not require extensive system modeling, its calibration was relatively easy, and it could utilize partial or imprecise information.

2.2 The Effects of Geometric Design on Safety in Merging Areas

Many studies have been performed to evaluate the effects of merging and diverging ramps on freeways. The studies include the effectiveness of different geometric, traffic, and crash features on efficiency and safety of freeways. In 1999 Bared et al. developed a model by using negative binomial regression to estimate crash frequency for ramps and their adjacent speed-change lanes as a function of ramp Average Annual Daily Traffic (AADT), mainline freeway AADT, deceleration lane length and ramp configurations. The researchers took 276 exit and 192 entrance ramp samples in Washington State in both rural and urban areas. Over a three-year-period, 1,452 crashes occurred, including 644 injury and fatal crashes. Several types of ramp configurations were studied such as diamond, parclo loop, free-flow loop, and outer connector. The study focused on the safety effects of the lengths of acceleration and deceleration lanes and they developed a model which shows that the rate of change of crash frequency on the freeway ramps is proportional with the ramp and freeway AADT and inversely proportional with the deceleration lane length. The analysis results of the study show that crash frequency will decrease by 4.8 percent for every increase of 100 ft in deceleration lane length. The final crash prediction model is given as follows:

$$\mathrm{N} = (\mathrm{RAADT})^{0.78}(\mathrm{FAADT})^{0.13}\exp(-7.27 + 0.45\mathrm{DIA} + 0.78\mathrm{PAR} - 0.02\mathrm{FF} + 0.69\mathrm{OC} \\ - 0.37\mathrm{RUR} + 0.37\mathrm{DECEL} - 2.59\mathrm{SCLEN} + 1.62\mathrm{RLEN})$$

Where:

- N is the expected number of total crashes in a three-year period on the entire ramp and speed-change lane,
- RAADT is the ramp AADT,
- FAADT is the mainline freeway AADT for the direction of travel in which the ramp is located,

- DIA, PAR and FF are dummy variables defined for diamond ramp, parclo loop ramp, and free-flow ramp, respectively,
- OC = 1 if the ramp is an outer connection ramp, 0 otherwise,
- RUR =1 if the area type is rural, 0 otherwise,
- DECEL is a dummy variable for off/on ramp (1 if the ramp is an off ramp, 0 otherwise),
- SCLEN is the speed-change lane length (miles), and
- RLEN is the ramp length (miles).

In 2010, Liu et al. conducted a study, which addressed two issues: "First, how the principles of lane balance and lane consistency are coordinated in the current practical engineering applications and second, what type of lane arrangement has the best safety performance." The freeway segments that were used for their study area were three sections named as A, B and C as shown in Figure 10.

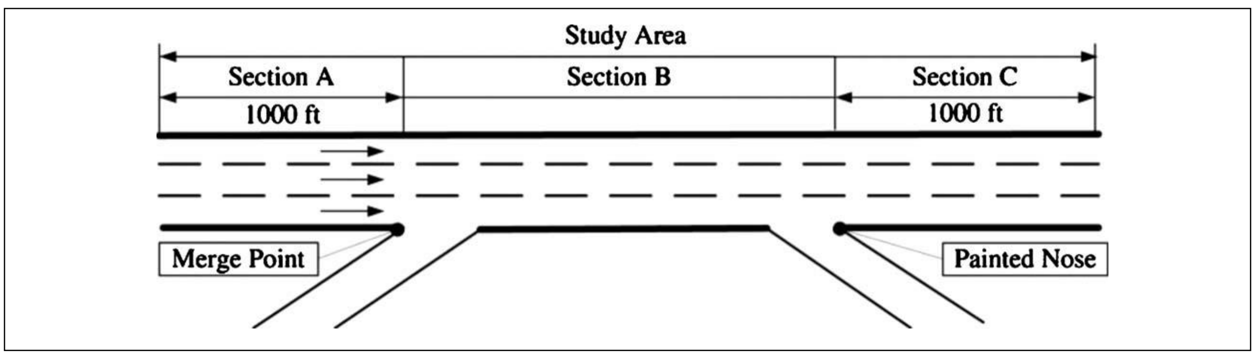


Figure 10: Study area (Liu et al, 2010)

The researchers reviewed 424 aerial photos of freeway segments in the state of Florida. According to their criteria, 66 sites were selected for further investigation. They found that seven different lane arrangements were being used in the current practical engineering applications in the state of Florida, which are designated as Type A to Type G as shown in Figure 11. The most commonly used lane arrangements between closely spaced freeway entrance and exit ramps were Types A, B and C. Therefore, only these types were considered in further crash data analysis. They conducted observational cross-sectional comparisons for crash frequency, crash rate, crash type, and crash severity between freeway segments with different types of lane arrangements. They developed crash prediction models to relate the crash counts reported at selected freeway segments to various explanatory variables such as traffic and geometric characteristics. Two types of crash prediction models were developed; first, a total crash model, which depended on the total number of crashes reported at each selected freeway segment per year, and second, a severe crash model, which depended on frequency of fatal and severe injury crashes reported at each selected freeway segment per year. Their final total crash model is shown below:

$$Y = 0.39 * \ L * \ ADT_e^{0.382} * \exp(0.379 * type_A * +0.757 * type_B + 0.009 * ADT_m + 0.723 * lanes + 0.852 * speed)$$

Where:

- Y = expected crash frequency for a freeway segment (crashes/year),
- L = length of the freeway segment (mile),
- ADT_e = entrance ramp average daily traffic in thousands,
- $type_A$ = indicator variable for Type A arrangement (=1 for type A arrangement, 0 otherwise),
- $type_B$ = indicator variable for Type B arrangement (=1 for type A arrangement, 0 otherwise),
- ADT_m = freeway mainline average daily traffic in thousands,

- Lanes = basic number of lanes on freeways, and
- Speed = indicator variable for posted speed limit on freeway mainlines (=1 if the posted speed limit equals 70 mph)

In the severe crash models, four independent variables were found as shown below:

$$Y_s = 0.96 * \ L * \ ADT_e^{0.387} * \exp(0.703 * type_B * +0.259 * lanes + 0.505 * speed)$$

Where:

- Ys= expected number of severe crashes for a freeway segment (crashes/year).
- L = length of the freeway segment (mile),
- ADT_e = entrance ramp average daily traffic in thousands,
- $type_B$ = indicator variable for Type B arrangement (=1 for type A arrangement, 0 otherwise),
- Lanes = basic number of lanes on freeways, and
- Speed = indicator variable for posted speed limit on freeway mainlines (=1 if the posted speed limit equals 55 mph, 0 if the posted speed limit equals to 70 mph)

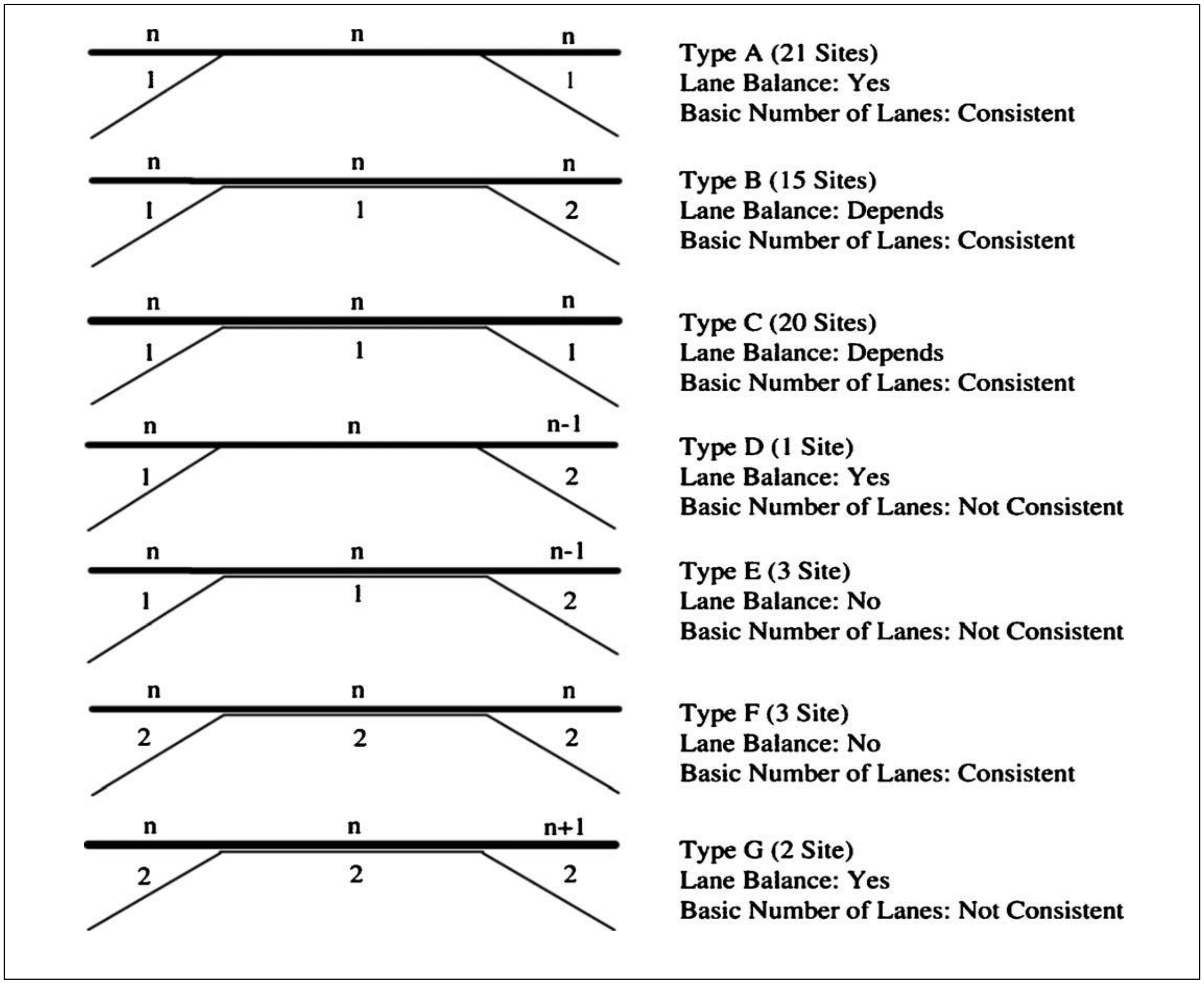


Figure 11: Coordination of lane balance and basic numbers of lanes (Liu et al. 2010)

In order to quantify the relationship between ramp spacing and freeway safety, in 2012, Le and Porter conducted a study “Safety Evaluation of Geometric Design Criteria for Spacing of Entrance-Exit Ramp Sequence and Use of Auxiliary Lanes.” The collected data in the study included three parts: freeway geometric features, traffic characteristics, and crash counts on 404 freeway segments in the states of California and Washington. The study focused only on segments with diamond interchanges, including basic diamonds as well as tight urban diamonds, half diamonds, and single-point urban interchanges. They explored the relationship between ramp spacing and safety by using a negative binomial regression modeling. The STATA software package was used to estimate the coefficients of the safety models. According to the results of the study, expected crash frequency increased as ramp spacing decreased. The proportion of the expected fatal and injured crash types decreased as ramp spacing decreased. The presence of an auxiliary lane was associated with a lower expected frequency of crashes for any given ramp spacing; the safety benefits of providing an auxiliary lane diminished as ramp spacing increased. They also developed three models for estimating total crashes, fatal and injury crashes, and multivehicle crashes.

2.3 Using Traffic Conflicts to Assess Safety in Merging and Diverging Area

In 2011, Yang and Ozbay conducted a study to develop a methodology for estimating the risk of traffic crashes based on investigating potential conflicts caused by the mandatory lane changes of merging vehicles. The methodology consisted of two major components: first, an estimation of the merging probability in which a merging decision depends on many factors such as gaps between vehicles, relative speed, and vehicle type. Second, for an evaluation of the risk of traffic crashes, they analyzed the microscopic vehicle behaviors from the view of traffic conflicts as a function of an indicator known as modified time-to-collision. To demonstrate the applicability of the proposed methodology of traffic conflicts, they used a field vehicle tracking data set called “I-101 Dataset,” which was generated by Next Generation Simulation (NGSIM). “NGSIM is a research project conducted by the Federal Highway Administration (FHWA) to provide resolution and high-quality driver behavior data and algorithms.” They collected data at a southbound segment of U.S. Highway 101 in the Universal City neighborhood of Los Angles, California. Their data came from video cameras with 0.1 second increments that included about 6,000 vehicle trajectories. The NGSIM vehicle trajectory data were analyzed by a statistical software package known as R. They developed a probability model on the basis on some estimated parameters as shown below:

$$\Pr(merge|x = position) = f(x) = \frac{exp\left[-\frac{1}{2}\left(\frac{\ln x - 5.3785}{0.9173}\right)^2\right]}{0.9173\sqrt{2\pi}\, x}$$

They also modeled the probability of conflicts by adopting an exponential decay function as shown below:

$$CP_i = \Pr(conflict|MTTC_i) = exp\left(\frac{-MTTC_i}{5.77}\right)$$

Where:

- MTTC is the modified time-to-collision.

In 2012, Atamo assessed the safety of traffic facilities by using a technique combining micro-simulation and automated conflict analysis. To perform statistical analysis of vehicle trajectory data, the researcher used SSAM that was developed by the FHWA and chose VISSIM as a traffic simulation modeling tool. A set of twenty-one interchanges that consisted of forty-two merging and forty-two diverging areas in Colorado were modeled under a.m peak-hour traffic conditions. The researcher imported trajectory output files from VISSIM and used them in SSAM software to identify traffic conflicts. In the study, five field tests for statistical validation were conducted to compare conflicts predicted by SSAM with actual crash records at merging and diverging influence areas. The researcher concluded that the technique used was valuable in assessing the relative safety performance of one design versus an alternative design. As a result, several prediction models were developed, which show the relation between the overall total crashes per year and other parameters including ADT for the mainline and the ramps, total conflicts, Peak Hour Volume (PHV) of the mainline and the ramp for both merge and diverge as shown in the Table 2. Several other prediction models were developed for rear-end types of crashes and conflicts.

Table 2: Atamo's prediction models by using SSAM software program (Atamo, 2012)

Merge: ADT & Crash	$\frac{\text{Total Crash}}{\text{Year}} = (2.12\text{E}-04) \text{ x } \text{ADT}_{\text{mainline}}{}^{0.773} \text{x } \text{ADT}_{\text{merge}}{}^{0.209}$
Diverge: ADT & Crash	$\frac{\text{Total Crash}}{\text{Year}} = 0.061 \text{ x } \text{ADT}_{\text{mainline}}{}^{0.058} \text{x } \text{ADT}_{\text{diverge}}{}^{0.478}$
Merge: Crash & Conflict	$\frac{\text{Total Crash}}{\text{Year}} = 1.072 \text{ x Total conflicts}_{\text{merge}}{}^{0.373}$
Diverge: Crash & Conflict	$\frac{\text{Total Crash}}{\text{Year}} = 2.617 \text{ x Total conflicts}_{\text{diverge}}{}^{0.204}$
Merge: Conflict & PHV	$\frac{\text{Total Crash}}{\text{Year}} = 0.071 \text{ x } \text{PHV}_{\text{mainline}}{}^{0.659} \text{x } \text{PHV}_{\text{merge}}{}^{0.394}$
Diverge: Conflict & PHV	$\frac{\text{Total Crash}}{\text{Year}} = (1.51\text{E}-05) \text{ x } \text{PHV}_{\text{mainline}}{}^{1.264} \text{x } \text{PHV}_{\text{diverge}}{}^{0.965}$

2.4 On-Ramp Merging Maneuvers and Driver Behavior

Studies have been conducted on exit and entrance ramps on freeways to understand the effects of the merging and diverging maneuvers on driver behavior, and efficiency and safety of freeways. In 2009, Kondyli conducted a Ph.D. dissertation, "Breakdown Probability at Freeway-Ramp Merges Based on Driver Behavior." The researcher considered three types of merging maneuvers (free, cooperative, and forced) based on the degree of interaction between the on-ramp and freeway vehicles. Breakdown probability models were obtained for all three types of merging maneuvers for freeway vehicle behavior and merging turbulence models were developed for the effect of merging maneuvers on the breakdown of freeway flow. The researcher realized three types of drivers (aggressive, cooperative, and conservative) depending on the driver actions during merging maneuvers (decelerate, change lanes, and do nothing). Two types of data were collected and used to calibrate the driver behavior models. First, for the purpose of understanding drivers' thinking during merging, the researcher asked some question to volunteer drivers (with different characteristics) and also she put cameras in the inside of a driver's vehicles to observe driver actions and reactions from the inside the vehicle for both the freeway and merging vehicles. The second type of data describe the vehicle's interaction in the traffic stream such as gaps, gap change rates between the lead/lag freeway vehicle and ramp vehicle, relative speeds, and accelerations.

Acceleration lanes, which provide access to freeways to transit low-speed ramp vehicles to high speed freeway vehicles, are the other important subject of the studies. In 2011, Calvi and De Blasiis evaluated driving performance on freeway acceleration lanes using a driver simulator. They investigated the effects of different design variables on driver behavior during merging maneuver. Thirty volunteer drivers performed driving simulation at the System Technology, Inc., driving simulator at the CRISS laboratory where realistic view of roads and surrounding environments were provided. Two scenarios were designed with two different lengths of acceleration lanes for two-lane freeway. In the first scenario, an Italian freeway acceleration-lane length formula was used as shown below:

$$L_a = \frac{V_{d2}^2 - V_{d1}^2}{2a}$$

Where:

- L_a is the distance between end of the curve ramp and the beginning of taper (300 m),
- V_{d1} is the design speed of ramp curve in m/s at beginning of acceleration lane (18 m/s),
- V_{d1} is 80 percent of the design speed in m/s of main lane (31 m/s), and
- a is the acceleration (1 m/s^2).

In the second scenario, the length of the acceleration lane was increased by adding a merging segment length (L_m = 225 m) into the previous Italian formula for the length of the acceleration lane, where L_m was the segment where drivers change the lane after accelerating. Three different freeway traffic volumes were used for each scenario, which were high traffic (3,000 vph), medium traffic (1,500 vph), and low traffic (1,000 vph). Statistical hypothesis tests and vehicle trajectory analyses were performed to understand the effectiveness of traffic flow and acceleration lane length on driver behavior during merging maneuvers. They concluded that driving performance during merging maneuvers was significantly affected by main lane traffic volumes, while it was not affected by acceleration lane length. Specifically, they mentioned that as the traffic volume increases, so does the merging length of the driver; the acceleration oscillations and the number of gaps rejected also increased.

In 2011, Brewer et al. studied driver behavior at freeway entry or exit maneuvers to assess existing design guidelines for speed change lanes in freeways. To identify behavioral patterns and influences of driver operations on freeway ramps, many detailed indirect measures of driver behavior were observed. The observed indirect measures were speed, acceleration and deceleration, using of throttle and brake pedals, drivers' glancing activity, and the presence of a leading vehicle during the merge or diverge maneuver. An instrumented vehicle equipped with multiple integrated systems was used to record various data relating to driver behavior, traffic conditions, and vehicle performance. Data acquisition systems on a central computer managed all onboard equipment. Video cameras were also used to provide adjacent traffic conditions and in-vehicle driver behaviors. They collected field data from 18 different locations of exit and entrance ramp locations in the metropolitan area of Dallas-Fort Worth, Texas. The result of the study showed that in uncongested or lightly congested conditions, drivers used at least half of the speed-change lane lengths during merging the freeways, while drivers seldom entered the speed change lane within the first 50 percent of the provided length during diverging the freeways. As a result, the researchers concluded that the AASHTO Policy on Geometric Design of Highways and Streets provides sufficient lengths of speed change lanes on freeway entrance ramps while it does not provide enough length for speed change lanes on freeway exit ramps.

2.5 Efficiency and Safety Evaluation of Ramp Metering Using Field Traffic Data

According to studies that have been done in California, Colorado, Minnesota, Oregon, and Washington, ramp metering has important benefits for traffic efficiency and safety on freeway. These studies showed that ramp metering increases the travel speeds of vehicles, helps smooth out peak demands, increases the throughput of a freeway, sustains greater traffic volumes than without metering, improves traffic flow by reducing the impacts of recurring congestion, reduces traffic crashes and reduces certain vehicle emissions. “The data from Denver, Detroit, Minneapolis, San Diego, and Seattle show mainline volumes well in excess of 2,100 vphpl on metered sections, and sustained volumes in the range of 5 to 6 percent greater than pre-metered conditions.” They also show that ramp meters reduce crash rates by 24 to 50 percent, increase throughout of 17 to 25 percent and increase mainline speeds by 16 to 62 percent (Piotrowicz & Robinson, 1995).

According to an FHWA survey (Meyer, 1997) that was made for seven ramp metering systems in the U.S. and Canada, ramp metering increased average highway speeds by 29 percent; they increased average speeds by 20 percent when delay on ramps was included. Table 3 shows the summary results of the Meyer’s study, which shows ramp metering impacts on speed, travel time, crashes, and traffic volumes from five locations in the United States. According to the table, speeds increased by an average of 12.5 mph and travel times decreased by an average of 41.5 percent after using ramp metering. The table also shows the benefits of ramp metering for safety and indicates that crashes in four states were reduced by an average of 28.5 percent.

Table 3: Summary of ramp metering impacts (Meyer, 1997)

States	Before speed	After speed	Travel time	Crashes	Volumes
Portland, OR	16 mph	41 mph	-61%	-43%	NA
Minneapolis	34 mph	46 mph	NA	-27%	+32%
Seattle	NA	NA	-48%	-39%	+62%
Denver	43 mph	50 mph	-37%	-5%	+19%
Long Island, NY	29 mph	35 mph	-20%	NA	0%

Liu and Wang (2013) assessed ramp-metering impacts on freeway operational safety near on-ramp entrances. They examined vehicular collisions for 19 ramp meters locations along several freeways in northern California, including US Routes 50 and 101, State Routes 85 and 99, and Interstates 5, 80, 205, and 580. To analyze the effects of ramp metering impacts on safety on the freeways for known traffic volumes of the on-ramps and freeways, collision data were collected for six years-three-years before and three-years after installing the ramp meters. Three indicators were introduced for the assessment: first, the percentage of reduction in collision numbers regardless of traffic volume; second, the percentage of the collision rate reduction by regarding different collision rates (in the unit of per million vehicles), and third, the rate of reduction by considering the number of interactions among the on-ramp and freeway vehicles. Depending on the results of the three indicators used for the evaluation, they concluded that ramp meters have positive effects on safety by reducing around 36 percent of freeway collisions near on-ramp entrances.

In 1997, Gaynor et al. evaluated the operational effectiveness of ramp metering systems on one of the Houston’s most congested freeways. They selected the Katy Freeway (I-10) to be the initial test site to return ramp metering to Houston. Eagle RMC300 controllers had been used which were capable of operation in a real-time traffic adaptive role; however, fixed-time control was used as the initial plan at each of the entrance ramps. Three-section head signal with cycles operating at 1.5 seconds green, one second amber, and one second red were used depending on the maximum metering traffic rate of 1,029 vehicles per hour. The controllers allowed the ramp meters to revert to the "dark phase" when queues backed up through cross streets. The comparison results on the 3.65 mile section of I-10 eastbound showed that the average travel time was decreased by 24 percent, and the average speed was increased by 9.4 mph. Travel time was not changed significantly in the westbound direction due to a major bottleneck at the entrance from the Sam Houston Tollway that controlled the freeway operations during the p.m. peak hours.

Some studies have been performed about the benefits of ramp-metering for increasing capacity for both cases of breakdown and non-breakdown activation. A study by Cassidy and Rudjanakanoknad in 2002 was conducted about the roles of ramp metering in the case of breakdown activation. They collected high-resolution traffic data during four afternoon rush hours when the site became an active bottleneck. The data were collected from loop detectors on a stretch of eastbound highway 22 (three-lane freeway) and its junction with the Fairview Avenue on-ramp (one-lane on-ramp) in Orange County, California. They also recorded individual vehicle arrival times at two sections of the freeway by using video cameras and used them as the primary performance for the analyses. According to the study results, the bottleneck activation originated on the shoulder lane and spread quickly to the other lanes, which impedes freeway discharge flows from the merge. It was also demonstrated that on-ramp metering reduced the total delay at the merge and increased freeway discharge flows by postponing the bottleneck activation and increasing service rates for the merge areas.

Certain issues concerning on-ramp metering and delay reduction have been clarified by Cassidy (2003). The researcher used a sports stadium as an analogy, which had some similarity in its geometrics with freeways. This hypothetical queuing system (as shown in the Figure 12) has been used to show that commuter delay is decreased by ramp metering to promote higher freeway outflows (higher off-ramp flow plus higher flows existing in the system's downstream-most freeway link). The researcher also explained that a metering logic that increases outflows at one freeway site could differ from the logic needed at another site. It was emphasized by showing that certain metering algorithms can increase delay and reduce outflows when the freeway is plagued by a diverge bottleneck. It also has been realized that on-ramp metering can be used to transfer freeway delay to on-ramps and nearby surface streets.

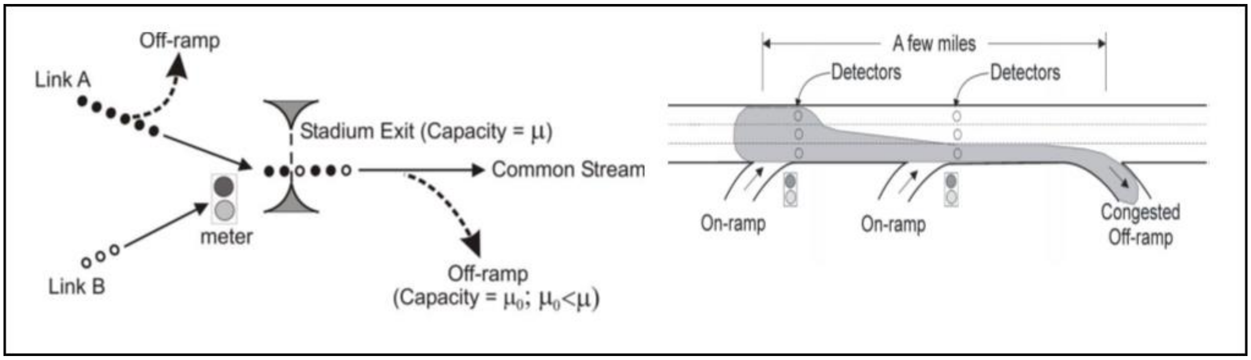


Figure 12: Diagram of simple queuing system and hypothetical freeway site, (Cassidy, 2003)

In 2009, Zhang and Levinson conducted a study about the effectiveness of ramp meters on the capacity of active freeway bottlenecks. They considered some geometric configurations on freeways as bottlenecks such as weaving sections, two major freeways with short joint sections (<1km), locations near bridges with narrow shoulder or inside tunnels, freeway sections with visually identifiable horizontal curves or uphill grade along the direction of travel, and lane drops. They identified and studied 27 active bottlenecks on freeways in the Minneapolis-St. Paul, Minnesota metropolitan area for two seven-week study periods (seven weeks with ramp metering and seven weeks without ramp metering). Queue activation when the upstream had uncongested flow conditions and the downstream was congested was considered as an active bottleneck. They proposed a methodology for identifying active freeway bottlenecks in a metropolitan area, and then a series of statistical hypothesis tests were developed to compare the relationship between ramp metering and the capacity of active bottleneck against empirical multi-bottleneck dataset. The researchers concluded three positive impacts of ramp metering, which resulted in increasing bottleneck capacity. First, ramp metering postponed and sometimes eliminated bottleneck activation; they noticed that the average duration of the pre-queue transition period across all studied bottlenecks was 73 percent longer with ramp metering than without. Second, the freeway accommodated higher flows during the pre-queue transition period than without metering; they noticed that the average flow rate during the transition period was 2 percent higher with metering than without. Third, the ramp meters increased queue discharge flow rates after breakdown. They noticed that the average queue discharge-flow-rate was 3 percent higher with metering than without.

In 2011, KDOT and MoDOT evaluated the effectiveness of ramp metering systems on I-435 in the Kansas City metropolitan area. The evaluation depended on several traffic elements, which were safety, traffic operations, ramp delay, compliance, incident management and community feedback. Crash data were collected for two years before and one year after operating the ramp meters. Safety results showed that the average number of crashes for two years before ramp metering installation was 44, while the number of crashes in the year after ramp metering installation was 16; this result suggested that ramp metering could decrease crash rates on I-435 by 64 percent. By using the Floating Car method, travel time and speed data were taken one year before and two years after ramp metering installation. Ramp meters increased speeds during rush hours on several segments of I-435; however, some speeds were decreased along the corridor during the afternoon rush hour period. Travel Times Index (TTI), which was equal to the average travel time divided by the free flow travel time, was used as an indicator to evaluate the effects of the ramp meters on the net overall freeway segments of I-435. Figure 13 shows that ramp metering decreased the TTIs along I-435 freeway segments, which indicates that ramp metering improved the net overall travel times. The results of their community feedback survey indicated that motorists had generally accepted the ramp meters. In the view of traffic incident management, the authors indicated that ramp metering would give faster incident clearance by emergency responders. As a conclusion, they mentioned that ramp metering was benefiting traffic flow on I-435 (KDOT and MoDOT, 2011).

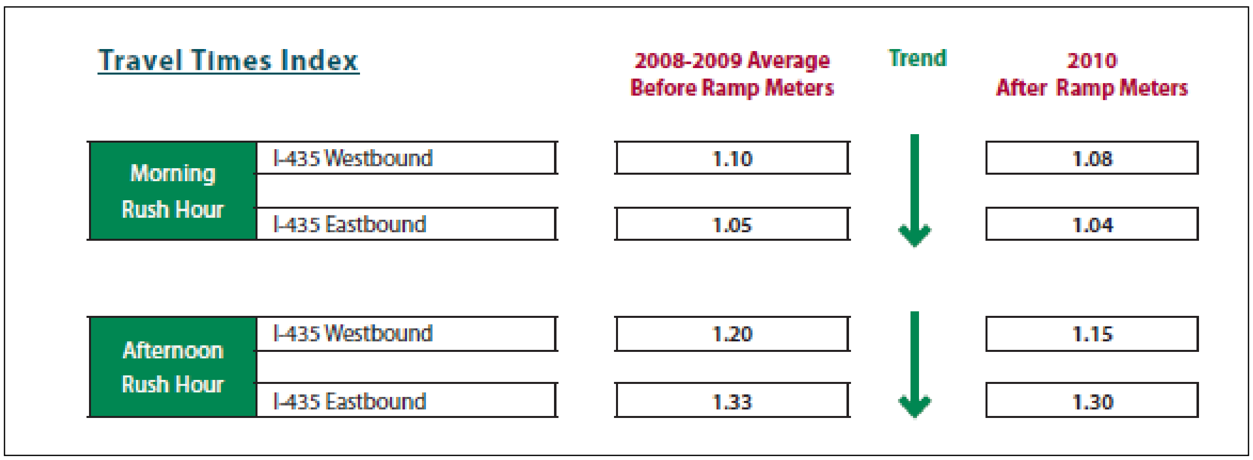


Figure 13: TTIs before and after implementing ramp metering on I-435 freeway (KDOT and MoDOT, 2011)

A study was performed to evaluate the benefits of changing ramp-metering strategies using traffic field data. Ahn S. et al. (2007) studied a true before and after evaluation of the benefits of a new System-wide Adaptive Ramp Metering System (SWARM) by using existing data stream, surveillance, and communications infrastructure in Portland, Oregon. An existing pre-timed ramp-metering system was replaced by SWARM on six major corridors. The study was to quantify the benefits of the SWARM system with respect to savings in delay, emissions and fuel consumption, and safety improvements on the freeways and ramps. They conducted a pilot study for two weeks in June 2006 on a seven-mile freeway corridor of OR-217 Southbound that contains 12 on-ramps, ten of which were controlled by ramp meters. Data were collected from loop detectors and video data from cameras for one week while the ramp meters were operating at the pre-timed rates and then, for one other week while the ramp meters were operating the SWARM system. Changes in the freeway concerning flow, speed, travel time, delay, vehicle miles traveled (VMT), and vehicles hours travelled (VHT) were measured. They found that the VMT increased marginally by 0.8 percent. However, the VHT and the average travel time increased by 6.0 percent and 5.1 percent, respectively, under the SWARM operation. The increased VMT and VHT corresponded to a significant increase of 34.7 percent in total freeway delay as shown in the Figure 14.

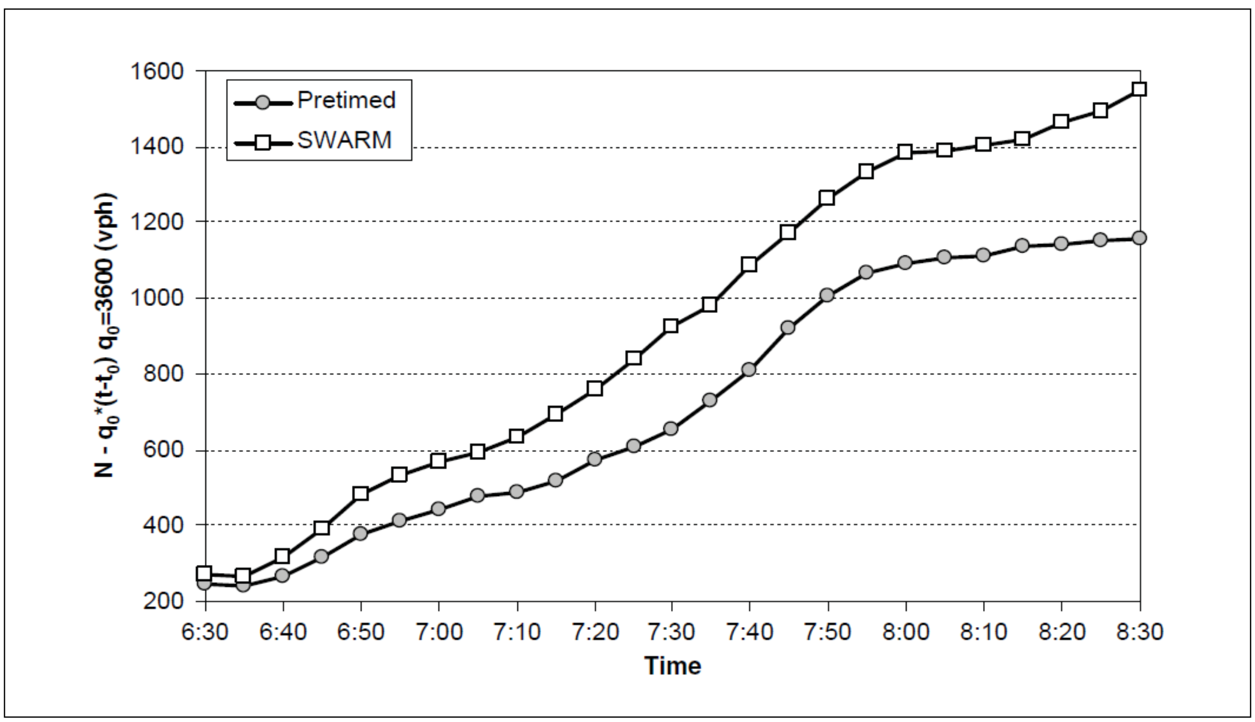


Figure 14: Changes in delay under SWARM in time-space plane. (Ahn S. et al. 2007)

2.6 Evaluation of Ramp Metering Systems Using Traffic Simulation

Microsimulation traffic software programs are playing an important role in transportation and traffic engineering studies. These microsimulation programs allow the engineers to compare different scenarios or designs and choose the best one prior to implementing the project. Several studies have been done on ramp metering performance evaluation using microsimulation. Many microscopic models are considered appropriate to be used for evaluating ramp metering such as AIMSUN2, CORSIM, MITSIM, PARAMICS, TRANSIM, and VISSIM. These microscopic models are able to calculate the state of individual vehicles continuously or discretely, and make predictions based on vehicle-vehicle interactions (Chu and Yang, 2003). The next section contains a review of literature focused on the operational impacts of ramp metering in which the signal rates are controlled by both fixed-time and algorithm systems using traffic simulation software programs.

2.6.1 Evaluation of Fixed-Time Ramp Metering System Using Traffic Simulation

Poorjafari and Yue (2013) used the AIMSUN microsimulation program to assess the probable impacts of fixed-time ramp metering on operational characteristics on an urban highway section. Two ramp control scenarios were used for the evaluation: no-control and a time-of-day metering plan. They developed three different metering scenarios based on the car releasing pattern for both peak and off-peak periods. The scenarios used included one-car-per-green, two-car platoon, and three-car platoon per green. Peak and off-peak hour traffic data were collected on a 400 meter-section of the Niyayesh Highway in Tehran's highway network where ramp metering had not been implemented. The measures of effectiveness as key operational parameter assessments were mean delay time, total travel time, density, and mean speed for the freeway and queue length for the on-ramp. The results showed that ramp metering could improve the highway performance especially in the peak-hours, but at the cost of increasing the delay for the ramp users. They concluded that ramp metering could not benefit the whole system; therefore, they recommended a thorough site investigation prior to implementing ramp metering.

Kesten et al. (2013) studied the effects of fixed-time ramp metering in alleviating the traffic congestion on an urban freeway. They studied the O1 route in Istanbul, Turkey, which connects highway traffic from Asia to Europe. The corridor was approximately seven kilometers in length where there were six entrance ramps and two exit ramps up to the Bosporus Bridge. The on-ramps had two different geometric characteristics: single and dual lanes. The video recorded data were used to simulate and calibrate model evaluation for a no ramp-metering scenario and several different fixed-time scenarios. By using VISSIM simulation software, a set of simulation experiments was designed to determine the optimal cycle time and green time and examine its effects on network performance. Headway and driver reaction times were altered as the model parameters for calibration until a qualitative and quantitative balance between the simulation and the observation was reached. Table 4 shows the efficiency performance of the fixed-time ramp metering.

The results show that fixed-time ramp metering (using 15 sec as the optimal cycle time) decreased the total travel time, the total delay and the number of stops by 32, 60 and 80 percent, respectively. It increased the average speed from 29.2 km/hour to 44.7 km/hr. They also analyzed the equity performance of the fixed-time ramp metering. They concluded that ramp control brought equity concerns for ramp users when the spot speeds were taken into account.

Table 4: Efficiency performance of fixed-time ramp metering (Kesten et al. 2013)

Measures of efficiency	No control	Fixed-time ramp metering
Total travel time [h]	4942	3368
Total delay time [h]	2910	1190
Number of stops	411,772	81,634
Average speed [km/h]	29.2	44.7
Total distance traveled [km]	144,406	150,460
Number of vehicles in the network	2189	1065
Number of vehicles that have left the network	26,696	27,718
Total stopped delay [h]	374	52
Average delay time per vehicle [s]	363	149
Average stopped delay per vehicle [s]	47	7
Average number of stops per vehicles	14	3

2.6.2 Evaluation of Ramp Metering Algorithm Systems Using Traffic Simulation

In 2002, Chu et al. evaluated three types of ramp-metering algorithms, including one local traffic-responsive algorithm ALINEA, and two coordinated algorithms, the Bottleneck and Zone algorithms. The PARAMICS microscopic traffic simulation program, which was enhanced by integrating complementary modules including a loop data aggregator, an actuated signal controller, and a time-based ramp controller, was used. A six-mile stretch of the northbound I-405 freeway in Orange County, California was studied, which included seven on-ramps, four off-ramps and one unmetered freeway-to-freeway ramp connecting I-405 with SR-133. The models were calibrated using travel demand data considering several parameters for the calibration of all algorithms such as geometry, vehicle type proportions, lane-usage, driver behavior, and vehicle characteristics. Four measures of effectiveness were used to evaluate the three ramp metering algorithms, which were: generalized total vehicle travel time, average mainline travel time, average on-ramp waiting time and average origin-destination travel time. They concluded that the two coordinated ramp metering algorithms (Bottleneck and Zone) were more efficient than both fixed-time control and the ALINEA algorithm. The Zone algorithm showed the best performance among the three ramp metering algorithms.

In 2004, Chu et al. used microsimulation to evaluate the performance of three adaptive ramp-metering algorithms, ALINEA, Bottleneck, and Zone, and two revised algorithms, Bottleneck-ALINEA and Zone- ALINEA. The PARAMICS simulation program was used to evaluate three measures of effectiveness: vehicle-hours traveled, average mainline travel time, and total on-ramp delay. They evaluated the ramp-metering algorithms for a six-mile stretch of northbound freeway I-405 in California under four scenarios: heavily congested morning peak-hour scenario (scenario 1), less-congested morning peak-hour scenario (scenario 2), severe incident scenario (scenario 3), and less-severe incident scenario (scenario 4). They calibrated the simulation models using the collected data from the field loop detectors. The results of the study showed that the adaptive ramp-metering algorithms reduced congestion on the freeway compared to fixed-time control; however, ramp-metering did not have a significant effect during severe congestion under incident scenarios. They also indicated that the ALINEA algorithm reduced freeway travel times under both recurrent and non-recurrent congestion scenarios while maintaining modest delays for on-ramp vehicles. The simulation results showed that the revised algorithms gave better performances than the original algorithms or ALINEA alone. Consequently, the revised Bottleneck algorithm showed the most robust performance under all scenarios.

Lee et al. (2005) supported the finding of the effects of ramp metering on safety. They observed the traffic flow changes using a microscopic traffic simulation model and they estimated crash potential for two types of freeway networks: the real freeway sections (9.2 mile section of I-880 in Hayward, California), and a hypothetical freeway sections. To examine the effects of isolated ramp metering without downstream bottleneck effects, they modeled a hypothetical freeway network. They used a local traffic-responsive ramp metering strategy, known as ALINEA ramp metering. PARAMICS microscopic traffic simulation was used to estimate the impacts of ramp metering on crash potential and traffic flow change. A real-time crash prediction model was used as a quantitative measure of freeway safety, based on short-term variations in traffic flow. They compared total crash potential between the no-control case and the ALINEA ramp-metering case to investigate the effectiveness of the ALINEA ramp-metering strategy. The results of the study demonstrated that the ALINEA ramp metering strategy improved safety by reducing total crash potential from 5 to 37 percent compared to the no-control case under the traffic condition of high ramp traffic volume. Despite its benefits, the study showed that its safety benefits are severely limited if a queue already existed downstream of the ramp.

Taylor et al. (1998) conducted a study about fuzzy ramp-metering algorithms and incorporating the fuzzy logic control into the microscopic freeway simulation model, FRESIM. A northbound section of I-5 in Seattle between NE Northgate Way and NE 175th street was chosen, which contains multiple ramps with recurrent and non-recurrent congestion. The freeway model was calibrated based on desired driver speeds and driver aggression. They took traffic data from loop detectors for every five minutes during unmetered peak conditions. Six different scenarios were tested using different traffic volumes, different freeway capacity, functioning ramp meters at different locations, and incidents. The fuzzy logic control was compared to three common controllers available within FRESIM, which were: clock, demand/capacity, and speed ramp-metering. Three performance criteria were used for the evaluation, which were: total kilometers traveled by all vehicles in the system, average system speed, and delay per vehicle-kilometer (including time waiting in ramp queues). The results of five scenarios out of six showed that the fuzzy logic control outperformed the other three metering and for no ramp controls. They mentioned that the more demand exceeded capacity, the more evident was the fuzzy logic control's advantage in balancing between mainline efficiency and ramp queues. They also recommended utilizing fuzzy logic control in locations that have ramp queue constraints due to limited alternative routes or political considerations.

Zhang et al. (2001) categorized and assessed 17 ramp-metering algorithms that ranged from simple local algorithms to complex integrated algorithms. The ALINEA, Bottleneck, SWARM, and Zone algorithms were further evaluated based on the qualitative assessment by using PARAMICS microscopic traffic simulations program. They made numerous simulation runs under different traffic demand patterns and coded the four selected ramp control strategies for a stretch of southbound Interstate 405 located in Orange County, California. They also considered a no ramp metering control case for the purpose of comparison. To compare the performance of control algorithms, t-test statistical analyses were performed on total vehicle travel times as measures of effectiveness. They concluded that ramp metering reduces the total travel time up to 7 percent compared with no ramp metering regardless of the ramp-metering algorithm type and travel demand load and pattern. The results showed that there were no significant performance differences among ALINEA, modified Bottleneck, modified SWARM with 1 time-step-ahead prediction, and Zone algorithms under the tested scenarios. The poorest performance among all tested algorithm was modified SWARM with five-step-ahead prediction, while SWARM with one-step prediction performed equally well as other tested algorithms. They also mentioned that the coordinated ramp metering algorithms did not necessarily perform better than local control algorithms if some of their key parameters are not well calibrated.

A study by Al-Obaedi and Yousif was done in 2012 about developing a microsimulation model for freeway merges with ramp-metering controls. The model was governed by the application of driver behavior such as car-following, lane-changing, and gap acceptance rules to deal with cooperative driver behavior type. They tested each part of the model against real traffic data. They also assessed three types of ramp metering algorithms (D-C, ALINEA, and ANCONA) after integrating them into the model. S-PARAMICS software was used to build and calibrate the model for different ramp and freeway traffic volumes. Three parameters were used to evaluate the effectiveness of the different ramp metering algorithms: saving total time spent on the main motorway (TTSM), saving total time spent of the ramp (TTSR), and saving total time spent (TTS = TTSM + TTSR). Figure 15 shows the result of their study for fixed freeway flow rate of 5250 vehicles per hour. They mentioned that significant reduction in (TTSM) have been obtained for all types of ramp metering algorithms; however, ANCONA algorithm gives better results than ALINEA, and D-C algorithms in terms of time saving. They also mentioned that ramp metering does not have any benefits for flow rates lower than the freeway capacity.

In 2012, Abdelfatah et al. utilized VISSIM microsimulation software to evaluate the effectiveness of ramp metering on The Emirates Road in Dubai R1000. They used six lanes as the predicted number of lanes in 2020. Five different volume/capacity (v/c) ratios on the freeway and on-ramp were used: 65, 80, 95, 110, and 120 percent. They assumed 2300 vehicles per hour lane, and 900 vehicles per hour per lane as the capacity of freeway and on-ramp, respectively. Two conditions of the freeway downstream were taken: with bottleneck and without bottleneck, while the queue in the ramp was not taken into account. They utilized the ALINEA ramp metering strategy by locating six detectors in the freeway and one detector in the ramp.

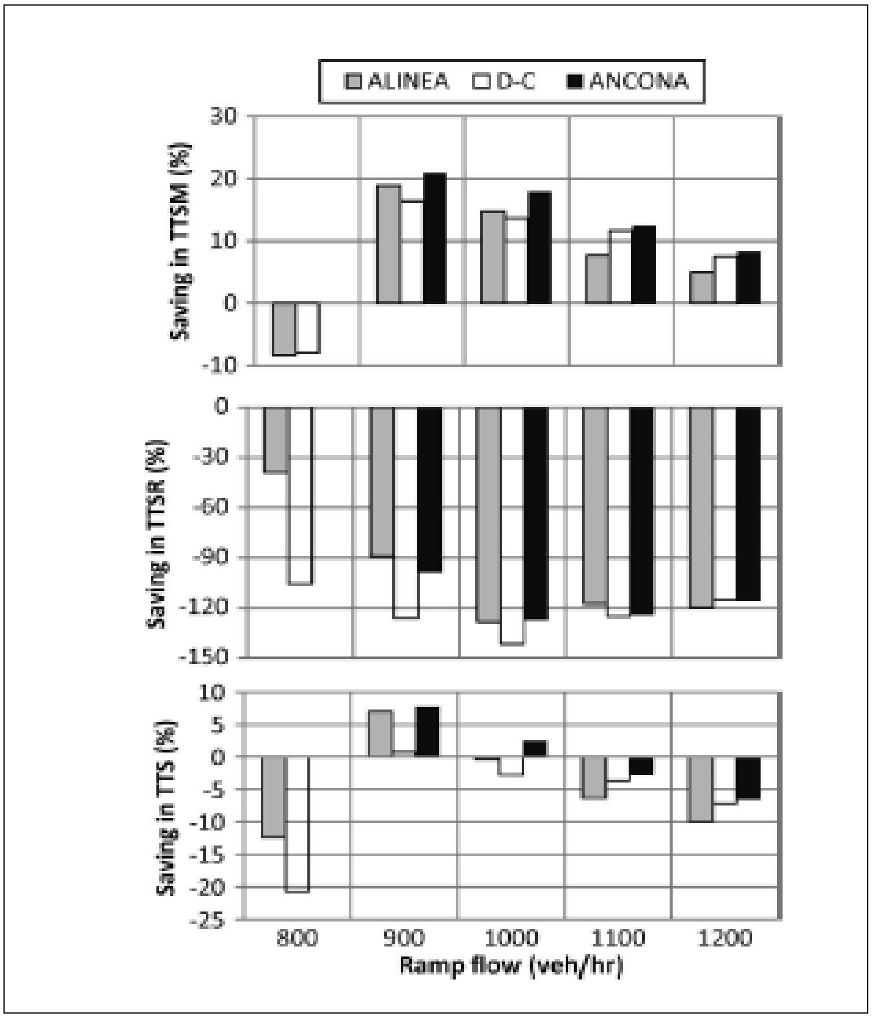


Figure 15: Effective of ramp metering (Al-Obaedi and Yousif, 2012)

VISSIM VAP (Vehicle Actuated Program) was used to interpret the control logic commands and create the signal control commands for VISSIM network based on the data from detectors. Three measures of effectiveness were used in the evaluation, which were travel time, speed, and density of the freeway. They concluded that in case of no bottleneck condition in the downstream of the freeway, ramp metering was not sensitive to low demand (65, 80, and 95 percent of the freeway capacity), while it had noticeable improvements for high levels of demand (110, and 120 percent of freeway capacity). In the case of a bottleneck condition in the downstream of the freeway, ramp metering showed significant improvements, especially for freeway v/c ratios of 80 percent.

In 2013, Greguric et al. conducted a study to improve the highway level of service of the Zagreb bypass freeway in Croatia. Thirty segments from the freeway were taken that contained several on- and off-ramps. An interactive freeway traffic macro simulator (CTMSIM), which was developed and run under the MATLAB program package, was used to simulate traffic flows in the study. An Adaptive Neural-Fuzzy inference system (ANFIS) algorithm was proposed for ramp metering control and compared to ALINEA, SWARM and no ramp metering scenarios. Productivity loss (PL), which was the number of lane-kilometers-hours on the highway lost due to reduced traffic flow, was used to assess level of service. Figure 16 shows the result of their study, which indicates that the ANFIS algorithm improved the level of service of the freeway; however, it did not show better results than the ALINEA algorithm. They mentioned that the ALINEA algorithm achieved the highest road lane usability compared to three other types.

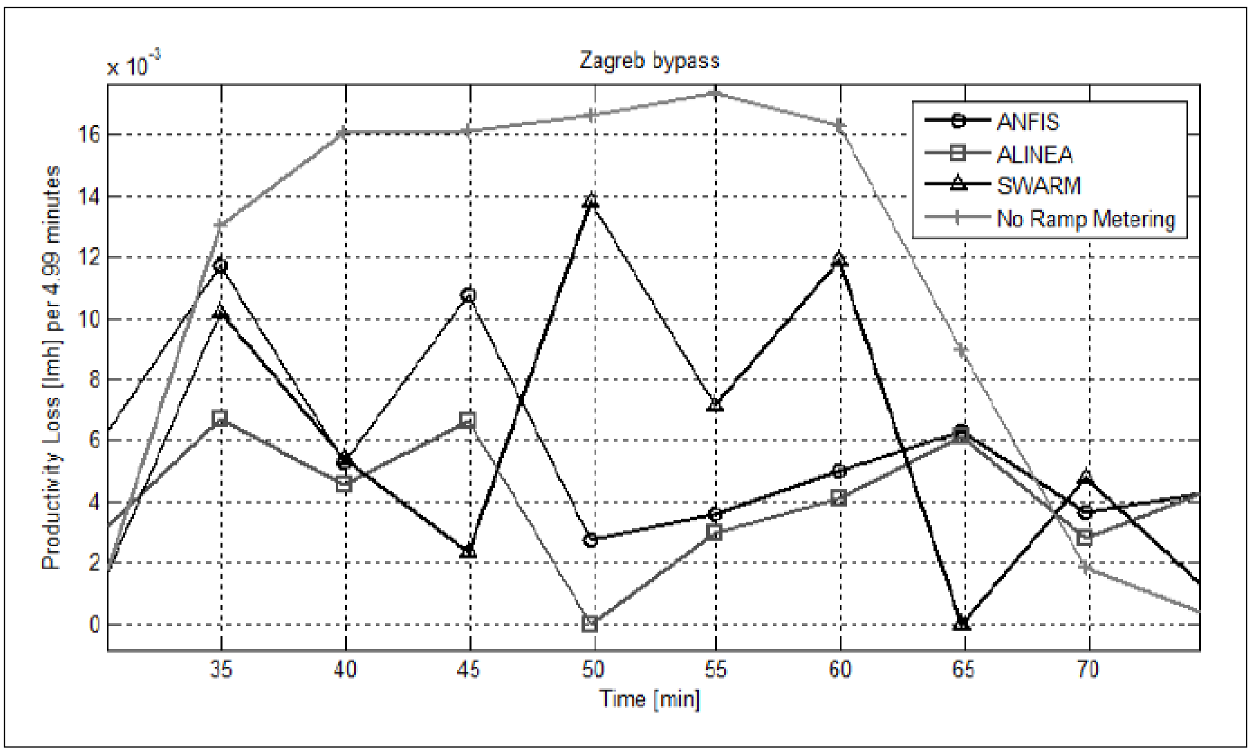


Figure 16: Productivity loss for evaluated ramp metering algorithms (Greguric et al. 2013)

2.7 Evaluation of Ramp Metering and Variable Speed Limit Using Traffic Simulation

Ramp metering systems and variable speed limits (VSL) can be used together as an intelligent transportation tool to improve the safety and efficiency of freeways. VSL is an emerging technology that is deployed immediately upstream of freeways to create some merging space for entering traffic from on-ramps. VSL strategy is used for two purposes: first, to improve safety by homogenizing speeds of the traffic flow, and second, to prevent traffic flow breakdown in freeways (Li and Ranjitkar, 2013). This section contains a review of studies of the effectiveness of using ramp metering system strategies with VSL on the safety and operation of freeways.

Dhindsa in 2005 conducted a study in Orlando, Florida about evaluating ramp meters and VSL to reduce the crash potential on congested freeways by using microsimulation. The researcher evaluated the effects of two strategies of ramp metering and VSL separately and collectively on improving safety conditions for different configurations of congested freeways. A nine-mile section of Interstate 4 in the Orlando metropolitan area was studied, which consisted of 17 loop detector stations, 11 on-ramps and 10 off-ramps. The researcher used the PARAMICS microsimulation software as a tool for modeling the freeway section and the ALINEA ramp metering algorithm to control ramp access. It was concluded that ramp metering could decrease crash risks and improve safety by decreasing the variance in speeds and decreasing average occupancy. It was also observed that safety was improved by increasing the number of ramp meters and using shorter signal cycle times. VSLs-when implemented along with ramp metering-provided safety benefits of up to 56 percent for the study section according to the safety benefit index, and it improved the network average speed besides by decreasing the overall network travel time by as much as 21 percent. However, VSLs were individually not as effective as ramp metering.

Abdel-Aty et al. conducted a study on Interstate 4 (I-4) freeway in Orlando, Florida. They evaluated safety, the travel time effects of ramp metering, and VSL for rear-end and lane change crash reductions along the freeway in real time based on new statistical and neural network models. The rear-end crash risk was based on the occurrence of crashes within one of two distinct traffic conditions: congested and uncongested, while the lane-change crash risk was based on a single neural network model. The microsimulation package PARAMICS was used for 36.25 miles of I-4 at three different loading scenarios, which were 60, 80, and 90 percent loadings. They used two types of ramp metering strategies for network simulation: the uncoordinated ALINEA algorithm, and the coordinated Zone algorithm. They examined two implementation methods of these algorithms: the traffic-cycle (TC) realization, and one-car-per-cycle (OCPC) realization. It was found that VSLs can be used to reduce crash risks and travel time at low traffic volume conditions, but it was not effective at reducing crash risks at congested traffic situations. They also indicated that both the Zone and ALINEA ramp metering algorithms could be applied to a congested freeway for successfully reducing real-time crash risks. Moreover, the study results showed that the traffic-cycle realization method provided better safety and operational benefits when applied with the ALINEA algorithm, especially with shorter cycle lengths. They found that the ALINEA algorithm was superior to the Zone algorithm in relation to reducing the crash risk; however, the Zone algorithm with longer cycle lengths was much better than the ALINEA algorithm in terms of the overall network travel time.

In 2013, Li and Ranjitkar used the AIMSUN microsimulation program to assess two ramp metering algorithms (ALINEA and HERO) individually and in combination with a VSL strategy. They used total travel time as a performance indicator for efficiency at the critical bottleneck section on State Highway One of the Auckland freeway in New Zealand. Network traffic data were obtained by the New Zealand Transport Agency's loop detectors that were installed at the on-ramps, off-ramps, and on the freeway mainline. The traffic data, which were accumulated over 30-second time intervals, were used for both calibration and validation of the simulation models. Six different control scenarios were tested systematically including a no control scenario, only VSL, ALINEA, ALINEA plus VSL, HERO, and HERO plus VSL. They used a no control scenario as a reference to measure improvements of the other five scenarios. Table 5 shows the results of the study, which shows that no significant improvement was obtained by using VSL only, while the greatest improvement was obtained by using HERO plus VSL. According to the table, the ALINEA strategy individually recorded a 12.5 percent improvement; however, the percentage slightly increased to 12.6 by using VSL with ALINEA. Similarly, the percentage of improvement was slightly increased from 17.9 to 18.3 when VSL was added to the HERO scenario. The authors tested the results statistically and they showed the significant results in the Table 5; however, if the benefit cost ratio is taken into account, VSL technology is not useful for these small differences.

Table 5: Total travel time for different scenarios (Li and Ranjitkar, 2013)

	No. Control	VSL		ALINEA		ALINEA+VSL		HERO		HERO+VSL	
		Value	% impr.	Value	% impr.	Value	% impr.	Value	% impr.	Value	% impr.
TTT (veh*h)	1669	1658	0.66	1461	12.46	1458	12.64	1370	17.91	1363	18.33
% impr. = percent of improvement compared to No-control option											

2.8 Ramp-Metering Benefit-Cost Ratio Assessment

Despite using ramp metering as an intelligent transportation system technique for improving freeway throughput, a benefit-cost ratio analysis should be conducted before ramp metering implementation to determine its cost effectiveness.

In 1999, Kang and Gillen studied the benefit-cost analysis of ramp metering by examining three different cost cases of ramp meters in the analysis. The costs of ramp meters, which include construction, signal, operation, and maintenance of the ramp meters were estimated from conversations with traffic engineers in the Caltrans Districts (case 1) and from the literature (case 2 and case 3). A cell-transmission traffic simulation model was used to simulate the impact of ramp metering on traffic behavior. They determined the travel demand that was saved by local, single entry traffic responsive ramp metering, and then they identified and quantified the benefits and costs of the ramp metering project. Traffic data were utilized on freeway and on-ramps during peak hours from the I-880 freeway, Alameda, California in 1993 as a typical travel pattern. They derived the benefits of the ramp metering based on travel time value and fuel consumption by saving in travel delay. According to the results of their study, the total net benefit value in the lifetime of ramp metering would be about $9.1 million, $9.9 million and $10.3 million for cases 1, 2, and 3, respectively. The benefit-cost ratio of each case was 7.85, 19.62, and 80.25, respectively. They conducted a sensitivity analysis by changing fuel economy, time value, ramp demand, and freeway demand. They also conducted a sensitivity analysis for different values of capacity reduction. Despite the fact that the ramp metering provided limited benefits in the sensitivity analyses, they concluded that ramp metering was still worthwhile implementing.

In 2000, Minnesota Department of Transportation (MnDOT) spent $651,600 for a study to evaluate the performance measures and safety impacts of the freeways associated with deactivated ramp metering system for a specific time of period. They selected four corridors including I-494, I-94, I-35W, and I-35E in the Twin Cities as representatives of all corridors throughout the metropolitan region. They also selected several parallel arterials to provide data on surface street conditions. They collected the data related to the effectiveness measures of two scenarios: ramp meters on and ramp meters set to "flashing yellow." After analyzing the data, they summarized benefits of ramp metering into: (a) after turning the ramp meters off, average traffic volume was reduced by nine percent on freeways; however, ramp meters did not change traffic volumes of the parallel arterials; (b) When ramp meters were turned off, average travel speed was decreased by 14 percent, and it resulted in increasing freeway travel time by 22 percent; (c) With no ramp metering system, peak period crashes were increased by 26 percent, which corresponds to four crashes per day in the entire freeway system; (d) Ramp meters resulted in an annual system-wide decrease of 1,160 tons of emissions, but ramp meters increased 5.5 million gallons of fuel consumption annually in the entire system; and (e) The benefit/cost ratio evaluation indicated that "Ramp metering benefits are five times greater than the cost of the entire congestion management system and over 15 times greater than the cost of the ramp metering system alone."

In 2011, Lu and Hadi used Intelligent Development Analysis System (IDAS) to propose a method to evaluate the impacts of ramp metering for different traffic conditions. IDAS is able to predict the ramp metering impact and convert its benefits to dollar values. The study procedures were based on modeling the probability of freeway traffic-breakdown elimination due to ramp metering. The I-95 corridor in Miami, Florida was evaluated assuming that the ramp metering would be deployed on three on-ramps along the freeway segments. They reproduced the traffic demand in the regional network based on field data of three hours of peak period. They assumed a freeway mainline capacity of 2,300 vphpl, a mean queue discharge flow-rate during breakdown conditions of 1,900 vphpl, and an on-ramp capacity of 1,500 vphpl. The study results showed that ramp metering increased capacity from zero to 15.1 percent using their proposed method. The benefit-cost ratio of the proposed method was 5.1.

2.9 Effects of Ramp Metering on Driver Behavior

Merging maneuver operations have been extensively investigated at the entrance to freeway junctions without using ramp metering; however, few researchers have investigated the effects of ramp metering system on merging maneuver operations. In 2007, Wu et al. conducted a study to evaluate the potential impacts of ramp metering on the driver behavior. They focused on whether ramp metering can reduce the stress of drivers at the on-ramp and can smooth the traffic flow in the downstream of the freeway. Seven merging maneuver behavior parameters, which were acceleration/deceleration, speed, headway, lane changing rates, gap acceptance, merge distance, and speed at merge, were used for the evaluation. Because of the difficulty to get the behavioral parameters, they equipped an instrumented vehicle with various devices to measure vehicle acceleration, speed, headway, time, coordinates, performance and reaction, and driver maneuvers. Sixteen drivers with different genders, ages, and driving experiences drove the instrumented vehicle on both the freeway and the on-ramp. They also employed 11 roadside video cameras to measure the interaction of the merging vehicles and freeway vehicles. Other devices were used such as an over-bridge camera (to measure lane change rate), and two loop detectors. They selected Junction 11 on the M27 freeway (3-lanes) in Southampton, England that had an average upstream freeway traffic flow of 3,800-4,000 vehicles per hour and 1,800-1,900 vehicles per hour from the on-ramp. The ALINEA algorithm with cycle times of 10, 12, 15, 20, 24, and 30 seconds was the used strategy in the M27-J11 freeway junction. Their investigation included a four week survey with ramp metering and four weeks without ramp metering. Driver behaviors were analyzed for three types of traffic: the upstream traffic, on-ramp merging traffic, and influence area freeway traffic. After using statistical analyses for their survey data, they concluded that there was no significant difference in driver behavior parameters before and after using ramp metering.

In 2007, Zheng and McDonald conducted a study to investigate the effects of ramp metering on the behavior characteristics of drivers during merging maneuvers on freeways. They compared dynamic merging process attributors such as eye movements and speed control of merging drivers, merging position, gap acceptance, and lane changing of passing traffic under both ramp metering on and ramp metering off conditions. A comprehensive observation of merging operation was carried out using a combination of an instrumented vehicle and camera technology recording at the roadside. The instrumented vehicle was equipped with two radars, a laser speedometer, a Global Positioning System (GPS) receiver, and three in-car cameras. Peak hour-time series states, such as, the position and speed of each vehicle, were recorded at a junction on the M27 with a normal-tapered merge. Merging operations were carried out under similar traffic flow conditions for both ramp metering off and on cases. A local traffic responsive ramp metering algorithm with two to three cars per green was implemented as a ramp metering signal timing. When queue length reached local streets, metering signal cycle length was changed to 34 seconds (20 seconds green, two seconds amber and ten seconds red). The results showed that the averages of driver eye movements were 3.8 and 4.2 times for ramp metering-off and ramp metering-on, respectively. The locations of eye movements were also changed for both cases of ramp metering off and on. It was noticed that the average speed of merging vehicles under ramp metering-off was much higher than that under ramp metering-on. There were no statistically significant differences for merging positions between ramps metering off and on. When ramp metering was in operation, 64 percent of drivers were able to merge into the original gaps; the remaining 36 percent were overtaken and merged into lag gaps. When ramp metering was not in operation, 87 percent of drivers were able to merge into the original gaps. These results indicate that gap acceptance becomes more difficult under ramp metering-on than ramp metering-off. In each five minutes, 9.2 vehicles changed to the outside lane when ramp metering was switched off, while 10.3 vehicles changed to the outside lane when ramp metering was on. These results indicate that merging operation under ramp metering-on causes a higher perceived deterrence to passing than under ramp metering-off. As a result, they concluded that merging maneuvers were more difficult under ramp metering control than with no control.

2.10 The Evaluation of the Effectiveness of Ramp Metering System on Air Pollution

Surface transportation is one of the major sources of air pollution, which affects global climate change. Intelligent Transportation Systems (ITS) have been playing an important role in reducing gas emissions and fuel consumptions. Studies have been done about the effectiveness of ramp metering system as one of the ITS. In 1999, Thornton et al. conducted a study to find the emission impacts of ramp metering strategies on the Atlanta freeway system. They collected traffic data in peak hours for both cases of turning ramp meters on and off by using video cameras and Nu-Metric devices. Howell Mill Road on-ramp, which has a 7 percent downgrade was chosen as the study site. The vehicle license plates were recorded to find the types and models of the cars, and then using them as inputs to find emission rates. Oxides of Nitrogen (NOx) and Carbon monoxide (CO) emissions were estimated by focusing on the changes in modal activity such as speeds and accelerations of vehicles. The results of the study showed that under metered conditions, average speeds on ramps decreased, maximum ramp acceleration rates increased and, both maximum and average speeds at mainline increased. These changes in accelerations and speeds of vehicles under metered conditions resulted in decreased NOx emissions on both the ramp and the mainline. CO emissions on the mainline decreased, but it increased on the ramp. They also mentioned that ramp metering affects driver behavior and emissions even when they are not in operation.

In 2012 Bae et al. performed a study to determine the effectiveness of ramp metering as one of the ITS technologies on reducing carbon dioxide CO_2 emissions. They took three traffic flow scenarios, with no ramp metering during peak hours, with ramp metering implementation, and ramp metering with the existence of a detour route. The adapted local ramp metering control algorithm was defined by the passing of four vehicles every 30 seconds. To measure the amount of CO_2 emissions, the Traffic Software Integrated System (TSIS) simulation program was used. The simulated results for both with and without ramp metering cases were compared with real traffic data to determine the accuracy of the simulation data. The CO_2 emissions were calculated from traffic volume and speed on the freeway links, off-ramps, and on-ramps based on traffic composition, fuel type, and the year of the vehicle models. The study results showed that the stop and go of the on-ramp vehicles in front of the meters caused more CO_2 emissions than free-flow traffic; however, ramp metering resulted in reducing CO_2 emissions in the on-ramp project area and the detour section as a whole system. They indicated that ramp metering reduced 818.4 kilograms of per hour of CO_2 emissions, which corresponds to 7.3 percent. They also estimated that 3,273.6 kilogram emissions per day or 1,1949.9 tons of emissions per year can be reduced by using ramp metering in the peak hour period.

2.11 Evaluation of Temporary Ramp Metering in Work Zones

Several studies have been done to evaluate efficiency and safety benefits of ramp metering systems, but few studies have been done about the effectiveness of temporary ramp metering on mainline freeways and on-ramp entrance in work zones.

In 2006, Pavithran used VISSIM microscopic traffic simulation to compare two types of merge metering strategies in the work zone: fixed-time and continuous merge metering, with the late merge strategy (i.e.: use either the open or closed lane until they reach the merge point at the lane closure taper rather than merging as soon as possible into the open lane). In the fixed-time strategy, three different cycle lengths were used: 30, 60, and 120 seconds, with the green times of 13, 28, and 58 seconds, respectively. They selected the best performance among the cycles of the fixed-time strategies for comparison purpose. In the continuous merge metering strategy, the vehicles in each lane had alternating green and red signals for one second each. The simulations were modeled for different traffic volumes and heavy vehicles scenarios. A section of 5.18 miles of I-75 in Cincinnati was used as the basis for the simulation study. The researcher modeled a two-lane freeway and a one-mile lane closure incorporating in the network at a distance of 3.21 miles from the start of the network. Delays and travel times were used as criteria for the comparisons. The results showed that both the fixed-time and continuous merge metering strategies produce less delay than the late merge strategy for all traffic volumes above capacity. For all traffic volumes that exceeded the capacity of a standard two-to-one lane closure, fixed-time and continuous merge metering strategies resulted in reducing travel time by 11.5 percent, and approximately 8 percent, respectively.

In 2009, Oner conducted a study to evaluate temporary entrance ramp metering control strategies in freeway work zones using digital simulation. The researcher published a set of guidelines based on two factors: first, the importance levels of freeway mainline throughput and local traffic access to the freeway, and second the hourly traffic volume levels for the freeway mainline and entrance ramps. The ramp metering effects were investigated for various hourly traffic volumes and truck percentages for freeway and entrance ramp. The researcher considered single lane (grade less than 3 percent) on-ramp and both signalized and non-signalized freeway entrance ramp designs. Microwave radar trailers were used to collect data at different freeway work zones in Ohio to generate the cumulative inter-arrival time. Two separate Arena simulation models were used to investigate the temporary freeway entrance ramp metering control strategy. The first one was developed to determine the spillback queue from the ramp metering signal back to the local street, and the second one was developed to determine the queue from the freeway mainline back to the ramp metering signal. Two situations of freeway work zones were taken: first, severe congestion in the work area and in the lane reduction area before the work area in freeway work zones; and second, severe congestion in the work area in freeway work zones. The results showed that the ramp metering signal intervals resulted in much shorter spillback queues from the ramp metering signal back to the local streets. The results of both signalized and non-signalized freeway entrance ramps indicated that ramp metering signal intervals did not increase the queue lengths from the freeway mainline back to the ramp metering signal even when the percentage of the trucks on both the mainline and the on-ramp was 10 percent.

In 2013, Sun et al. conducted a study to evaluate the effectiveness of temporary ramp metering deployment in work zones. They deployed seven temporary ramp meters at work zones, which were near ramps in Colombia, Missouri. The work zones had different characteristics such as configuration, location with respect to ramp, ramp traffic volume, grade and length of the entrance's ramp, and truck percentages. They were located on access-controlled high-speed facilities on Interstate 70 or U.S. Highway 63. Four cameras and two speed radars were used at each work zone to extract the safety and the mobility measures. Because of the lack of crash data during the time of the study, surrogate safety measures were used to assess the safety evaluation, such as driver compliance rates, speed statistics of the mainline and ramp traffic, speed differences between merging vehicles and mainline vehicles, ramp platooning, merging headways, lanes changes, and braking events. The traffic microscopic simulation software VISSIM was used and calibrated to obtain the total delay experienced by all vehicles to investigate the mobility effects of ramp metering on work zones. Adequate calibration for driver behavior and vehicle characteristics was done by using the collected field data. The results show that temporary ramp meters could save delay only at congested work zone locations, while ramp metering implementation was not beneficial for non-congested conditions. The major issue from the safety view in the deployment of temporary ramp metering was the lack of compliance by the drivers; however higher compliance rates can be achieved using three-section signal head instead of two-section signal heads.

2.12 Study of Ramp Metering Components Design

Many states currently have standard guidelines for designing, installing, and operating ramp metering systems. In order to develop design guidelines of ramp metering, studies have been conducted on the design elements of ramp metering systems.

In 1970, Cook et al. evaluated the effectiveness of ramp metering on traffic operations, safety, and violations after installing or modifying traffic control signs. They changed traffic controls on eight metered freeway ramps in Detroit, Michigan to try to reduce the violation rate. The violation rate was 40 percent before additional control devices were installed or modified. After installing “on green one car only” sign, the rate of violations was reduced to 10 percent. They recommended to put a sign of "ramp metering when flashing" to improve safety. They obtained that the number of crashes were not changed after installing ramp metering. However, the authors pointed out that their new strategy of ramp metering reduced travel time by 30 vehicle hours per day compared to the previous operation of the metering system, and it resulted a smaller proportion of ramp congestion during the peak hour.

In 2002, Chaudhary and Messer conducted a study to develop a design criterion for metered ramps with excessive queue detectors. They used excessive queue detectors to monitor ramp queues from spilling back into the upstream traffic signal. Three distance requirements for freeway on-ramps were considered in the study. First was the safe stopping distance provided for vehicles to discharge from the upstream signal to stop safely behind the maximum queue of the vehicles being metered. Second was the storage distance provided to store the resulting cyclic queue of vehicles without blocking an upstream signalized intersection. Third was the acceleration distance that was the distance provided for the stopped vehicles at the meter to accelerate and attain safe merge speeds. As the result of the study, they recommended different distances from ramp meter to freeway merge point for various merge speeds and ramp grades as shown in Table 6. They also recommended the distances from the cross street to the ramp meter for the metering strategies as shown in the Table 7. The recommended total ramp distance can be obtained by adding appropriate values from Tables 6 and 7.

Table 6: The travel distance from ramp meter to freeway merge point for various freeway entry speeds (meter) (Chaudhary and Messer, 2002)

Merge speed (km/h)	**Travel distance (meter) by ramp grade**		
	-3 %	**0 %**	**+3 %**
60	90	112	150
70	127	158	208
80	180	228	313
90	248	323	466
100	331	442	665

Table 7: Recommended distance (m) from cross street to ramp meter by metering strategy (Chaudhary and Messer, 2002)

Ramp volume (vph)	**Single lane**	**Bulk metering**	**Dual lane**	**General model**
0	75	75	75	75
300	153	153	153	145
600	196	194	194	200
900	244	224	221	241
1200	---	265	235	269
1500	---	---	257	284

2.13 Summary of Literature Review

Several studies were reviewed in the literature from which, many important considerations were obtained and will benefit this dissertation. The researchers used traffic field data and/or traffic simulators to evaluate the traffic parameters at the ramp-freeway junctions with and/or without using ramp metering systems.

- Four studies showed that geometric design of ramps and freeways affects the safety in merging areas. Bared et al. (1999) and Liue et al. (2010) indicated that the rate of change of crash frequency on the freeway ramps was inversely proportional with the acceleration and deceleration lane lengths. Le and Porter (2012) concluded that expected crash frequency increased as space between ramps decreased, while the proportion of the expected fatal and injury crash types decreased as ramp spacing decreased. However, Calvi and De Blasiis (2011) indicated that driving performance during merging maneuvers was not affected by the acceleration lane length.

- Several studies showed that ramp metering provided better efficiency and safety for the freeways, but they obtained different percentages of the improvement change. Piotrowicz and Robinson (1995) showed that ramp metering increased mainline speed by 16 to 62 percent, and reduced crash rate by 24 to 50 percent. Meyer (1997) showed that average freeway speeds increased by 29 percent, and crashes were reduced by 28.5 percent after using ramp metering. In Liu and Wang's study (2013), ramp metering affected safety positively by reducing around 36 percent of freeway collisions near on-ramp entrances. KDOT and MoDoT (2011) obtained a greater percentage of crash reduction (64 percent) due to using ramp metering. Significant improvements for the freeway efficiency and safety were obtained by using a fixed-time ramp metering system. Gaynor et al. (1997) indicated that a fixed-time ramp metering system increased the average speed by 9.4 percent, while according to the study of Kesten et al. (2013), it increased the average speed by 53 percent. On the other hand, Poorjafari and Yue (2013) found that the fixed-time ramp metering systems could improve the freeway performance especially in the peak-hours; however, it could not benefit the whole system. Therefore, they recommended a thorough site investigation before implementing ramp metering.
- Two contrary results were obtained about the role of ramp metering in case of break down or bottleneck activation. The studies of Cassidy and Rudjanakanoknad (2002),

Zhang Levinson (2009), and Abdelfatah (2012) showed that ramp metering resulted in increasing freeway discharge flows or bottleneck capacity; on the other hand, Gaynor et al. (1997) concluded that ramp-metering systems did not change the capacity significantly, when bottlenecks controlled the freeway operations.

- Almost all of the studies reviewed in the literature agreed up on the beneficial effects of the ramp metering algorithms, which were designed for variable metering rates. Lipp et al. (1991) showed that after implementing Helper ramp metering algorithm, the freeway speed increased by 58 percent. In the study by Taylor et al. (1998), the Fuzzy ramp metering algorithm would provide significant balance between mainline efficiency and ramp queues, especially when the demand exceeded capacity. Lee et al. (2005) demonstrated that the ALINEA ramp metering algorithm would decrease total crash potential from 5 to 37 percent under high ramp traffic volume conditions.
- The limited benefit of ramp metering pushed some researchers to analyze benefit-cost ratios of the ramp metering systems. The analysis results showed that ramp metering benefits were five times greater than the cost of the ramp metering system (Kang and Gillen 1999), (MnDOT, 2000), and (Lu and Hadi, 2011).
- Driver behaviors during merging maneuver operations, and the effects of ramp metering on merging maneuvers have been investigated by many researchers. Kondyli (2009) classified drivers into three types: aggressive, cooperative, and conservative based on the actions during merging maneuvers (decelerate, change lanes, and do nothing). The studies about the effects of ramp metering on merging maneuvers provided contrary results; for example, Wu et al. 2007 concluded that there was no significant difference in driver behavior before and after using ramp metering.

However, Zheng and McDonald (2007) realized that there were statistically significant differences for merging maneuvers in terms of driver behaviors. They also concluded that ramp metering makes the merging maneuvers more difficult than with no control case.

The studies that were reviewed in the literature illustrated the results of efficiency and safety effects of ramp metering on freeways. The studies covered several aspects, such as using fixed-time ramp meters signal, ramp metering algorithms, ramp metering benefit-cost ratio, driver behaviors at merging areas, and using ramp meters at work zones. A subject that the researchers have not explored is the effectiveness of ramp metering on freeway efficiency and safety at specific situations combining geometric configuration of the ramp-freeway junctions, different traffic volumes of the freeway and the ramp, and ramp meters signal timings. This study has been done to fill this gap.

The information in the literature review was useful for designing the major components of the evaluation processes in this study regarding the performance of ramp metering, geometric design of ramp-freeway junctions, traffic volumes that cause breakdown and non-breakdown conditions on the freeway, and the parameters of the VISSIM traffic simulation program. The study procedures including this benefited information are presented in Chapters 3 and 4.

CHAPTER 3: RESEARCH METHODOLOGY-DEVELOPMENT OF SIMULATION BASED STUDY

This chapter illustrates types of data, traffic simulation programs, calibration process, and mechanisms that were required to explore the efficiency, Level of Service, and safety of an on-ramp connection to a freeway using a ramp metering system.

3.1 Parameters Affecting the Efficiency, Level of Service, and Safety of the Freeway

According to the previous studies mentioned in Chapter 2, many parameters affect the efficiency, capacity, and safety of an on-ramp connection to a freeway with and without using ramp meters such as:

- Design and 85th percentile speeds on the freeways downstream and upstream;
- Driver behavior on the ramp and the freeway vehicles;
- Geometric configuration of the ramp;
- Grade of the ramp;
- Length of the ramp;
- Platoon in the upstream of the ramp due to traffic control type of arterial or local road (traffic signal, yield controlled, stop controlled or uncontrolled);
- Queue length of the ramp and the freeway;
- Ramp vehicle speed;
- Signal timing design of ramp metering;
- Space between the ramp and the adjacent ramps;
- Traffic composition of the ramp and the freeway vehicles;
- Traffic flow on the ramp and the freeway;
- Type of acceleration lane such as parallel and taper; and

- Type of road (arterial or local) that are connected to the upstream ramp

The same data parameters were collected and used for both the calibration process, and building models.

3.2 Simulation Models for Evaluating Ramp Metering

Two methods can be used to evaluate performance and safety of ramp metering: field operational testing and computer simulation. Field operational evaluation can provide more realistic results than computer simulation, but it is more time consuming and less economical compared to simulation. Field operational testing is impractical for evaluating different alternative designs or scenarios. Traffic simulation models have become powerful tools to assess the benefits of intelligent transportation systems in the planning mode, generating different scenarios, optimizing control, and predicting network behavior at an operational level.

Traffic simulation models can be classified into three types: microscopic, macroscopic, and mesoscopic. Microscopic models predict the state of individual vehicle movements in discrete-time periods based on vehicle-to-vehicle interaction; for example, the speed of individual vehicles at its location. PARAMICS, CORSIM, VISSIM, AIMSUN2, TRANSIM, and MITSIM are examples of microscopic models. Macroscopic models measure traffic flow aggregately such as speed, density, and flow. FREFLO, AUTOS, METANET, and VISUM are examples of macroscopic models. Mesoscopic models are the mixture of both the microscopic and macroscopic models. DYNASMART, DYNAMIT, INTEGRATION and METROPOLIS are examples of mesoscopic models.

Moreover, traffic simulation models can be classified according to functionality such as signal, freeway, or integrated (Horowitz et al. 2004) (Chu et al. 2002). Each traffic simulation model is designed with special features and used for specific purposes; Table 8 shows nine traffic simulation models with different Intelligent Transportation System (ITS) features. The highlighted row in the table indicates those simulation programs that can be used for ramp metering evaluation.

Table 8: Summary of simulator models based on in-depth criteria (Horowitz et al, 2004)

ITS features Modeled	**AIMSUN 2**	**CONTRAM**	**CORFLO**	**CORSIM**	**FLEXYT II**	**HUTSIM**	**INTEGRATION**	**PARAMICS**	**VISSIM**
Adaptive traffic signals	X	X		X	X	X	X	X	X
Congestion pricing						X		X	
Coordinated traffic signals	X	X		X	X	X	X	X	X
Driver behavior	X			X	X		X	X	
Graphical network builder	X	X			X	X			X
Graphical presentation of results	X	X		X	X	X	X	X	X
Incidents	X		X	X	X	X	X	X	X
Integrated simulation	X	X		X		X	X	X	X
Interface w/other ITS algorithms	X								
Network conditions	X					X		X	
Network flow pattern predictions					X	X	X	X	X
Other properties									
Queue spillback	X			X	X	X	X	X	X
Ramp metering	X			X	X	X	X	X	X
Route guidance									
Runs on a PC	X	X		X	X	X	X	X	X
Traffic calming					X	X	X	X	X
Traffic devices	X						X	X	
Traffic device functions	X						X	X	
Vehicle interaction	X			X	X		X	X	
Well documented	X	X	X	X	X	X	X	X	X

3.3 Efficiency Evaluation

In this study, a microscopic traffic flow-simulation software, VISSIM was employed to evaluate the operational effectiveness of the ramp metering and to obtain trajectory files for using them in the SSAM program, which was used to evaluate the safety of the freeway using a ramp metering system.

3.3.1 VISSIM

According to the VISSIM User Manual 2011's definition, VISSIM is a microscopic, time step and behavior-based simulation model developed to model urban traffic and public transport operations and flows of pedestrians (VISSIM 5.30-05 User Manual, 2011). The model was developed at University of the Karlsruhe, Germany during the early 1970s. The program is a useful tool to evaluate the effectiveness of various alternatives because of its ability to analyze private and public transport operations under constraints such as lane configuration, vehicle composition, traffic and signals. Multiple field measurements at the University of Karlsruhe were taken to calibrate the model. VISSIM is a traffic flow simulator, which considers the car following and lane change logic. VISSIM allows importing aerial photographs or images to build the network system. In VISSIM, traffic flow is simulated by moving "driver-vehicle-units" through a network. The driver behavior and vehicle performance characteristics are accounted for in VISSIM, with specific driver behavior characteristics assigned to each vehicle. According to VISSIM user manual 2011, attributes characterizing each driver-vehicle unit can be discriminated into three categories:

> (1) Technical specification of the vehicle, for example, length, maximum speed, potential acceleration, actual position in the network, and actual speed and acceleration. (2) Behavior of driver-vehicle units for example, psycho-physical sensitivity thresholds of the driver, memory of driver, and acceleration based on current speed and driver's desired speed (ability to estimate, aggressiveness). (3) Interdependence of driver-vehicle units, for

example, reference to leading and following vehicles on own lane and adjacent travel lanes, reference to current link and next intersection, and reference to next traffic signal.

VISSIM was not designed to analyze highway safety but proper trajectory output files related to conflict analysis can be obtained from VISSIM software; therefore, an additional software tool SSAM was required to perform the safety analysis in the study. A description of the SSAM software program is described in Section 3.4.2.

3.3.2 Calibration and Validation Processes

"Calibration is defined as the adjustment of computer simulation model parameters to accurately reflect prevailing conditions of the roadway network" (Woody, 2006). Several parameters can be adjusted such as driver lane-change aggressiveness, car following behavior, lane-change gap acceptance, route choice, and speed and acceleration distributions. To identify validated parameters such as baseline settings that reflect the overall driving behavior and operational characteristics, a validation process is necessary. "Validation is defined as the process of comparing simulated model results with field measurements in order to determine the accuracy of the simulation model" (Woody, 2006). In the calibration and the validation processes, the vehicle and driver behavior parameters are altered until a quantitative and/or qualitative balance between the simulation and the observation parameters are reached by using statistical analysis tests.

In VISSIM, the key parameters for the freeway model to be calibrated are system and operational calibration parameters. System calibration parameters are high level parameters such as the size of the model study area, traffic demand, vehicle routing, and geometry and network inputs. Operational calibration parameters control the driver behavior characteristics of individual vehicles in the simulation model. In order to reflect realistic driver behavior, three main operational calibration parameters should be calibrated that are car following behavior, necessary lane change behavior, and lane change distance (Woody, 2006).

In this study, both the system and the operational parameters were calibrated for a model. In later steps, the calibrated parameters were used in the model scenarios that were built for the evaluation processes.

3.4 Safety Evaluation

Traditionally, crash data statistics are used to evaluate highway traffic safety. Frequency, type, and severity of traffic crashes, which can be obtained from traffic police reports, are direct indicators for measuring highway safety, (FHWA conflict manual, 1989). Crash data depend on the report forms of traffic crashes, which are filled by traffic police. The crash report forms contain much information about the crashes, such as:

- Collision type (rear end, cross, head on, sideswipe, angle, etc.);
- Crash class (overturned, railway train, pedestrian, fixed object, bicycle, etc.);
- Crash severity (property damage only, injury, fatal);
- Crash location (non-intersection, intersection, interchange, etc.) ;
- Time of crash (day or night);
- Weather condition (no adverse condition, rain, sleet, snow, fog);
- Surface condition (dry, wet, ice, mud) ;
- Driver condition (drinking alcohol, normal);
- Driver age; and
- Other information related to the road, vehicle, environment, and driver, of the accidents (Mulinazzi and Russell, 1994).

Crash data are associated with numerous problems, Laureshyn et al. (2010) summed up the problems into the following aspects:

- Compared to other events in traffic, crashes are exceptional in the sense that they are the results of a series of unhappy realizations of many small probabilities;
- Crashes are rare events, making it troublesome to base traffic safety analyses at individual sites on crashes only;
- Not all crashes are reported and the level of underreporting depends on the crash's severity and types of road users involved; and
- Information on the behavioral aspects preceding the crashes are seldom available.

Because of the reasons that are mentioned above, traffic conflict data can be used as appropriate surrogates for traffic crash data to evaluate highway safety. Before and after studies or new design alternatives need a long time to collect crash data after implementation. For example, if several new alternative designs are evaluated from the view of safety, three years of crash data are needed after implementing each new design in the field, which is not practical, as well as not economical. Moreover, it is not easy to assess safety in new and innovative traffic treatments.

In this dissertation, seven different signal timing scenarios for different traffic volume scenarios on the ramp and the freeway were proposed. Therefore, traffic conflicts were used to evaluate safety. Detailed descriptions of movements were obtained by using trajectory files, which were taken from VISSIM as input files, and were analyzed them in the SSAM software program.

3.4.1 Traffic Conflict

According to the FHWA Manual for Traffic Conflict Techniques, "A traffic conflict is an event involving two or more road users, in which the action of one user causes the other user to make an evasive maneuver to avoid a collision." The FHWA Manual for Traffic Conflict Techniques, classified conflicts into six main types and subdivided them into 15 secondary types as follows:

1. Same-direction conflicts
    a. Left-turn, same-direction conflicts;
    b. Right-turn, same-direction conflicts;
    c. Slow-vehicle, same-direction conflicts; and
    d. Lane-change conflicts
2. Opposing left-turn conflicts
3. Cross-traffic conflicts
    a. Right-turn, cross-traffic-from-right conflict;
    b. Left-turn, cross-traffic-from-right conflict;
    c. Through, cross-traffic-from-right conflict;
    d. Right-turn, cross-traffic-from-left conflict;
    e. Left-turn, cross-traffic-from-left conflict; and
    f. Through, cross-traffic-from-left conflict
4. Right-turn-on-red conflicts
    a. Opposing right-turn-on-red conflict; and
    b. Right-turn-on-red-from-right conflict
5. Pedestrian conflicts
6. Secondary conflicts

3.4.2 SSAM Software Program

In this study, overall conflict frequency, types of conflicts and severity of conflicts were used to evaluate the effectiveness of ramp metering on freeway safety. The overall conflict frequency and type of conflicts were obtained by using SSAM to analyze the trajectory files that were obtained as output in VISSIM. According to Pu and Joshi (the SSAM software manual, 2008), SSAM is a software application designed to perform statistical analysis of vehicle trajectory data output from microscopic traffic simulation models. SSAM is compatible with many traffic simulators such as AIMSUN, PARAMICS, TEXAS, VISSIM, etc. Surrogate measures of safety corresponding to each vehicle-to-vehicle interaction are calculated and deemed to be conflicts by the SSAM software program. SSAM classifies the vehicle-to-vehicle interaction as a conflict by using two threshold values that are Time-To-Collision (TTC) and Post-Encroachment Time (PET) (Gettman et al. 2008). SSAM identifies many surrogate measures for the conflict points such as PET, TTC, Max S (maximum speed between the two conflicting vehicles), and Delta S (The speed difference between the two conflicting vehicles). Figure 17 illustrates the surrogate measures on a conflict point occurring between a turning vehicle and a thorough vehicle at a typical intersection. According to the figure, the difference between the encroachment end time of the turning vehicle and the projected arrival time of the thorough vehicle (t_4-t_3) is the TTC. The time between the departure of the encroaching vehicle from the conflict point and the arrival of the vehicle (t_5-t_3) is the PET (Gettman and Head, 2003).

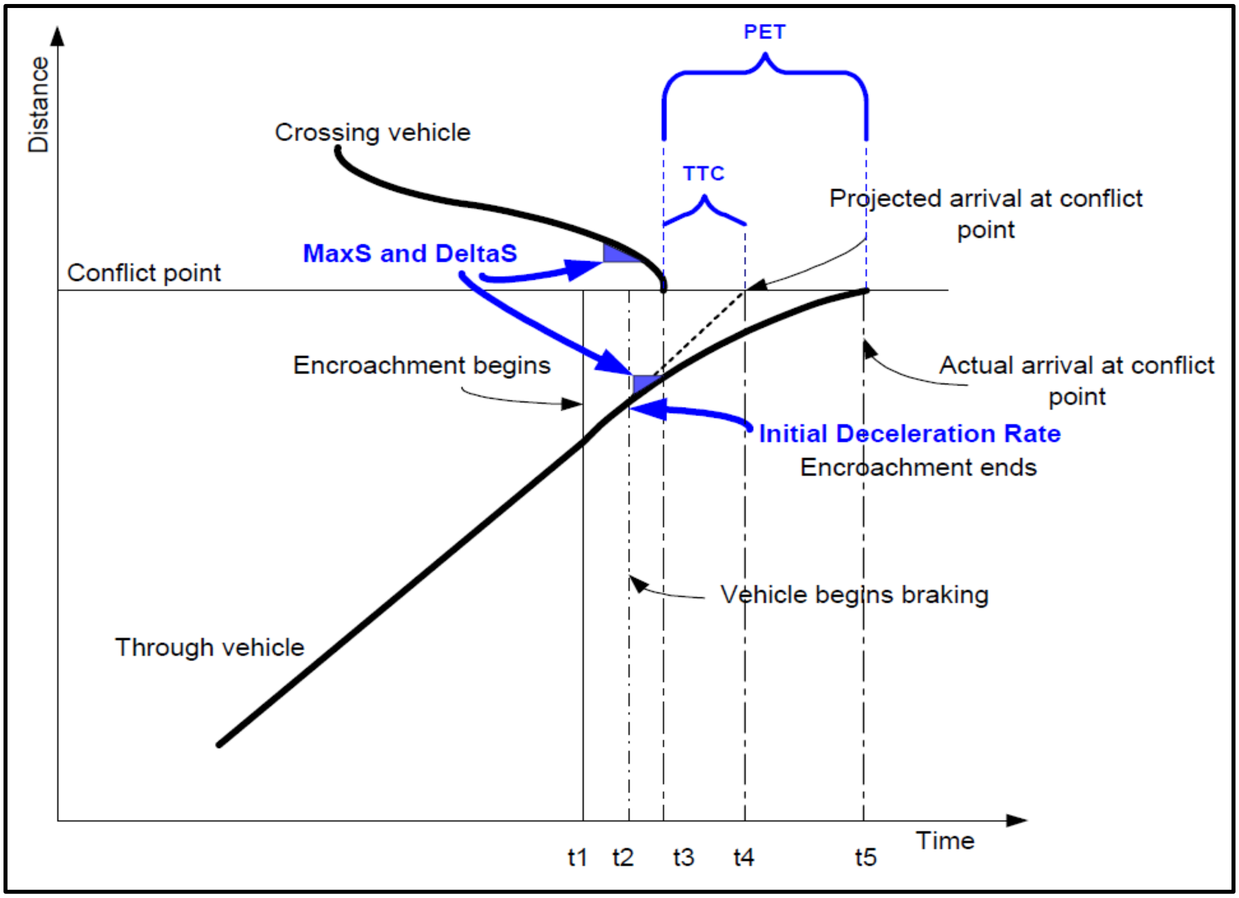


Figure 17: Surrogate measures on conflict point diagram (Gettman and Head, 2003)

Depending on the conflict angle between the two vehicles, SSAM classifies the conflicts into four types: crossing, rear-end, lane-change and unclassified; however, the link and lane information that are obtained from the VISSIM trajectory files affect the classification. If the two vehicles conflict at the same link and lane, SSAM classifies the conflict as a rear-end type regardless of the conflict angle. If the two vehicles are on the same link and one of them changes its lane, SSAM classifies the conflict as a lane-change type crash regardless of the conflict angle. In some cases, SSAM does not use link and lane information, for example if the information is not provided in the trajectory file, or if the vehicles are on different links. In such cases, SSAM uses conflict angles to classify the conflict types as follows (Pu L. and Joshi R, 2008):

- Unclassified: if the conflict angle/s unknown;
- Crossing: if the conflict angle greater than 80º;
- Rear-end: if the conflict angle is less than 30º; and
- Lane-change: if the conflict angle is between 30º and 80º

SSAM classifies the conflicts into four severity levels depending on TTC values. The severity levels start from the high severity to low severity as follows: conflicts with TTC equal to zero second, conflicts with TTC less and equal to 0.5 second, conflicts with TTC less and equal to 1.0 second, and conflicts with TTC less and equal to 1.5 seconds (Pu L. and Joshi R, 2008).

3.4.3 Time-To-Collision

Time-To-Collision (TTC) is used as a micro-level behavior indicator to classify the severity of conflicts. TTC is defined as the required time for two vehicles to collide if they continue at their present speed and along the same path (Laureshyn et al. 2010). Essentially TTC is calculated by assuming that the road users' trajectories cross at a right angle or they are parallel, as shown in Figure 18 and the following equations.

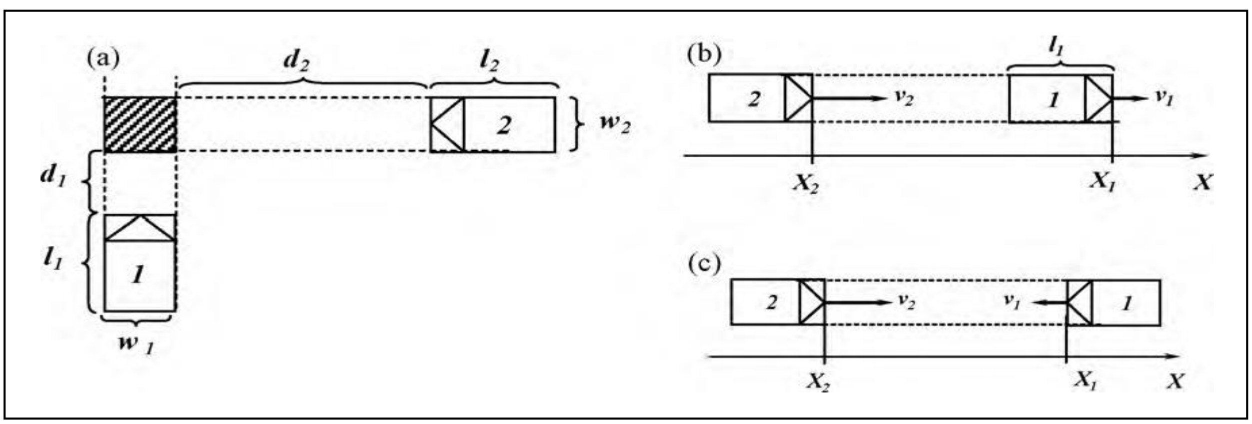


Figure 18: Calculation of TTC for perpendicular and parallel trajectories (Laureshyn et al. 2010)

Right angle approach:

$$\mathrm{TTC} = \frac{d_2}{v_2}, \qquad \text{if } \frac{d_1}{v_1} < \frac{d_2}{v_2} < \frac{d_1 + l_1 + w_2}{v_1}$$

$$\mathrm{TTC} = \frac{d_1}{v_1}, \qquad \text{if } \frac{d_2}{v_2} < \frac{d_1}{v_1} < \frac{d_2 + l_2 + w_1}{v_2}$$

Rear-end collision:

$$\mathrm{TTC} = \frac{X_1 - X_2 - l_1}{v_1 - v_2}, \qquad \text{if } v_2 > v_1,$$

Head-on collision:

$$\mathrm{TTC} = \frac{X_1 - X_2}{v_1 + v_2}$$

Where: d_1 and d_2 are distances from the fronts of vehicles 1 and 2, respectively; l_1, l_2 , and w_1, w_2 are the lengths and widths of vehicles 1 and 2, respectively; v_1, and v_2 are the vehicle speeds; X_1, and X_2 are the positions of vehicles 1 and 2, respectively (Laureshyn et al. 2010).

3.4.4 Severity of Conflict

As mentioned in Section 3.4.2, the SSAM program cannot classify the conflicts according to the severity types, but it can separate the conflicts into four different levels according to their TTC ranges. Many studies have been done to classify the severity of conflicts; however, most of the studies classified the conflict severities based on TTC. Sayed and Zein (1998) conducted a study to estimate traffic safety at signalized and unsignalized intersections throughout British Columbia by applying the traffic conflict technique. They separated TTCs into three different ranges to determine TTC and Risk of Collisions (ROC) scores. They classified ROC into three types: low, moderate, and high risks based on the classified TTC and ROC scores as shown in Table 9.

Table 9: TTC and ROC scores (Sayed and Zein, 1998)

TTC and ROC scores	Time To Collision (Seconds)	Risk Of Collision
1	1.6-2.0	Low risk
2	1.0-1.5	Moderate risk
3	0.0-0.9	High risk

Hyden (1987) developed a method for traffic safety evaluation based on traffic conflicts. The researcher classified the severity of conflicts into serious and non-serious types by drawing a new border line in the conflicts' speed-TTC diagram. The researcher separated the conflicts' speed-TTC diagram into six uniform severity zones and levels. The uniform separated zones went from one to six representing the low severe to high severe conflicts. Figure 19 shows the separated uniform zones above the new border line.

In 2012, Souleyrette and Hochestein conducted a study to develop a conflict analysis methodology by using the SSAM software program. They evaluated and compared the safety consequences of three alternative high-speed rural expressway intersection designs in Floyd, Iowa by modelling the expressway in VISSIM and examining the conflicts in SSAM. The severity scores were obtained to evaluate safety using three measures of conflicts: TTC, PET and MaxDeltaV.

In this study, the method that Souleyrette and Hochestein's used in their study was employed to classify the conflict severity.

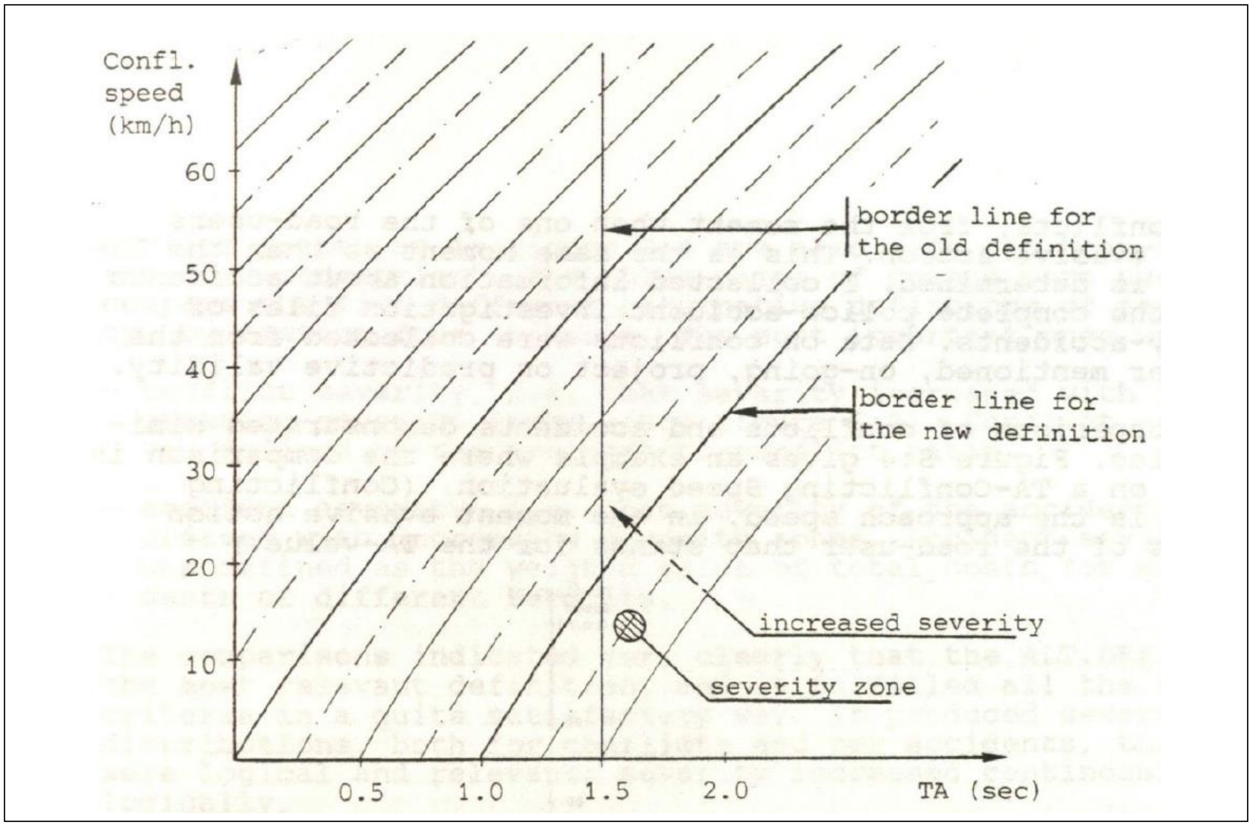


Figure 19: Uniform severity level and severity zones developed by Hyden (1987)

3.4.5 Equivalent Property Damage Only (EPDO)

Traffic crashes are classified into fatal, injury, and property damage only according to severity of accidents. Traffic safety can be evaluated by using an indicator of crash severity that consists of assigning a weighting factor to the number of Fatal (F) and Injury (I) and Property Damage Only (PDO) crashes. The addition of the weighted number of injury and fatal crashes to the number of PDO crashes is called an Equivalent Property Damage Only (EPDO) number (Mulinazzi and Russell, 1994). The weighting factors are often calculated depending on the costs of crashes that include direct and indirect crash costs. "Direct crash costs include ambulance service, police and fire services, property damage, or insurance. Indirect crash costs include the value society would place on pain and suffering or loss of life associated with the crash" (HSM, 2010). The weighting factors of fatal, injury, and PDO crash severities are calculated depending on the average crash costs using these formulas below (Herbel et al. 2010):

$$\text{Fatality Weighting Factor} = \mathrm{F_w} = \frac{\text{Average Fatal Crash Cost}}{\text{Average PDO Crash Cost}}$$

$$\text{Injury Weighting Factor} = \mathrm{I_w} = \frac{\text{Average Injury Crash Cost}}{\text{Average PDO Crash Cost}}$$

$$\text{PDO Weighting Factor} = \mathrm{P_w} = 1.0$$

Then, EPDO can be calculated by putting these factors into the following equation:

$$\mathrm{EPDO_i} = \mathrm{K_F(F_w) + K_I(I_w) + K_{PDO}(P_w)}$$

Where: $\mathrm{K_F}$ fatal crashes frequency, $\mathrm{K_I}$ is injury crash frequency, and $\mathrm{K_{PDO}}$ is PDO crash frequency.

Different agencies use different weighting factors to estimate EPDO. The KDOT applies six as a weighting factor for each fatal and injury crash; it means a fatal or an injury crash is considered as six PDO crashes as shown in the equation below (Mulinazzi and Russell, 1994):

$$\mathrm{EPDO_{Kansas}} = 6(\mathrm{F} + \mathrm{I}) + 1\ \mathrm{PDO}$$

The Virginia Department of Transportation (VDOT) uses 12 as a weighting factor for fatal crashes and three as a weighting factor for injury crashes as shown below (Nichols, 2006):

$$\mathrm{EPDO_{virginia}} = 12\ \mathrm{F} + 3\ \mathrm{I} + 1\ \mathrm{PDO}$$

The Massachusetts Department of Transportation (MassDOT) uses ten as a weighting factor for fatal crashes and five as a weighting factor for injury crashes as shown in the equation below (Cape Cod Commission, 2012):

$$\mathrm{EPDO}_{\mathrm{massachusetts}} = 10\ \mathrm{F} + 5\ \mathrm{I} + 1\ \mathrm{PDO}$$

Some agencies use more complicated formulas for finding EPDO; for example, the Kentucky Transportation Cabinet separates injury crashes into three types A, B and C depending on the severity of the injuries sustained, and uses the formula shown below to calculate EPDO (Zegeer and Deen, 1977) (Deacon, 1974).

$$\mathrm{EPDO}_{\mathrm{kentucky}} = 9.5\ (\mathrm{F} + \mathrm{A}) + 3.5\ (\mathrm{B} + \mathrm{C}) + \mathrm{PDO}$$

In this study, the serious, slight, and potential conflict types were considered to correspond to the fatal, injury, and PDO crash types, respectively. The same concept for calculating EPDO was used to calculate a new conflict severity criterion, which was named as Equivalent Potential Conflict (EPC) number. Three models were used to calculate EPC: the Kansas, Virginia, and Massachusetts formulas. The Massachusetts model gave the median values of EPC numbers among the models; therefore, it was used to evaluate effectiveness of the ramp meters on freeway safety for all the designed scenarios.

3.4.6 Crash Modification Factors

Crash Modification Factors (CMF) can be used to estimate the potential change in the crash frequency of a site after implementing a particular countermeasure, an intervention, or a design alternative. CMFs can be obtained by dividing the crash frequency after implementing a new design or a treatment to the crash frequency before implementing a new design or a treatment. If the CMF is equal to one, it indicates that the new design did not provide any improvement in the view of safety. When the CMF is less than one, the implementation of the new design resulted in improvement of safety by reducing the crash frequency. When the CMF is greater than one, the numbers of crashes after implementing the new design increased.

In this study, conflict modification factors (cMF) was used as an alternative to CMFs for estimating the potential change in conflict frequency, type and/or conflict severity after using ramp metering. The number, type, and severity of the conflicts were obtained from SSAM, while the cMFs were calculated for all designed scenarios using the following formula:

$$\text{Conflict Modification Factor} = \frac{\text{Conflicts with Ramp Metering}}{\text{Conflicts without Ramp Metering}}$$

CHAPTER 4: RESEARCH METHODOLOGY CONTIUNED-SITE SELECTION AND CHARACHTERISTICS

This chapter includes the detailed procedure that have been completed in the dissertation relating to site selection, geometric configuration designs, data collection, building models, calibration of a model, and the analysis methods that were used. The effects of ramp metering system on local streets are also shown in this chapter. In addition, the procedure of the sensitivity analysis is illustrated in detail.

4.1 Ramp Meters Site Selection and Geometric Configuration Design of the Study

For evaluating safety, efficiency, and Level of Services of both cases with and without ramp metering, several freeway sites in the Kansas City metropolitan area, having different geometric features, were investigated. The ramp meters are located on the I-435 freeway in Kansas City as illustrated in Figure A.1 in Appendix A. There are 16 metered ramps, which are located on the interchanges of I-435 connected with local streets. The connected streets are: Metcalf Avenue, Nall Avenue, Roe Avenue, State Line Road, Wornall Road, Holmes Road, and 103rd/104th Street. According to the Kansas City Scout, the ramp meters were installed in 2009 (KDOT and MoDOT, 2011). Nall Avenue and one of the metered ramps at Roe Avenue were not selected for this study because they are not connected to the freeway directly and they do not affect the movements on the freeway. Two sites of the ramp meters, State Line Road and Wornall Road, were not chosen as indicated by white circles in Figure A.1 in Appendix A, because their movements are weaving maneuvers. Eight of the ramps have ramp meters were used in this study. The eight ramp-freeway junctions were divided into three types depending on the geometric configuration. Two of the junctions are four-lane freeways with two-lane on-ramps, which are located on Metcalf Avenue and State Line Road. Two of the junctions were four-lane freeways with one-lane ramps, which are located on Holmes Road, and $103^{rd}/104^{th}$ Connector. Three of them are four-lane freeways with one-lane loop ramps, which are located on State Line Road and Holmes Road. One of the junctions is a five-lane freeway with a two-lane ramp that is changed to a one-lane beyond the ramp meter. All of the junctions have auxiliary lanes with different lengths. The number of lanes on the freeway main line at the I-435/Roe Avenue junction was reduced from five to four. Despite the new geometric configuration at I-435 Roe Avenue, the junction does not represent any specific real-world freeway-ramp junction, it was considered to reflect typical characteristics associated with isolated on-ramps and to build generic models for the evaluation study. Figure 20 shows the three samples selected from the eight ramps for collecting data, building models, and evaluating traffic parameters. The selected freeway-ramp junctions have different geometric configurations and traffic signal designs. The number of lanes for the freeway mainline of the Metcalf Avenue junction in the upstream is four, while in the downstream it is five. Lane numbers of the freeway mainline are the same for the upstream and downstream of both Holmes Road and Roe Avenue. The number of on-ramp lanes on the Holmes Road is one, while for Metcalf it is two. Roe Avenue has a different geometric configuration for on-ramp lanes; it has two lanes from the local street-ramp junction until the ramp meters, then one of the lanes is reduced from the ramp meters until the ramp-freeway junction. Figures A.2, A.3, and A.4 in Appendix A show the google images of I-435 freeway connected to the Metcalf Avenue, Roe Avenue, and Holmes Road, respectively.

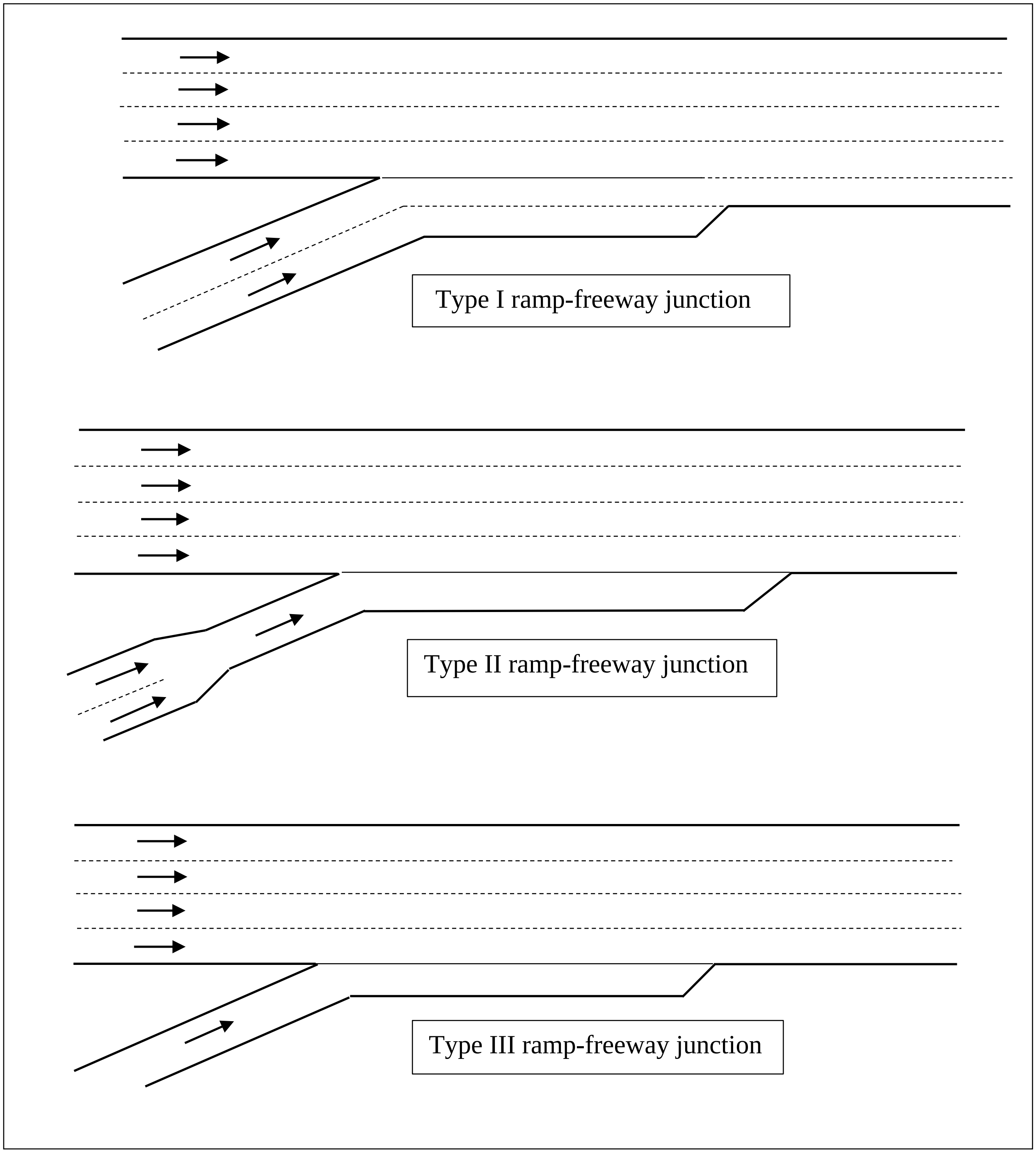


Figure 20: Three types of the selected geometric configurations for on-ramps and freeways

4.2 Field Data Collection

In order to calibrate driver behavior parameters in the VISSIM model, traffic data were obtained from video cameras and Kansas City Scout detectors. Four video cameras were installed in the morning and evening peak periods on Friday, September 12, 2014 to collect traffic data, such as traffic composition, traffic flow, queue length, and signal timing from Metcalf Avenue and Holmes Road-I-435 freeway junctions. Figure 21 shows the positions of the cameras. Speed and flow data on the upstream and downstream of the freeway and on the onramp were retrieved for the same time interval from Kansas City Scout. Traffic flow and speed data were collected from video cameras and Kansas City Scout for each lane of the freeway as numbered from one for the center lane to four for the shoulder lane as illustrated in Figure 21. Data were not taken at the Roe Avenue interchange because of two reasons: first, the interchange was being maintained during the study's data collection period and second, the freeway at this segment has five lanes, which was reduced to four lanes in the study. Therefore, the same driver behavior characteristics at the Holmes Road and Metcalf Avenue junction with the I-435 freeway were used for Roe Avenue and I-435 freeway junction. After three days of observation, the ramp meters on Holmes Road were realized to be in operation for short periods; therefore, only Metcalf Avenue data were used for driver behavior calibration and its data were applied to the freeway. The Holmes Road ramp and freeway traffic data were still taken to compare with Kansas City Scout detectors’ data.

4.2.1 Upstream Traffic Flow Data for I-435 Freeway

Camera number one was used to collect data from the upstream lanes in the freeway as shown in Figure 21. Traffic volumes, compositions, and lane proportions of the I-435 freeway connected to Metcalf Avenue are shown in Table 10 and Table 11. The data in Table 10 show that 97.2 percent of the vehicles are passenger cars and 2.8 percent of the vehicles are trucks and buses.

Table 11 shows that lane number four had the highest proportion (29 percent) of the traffic flow among the lanes. The data in both tables were used for calibration purposes. The collected data on the freeway upstream connected to Holmes Road are shown in Tables A.1 and A.2 in Appendix A.

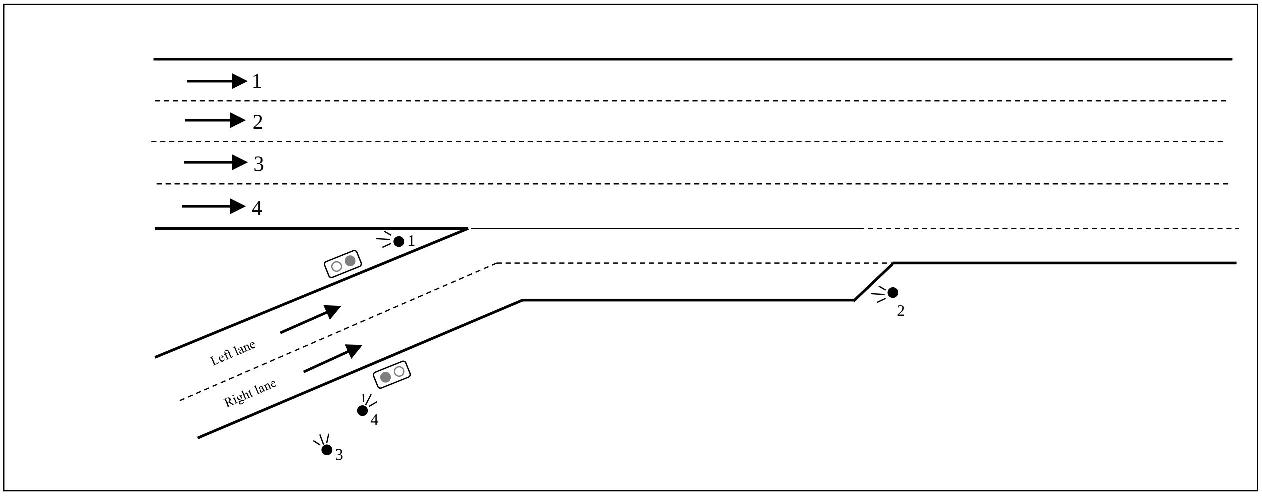


Figure 21: Locations of video camera in the I-435 freeway-Metcalf Avenue

Table 10: Upstream freeway traffic flow and composition in PM peak hour (I-435 freeway-Metcalf Avenue)

Lane 4				**Lane 3**				**Lane 2**				**Lane 1**			
Car	**Bus**	**Truck**	**Total**	**Car**	**Bus**	**Truck**	**Total**	**Car**	**Bus**	**Truck**	**Total**	**Car**	**Bus**	**Truck**	**Total**
1617	0	17	1634	1303	2	45	1350	1206	0	86	1292	1345	0	10	1355
99%	0%	1%	100%	96.5%	0.1%	3.4%	100%	93.3%	0%	6.7%	100%	99.3%	0%	0.7%	100%
Overall percentages: Car = 97.16 % Bus = 0.04 % Truck = 2.8 %															

Table 11: Proportions of freeway lanes in PM peak hour (I-435 freeway-Metcalf Avenue)

Lane number	**4**	**3**	**2**	**1**	**Total**
Traffic flow (vehicle per hour)	1634	1350	1292	1355	5631
Proportion	29 %	24 %	23 %	24%	100%

The percentages of differences between the Kansas City Scout traffic data and the field traffic data that were taken by cameras for the I-435 freeway connection to Holmes Road are shown in Table A.3 in Appendix A. The overall difference of traffic flow was 3.6 percent; however, the traffic flow difference was 9.9 percent for lane number four. These differences likely occurred due to detector error. Therefore, the field data from cameras were preferred for the calibration process and only the speed data were applied from the Kansas City Scout detectors.

4.2.2 On-ramp Traffic Flow Data

Table 12 shows the PM peak-hour traffic flow, composition, and proportions for both right and left lanes of the Metcalf Avenue ramp, which were taken from camera number two. The table shows that the overall percentage of passenger cars was 99.4, while the overall percentage of the trucks was only 0.6. The traffic flow at the right lane, 520 vph, was lower than the traffic flow at the left lane, 611 vph. The data from this table were used for calibration. Table A.4 in Appendix A shows the ramp traffic data for Holmes Road, which was used only for comparison.

Table 12: Traffic composition of the ramp in Metcalf Avenue at PM peak hour

	Right lane				**Left lane**				**Total**			
	Car	Bus	Truck	Total	Car	Bus	Truck	Total	Car	Bus	Truck	Total
Traffic flow	517	0	3	520	607	0	4	611	1124	0	7	1131
Proportion	99.4 %	0 %	0.6 %	100 %	99.3 %	0 %	0.7 %	100 %	99.4 %	0 %	0.6 %	100 %

4.2.3 Ramp Traffic Queue

Another type of traffic data used for calibration was on-ramp queue length behind the ramp meters. Camera number three was used to record the queue vehicle lengths that occurred on the onramps. The numbers of queued vehicles were counted every 30 seconds for both right and left lanes. The queues were measured from the signalized controlled intersection of the arterial street upstream of the ramp to the ramp meter's stop line. Table A.5 in appendix A shows the results of the ramp queue lengths. The number of queued vehicles was converted to queue length in feet after multiplying the numbers by 25, based on the Highway Capacity Manual. The mean queue length for the left lane was 132.2 feet, while it was 75.8 feet for the right lane. The average of the mean queue lengths for both right and left lanes was 104 feet.

4.2.4 Ramp Metering Traffic Signal Rates

Sixteen ramp meters on a 5.5 mile segment of the freeway I-435 starting from Metcalf Avenue to 103rd/104th street in Kansas City were deployed on seven interchanges of the Kansas City Scout system. To control the ramp meter signal rates, a new Corridor Adaptive Ramp Metering Algorithm (CARMA) was used, which allows ramp meters to be activated based on traffic demand (Sims, 2011). The algorithm system, CARMA, computes metering rates at each mainline vehicle detector station based on smoothed mainline speeds. The CARMA algorithm provides interconnection among the ramps based on downstream conditions, maximum and minimum rates, ramp queues, and hours of operations. According to the Kansas City Scout data, meters turn on when mainline speeds are below the threshold for at least three minutes; then the system adjusts metering rates depending on the mainline speed. The meters turn off in two cases; first, when speeds exceed the threshold for at least three minutes, and second, when the queue of vehicles on the ramp spills back into the upstream traffic signal on the arterial or the local streets. Metering rates are designed to be limited in the CARMA system by considering several minimum and maximum ramp, freeway, and signal parameters as shown in Table 13.

Table 13: Ramp meters parameters for CARMA algorithm in I-435 freeway in Kansas City (KC Scout Data)

	Metcalf Avenue	**Roe avenue**	**Holmes Road**
Vehicle per green	2	1	1
Min green (seconds)	2.7	1.5	1.5
Max green (seconds)	5	2.5	2.5
Min red (seconds)	2.5	2.5	2.5
Min rate (vphpl)	850	720	720
Max rate (vphpl)	1385	900	900
Min threshold speed (mph)	30	30	30
Max threshold speed (mph)	50	50	50

4.2.4.1 Traffic Signal Metering Rates

Camera number four was used to collect data from the right and the left lane ramp meters on the Metcalf Avenue ramp. The metering rates were operating based on the CARMA algorithm as illustrated in Section 4.2.4. The ramp meters' green and red times in the right and the left lanes were working reciprocally. When the left lane signal became red, the right lane signal was green and vice versa. Two seconds of all red signals existed in each cycle. In addition, the green and the red times were different for each cycle. The green-time periods, during the PM peak hour, were recorded precisely and separated for both the left and the right lanes. Table A.6 in Appendix A shows the results of the right lane traffic signal metering rates. The table shows that the total green-time period for the peak hour was 1,221.3 seconds, and the average value of the green-time period was 4.4 seconds. Table A.7 in Appendix A, shows the results of the left lane traffic signal metering rates. The total green-time period for the left lane was 1,354.4 seconds, which indicates a different value to the right lane. The average value of the left lane green-time periods was also 4.4 seconds.

Sometimes the left lane signal turned to green twice, while the right lane signal stayed in the red phase and turned to green signal only once. These resulted in making different cycle numbers for each of the lanes. As shown in the Tables A.6 and A.7, the number of signal cycles in the right lane was 278, while in left lane it was 307 cycles.

4.2.4.2 Violating Vehicles in the Ramp Metering

During the data reduction of vehicle numbers and signal timing, a significant number of violating vehicles that did not stop during the red-time intervals were noticed. As illustrated in Section 4.2.4.1, the cycle length period, and the green and the red time intervals were not constant because they were changing every 30 seconds for both lanes. The continuous change of the signal timings may have resulted in driver hesitation and the observed violation rate, which are shown in Table 14. The number of violating vehicles in the right lane was 69 out of 520 vehicles, which corresponds to a 13.3 percent violation rate. The number of violating vehicles in the left lane was 60 out of 611 vehicles, which corresponds to a 9.8 percent violation rate. The overall number of violating vehicles was 129 out of 1131 vehicles during the peak hour, which corresponds to an 11.4 percent rate. The violating vehicles had effects on the operation of the freeway mainline because the percentage of violating vehicles was not small; therefore, the violating vehicles were compensated for by a design with increased green-time intervals in the calibration processes.

Table 14: Number and percentage of violating vehicles on the Metcalf Avenue ramp metering during the peak hour

	Right lane	**Left lane**	**Total**
Traffic flow (vehicles per hour)	520	611	1131
Number of violating vehicles	69	60	129
Percentage of violating vehicles	13.3 %	9.8 %	11.4 %

4.2.4.3 The Signal Timing Design Used in the Calibration

Table 15 shows the summary of field traffic signal data on right and left lanes of the Metcalf Avenue ramp. The proposed design of the signal timing periods that were used for the calibration is shown in Table 16. Cycle timing lengths of 12 seconds were used, but the green-time periods for the right and the left lanes were different. The number of vehicles in the left lane was greater than the number of vehicles in the right lane; therefore, four seconds was used for the green-time period in the right lane and five seconds was used for green-time period in the left lane.

Table 15 Summary of field traffic signal for both lanes of the Metcalf Avenue ramp

Lane	Total green time (seconds) in peak hour	No. of violating vehicles	Number of cycle
Right	1,221.3	69	278
Left	1,354.4	60	306

Table 16: Proposed design of signal timing periods for the calibration

Lane	Design of signal timing periods	Cycle timing length
Right	4 s Green + 2 s All Red + 5 s Red + 1 s All Red	12 seconds
Left	4 s Red + 2 s All Red + 5 s Green + 1 s All Red	12 seconds

The total proposed design green-time periods for the one hour calibration model in the right and the left lanes correspond to 1,200 and 1,500 seconds, respectively, as shown in Table 17. The values of the proposed total green-time periods for one hour were very close to the field signal green-time periods for the right and left lanes, which were 1221.3 and 2575.7 seconds, respectively. The total green-time period difference for both of the lanes between field values and the proposed traffic signal was 124.3 seconds, which was used to modify the effects of 129 violating vehicles by allocating 0.96 second for each vehicle. The effect of violating vehicles on the freeway was modified by adding 124.3 seconds to the total green-time. In addition, only integer numbers can be used for fixed-time signal in the VISSIM program. As a result, the proposed signal-time periods shown in Table 16 were used as the best fit for the calibration process.

Table 17: Proportions of designed green-time for the calibration

Lane	Total green-time periods (seconds) designed for the calibration	Total green-time periods (seconds) in field during peak hour	No. of the violating vehicles
Right	1200	1221.3	69
Left	1500	1354.4	60
Total	2700	2575.7	129
Difference	124.3		

4.2.5 Traffic Flow and Speed Data Selection

Table 18 shows the traffic flow and speeds for each lane of the freeway segment and the ramp lanes, which were taken from the Kansas City Scout detectors. The average speed of the four lanes of the freeway was 44 mph and 35 mph on upstream and downstream, respectively. The peak-hour speed data of the freeway indicate a reduction in the speed of the freeway, which was reduced from 65 mph to 35 mph in the downstream. The average speed of both lanes in the ramp was 37.0 mph. As observed before for the Holmes Road and I-435 freeway junction, as shown in Table A.3 in Appendix A, there were differences between the detectors and cameras traffic flow data on the freeway. The Metcalf Avenue and I-435 freeway junction's data obtained from Kansas City Scout and the cameras show the differences between them too, as shown in Table 19. The overall total difference was -7.2 percent; however, for lanes number two and four the differences were considerable -24.3 percent and +19.6 percent. Therefore, the traffic flow data obtained from the Kansas City Scout were not used for the calibration. As a result, the speed data from Kansas City Scout detectors and traffic flow data from cameras were used as inputs for the calibration process for both the ramp and the freeway in the study. Kansas City Scout’s speed data and the traffic flow data from the cameras were taken at the same time at the PM peak period from 4:30 to 5:30.

Table 18: Kansas City Scout’s data at the Metcalf Avenue and I-435 junction

		Traffic volume, vph					**Speed, mph**				
Sep. 12, 2014 4:30-5:30 PM	**Lanes**	**1**	**2**	**3**	**4**	**All**	**1**	**2**	**3**	**4**	**All**
	Upstream mainline	1570	1606	1549	1314	6039	38.8	36.8	44.8	59.3	44
	Downstream mainline	1785	1341	1406	1564	6096	54.5	22	29.5	33	35
	Ramp	495				495	36.8				37

Table 19: Kansas City Scout detector and camera’s data on upstream of the freeway connected to (Metcalf Avenue), PM peak hour

Lanes	**1**	**2**	**3**	**4**	**Total**
Kansas Scout data	1570	1606	1549	1314	6039
Field data	1355	1292	1350	1634	5631
Difference	-215	-314	-199	+ 320	-408
% of difference	-15.9 %	-24.3 %	-14.7 %	+19.6 %	-7.2 %

4.3 A Model Calibration Process

A model, which was calibrated for both of the system and the operational calibration parameters, was used to analyze the scenarios that were built for the evaluation processes. Traffic and geometric data that were collected from the cameras and Kansas City Scout, as illustrated in Section 4.2, were used as the system calibration parameters to develop a baseline for the simulation model. The ramp-freeway junction was divided into five areas: freeway upstream, downstream, ramp influence area, auxiliary lane and the ramp as shown in Figure 22.

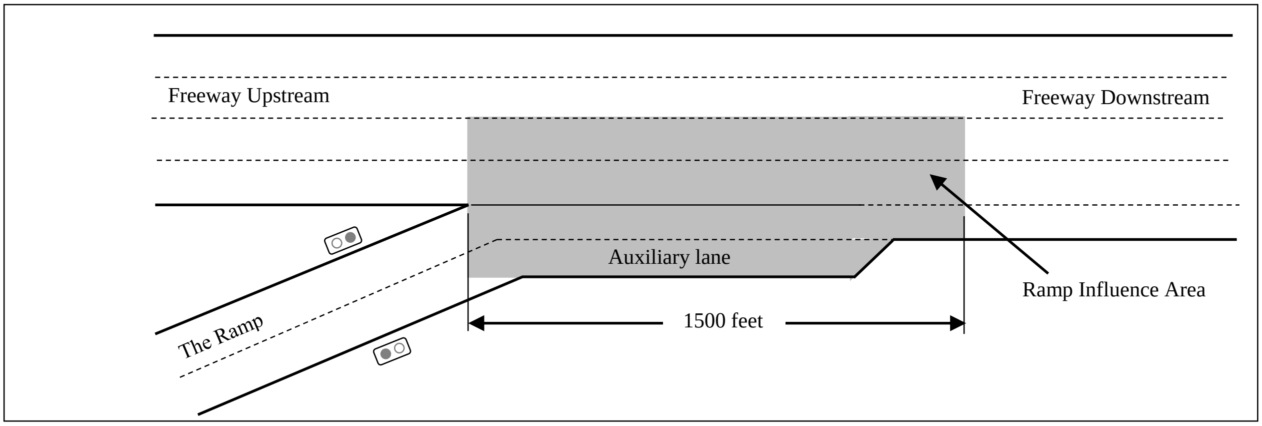


Figure 22: The freeway of I-435 connected to the Metcalf Avenue

The Wiedemann 99 model was selected as the car following model to set the operational calibration parameters. The Wiedemann 99 parameters in the VISSIM microsimulation program include ten parameters as shown in Table B.1 in Appendix B. The target key parameters among the operational calibration parameters were headways for all of the specified areas. The calibrated values for car-following headways on the freeway upstream and downstream, the ramp and the auxiliary lanes, and the ramp influence area, which were 2.24, 4.29, and 1.10 seconds, respectively.

The model was run five times with different seeds, and then tested statistically to calibrate a 3000 feet segment of the freeway and on-ramp junction. Each model was run for one hour with the model simulation resolution of five. The used multi-run seeds in VISSIM software were 19, 47, 75, 103, and 131 that were selected randomly. The increment, which was 28, also was selected randomly. The numbers of samples were checked statistically using 95% as the confidence level. Speeds upstream and downstream of the mainline and queue length on the ramp were taken as measures of effectiveness for the calibration, as shown in Table 20. The average values of the mean speeds in both the upstream and the downstream mainline lanes were used for the calibration. The statistical one-sample t-test was used to test the null hypotheses for both the freeway upstream and downstream speeds. The null hypothesis to test the freeway upstream average speed that were obtained from running the simulated model and from the Kansas City Scout detectors is shown below:

$$H_o: \mu_{VISSIM} = 44\ mph$$

$$H_a: \mu_{VISSIM} \neq 44\ mph$$

The average speed at the upstream mainline in the simulated model was 44.7 mph; that was close to the Kansas City Scout upstream mainline speeds, which was 44mph. The p-value was equal to 0.077, which was greater than 0.05. The null hypothesis was not rejected because it was not located in the rejection region. Therefore, it can be said that the average speed at the upstream mainline in the simulated model was equal to 44 mph.

The null hypothesis to test the freeway downstream average speed in the simulated model and the average speed in the field is shown below:

$$H_o: \mu_{VISSIM} = 35\ mph$$

$$H_a: \mu_{VISSIM} \neq 35\ mph$$

The average speed at the downstream mainline in the simulated model was 33.3 mph, while in the field it was 35 mph. The null hypothesis was not rejected because the p-value was equal to 0.068, which was greater than 0.05.

Average values of queue lengths for both of the right and the left lanes were taken at every 30 seconds during the peak hour in the simulation. The average of mean queue lengths was calculated for the five different seeds as shown in Table B.2 in Appendix B. The simulated average value of the queue lengths for both of the right and the left lanes and for the peak hour period was 116.3 feet that compared to the average values obtained from field cameras (Table A.5 in Appendix

A) of 104 feet. Statistical two-sample t-test was used to test the null hypothesis of the queue lengths on the ramp from the simulated model and the field.

$$H_o: \mu_{VISSIM} = \mu_{Field}$$

$$H_a: \mu_{VISSIM} \neq \mu_{Field}$$

Because the p-value was equal to 0.189, which was greater than 0.05, the null hypothesis was not rejected. Therefore, the platoons of the vehicles that came from the signalized controlled intersection upstream of the ramp were formed based on the calibrated queue lengths on the ramp. The tolerance for the average speed and the average queue length in the statistical sample tests were assumed as 2 mph, and 25 feet, respectively.

The calculation of the sample checking in Table 20 shows that queue length criterion, which was 3.11, controls the number of the sample. Therefore, running four models was appropriate for the safety and efficiency evaluation process; however, it was preferred to run five models with the selected seeds for both calibration and evaluation processes.

Table 20: Comparison between simulated and field data for calibration

			The ramp	The freeway					
	Run No.	**Seed No.**	**Average queue length, ft**	**Lane No.**	**Upstream speed, mph**	**Upstream average speed, mph**	**Lane No.**	**Downstream speed, mph**	**Downstream average speed, mph**
VISSIM simulated	1	19	84.3	1	44.6	44.5	1	34.9	34.9
				2	43		2	35.4	
				3	44.8		3	35.7	
				4	45.7		4	35.0	
							5	33.7	
	2	47	116.3	1	44.9	44.9	1	32.3	31.4
				2	43.6		2	32.3	
				3	45		3	31.2	
				4	46		4	30.3	
							5	30.8	
	3	75	108.9	1	44.6	43.7	1	34.4	33.7
				2	41.9		2	34.5	
				3	43.3		3	33.9	
				4	45.1		4	32.8	
							5	32.9	
	4	103	145.1	1	46	45.2	1	35.7	33.8
				2	43.9		2	34.9	
				3	44.9		3	33.5	
				4	45.9		4	32.8	
							5	31.9	
	5	131	127.1	1	45.5	45.2	1	35	34.0
				2	44		2	34.8	
				3	44.8		3	34.2	
				4	46.3		4	33	
							5	33.2	
	Average simulated		116.3			44.7			33.6
	Standard deviation		22.50			0.60			1.32
	Field		104			44			35
	p-value		0.189			0.077			0.068

Calculation of sample checking:
Confidence level = 95%
e = Tolerance
$n = (3.84 * SD^2)/(e^2)$
e = 2 mph for the speeds (assumed)
e = 25 feet for the queue (assumed)
Random seed starting point = 19
Random seed incremental point = 28
For queue → $n = 3.84*(22.5^2)/25^2 = 3.11$
For upstream speed → $n = 3.84*(0.6^2)/2^2 = 0.35$
For downstream speed → $n = 3.84*(1.32^2)/2^2 = 1.26$

4.4 Building Models and Assumptions

In this study, VISSIM 5.40 was used as the tool to build the models. To evaluate safety and efficiency of the ramp metering, 280 different scenarios were modeled including three different geometric configurations, various traffic signal timing designs, and different ramp and mainline traffic flows. In addition, 40 different scenarios were modeled to analyze the sensitivity of car following headways in the ramp influence area and traffic composition of the vehicles in the freeway.

4.4.1 Geometric Configurations

As shown in Figure 20, three different geometric configurations of ramp-freeway junctions were coded in VISSIM. To reflect typical characteristics and building generic models, the geometric configuration of the ramp-freeway junctions were defined as Type I, Type II, and Type III for the Metcalf Avenue, Roe Avenue, and Holmes Road ramps connected to I-435 freeway, respectively. These were modeled after the field sites on I-435 where field data were collected. In each version of the model, there were four main lanes on the freeway. Single and dual lane scenarios were modeled for the ramps with two different geometric configurations. The ramps and freeway segments were assumed to have zero slope. The parallel type of auxiliary lanes was selected. All of the freeway and ramp lanes were assumed to have twelve feet width. Figure 23 shows a sample of Type I ramp-freeway junction during running of the VISSIM program.

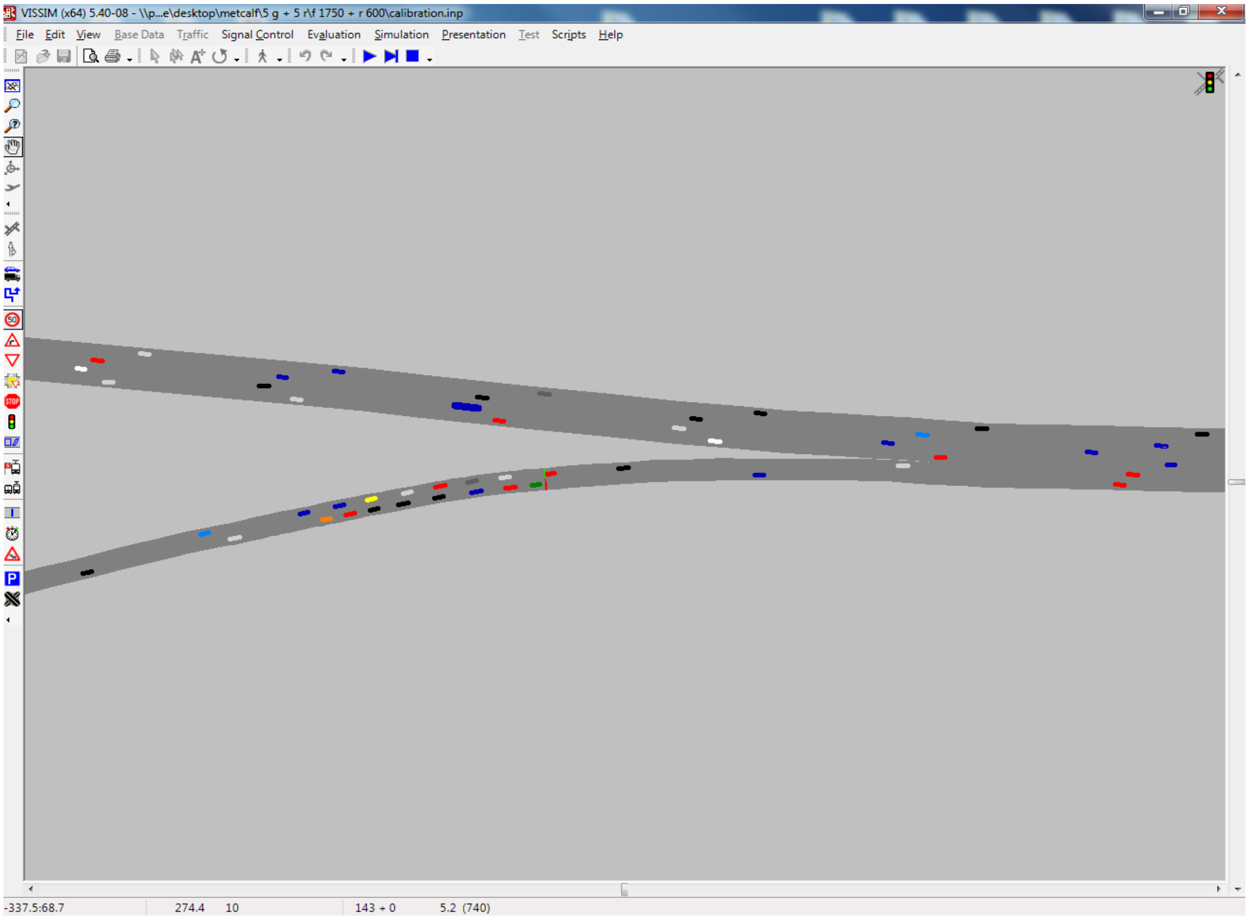


Figure 23: A view of the Type I ramp-freeway junction during running the program

4.4.2 Traffic Volume

Different traffic volume scenarios for the mainline and on-ramps were used to evaluate the effects of different traffic flow conditions on the ramp influence area and the downstream on the freeway. The traffic volume scenarios in the upstream mainline were assumed from 500 to 2,000 vphpl representing low to high traffic flow conditions. The traffic volume scenarios for the ramps ranged from 400 to 1000 vphpl. The traffic volume increments for the freeway mainline were 500 vphpl, while for the ramp were 200 vphpl. The assumed traffic volume scenarios represent many traffic flow conditions of the freeways and the ramps such as traffic flow breakdown and non-breakdown for the freeways, queue length spillback for the ramp vehicles, and qualitative traffic flow situations in the freeway downstream (congestion). Table 21 shows the upstream mainline freeway and on-ramp traffic volume scenarios that were modeled in the study.

Table 21: Traffic flow scenarios used in the study

		Ramp volume (vehicle per lane per hour)			
Freeway volume vehicles per lane per hour		400	600	800	1000
	500				
	750				
	1000				
	1250				
	1500				
	1750				
	2000				

4.4.3 Signal Design

Seven different signal timing scenarios were designed for the ramp meters based on the geometric configuration of the ramps and their traffic flow characteristics such as the number of cars per green and the number of lanes before and behind the ramp meters. Table 22 shows the different signal timing scenarios of the ramp meters that were compared with the base case (no ramp metering) to evaluate the effectiveness of the different designed volume scenarios. Two signal timing scenarios were designed for the ramp meters for each of the Type I and the Type II ramp-freeway junctions, while three signal timing scenarios were designed for the ramp meters for the Type III ramp-freeway junction. The signal timing scenarios were tested visually in the VISSIM program to allow one or two cars per green, as indicated in the Table 22. Only red and green signals were chosen without using an amber signal. Ten different signal timing scenarios were run for each of the traffic volume scenarios of the freeway and the ramp, which resulted in running 280 different models.

Table 22 Signal timing designs for different ramp geometric configurations

Type of junction	**Number of car/s per green per lane**	**First lane**	**Second lane**
Type I	--	No ramp meter	No ramp meter
	1	2G+1AR+2R+1AR	2R+AR+2G+1AR
	2	5G+1AR+5R+1AR	5R+1AR+5G+1AR
Type II	--	No ramp meter	No ramp meter
	1	2G+1AR+2R+1AR	2R+1AR+2G+1AR
	2	5G+1AR+5R+1AR	5R+1AR+5G+1AR
Type III	--	No ramp meter	No ramp meter
	1	2G+2R	
	2	4G+4R	
	1	2G+4R	
Note : G= Green, R= Red, AR= all red (all periods are in seconds)			

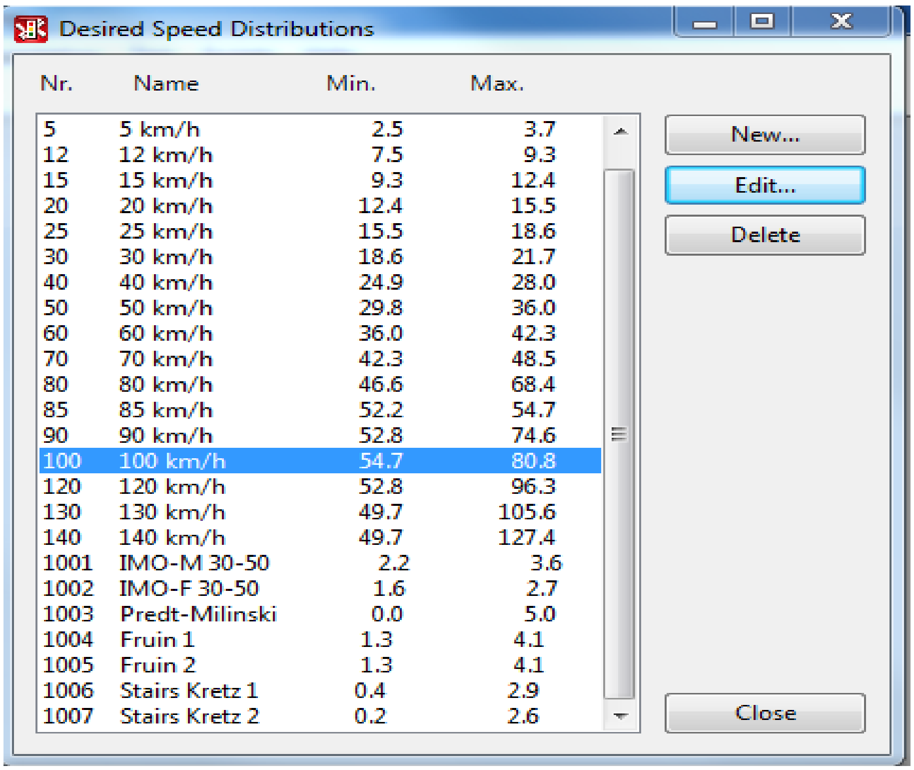
4.4.4 Traffic Data Assumptions

As mentioned in the previous sections, traffic volume, geometric configuration, and ramp signal timing were assumed in the study; moreover, many other traffic data were assumed such as speed limit, desired speed, traffic composition, and lane change behavior. The assumed speed limit for the freeway upstream was 62.2 mph (100 km/hr) with the assumed desired speed profile ranges from 54.7 mph to 80.8 mph; while assumed speed limit for the ramp was 43.5 mph (70 km/hr) with the assumed desired speed profile ranges from 42.3 mph to 48.5 mph. The assumed speed limits for the freeway upstream and the ramp were chosen as 62.2 mph and 43.5 mph, respectively because the speed limits on VISSIM were designed by using metric units while the maximum and minimum speeds of the desired speeds were designed by English unit (mph) as shown in Figure

24. The peak-hour traffic composition data at the I-435-Metcalf Avenue interchange, which were collected for the calibration process, were used for running the models. The assumed traffic composition consisted of 97 percent of passenger cars and 3 percent of buses and trucks. Ramp meters were set at two-thirds of the distance from the upstream of the ramp. Flashing yellow beacon was set in the beginning of the upstream of the ramp. The calibrated driver behavior and route decision characteristics were applied to all of ramp-freeway junctions. Wiedmann 1999 and free lane change option were chosen as the car following and the lane change behavior model. Each simulated model was run five times with different running seeds of 19, 47, 75,103, and 131 based on the calibrated queue lengths in the ramps to form platoons. Each model was run for one hour and five minutes. The first five minutes of the models' running were required for vehicles to settle in the system to avoid any data bias. The outputs of the first five minutes were not taken for the evaluation, only the outputs of the last hour were used into account. The simulated models were tested visually for realistic and reasonable vehicle behavior movements.

4.5 Efficiency Evaluation

In ramp metering control strategies, efficiency can be measured as a function of two parameters: input and output. Input consists of the cost of ramp metering implementation such as installation, maintenance and operation of the ramp meters. Output determines the benefits that gain from the implementation of ramp meters such as reduction of total travel time, delay, fuel consumption, and emissions, and/or changes in total traffic volume and speed. In this study, four operational factors were used as measures of effectiveness to evaluate the effects of ramp metering on efficiency: speed, travel time, density, and the level of service of the freeway. Queue lengths behind the ramp meters were also considered in the study in order to avoid the negative effects of the ramp meters on the surrounding street network. Figure 25 illustrates how the VISSIM outputs were measured to evaluate the effectiveness of ramp metering on the efficiency of the freeways.


Desired Speed Distributions

Nr.	Name	Min.	Max.
5	5 km/h	2.5	3.7
12	12 km/h	7.5	9.3
15	15 km/h	9.3	12.4
20	20 km/h	12.4	15.5
25	25 km/h	15.5	18.6
30	30 km/h	18.6	21.7
40	40 km/h	24.9	28.0
50	50 km/h	29.8	36.0
60	60 km/h	36.0	42.3
70	70 km/h	42.3	48.5
80	80 km/h	46.6	68.4
85	85 km/h	52.2	54.7
90	90 km/h	52.8	74.6
100	100 km/h	54.7	80.8
120	120 km/h	52.8	96.3
130	130 km/h	49.7	105.6
140	140 km/h	49.7	127.4
1001	IMO-M 30-50	2.2	3.6
1002	IMO-F 30-50	1.6	2.7
1003	Predt-Milinski	0.0	5.0
1004	Fruin 1	1.3	4.1
1005	Fruin 2	1.3	4.1
1006	Stairs Kretz 1	0.4	2.9
1007	Stairs Kretz 2	0.2	2.6

Figure 24: Desired speed distribution in VISSIM program

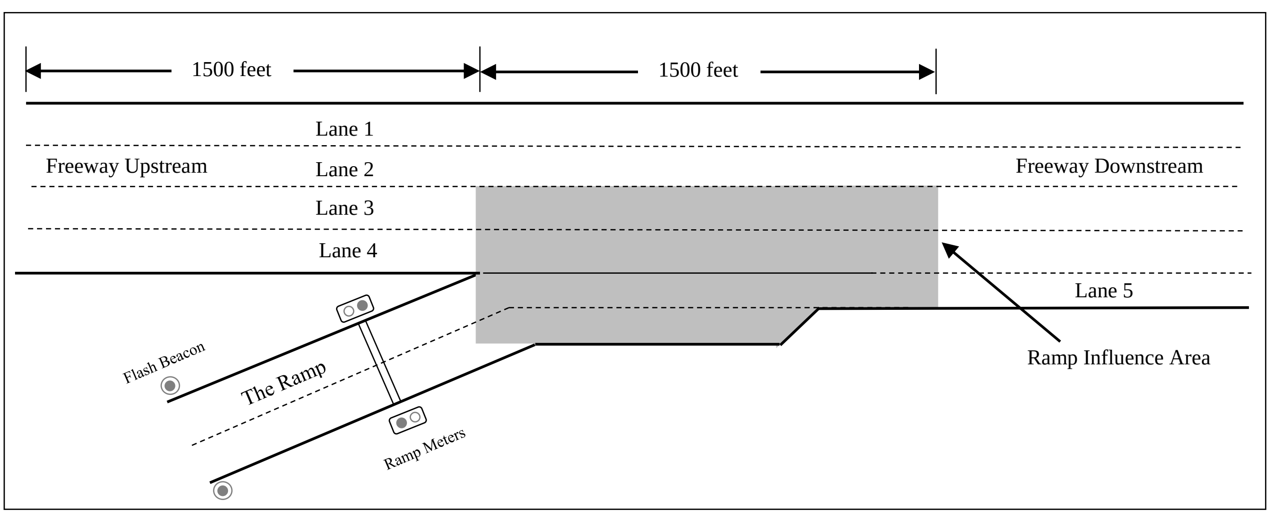


Figure 25: Determination of measures of effectiveness at the Type I ramp-freeway junction

The detailed processes of measuring the outputs are shown below:

- Speed: The average of mean speeds of all lanes from one through five in the ramp influence area indicated in Figure 25 were compared in both cases of with and without using ramp meters.
- Travel time: as a measure of the overall traffic condition on the freeway mainline, the average of the total travel time was compared for both cases of with and without using ramp meters. The average of total travel times of the vehicles passing the 3,000 feet of the freeway segment was taken. The freeway segment started from the beginning of the freeway upstream to the beginning of the freeway downstream as illustrated in Figure 25.
- Density: average of the mean densities in lanes number three, four, and five in the ramp influence area were compared for both cases of with and without using ramp meters.
- Level of service: based on the average density of the ramp influence area, the levels of service on the freeway segments were found to determine if the ramp meters were able to change the levels of service or not.
- Queue behind the ramp meter: queue spillback blocks the traffic movements in the traffic signal from upstream of the ramp and it may cause delay on the adjacent street network; therefore, the average queue length behind the ramp meters was taken into account for the evaluation.

4.6 Safety Evaluation

Having different scenarios of geometric configurations, ramp metering signals, and traffic volumes was a reason to use traffic conflicts as appropriate surrogates for traffic crashes. Traffic conflicts regarding frequency, type, and severity of the conflicts that occurred among vehicles on 3,000 ft of the freeway were used as measures of effectiveness of the ramp meters on safety. The SSAM software program was used to find the frequency and type of the conflicts. In addition, the excel output files were used to classify the conflicts according to severity. Five seconds was used for both maximum (TTC) and maximum (PET) values; while, the default values were used for the rest of the parameters.

CMFs were calculated by dividing the conflict numbers with using ramp metering into the conflict numbers without using ramp metering. The CMFs were obtained for all geometric, signal, and traffic volume scenarios in terms of frequency, type, and severity of conflicts.

4.6.1 Classification of Conflicts According to Types

The conflicts were classified into four different types: crossing, rear end, lane change, and unidentified by using the default values of the conflict angles criteria in the SSAM software program. The cross and unidentified conflict types were neglected and considered as zero, because their numbers were very small; therefore, only rear-end and lane-change conflict types were taken into account.

4.6.2 Classification of Conflicts According to the Severity

The severities of the conflicts were specified based on TTC and MaxDeltaV (maximum speed difference between conflicting vehicles). The method of classifying severity of conflicts used in the Souleyrette and Hochestein study was also employed. The conflicts were classified according to the severity for all of the assumed 280 scenarios including the five running seeds. A classification of the conflict severity for one scenario is illustrated below in which the freeway and the ramp traffic volumes were 1,500, and 600 vphpl, respectively, for the no ramp meter-seed 19 case. The classification process depended on severity scores that were used as final indicators to separate the severity of the conflicts. The severity scores were obtained by adding the TTC score and the ROC score. The first step to separate the severity of the conflicts was to determine the TTC score by drawing a cumulative frequency distribution line for TTC as shown in Figure 26. The first inflection point, shown at about 1.2 seconds in the diagram, was selected as a critical point for the extreme collision propensity level and corresponds to TTC score of 3 as shown in Table 23.

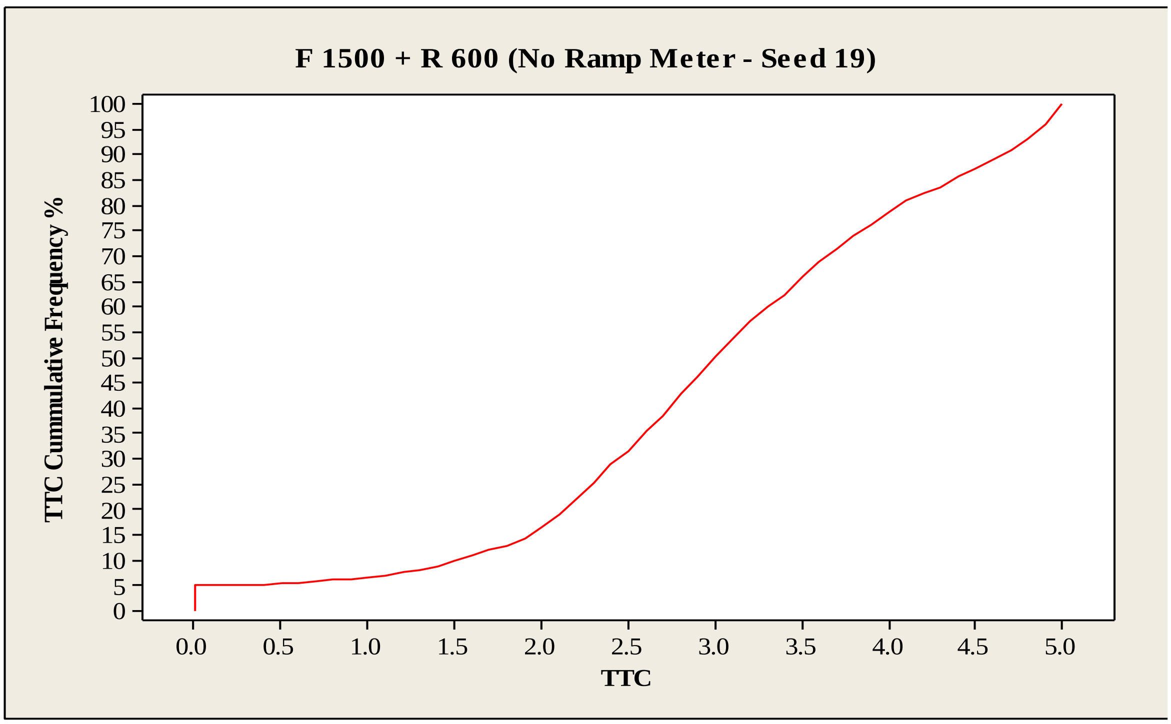


Figure 26: Cumulative frequency percentage for TTC

Table 23: Assigned TTC (collision propensity) score F1500+R600 (No ramp meter) - seed 19

TTC score	TTC range (sec)	Sample size, (%)	Collision propensity level
0	3.6 < TTC	499, (30.9)	Low
1	2.7 < TTC ≤ 3.6	497, (30.8)	Moderate
2	1.2 < TTC ≤ 2.7	497, (30.8)	High
3	TTC ≤ 1.2	121, (7.5)	Extreme
	Total	1614, (100)	

The other points of collision propensity level (high, moderate, and low) were indicated by dividing the TTC cumulative frequency percentage ranges equally (roughly 30 percent); then for each of the conflict severity, a TTC score was specified from zero to three as shown in Table 23.

The second step was drawing a cumulative frequency curve for the MaxDeltaV. The 85th percentile was selected as a critical point to determine the ROC score. The ROC score is one for conflicts when the MaxDeltaV is under the 85th percentile in the cumulative frequency curve. The ROC score is two for conflicts that have a MaxDeltaV above the 85th percentile in the cumulative frequency curve. Souleyrette and Hochestein (2012) set the ROC scores as three for those conflicts that had MaxDeltaV greater than 40 mph. Figure 27 shows the curve of the cumulative frequency for the MaxDeltaV (mph) for one example scenario.

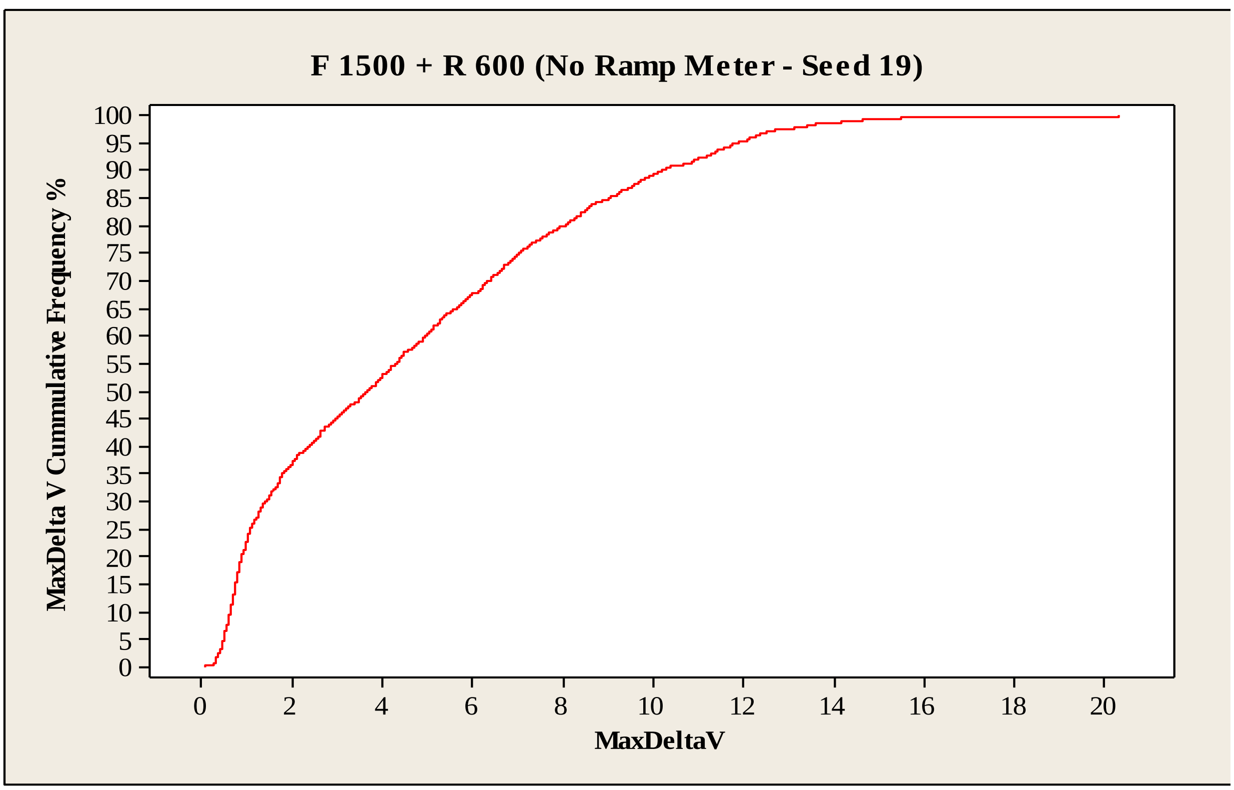


Figure 27: Cumulative frequency percentage for MaxDeltaV (mph)

In Figure 27, the 85th percentile for MaxDeltaV was 9 mph that is determined as a critical value for separating ROC scores. A MaxDeltaV less than 9 mph were determined as a low potential collision severity level. When MaxDeltaV was between 9 and 40 mph, the level was considered as moderate; while a MaxDeltaV is greater than 40 was considered as high, as shown in Table 24.

Table 24: Assigned ROC score based on MaxDeltaV (F1500+R600)–(no ramp metering)-seed 19

ROC score	**MaxDeltaV range (mph)**	**Potential collision severity level**
1	MaxDeltaV ≤ 9 (85th percentile)	Low ≈ PDO
2	9 < MaxDeltaV < 40	Moderate ≈ Injury
3	MaxDeltaV ≥ 40	High ≈ Fatal

The last step was finding the severity score by adding the obtained TTC and ROC scores. The severity scores were numbers starting from one to six in which conflicts with high scores were more severe than the conflicts with low scores. Conflicts were classified into three severity levels: potential conflicts with severity scores of one or two; slight conflicts with severity scores of three or four; and serious conflicts with severity scores of five or six. Table 25 and Figure 28 show the results of classification of the severity scores of the illustrative scenario. In Figure 28, the black and red colors indicate the potential conflicts; green and blue colors indicate slight conflicts; and the orange color indicates serious conflicts.

Table 25: Severity score for F1500+R600 (no ramp metering) – seed 19

Severity score	Collision number	Sum	Type
1	495	979	Potential
2	484		
3	433	597	Slight
4	164		
5	38	38	Serious
6	0		

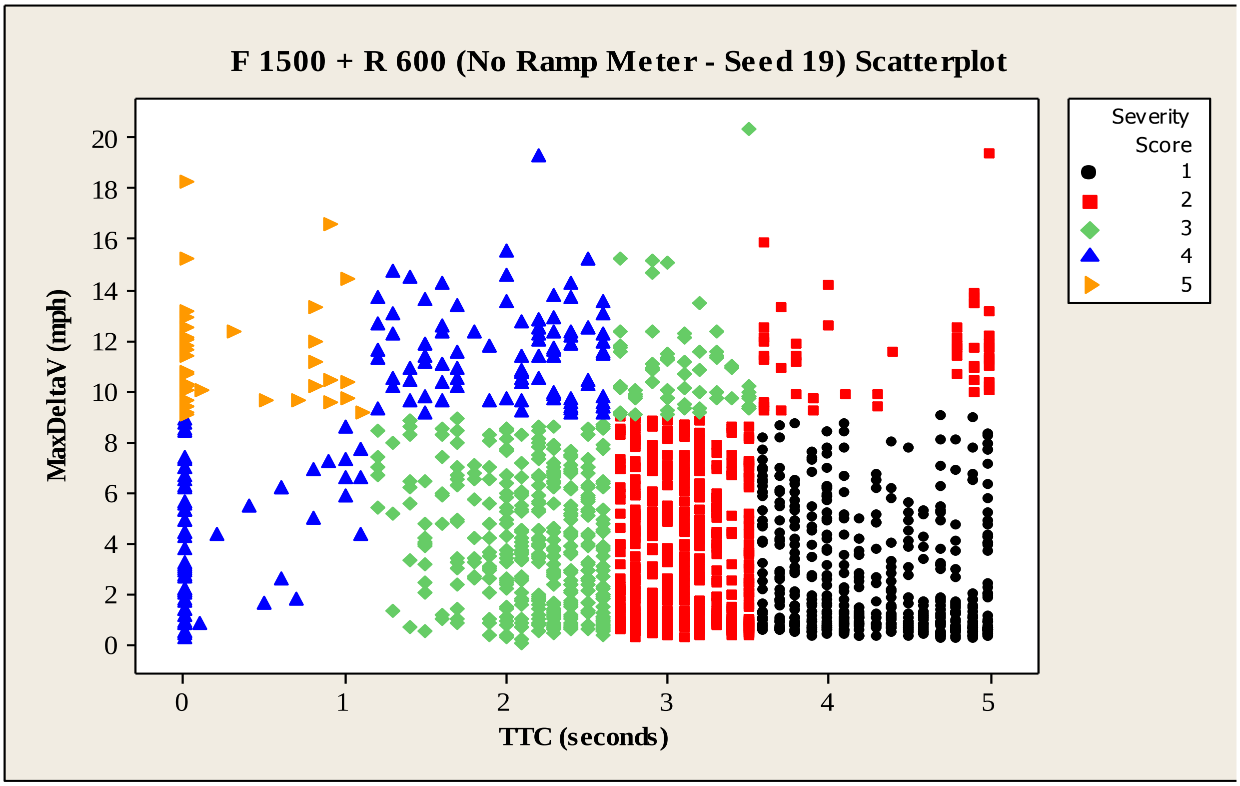


Figure 28: Conflicts showing the severity of the conflicts of F1500+R600 (no ramp metering case) – seed 19

4.6.3 Equivalent Potential Conflicts

A new conflict severity criterion was introduced and named the Equivalent Potential Conflicts (EPC) by using the same equations of EPDO as mentioned in Section 3.4.5. The fatal, injury, and PDO crash terms were altered by the serious, slight, and potential conflicts. The Massachusetts model, which provided the median values of the EPC numbers among the models, was used as the major model to evaluate the effectiveness of the ramp metering on freeway safety.

4.7 Sensitivity Analysis

To examine the effects of change in two of the assumptions on the evaluation of the ramp metering, a sensitivity analysis was conducted. Also, the effects of the assumptions change on the efficiency and safety of the ramp metering were tested for specific traffic volume scenarios. Sensitivity analyses for different car following headways and traffic composition scenarios were done in the Type III geometric configuration of a ramp-freeway junction. The freeway traffic volume was fixed 1,750 vphpl and the ramp traffic volumes were taking as 400, 600, 800, and 1,000 vphpl representing the peak hour period in the field. Five different car following headways in the ramp influence area (0.9, 1.0, 1.1, 1.2, and 1.3 sec) were examined for the sensitivity analysis. In addition, five different percentages of buses and trucks were examined representing different traffic composition. The percentages of buses and trucks, which were examined in the sensitivity analysis, were 3, 5, 7, 9, and 11 percent. The effects of the assumptions on changes in the sensitivity analysis were assessed statistically using F tests. The Minitab statistical software program was used to test the null hypotheses with 95 percent as the level of significance.

The methodology and the research approach presented in Chapters 3 and 4 were prepared to explore the effectiveness of ramp metering on the efficiency, Level of Service, and safety of freeway. The evaluation results are presented in Chapter 5.

CHAPTER 5: EVALUATION RESULTS

This chapter presents the results of the exploratory analysis of the effects of ramp metering on freeway efficiency, Level of Service, and safety. The effects of the ramp meters on local streets were considered by taking into account the queue spillback from the ramp into the local streets. The evaluation results for the different traffic volume scenarios of the ramp and the freeway are explained regarding different ramp metering signal timing scenarios in three different geometric configurations of ramp-freeway junctions. The evaluation of the freeway efficiency was based on average speed in the ramp influence area and the average travel time of the vehicles on a 3000 ft freeway segment adjacent to the ramp. The average density of the vehicles in the ramp influence area was used to indicate the level of service of the freeway. All of the parameters that were used as measures of effectiveness in the efficiency and Level of Service evaluation were obtained as outputs from running micro-simulation VISSIM. Conflict Modification Factors (cMF) were used as indicators to evaluate the effectiveness of ramp meters on freeway safety. The cMFs were obtained from simple calculations of the vehicle conflicts that occurred during the periods when the ramp meters are on and off. Traffic conflicts were obtained from the analyses of the VISSIM trajectory files by using the SAAM software program. The cMFs were counted for overall conflicts, types, and severity of conflicts. The conflicts were classified as rear-end and lane-change type conflicts; the conflicts were classified according to the severity of the conflict: potential, slight, and serious severity conflicts. Only the effectiveness of the ramp meters on freeway safety was evaluated by taking the conflicts that occurred in the 3,000 feet segment of the freeway adjacent to each ramp. The conflicts that occurred on the on-ramp were not taken into account in the safety evaluation. The average queue length of the vehicles in the onramp was used as a measure of the negative effects of the ramp meters on local streets. The ramp metering signal timing rates were designed based on the average lengths of the vehicles in the onramps, number of cars per green interval, and the geometric configuration of the ramp. The Anderson-Darling normality test was used to test whether the outputs were normal or not. This chapter also explains the results of the sensitivity analysis, which include the effects of the changes of two assumptions on the results' outputs. Car-following headway and traffic composition of the vehicles were the two assumptions that were used in the sensitivity analysis. The statistical hypothesis F-test was used to determine the effects of the assumptions changes on the outputs.

5.1 Evaluation of the Effectiveness of Ramp Metering on Freeway Efficiency

This section includes the evaluation results of the effectiveness of ramp metering on the efficiency of the freeway based on the average speed and the average travel time of the freeway vehicles. It explains the evaluation results for the different assumed traffic volumes and the designed ramp meters signal timing scenarios applied to the geometric configurations of Type I, Type II, and Type III ramp-freeway junctions. The average speeds of the vehicles were taken from the lanes in the ramp influence area. Both the average speeds and the average travel times were obtained from the outputs of running five different seeds in VISSIM.

5.1.1 Effects of Ramp Metering on Freeway Efficiency of Type I Ramp-Freeway Junction

Tables C.1, C.2, and C.3 in Appendix C show the results of the VISSIM output of average speeds for the base case and the two designed ramp meter signal timing scenarios at the ramp influence area of the Type I ramp-freeway junction. Table 26 shows the summarized values of the average speeds, designed signal timings, and assumed traffic volumes; it also includes the percentages of average speed change due to use of the two designed ramp metering signal timing scenarios. The first signal timing scenario was 2 seconds red, 1 second all red, 2 seconds green, and 1 second all red (2R+1AR+2G+1AR), and the second signal timing scenario was 5 seconds red, 1 second all red, 5 seconds green, and 1 second all red (5R+1AR+5G+1AR). The highlighted cells indicate that the ramp meters could increase the average speeds in the ramp influence area by more than five percent. The table shows that the ramp meters increased average speeds in the ramp influence area when the ramp traffic volume was greater or equal to 800 vphpl and the freeway traffic volume greater or equal to 1,250 vphpl, simultaneously. The signal timing scenario of (5R+1AR+5G+1AR) provided better results than the signal timing scenario of (2R+1AR+2G+1AR). For example, when the freeway traffic volume was 2,000 vphpl and the ramp traffic volume was 1,000 vphpl (F2000+R1000), the signal timing scenario of (2R+1AR+2G+1AR) increased the average speed in the ramp influence area by 130.5 percent (from 21.3 to 49.1 mph), whereas the signal timing scenario of (5R+1AR+5G+1AR) increased the average speed by 133.3 percent (from 21.3 to 49.7 mph). The more traffic volume on the ramp and the freeway, the more freeway traffic efficiency was provided by the ramp meters. Non-highlighted cells in the table indicate that the ramp metering could not provide better freeway efficiency than the base case. Sometimes the ramp metering resulted in decreasing the average speeds of the ramp influence area in some traffic volume scenarios. For example, in (F2000+R400) traffic volume scenario, the average speed in the no ramp metering scenario was 54.4 mph, while in signal timing scenarios of (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR) were 53.8 and 53.6 mph, respectively.

Table 26: Average speed (mph) and percent change at the ramp influence area of the Type I ramp-freeway junction before and after using ramp metering

Freeway volume (vehicle / hour lane)	Signal design	Ramp volume (vehicles / hour lane) 400 speed	400 % change	600 speed	600 % change	800 speed	800 % change	1000 speed	1000 % change
500	No ramp meter	60.4		59.6		59.0		59.0	
	2R+1AR+2G+1AR	60.4	0.0	59.7	0.2	59.6	1.0	59.6	1.0
	5R+1AR+5G+1AR	60.5	0.2	59.6	0.0	59.6	1.0	59.6	1.0
750	No ramp meter	60.1		59.1		58.6		58.5	
	2R+1AR+2G+1AR	60.2	0.2	59.3	0.3	59.3	1.2	59.3	1.4
	5R+1AR+5G+1AR	60.1	0.0	59.3	0.3	59.2	1.0	59.2	1.2
1000	No ramp meter	59.4		58.5		57.2		56.9	
	2R+1AR+2G+1AR	59.4	0.0	58.5	0.0	58.5	2.3	58.5	2.8
	5R+1AR+5G+1AR	59.4	0.0	58.4	-0.2	58.4	2.1	58.4	2.6
1250	No ramp meter	56.9		54.8		48.5		47.7	
	2R+1AR+2G+1AR	57.1	0.4	54.7	-0.2	54.1	11.5	54.0	13.2
	5R+1AR+5G+1AR	56.9	0.0	54.6	-0.4	54.6	12.6	54.1	13.4
1500	No ramp meter	54.6		49.9		36.1		23.9	
	2R+1AR+2G+1AR	54.2	-0.7	49.6	-0.6	49.6	37.4	49.6	107.5
	5R+1AR+5G+1AR	54.3	-0.5	49.9	0.0	49.7	37.7	49.7	107.9
1750	No ramp meter	54.4		49.9		36.0		23.9	
	2R+1AR+2G+1AR	53.9	-0.9	49.6	-0.6	49.6	37.8	49.2	105.9
	5R+1AR+5G+1AR	53.6	-1.5	50.2	0.6	50.2	39.4	49.7	107.9
2000	No ramp meter	54.4		49.9		36.0		21.3	
	2R+1AR+2G+1AR	53.8	-1.1	49.7	-0.4	49.4	37.2	49.1	130.5
	5R+1AR+5G+1AR	53.6	-1.5	49.8	-0.2	49.7	38.1	49.7	133.3

The average travel time outputs of VISSIM in the assumed traffic volume and signal timing scenarios are shown in Tables C.4, C.5, and C.6 in Appendix C. Table 27 shows the summary of the average travel time results of the designed scenarios. The table also includes the percentage changes of average travel time that resulted after using ramp metering. It was considered that the ramp meters increased the efficiency of the freeway if the percentage change of the average travel time was equal or greater than five percent, as indicated in the highlighted cells. The results obtained from the travel time analyses support the results that were obtained in the speed analyses. Table 27 shows that ramp metering on Type I ramp-freeway junctions increased the traffic efficiency of the freeway when the traffic volume of the freeway was equal or greater than 1,250 vphpl and the traffic volume of the ramp equal or greater than 800 vphpl, simultaneously. While the two designed signal timings were very close in operation, it can be said that the (5R+1AR+5G+1AR) signal timing scenario provided better results than the (2R+1AR+2G+1AR) signal timing scenario. For example, in (F1250+R800) traffic volume scenario, the percentage change of the average travel time in the (2R+1AR+2G+1AR) signal timing scenario was 8.1, while in the (5R+1AR+5G+1AR) signal timing scenario, it was 9.9. The difference of the percentage change between the two signal timing scenarios was 1.8 percent, which can be considered as a significant difference if all the vehicles that pass the freeway during peak hour are taken into account. As can be seen in the table, using ramp meters provided negative effects in several situations because the travel times increased after using the ramp metering such as shown in the positive numbers of percentage changes in the non-highlighted cells. Moreover, the negative effects of the ramp meters in the non-highlighted cells become greater if the benefit cost ratio analysis is conducted or the delayed time of the ramp vehicles is regarded. Consequently, it can be said that ramp metering is useful for increasing the freeway efficiency of the geometric configuration of Type I ramp-freeway junction only during the peak period, or specifically when the traffic volume of the ramp is equal or greater than 800 vphpl and the traffic volume of the freeway is equal or greater than 1,250 vphpl, simultaneously.

Table 27: Travel time (second) and percent change on the freeway of the Type I ramp-freeway junction before and after using ramp metering

Freeway volume (vehicle / hour lane)		Signal design	Ramp volume (vehicles / hour lane)							
			400		600		800		1000	
			TT	% change	TT	% change	TT	% change	TT	% change
	500	No ramp meter	31.5		31.6		31.7		31.7	
		2R+1AR+2G+1AR	31.6	0.3	31.8	0.6	31.9	0.6	31.9	0.6
		5R+1AR+5G+1AR	31.6	0.3	31.8	0.6	31.8	0.3	31.8	0.3
	750	No ramp meter	32.0		32.2		32.3		32.3	
		2R+1AR+2G+1AR	32.1	0.3	32.3	0.3	32.4	0.3	32.4	0.3
		5R+1AR+5G+1AR	32.2	0.6	32.3	0.3	32.3	0.0	32.3	0.0
	1000	No ramp meter	33.1		33.4		34.1		34.3	
		2R+1AR+2G+1AR	33.2	0.3	33.6	0.6	33.6	-1.5	33.6	-2.0
		5R+1AR+5G+1AR	33.2	0.3	33.6	0.6	33.5	-1.8	33.5	-2.3
	1250	No ramp meter	39.5		40.9		45.6		46.4	
		2R+1AR+2G+1AR	39.4	-0.3	41.2	0.7	41.9	-8.1	42.0	-9.5
		5R+1AR+5G+1AR	39.8	0.8	41.1	0.5	41.1	-9.9	41.8	-9.9
	1500	No ramp meter	43.7		47.4		58.3		73.2	
		2R+1AR+2G+1AR	43.8	0.2	47.8	0.8	48.0	-17.7	47.9	-34.6
		5R+1AR+5G+1AR	43.5	-0.5	47.6	0.4	47.8	-18.0	47.8	-34.7
	1750	No ramp meter	43.5		47.2		58.6		73.2	
		2R+1AR+2G+1AR	43.9	0.9	48.2	2.1	47.7	-18.6	48.1	-34.3
		5R+1AR+5G+1AR	44.3	1.8	47.3	0.2	47.3	-19.3	47.9	-34.6
	2000	No ramp meter	43.7		47.2		58.5		75.2	
		2R+1AR+2G+1AR	44.3	1.4	48.0	1.7	48.4	-17.3	48.1	-36.0
		5R+1AR+5G+1AR	44.3	1.4	47.7	1.1	47.7	-18.5	47.7	-36.6

5.1.2 Effects of Ramp Metering on Freeway Efficiency of Type II Ramp-Freeway Junction

The results of the VISSIM output of the average speeds (mph) of the base case and the two designed signal timing scenarios at the ramp influence area of the Type II ramp-freeway junction are shown in Tables C.7, C.8, and C.9 in Appendix C. Table 28, which is the summary table for the three previous tables, includes the average speeds in the ramp influence area and the percentage of average speed change that resulted after using the ramp metering. Table 28 shows that the ramp metering did not increase the efficiency of the freeway; on the contrary, the ramp metering decreased the efficiency of the freeway. The efficiency of the freeway was decreased by a large percentage under some of the designed scenarios. As an example, when the freeway traffic volume was 1,250 vphpl, the average speeds after using the ramp metering decreased by roughly 20 percent, as indicated by bold letters. The difference between Type I ramp-freeway junction and Type II ramp-freeway junction is the number of lanes in the downstream of the freeway; the freeway in Type I junction has five lanes in the downstream, while the freeway in Type II junction has four lanes in the downstream. The number of lanes in the freeway downstream affects the effectiveness of the ramp metering. In the Type I ramp-freeway junction, the vehicles that entered the freeway were distributed over five lanes, while in the Type II ramp-freeway junction, they distributed over four lanes. The distribution of the vehicles in the freeway in Type II junction over four lanes caused more traffic congestion in the freeway downstream than in the freeway in Type I junction. As a result, the queue of congested vehicles on the freeway of Type II junction reached the upstream of the freeway, specifically when the traffic flow of the ramp and the freeway was similar to the traffic flow of peak hour. The negative effectiveness of the ramp metering was much greater if the benefit-cost ratio was analyzed; therefore, ramp metering is not suggested for use in the geometric configuration of Type II ramp-freeway junction.

Table 28: Average speed (mph) and percent change at the ramp influence area of the Type II ramp-freeway junction before and after using ramp metering

Freeway volume (vehicle / hour lane)	Signal design	400 speed	400 % change	600 speed	600 % change	800 speed	800 % change	1000 speed	1000 % change
		Ramp volume (vehicles / hour lane)							
500	No ramp meter	59.3		59.0		59.2		59.0	
	2R+1AR+2G+1AR	58.8	-0.8	58.8	-0.3	58.7	-0.8	58.8	-0.3
	5R+1AR+5G+1AR	59.4	0.2	58.6	-0.7	58.7	-0.8	58.7	-0.5
750	No ramp meter	59.0		58.7		58.6		58.6	
	2R+1AR+2G+1AR	59.0	0.0	58.3	-0.7	58.2	-0.7	58.4	-0.3
	5R+1AR+5G+1AR	59.0	0.0	58.3	-0.7	58.3	-0.5	58.4	-0.3
1000	No ramp meter	57.9		57.5		57.5		57.5	
	2R+1AR+2G+1AR	57.6	-0.5	57.0	-0.9	56.9	-1.0	56.9	-1.0
	5R+1AR+5G+1AR	57.8	-0.2	57.1	-0.7	57.0	-0.9	57.1	-0.7
1250	No ramp meter	25.6		22.8		22.3		22.4	
	2R+1AR+2G+1AR	24.4	-4.7	18.4	**-19.3**	18.3	**-17.9**	17.9	**-20.1**
	5R+1AR+5G+1AR	24.5	-4.3	17.9	**-21.5**	18.1	**-18.8**	18.6	**-17.0**
1500	No ramp meter	14.2		14.0		14.1		14.0	
	2R+1AR+2G+1AR	13.9	-2.1	13.9	-0.7	13.7	-2.8	13.8	-1.4
	5R+1AR+5G+1AR	13.9	-2.1	13.8	-1.4	13.7	-2.8	13.7	-2.1
1750	No ramp meter	14.2		14.1		14.1		14.0	
	2R+1AR+2G+1AR	13.9	-2.1	13.8	-2.1	13.8	-2.1	13.9	-0.7
	5R+1AR+5G+1AR	13.9	-2.1	13.8	-2.1	13.8	-2.1	13.7	-2.1
2000	No ramp meter	14.1		14.0		14.1		13.9	
	2R+1AR+2G+1AR	14.0	-0.7	13.7	-2.1	13.8	-2.1	13.7	-1.4
	5R+1AR+5G+1AR	13.8	-2.1	13.9	-0.7	13.8	-2.1	13.8	-0.7

Tables C.10, C.11, and C.12 in Appendix C show the results of the VISSIM outputs of the average travel time on the freeway segment of Type II junction. Table 29, which includes the summary of the average travel times of the three previous tables, supports the consequences obtained from the speed analyses in which ramp meters increased average travel times of the vehicles in all of the assumed traffic volume and the designed signal timing scenarios. Thus, in the light of the speed and travel time results, using ramp metering in the geometric configuration of Type II ramp-freeway junctions is not recommended.

Table 29: Travel time (second) and percent change on the freeway of the Type II ramp-freeway junction before and after using ramp metering

Freeway volume (vehicle / hour lane)	Signal design	Ramp volume (vehicles / hour lane)							
		400		600		800		1000	
		TT	% change	TT	% change	TT	% change	TT	% change
500	No ramp meter	31.8		31.7		31.7		31.8	
	2R+1AR+2G+1AR	31.8	0.0	31.8	0.3	31.8	0.3	31.8	0.0
	5R+1AR+5G+1AR	31.8	0.0	31.8	0.3	31.7	0.0	31.8	0.0
750	No ramp meter	32.3		32.3		32.3		32.3	
	2R+1AR+2G+1AR	32.3	0.3	32.3	0.0	32.3	0.0	32.3	0.0
	5R+1AR+5G+1AR	32.3	0.3	32.6	0.9	32.3	0.0	32.3	0.0
1000	No ramp meter	33.4		33.6		33.7		33.6	
	2R+1AR+2G+1AR	33.7	0.9	33.8	0.6	33.9	0.6	33.8	0.6
	5R+1AR+5G+1AR	33.7	0.9	33.7	0.3	33.7	0.0	33.8	0.6
1250	No ramp meter	68.1		71.9		73.5		72.7	
	2R+1AR+2G+1AR	70.3	3.2	80.9	**12.5**	81.2	**10.5**	82.0	**12.8**
	5R+1AR+5G+1AR	69.6	2.2	82.3	**14.5**	81.8	**11.3**	79.9	**9.9**
1500	No ramp meter	106.0		106.2		106.5		106.8	
	2R+1AR+2G+1AR	107.2	1.1	107.0	0.8	107.7	1.1	107.5	0.7
	5R+1AR+5G+1AR	106.8	0.8	108.0	1.7	108.5	1.9	108.1	1.2
1750	No ramp meter	105.9		106.3		106.1		106.1	
	2R+1AR+2G+1AR	107.0	1.0	107.9	1.5	107.9	1.7	107.8	1.6
	5R+1AR+5G+1AR	106.3	0.4	107.6	1.2	107.5	1.3	107.6	1.4
2000	No ramp meter	106.0		106.6		106.5		106.2	
	2R+1AR+2G+1AR	106.8	0.8	108.2	1.5	107.8	1.2	107.9	1.6
	5R+1AR+5G+1AR	107.2	1.1	107.1	0.5	107.6	1.0	107.9	1.6

5.1.3 Effects of Ramp Metering on Freeway Efficiency of Type III Ramp-Freeway Junction

To evaluate the effectiveness of ramp metering on freeway efficiency for a Type III ramp-freeway junction, three signal timing scenarios were designed. Tables C.13, C.14, C.15, and C.16 show the results of the VISSIM outputs of the average speed at the influence area of a Type III ramp-freeway junction. Table 30 shows the summary results of the average speeds in the ramp influence area for the base case and designed signal timing scenarios. The table also includes the percentages of the average speed change that resulted from using the designed ramp metering signal timing scenarios. The results of the first two signal timing scenarios (2R+2G) and (4R+4G) show that ramp metering decreased the average speed of the vehicles in the ramp influence area in almost all traffic volume scenarios, which means decreasing the efficiency of the freeway. On the other hand, under the designed signal timing scenario (4R+2G), ramp metering increased the average speed of the vehicles in the ramp influence area when the freeway traffic volume was greater or equal than 1,250 vphpl and the ramp traffic volume was equal or greater than 600 vphpl, simultaneously. Under the circumstances of the (4R+2G) signal timing scenario, the ramp meters provided the greatest positive effects on efficiency when the freeway traffic volume was 1,250 vphpl. For example, the ramp meters increased the average speed of the vehicles in the ramp influence area from 21.5 mph to 40.9 mph (a 90.2 percent increase) in the (F1250+R1000) traffic volume scenario. Tables C.17, C.18, C.19, and C.20 in Appendix C, show the results of the average travel time on the freeway segment for the designed scenarios. The results of the average travel times of the base case and the three designed signal timing scenarios are summarized in Table 31. The table also includes the percentage of the travel time changes resulted by using ramp metering.

Table 30: Average speed (mph) and percent change at the ramp influence area of the Type III ramp-freeway junction before and after using ramp metering

Freeway volume (vehicle / hour lane)		Signal design	Ramp volume (vehicles / hour lane) 400 speed	400 % change	600 speed	600 % change	800 speed	800 % change	1000 speed	1000 % change
	500	No ramp meter	58.6		57.8		57.4		57.5	
		2R + 2G	58.6	0.0	58.0	0.3	57.4	0.0	57.2	-0.5
		4R + 4G	58.5	-0.2	57.9	0.2	57.6	0.3	58.5	1.7
		4R + 2G	58.4	-0.3	57.9	0.2	57.9	0.9	58.0	0.9
	750	No ramp meter	58.2		57.7		57.0		56.9	
		2R + 2G	58.3	0.2	57.7	0.0	57.1	0.2	57.0	0.2
		4R + 4G	58.4	0.3	57.6	-0.2	57.1	0.2	57.0	0.2
		4R + 2G	58.3	0.2	57.6	-0.2	57.5	0.9	57.5	1.1
	1000	No ramp meter	57.7		56.9		56.2		55.8	
		2R + 2G	57.7	0.0	56.8	-0.2	56.1	-0.2	55.9	0.2
		4R + 4G	57.6	-0.2	56.8	-0.2	56.2	0.0	56.0	0.4
		4R + 2G	57.3	-0.7	56.7	-0.4	56.8	1.1	56.8	1.8
	1250	No ramp meter	52.1		36.7		23.2		21.5	
		2R + 2G	52.2	0.2	37.5	2.2	22.3	-3.9	19.1	-11.2
		4R + 4G	53.1	1.9	37.3	1.6	22.2	-4.3	19.9	-7.4
		4R + 2G	51.7	-0.8	40.9	11.4	38.0	63.8	40.9	90.2
	1500	No ramp meter	28.0		20.6		17.7		17.8	
		2R + 2G	27.8	-0.7	20.6	0.0	17.8	0.6	17.6	-1.1
		4R + 4G	27.8	-0.7	20.5	-0.5	17.8	0.6	17.4	-2.2
		4R + 2G	28.2	0.7	21.5	4.4	21.0	18.6	21.3	19.7
	1750	No ramp meter	27.9		20.4		17.8		17.6	
		2R + 2G	28.0	0.4	20.5	0.5	17.9	0.6	17.6	0.0
		4R + 4G	27.8	-0.4	20.6	1.0	17.8	0.0	17.5	-0.6
		4R + 2G	28.2	1.1	21.4	4.9	21.1	18.5	21.2	20.5
	2000	No ramp meter	28.0		20.6		17.7		17.8	
		2R + 2G	27.8	-0.7	20.7	0.5	17.6	-0.6	17.6	-1.1
		4R + 4G	28.3	1.1	20.7	0.5	17.8	0.6	17.6	-1.1
		4R + 2G	28.3	1.1	21.6	4.9	21.2	19.8	21.1	18.5

Table 31: Average travel time (second) and percent change on the freeway of the Type III ramp-freeway junction before and after using ramp metering

Freeway volume (vehicle / hour lane)	Signal design	400 TT	400 % change	600 TT	600 % change	800 TT	800 % change	1000 TT	1000 % change
500	No ramp meter	31.3		31.4		31.5		31.6	
	2R + 2G	31.3	0.0	31.4	0.0	31.5	0.0	31.5	-0.3
	4R + 4G	31.3	0.0	31.4	0.0	31.5	0.0	31.3	-0.9
	4R + 2G	31.4	0.3	31.5	0.3	31.5	0.0	31.5	-0.3
750	No ramp meter	31.7		31.9		32.1		32.1	
	2R + 2G	31.7	0.0	31.9	0.0	32.1	0.0	32.1	0.0
	4R + 4G	31.7	0.0	31.9	0.0	32.0	-0.3	32.1	0.0
	4R + 2G	31.8	0.3	32.0	0.3	32.0	-0.3	32.0	-0.3
1000	No ramp meter	32.7		33.0		33.7		33.9	
	2R + 2G	32.8	0.3	33.2	0.6	33.7	0.0	34.1	0.6
	4R + 4G	32.7	0.0	33.2	0.6	33.7	0.0	33.8	-0.3
	4R + 2G	32.8	0.3	33.1	0.3	33.1	-1.8	33.3	-1.8
1250	No ramp meter	40.3		52.0		76.1		80.8	
	2R + 2G	40.1	-0.5	52.4	0.8	78.5	3.2	89.3	**10.5**
	4R + 4G	40.0	-0.7	52.6	1.2	78.6	3.3	86.5	**7.1**
	4R + 2G	40.5	0.5	50.0	-3.8	52.6	-30.9	53.0	-34.4
1500	No ramp meter	79.0		91.7		99.9		100.4	
	2R + 2G	79.2	0.3	91.5	-0.2	100.1	0.2	101.1	0.7
	4R + 4G	79.2	0.3	91.6	-0.1	100.2	0.3	101.6	1.2
	4R + 2G	79.1	0.1	91.6	-0.1	92.1	-7.8	92.3	-8.1
1750	No ramp meter	79.3		92.0		100.0		100.7	
	2R + 2G	78.9	-0.5	91.7	-0.3	100.4	0.4	101.3	0.6
	4R + 4G	79.2	-0.1	91.8	-0.2	100.2	0.2	101.3	0.6
	4R + 2G	79.2	-0.1	91.5	-0.5	92.3	-7.7	92.1	-8.5
2000	No ramp meter	79.2		91.6		100.1		100.3	
	2R + 2G	79.1	-0.1	91.4	-0.2	100.4	0.3	101.2	0.9
	4R + 4G	78.7	-0.6	91.5	-0.1	100.1	0.0	101.4	1.1
	4R + 2G	79.1	-0.1	91.6	0.0	92.4	-7.7	92.3	-8.0

Ramp volume (vehicles / hour lane)

The ramp meters decreased the average travel times only in the signal timing scenario of (4R+2G) when the traffic volume of the ramp was equal or greater than 800 vphpl and the freeway traffic volume was equal or greater than 1,250 vphpl, simultaneously. The ramp meters in the first two signal timing scenarios did not provide any beneficial effects; on the contrary, they provided negative effects when the traffic volume of the freeway was equal or greater than 1,250 vphpl and the traffic volume of the ramp was equal or greater than 600 vphpl.

In conclusion, it was determined that ramp metering was able to increase the efficiency of the freeway only under the designed signal timing scenario of the (4R+2G) when the traffic volume of the freeway is equal or greater than 1,250 vphpl and the traffic volume of the ramp is equal or greater than 800 vphpl. Increasing 2 seconds for the red-time period in the (4R+2G) scenario transferred the delay of the vehicles from the freeway to the ramp and resulted in increasing the average speed and decreasing the average travel time of the vehicles on the freeway.

5.2 Evaluation of the Effectiveness of Ramp Metering on Level of Service of the Freeways

Density of the vehicles in the ramp influence area was used to find the level of service of the freeway as a measure of the capacity evaluation. VISSIM provides and separates vehicle density for every lane as an output with units of vehicles-per-mile-per-lane (vpmpl). The average vehicle density of the five different seeds was used to find the level of service (LOS) of the freeway by using the method from the HCM. Table 32 is the HCM’s table that was used to specify the level of service of the freeway.

Table 32 Level of service criteria for freeway merge and diverge segments (HCM, 2010)

LOS	Density (pc/mi/ln)	Comments
A	≤ 10	Unrestricted operations
B	> 10 - 20	Merging and diverging maneuvers noticeable to drivers
C	> 20 - 28	Influence area speeds begin to decline
D	> 28 - 35	Influence area turbulence becomes intrusive
E	> 35	Turbulence felt by virtually all drivers
F	Demand exceeds capacity	Ramp and freeway queues form

As it is seen in the table, density with pcpmpl unit was used to indicate the LOS of freeway merge and diverge segments. In order to change the unit from vpmpl to pcpmpl, the following traffic parameters were assumed as the adjustment factors: 0.92 as a Peak Hour Factor (PHF), three percent buses and trucks as a traffic composition, level ground as a type of terrain, no recreational vehicles, and familiar driver commuters. The LOSs were obtained for the designed and assumed scenarios including different geometric configurations, signal timings, and ramp and freeway traffic volumes in both cases of ramp metering off and on.

5.2.1 Effects of Ramp Metering on Freeway Level of Service of Type I Ramp-Freeway Junctions

Tables D.1, D.2, and D.3 in Appendix D show the VISSIM output results of the average vehicle densities in the influence area of Type I ramp-freeway junctions. Table D.4 summarized the results of the average densities at the ramp influence area, which were obtained from using the three designed signal timing scenarios. The freeway LOSs cannot be obtained from the density values of Table D.4 because the units of the densities are vpmpl. Table 33 shows the summary results of the average densities after converting the units from vpmpl to pcpmpl. The results in Table 33 show that ramp metering changed the LOSs of the freeway from the low LOS of E or F into high

LOS of C when the freeway traffic volume was equal or greater than 1,250 vphpl and the ramp traffic volume was equal or greater than 800 vphpl. The highlighted cells show the traffic volume scenarios in which ramp meters increased the freeway capacity by raised their LOS. The ramp meters provided considerable positive effects when traffic volumes on the freeway were equal or greater than 1,500 vphpl. The greatest benefits that the ramp meters provided to the freeway was in the scenario of (F1500+R1000) AND (F2000+R1000) in which the LOSs were raised from F to C. The two designed signal timing scenarios almost provide the same positive effects. As a conclusion, using ramp metering with the two designed signal timing scenarios in the geometric configuration of Type I ramp-freeway junction under the circumstance of high freeway and ramp traffic volumes was found to be beneficial, specifically when the traffic volume of the freeway is equal or greater than 1,250 vphpl and the traffic volume of the ramp is equal or greater than 800 vphpl. Using of ramp metering is not beneficial for the freeway capacity under the circumstances of low traffic volume of the freeway and/or low traffic volume of the ramp.

5.2.2 Effects of Ramp Metering on Freeway Level of Service of Type II Ramp-Freeway Junctions

Tables D.5, D.6, and D.7 in Appendix D show the VISSIM output results of the average densities at the influence area of Type II ramp-freeway junction. Table (D.8) includes the summarized results of the average densities of the ramp influence areas for the three designed signal timing scenarios. The units of the average densities in the Table D.8 are vpmpl that cannot be used for finding the freeway LOSs in the HCM’s table; therefore, the average density units were converted to pcpmpl in Table 34.

Table 33: Average density (passenger car per mile per lane) at the ramp influence area and freeway LOS of Type I ramp-freeway junction

Freeway volume (vehicle / hour lane)	Signal design	Ramp Volume (vehicles / hour lane) 400 Density	400 LOC	600 Density	600 LOC	800 Density	800 LOC	1000 Density	1000 LOC
500	No ramp meter	8.7	A	10.5	B	12.1	B	12.4	B
	2R+1AR+2G+1AR	8.7	A	10.4	B	10.5	B	10.5	B
	5R+1AR+5G+1AR	8.7	A	10.4	B	10.4	B	10.5	B
750	No ramp meter	11.4	B	13.3	B	14.9	B	15.2	B
	2R+1AR+2G+1AR	11.6	B	13.3	B	13.3	B	13.3	B
	5R+1AR+5G+1AR	11.6	B	13.2	B	13.3	B	13.3	B
1000	No ramp meter	14.5	B	16.2	B	18.1	B	18.7	B
	2R+1AR+2G+1AR	14.5	B	16.2	B	16.3	B	16.3	B
	5R+1AR+5G+1AR	14.5	B	16.3	B	16.3	B	16.3	B
1250	No ramp meter	17.8	B	20.4	C	28.8	D	30.7	D
	2R+1AR+2G+1AR	17.7	B	20.4	C	20.7	C	20.8	C
	5R+1AR+5G+1AR	17.8	B	20.4	C	20.4	C	20.8	C
1500	No ramp meter	20.3	C	26.6	C	53.7	E	90.3	F
	2R+1AR+2G+1AR	20.7	C	27.4	C	27.5	C	27.0	C
	5R+1AR+5G+1AR	20.4	C	26.7	C	26.5	C	26.5	C
1750	No ramp meter	20.5	C	26.4	C	55.1	E	89.3	F
	2R+1AR+2G+1AR	21.0	C	27.2	C	26.8	C	28.2	D
	5R+1AR+5G+1AR	21.0	C	26.4	C	26.4	C	27.2	C
2000	No ramp meter	20.5	C	26.5	C	54.3	E	97.6	F
	2R+1AR+2G+1AR	20.8	C	26.9	C	28.0	C	27.6	C
	5R+1AR+5G+1AR	21.0	C	26.9	C	27.1	C	27.1	C

Table 34: Average density (passenger car per mile per lane) at the ramp influence area and freeway LOS of Type II ramp-freeway junction

Freeway volume (vehicle / hour lane)	Signal design	Ramp volume (vehicles / hour lane)							
		400		600		800		1000	
		Density	LOC	Density	LOC	Density	LOC	Density	LOC
500	No ramp meter	10.5	B	10.8	B	10.8	B	10.9	B
	2R+1AR+2G+1AR	11.3	B	11.3	B	11.4	B	11.3	B
	5R+1AR+5G+1AR	10.5	B	11.3	B	11.3	B	11.3	B
750	No ramp meter	13.9	B	14.4	B	14.3	B	14.4	B
	2R+1AR+2G+1AR	13.9	B	14.8	B	14.8	B	14.8	B
	5R+1AR+5G+1AR	13.9	B	14.8	B	14.8	B	14.8	B
1000	No ramp meter	17.6	B	17.9	B	17.9	B	17.9	B
	2R+1AR+2G+1AR	17.6	B	18.5	B	18.6	B	18.5	B
	5R+1AR+5G+1AR	17.6	B	18.5	B	18.5	B	18.5	B
1250	No ramp meter	72.1	F	80.6	F	81.9	F	81.9	F
	2R+1AR+2G+1AR	76.1	F	96.1	F	96.1	F	97.1	F
	5R+1AR+5G+1AR	75.6	F	96.6	F	97.0	F	94.3	F
1500	No ramp meter	115.9	F	116.4	F	116.2	F	116.3	F
	2R+1AR+2G+1AR	117.0	F	117.6	F	118.0	F	117.7	F
	5R+1AR+5G+1AR	116.7	F	117.8	F	117.6	F	117.7	F
1750	No ramp meter	115.8	F	116.5	F	116.5	F	116.6	F
	2R+1AR+2G+1AR	117.0	F	117.9	F	117.7	F	117.7	F
	5R+1AR+5G+1AR	117.2	F	117.4	F	117.4	F	117.9	F
2000	No ramp meter	115.8	F	116.4	F	116.3	F	116.7	F
	2R+1AR+2G+1AR	116.7	F	117.7	F	117.7	F	117.8	F
	5R+1AR+5G+1AR	117.5	F	117.6	F	117.7	F	117.7	F

The results show that ramp metering in this type of geometric configuration is not preferred because the freeway's LOSs before using the ramp metering were the same as the freeway's LOSs after using the ramp metering. In other words, the ramp meters could not raise the LOSs for any of the designed signal timing and traffic volume scenarios. Although the ramp meters did not decline the freeway's LOSs, using of ramp metering is not recommended in the geometric configuration of the Type II ramp-freeway junction. Using of ramp metering is a disadvantageous engineering decision under this geometric configuration because of the ramp metering costs for implementation and maintenance, delay times of the ramp vehicle, and the negative effects of the ramp meters on the local streets.

5.2.3 Effects of Ramp Metering on Freeway Level of Service of Type III Ramp-Freeway Junctions

Tables D.9, D.10, D.11, and D.12 in Appendix D show the VISSIM output results of the average densities at the ramp influence area of the Type III ramp-freeway junction. The average densities at the ramp influence area under the designed scenarios are summarized in Table D.13 with units of vpmpl. Table 35 shows the summary results of average densities (after converting the units to pcpmpl) and LOSs under the case of no ramp metering and the two designed signal timing scenarios. The results of the Level of Service analysis do not coincide with the results that were obtained from the speed and travel time analyses. The highlighted cells show the traffic volume scenarios in which ramp meters could raise Level of Service of the freeway.

Despite raising the LOSs in some traffic volume scenarios, the results are not significant because the changes in the average densities were small. As an illustrative example, under the traffic volume scenario of (F500+R800), the average density that was obtained from the base case was 10.3 pcpmpl, while in the (4R+2G) signal timing scenario was 9.3 pcpmpl. As can be seen, in the (4R+2G) signal timing scenario, the change in average density was 1.0 pcpmpl; however, the LOS was raised to a higher level.

Table 35: Average density (passenger car per mile per lane) at the ramp influence area and freeway LOS of Type III ramp-freeway junction

Freeway volume (vehicle / hour lane)	Signal design	400 Density	400 LOC	600 Density	600 LOC	800 Density	800 LOC	1000 Density	1000 LOC
500	No ramp meter	8.2	A	9.3	A	10.3	B	10.5	B
	2R + 2G	8.2	A	9.3	A	10.4	B	10.7	B
	4R + 4G	8.2	A	9.3	A	10.3	B	8.2	A
	4R + 2G	8.2	A	9.2	A	9.3	A	9.3	A
750	No ramp meter	11.4	B	12.6	B	13.6	B	13.8	B
	2R + 2G	11.4	B	12.6	B	13.6	B	14.0	B
	4R + 4G	11.4	B	12.5	B	13.6	B	13.9	B
	4R + 2G	11.4	B	12.5	B	12.5	B	12.5	B
1000	No ramp meter	15.1	B	16.2	B	17.3	B	17.6	B
	2R + 2G	15.1	B	16.2	B	17.4	B	17.9	B
	4R + 4G	15.1	B	16.2	B	17.3	B	17.7	B
	4R + 2G	15.1	B	16.2	B	16.2	B	16.3	B
1250	No ramp meter	20.7	C	35.3	E	67.2	F	78.1	F
	2R + 2G	20.6	C	34.6	D	69.4	F	81.4	F
	4R + 4G	20.1	C	34.9	D	69.0	F	78.7	F
	4R + 2G	21.0	C	31.5	D	35.3	E	32.2	D
1500	No ramp meter	56.8	E	76.8	F	88.6	F	88.8	F
	2R + 2G	57.1	E	76.4	F	86.8	F	87.2	F
	4R + 4G	57.2	E	76.5	F	87.2	F	87.9	F
	4R + 2G	56.7	E	76.7	F	89.1	F	77.9	F
1750	No ramp meter	57.1	E	77.4	F	88.6	F	89.3	F
	2R + 2G	56.4	E	76.8	F	86.1	F	87.4	F
	4R + 4G	56.8	E	76.1	F	86.6	F	87.4	F
	4R + 2G	57.0	E	77.2	F	78.0	F	78.2	F
2000	No ramp meter	56.9	E	76.7	F	88.9	F	88.9	F
	2R + 2G	56.9	E	76.6	F	87.6	F	87.7	F
	4R + 4G	56.2	E	76.4	F	86.8	F	87.4	F
	4R + 2G	56.9	E	76.8	F	77.7	F	78.0	F

This improvement in LOS was obtained because 9.3 pcpmpl is located in LOS A, while 10.6 pcpmpl is located in LOS B. In some traffic volume scenarios, the average density decreased due to use of the ramp metering; however, the LOS remained at the same level. For example, in the traffic volume scenario of (F2000+R800), the average density in the ramp influence area decreased from 88.9 pcpmpl in the base case to 77.7 pcpmpl in the (4R+2G) signal timing scenario; however, the LOSs for both the base case and the (4R+2G) were F. Therefore, both density and LOS should be taken into account during the evaluation of the capacity. Based on the results obtained from the speed and travel time measures of effectiveness, it is recommend that ramp metering be used only in case of high traffic volumes on the freeway (≥ 1,250 vphpl) and high traffic volumes of the ramp (≥ 800 vphpl) under signal timing scenario of (4R+2G).

5.3 Evaluation of the Effectiveness of Ramp Metering on Safety of the Freeway

In this study, the effectiveness of ramp metering on freeway safety was evaluated by comparing the cMFs that were obtained in the base case and by using ramp metering with the designed signal timing scenarios. The overall conflicts and the types of conflicts for the five different seeds on the 3,000 ft freeway segment were obtained by analyzing VISSIM trajectory files in the SSAM software program. The conflicts were separated according to their severity by using the procedure mentioned in Section 4.5.3. The cMFs were calculated by dividing the frequency of conflicts when the ramp meters were in operation to the frequency of the conflicts when the ramp meters were not in operation. The cMFs were used as measures of safety effectiveness for the geometric configurations. The conflicts and the cMFs were obtained from running five different seeds. It was assumed that the ramp meters were advantageous for the freeway safety if the number of conflicts decreased by five percent or more. As an illustrative example of the normality test for the cMFs, a traffic volume scenario of (F2000+R1000) for Type I ramp-freeway junctions was used to test whether the cMFs were distributed normally or not. Table 36 shows the conflict frequency after running five different seeds in the base case and (2R+2G) signal timing scenario. The cMF values, which were obtained from running different seeds, are close to each other because the conflict frequencies, which were obtained from running different seeds, are close to each other too. The seed number 103 provided the smallest cMF value, which was 0.18, the seed number 47 provided the greatest cMF value that was 0.29, and the average cMF value was 0.23.

Table 36: Conflict frequencies and cMFs for each seed on the freeway of Type I ramp-freeway junction-using traffic volume scenario of (F2000+R1000)

Seeds	No ramp metering	2 R + 2 G	cMF
19	6757	1918	0.28
47	6168	1812	0.29
75	6657	1463	0.22
103	7988	1400	0.18
131	7615	1416	0.19
Average	7037	1602	0.23

The Minitab statistical software program was used to analyze the cMFs' normality test. The Darling-Anderson method, with 95 percent level of confidence, was used to test the normality of the cMFs. Figure 29 shows the normality test and descriptive statistics summary results for the cMFs under the traffic volume scenario of (F2000+R1000) and the base case. According to the test results, it cannot be said that the cMFs are not distributed normally because the p-value is equal to 0.308, which is greater than 0.05 and the following null hypnosis is not rejected:

$$\mathrm{H_0}\text{: The cMFs follow the normal distribution}$$

$$\mathrm{H_a}\text{: The cMFs do not follow the normal distribution.}$$

However, the statistical test showed that the cMFs follow the normal distribution; statistically confidence interval for the means of the cMF was not used to specify the limits of the beneficial effects of the ramp meters because we only have a few data points (5 points), which is less than the recommended 15 data points; therefore, it is hard to tell if normality exists as a practical matter. In addition, the same percentage of conflict change should be applied on all of the points to know whether the ramp meters provide positive effects or not. Therefore, the ramp meters were assumed able to improve the freeway safety if they can reduce the numbers of conflicts by five percent or more (the cMFS are equal to 0.95 or less).

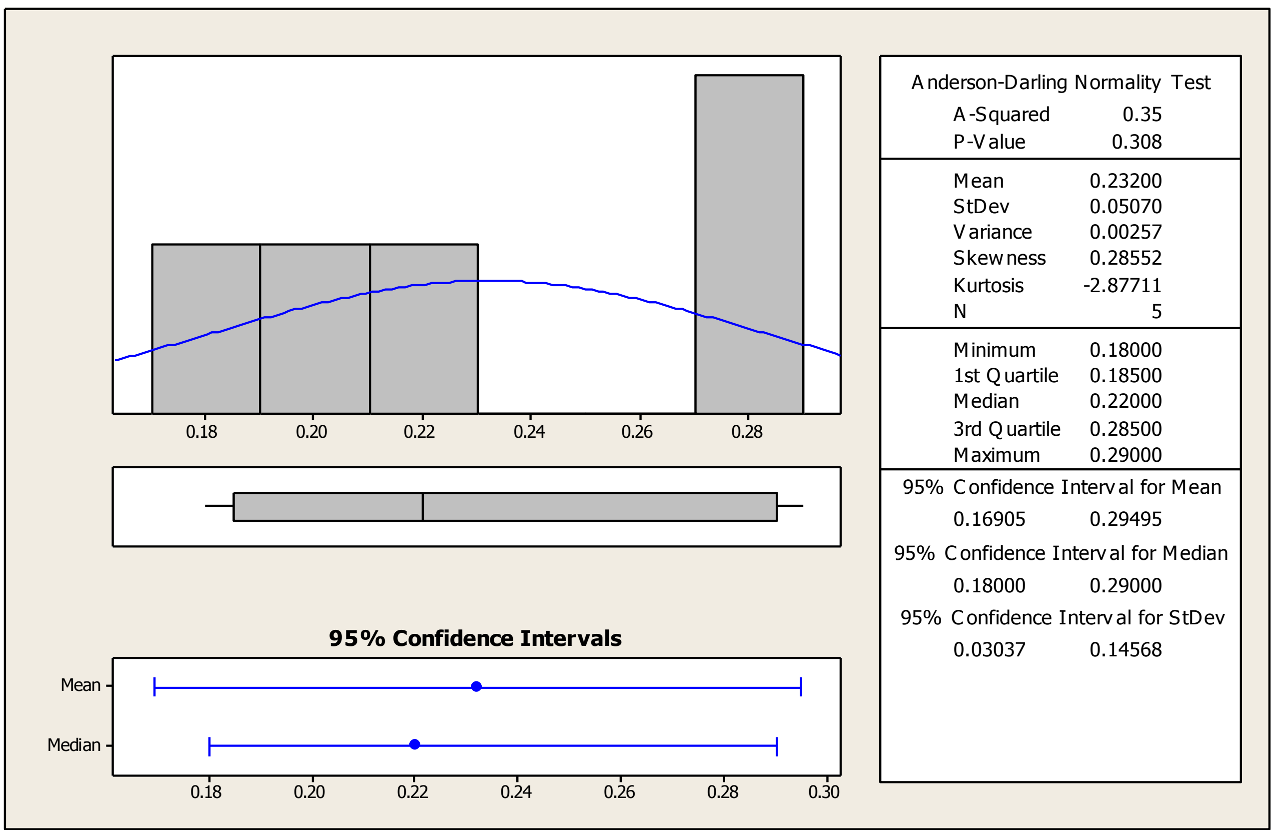


Figure 29: Summary of the normality test for the cMFs in the freeway of the Type I ramp-freeway junction-No ramp meter and traffic volume scenario of (F2000+R1000)

5.3.1 Effects of Ramp Metering on Freeway Safety of Type I Ramp-Freeway Junctions

This section illustrates the results of the effectiveness of ramp metering on freeway safety in the geometric configuration of the Type I ramp-freeway junction. Safety was evaluated by comparing the cMFs based on overall, type, and severity of traffic conflicts for both scenarios of with and without ramp metering.

5.3.1.1 The Overall cMFs at the Freeway of Type I Ramp-Freeway Junction

The results of the SSAM output of the overall conflict numbers that occurred on the freeway segment were determined by using the base case and the designed signal timing scenarios of (2R+1AR+2G+1AR), and (5R+1AR+5G+1AR) are shown in tables E.1, E.2, and E.3 in Appendix E, respectively. Table 37 shows the results of the average values of the overall cMFs of the (2R+1AR+2G+1AR) designed signal timing scenario for the assumed freeway and ramp traffic volumes. The results of the cMFs show that ramp meters significantly improved the freeway safety when the traffic volume of the freeway was equal or greater than 1,000 vphpl and the traffic volume of the ramp was equal or greater than 800 vphpl, simultaneously.

Table 37: Overall cMFs on the 3,000 ft freeway segment of Type I junction-(2R+1AR+2G+1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicle / hour. lane)	500	1.30	1.23	1.76	1.89
	750	1.07	0.95	0.95	1.19
	1000	1.06	0.98	0.38	0.26
	1250	0.71	0.98	0.32	0.28
	1500	1.13	1.09	0.40	0.22
	1750	1.10	1.09	0.36	0.25
	2000	1.15	1.08	0.43	0.23

Table 38 shows the results of the average values of the overall cMFs for the designed signal timing scenario of (5R+1AR+5G+1AR). The ramp metering with signal timing scenario of (5R+1AR+5G+1AR) provided similar results that were obtained in the signal timing scenario of (2R+1AR+2G+1AR). The signal timing scenario of (5R+1AR+5G+1AR) also showed that the ramp meters improved traffic safety of the freeway when the traffic volume of the freeway was equal or greater than 1,000 vphpl and the traffic volume of the ramp was equal or greater than 800 vphpl, simultaneously. The signal timing scenario of (5R+1AR+5G+1AR) provided smaller cMFs compared to the signal timing scenario of (2R+1AR+2G+1AR); however, the differences were slight. The results of the cMFs in the specified freeway and ramp traffic volumes show that the ramp meters provided significant positive effects on the freeway traffic safety. For example, in the traffic volume scenario of (F2000+R1000), the cMF was 0.21, which means the ramp meters decreased average traffic conflicts by five times from 7,037 to 1,474.

Table 38: Overall cMFs on 3000 feet freeway segment of Type I junction-(5R+1AR+5G+1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicle / hour lane)	500	1.60	1.59	1.64	1.58
	750	1.22	0.91	1.03	1.13
	1000	1.04	1.07	0.38	0.23
	1250	0.83	0.99	0.23	0.23
	1500	0.90	1.04	0.39	0.21
	1750	1.13	0.98	0.36	0.24
	2000	1.14	1.06	0.39	0.21

Based on the results obtained from using the designed signal timing scenarios, it is recommended to use ramp metering when the traffic volume of the freeway is equal or greater than 1,000 vphpl and the traffic volume of the ramp is equal or greater than 800 vphpl.

5.3.1.2 The cMFs According to Conflict Type for the Freeway of Type I Ramp-Freeway Junctions

The conflicts were separated into two types: lane change and rear end conflicts. The frequencies of cross-type conflicts were equal to zero in almost all of the traffic volume scenarios; therefore, the cross-type conflicts were not considered in this study. In addition, unclassified type conflicts (as discussed in Section 3.4.2) were also removed from consideration for this study.

5.3.1.2.1 The Lane Change cMFs for the Freeway of Type I Ramp-Freeway Junctions

Tables E.4, E.5, and E.6 in Appendix E show the SSAM output results of the lane change conflict numbers occurred on freeway segment of the Type I ramp-freeway junction by using no ramp metering, (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR) signal timing scenarios. Table 39 and Table 40 show the results of the average values of the lane change cMFs using signal timing scenarios of (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR). The highlighted cells indicated that the ramp meters reduced the number of lane change conflicts when the traffic volume of the freeway was equal or greater than 1,000 vphpl and the traffic volume of the ramp was equal or greater than 800 vphpl.

Table 39: Lane change cMF on a 3,000 ft freeway segment of Type I junction (2R+1AR+2G+1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicle / hour lane)	500	1.52	1.45	2.23	2.54
	750	1.09	1.24	1.1	1.37
	1000	1.12	1.08	0.91	0.71
	1250	0.95	1.18	0.44	0.44
	1500	0.99	1.23	0.60	0.49
	1750	1.12	1.17	0.48	0.49
	2000	1.05	1.13	0.62	0.49

The non-highlighted cells indicate the traffic volume scenarios in which ramp metering did not decrease the lane change conflicts on the freeway segment; therefore, it is not recommended to use ramp metering under circumstances of low traffic volume of the freeway and/or low traffic volume of the ramp, as it provides no safety improvements. As an illustrative example, in the traffic volume scenario of (F500+R800) with signal timing scenario of (5R+1AR+5G+1AR), the cMF is 2.06, which indicates that the ramp meters increased the number of the lane change conflicts by more than two times.

Table 40: Lane change cMFs on a 3,000 ft freeway segment of Type I junction- (5R+1AR+5G+1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	1.80	1.93	2.06	1.96
	750	1.25	1.07	1.15	1.43
	1000	1.03	1.08	0.94	0.60
	1250	0.98	1.01	0.40	0.39
	1500	0.94	1.13	0.53	0.44
	1750	1.16	1.17	0.46	0.50
	2000	1.04	1.18	0.57	0.40

5.3.1.2.2 The Rear End cMFs for the Freeway Type I Ramp-Freeway Junctions

Tables E.7, E.8, and E.9 in Appendix E show SSAM output results of the rear end type conflict numbers, which occurred on freeway segment of the Type I ramp-freeway junction. Table 41 shows the results of the average values of the rear end type cMFs obtained by using the signal timing scenario of (2R+1AR+2G+1AR). The results show that the ramp meters decreased the numbers of the rear end conflicts when the traffic volume of the ramp was equal or greater than 800 vphpl regardless of the traffic volume of the freeway. The ramp meters also decreased the number of the rear end conflicts when the traffic volume of the freeway was equal or less than 1,000 vphpl and traffic volume of the ramp was 600 vphpl. The highlighted cells in Table 41 show that the ramp meters improved the safety of the freeway regarding rear end conflicts. In other words, the ramp meters decreased the rear end conflicts by 5 percent or more. As a conclusion, it is recommended to use ramp metering in the highlighted scenarios for those freeway segments where the ratio of the rear end collision is high.

Table 41: Rear end cMFs on a 3,000 ft freeway segment of Type I junction-(2R+1AR+2G+1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	0.20	0.60	0.71	0.50
	750	1.00	0.38	0.59	0.75
	1000	0.96	0.91	0.24	0.16
	1250	0.61	0.95	0.31	0.27
	1500	1.15	1.08	0.39	0.21
	1750	1.10	1.08	0.35	0.24
	2000	1.16	1.08	0.42	0.22

Table 42 shows the results of the average values of the rear end cMFs obtained by using the signal timing scenario of (5R+1AR+5G+1AR). The results show that ramp meters decreased the rear end conflicts on the freeway segment when traffic volume of the ramp was equal or greater than 800 vphpl regardless of the traffic volume of the freeway. The ramp meters also improved rear end safety on five other scenarios as shown in the highlighted cells in Table 42. Based on the results of the cMFs, it is recommended to use ramp meters in the highlighted scenarios for those segments of the freeways where a high ratio of the rear end traffic collisions have been recorded.

Table 42: Rear end cMFs for a 3,000 ft freeway segment of Type I junction-(5R+1AR+5G+1AR)

		Ramp Volume (vehicles / hour lane)			
		400	600	800	1000
Freeway Volume (vehicles / hour lane)	500	0.60	0.60	0.71	0.75
	750	1.08	0.59	0.78	0.40
	1000	1.06	1.06	0.22	0.15
	1250	0.77	0.99	0.21	0.22
	1500	0.90	1.03	0.38	0.20
	1750	1.12	0.97	0.35	0.23
	2000	1.15	1.05	0.38	0.20

5.3.1.3 The cMFs According to Conflict Severity of Freeway of Type I Ramp-Freeway Junctions

This section includes the results of the conflicts and cMFs according to severity types that occurred on the freeway segment of Type I ramp-freeway junction. The numbers of the potential, slight, and serious conflicts by using base case and signal timing scenarios of (2R+1AR+2G+1AR) and (2R+1AR+2G+1AR) are shown in Tables E.10, E.11, and E.12 in Appendix E, respectively. The potential, slight, and serious conflicts in the tables were converted to Equivalent Potential Conflict (EPC) numbers according to the Kansas, Massachusetts, and Virginia models as shown in the Tables E.13 through E.21 in Appendix E. The cMFs were calculated by dividing the EPCs when the ramp meters were not in operation by the EPCs when ramp meters were in operation. The cMF values of the EPCs of the different models are similar to each other; however, the Massachusetts model provided the median EPC values among the three models; therefore, only the Massachusetts model was used in the geometric configuration of the Type I ramp-freeway junction. Tables E.22 through E.25 in Appendix E show the results of the cMFs that were obtained by the Kansas and Virginia models, respectively, by using the signal timing scenarios of (2R+1AR+2G+1AR) and (2R+1AR+2G+1AR). Table 43 shows the results of the average values of the cMFs obtained by using the Massachusetts model under the signal timing scenario of (2R+1AR+2G+1AR). The highlighted cells show the traffic volume scenarios in which the ramp meters decreased the number of EPCs regarding the severity of the conflicts. In the light of the results, ramp meters improved safety of the freeway regarding the severity of the conflicts when the traffic volume of the ramp was equal or more than 800 vphpl and the traffic volume of the freeway was equal or greater than 1,000 vphpl, simultaneously.

Table 43: cMFs for EPC in the freeway of Type I junction-(2R+1AR+2G+1AR) - Massachusetts model = 10F + 5I + 1PDO

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	1.24	1.38	1.89	1.75
	750	1.08	0.98	0.86	1.69
	1000	1.06	1.03	0.49	0.34
	1250	0.77	0.99	0.34	0.31
	1500	1.12	1.08	0.44	0.25
	1750	1.09	1.09	0.38	0.28
	2000	1.17	1.10	0.46	0.25

Table 44 shows the results of the average values of the cMFs obtained by using the Massachusetts model under the signal timing scenario of (5R+1AR+5G+1AR). The values of the cMFs in the (5R+1AR+5G+1AR) signal timing scenario are smaller than the values of the cMFs that were obtained in the (2R+1AR+2G+1AR) signal timing scenario; however, the differences were small. Although the values of the cMFs were different when the Kansas and Virginia models were used, areas where beneficial cMFs were observed were the same for all three models- namely when ramp traffic volumes were 800 vphpl or above and when freeway traffic volumes were 1,000 vphpl or above. This can be seen in Tables E.22 through E.25 in Appendix E.

Table 44: cMFs for EPC in the freeway of Type I junction-(5R+1AR+5G+1AR) - Massachusetts model = 10F + 5I + 1PDO

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	1.51	1.64	1.73	1.45
	750	1.31	1.04	1.02	1.20
	1000	1.02	1.00	0.47	0.30
	1250	0.87	0.98	0.25	0.25
	1500	0.89	1.04	0.42	0.24
	1750	1.13	0.98	0.38	0.26
	2000	1.13	1.07	0.42	0.23

5.3.2 Effects of Ramp Metering on Freeway Safety of Type II Ramp-Freeway Junctions

The effectiveness of ramp metering on freeway safety of Type II ramp-freeway junction is explained in this section. The freeway safety was evaluated based on the cMFs obtained from the overall, type, and severity of conflicts that occurred on the 3,000 ft freeway segment near the ramp junction. The cMFs were calculated by dividing the conflicts that occurred when the ramp meters on to the conflicts that were occurred when the ramp meters off. The conflicts were obtained by analyzing five different seeds of the VISSIM trajectory files in the SSAM software program.

5.3.2.1 The Overall cMFs at the Freeway of Type II Ramp-Freeway Junctions

Tables E.26, E.27, and E.28 in Appendix E show the SSAM output results of the overall numbers of conflicts that occurred on the freeway segment of Type II ramp-freeway junction under the base case and the designed signal timing scenarios of (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR). Table 45 shows the average values of the cMFs that were obtained from using ramp meters with the designed signal timing scenario of (2R+1AR+2G+1AR). The table shows that the ramp meters did not provide improvements regarding safety in the assumed volume scenarios because almost all of the cMFs are greater than one.

Table 45: Overall cMFs on a 3,000 ft freeway segment of Type II junction-(2R+1AR+2G+1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	1.28	1.13	1.09	0.82
	750	1.28	1.48	1.00	1.00
	1000	1.82	1.53	1.59	1.08
	1250	1.08	1.24	1.21	1.25
	1500	1.01	1.00	1.00	1.02
	1750	1.02	1.03	1.02	1.02
	2000	0.99	1.02	1.02	1.02

Table 46 shows the results of the average values of the cMFs obtained by using ramp meters under the designed signal timing scenario of (5R+1AR+5G+1AR). The ramp meters decreased the number of the overall conflicts only in three traffic volume scenarios as indicated in the highlighted cells, while in the other traffic volume scenarios the ramp meters adversely affected the safety of the freeway by increasing the number of the overall conflicts. In the light of the results, it is recommended to use ramp meters in the geometric configuration of Type II ramp-freeway junction.

Table 46: Overall cMFs on a 3,000 ft freeway segment of Type II junction-(5R+1AR+5G+1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	1.17	1.17	1.15	0.91
	750	1.30	1.14	0.98	0.92
	1000	1.59	0.93	1.03	1.04
	1250	1.06	1.25	1.24	1.21
	1500	1.02	1.00	1.02	1.02
	1750	1.01	1.01	1.02	1.01
	2000	1.00	1.02	1.01	1.02

5.3.2.2 The cMFs According to Conflict Type for the Freeway of Type II Ramp-Freeway Junctions

The overall number of conflicts that occurred on the freeway segment of Type II ramp-freeway junction affected the lane change and rear end conflict types because the same number of the overall conflicts was divided into lane change and rear end conflicts. This section explains the number of conflicts and cMFs according to the type of conflicts. Even though, it was not recommended to use ramp meters in the geometric configuration of Type II ramp-freeway junction.

5.3.2.2.1 The Lane Change cMFs for Freeway of Type II Ramp-Freeway Junctions

Tables E.29, E.30, and E.31 in Appendix E show the SSAM output results of the lane-change conflict numbers, which occurred on freeway segment of Type II ramp-freeway junction. The average values of the conflicts were taken by calculating cMFs under base case, and the designed signal timing scenarios of (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR). Table 47 and Table 48 show the results of the average values of the lane change cMFs for the designed signal timing scenarios of (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR), respectively.

Table 47: Lane change cMFs on a 3,000 ft freeway segment of Type II junction-(2R+1AR+2G+1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	1.21	1.14	1.07	0.83
	750	1.29	1.19	0.98	1.07
	1000	0.97	1.09	1.40	0.88
	1250	1.09	1.11	1.17	1.22
	1500	1.05	0.97	0.98	1.01
	1750	0.96	1.11	0.99	0.98
	2000	1.03	1.10	1.03	1.13

Table 48: Lane change cMFs on a 3000 ft freeway segment of Type II junction (5R+1AR+5G+1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	2.00	1.00	2.00	1.00
	750	2.20	8.00	0.33	1.00
	1000	2.23	0.92	1.07	1.10
	1250	1.05	1.26	1.24	1.21
	1500	1.01	1.01	1.01	1.02
	1750	1.02	1.01	1.02	1.01
	2000	1.00	1.02	1.01	1.02

Each of the tables includes two different highlighted traffic volume scenarios in which the ramp meters decreased the numbers of lane change conflicts. The ramp meters in the non-highlighted traffic volume scenarios did not provide any improvement of the freeway safety regarding the lane change conflicts; even in some of the traffic volume scenarios, the ramp meters provide adverse safety impacts. In reality, it is not practical to use ramp meters only under one or two specific traffic volume scenarios of freeway and ramp. Therefore, ramp meters are not recommended for use as an intelligent transportation system device to decrease the numbers of lane change conflicts or collisions for this type of geometric configuration.

5.3.2.2.2 The Rear End cMFs for the Freeway Type II Ramp-Freeway Junctions

Tables E.32, E.33, and E.34 in Appendix E show the SSAM output results of rear end conflicts that occurred on freeway segment of Type II ramp-freeway junction under the circumstances of base case and the two designed signal timing scenarios of (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR), respectively. Table 49 shows the results of the average values of the rear end cMFs that were obtained by using the signal timing scenario of (2R+1AR+2G+1AR). The results of Table 49 show that the ramp meters decreased the number of rear end conflicts when the traffic volume of the ramp was equal to 1,000 vphpl and the traffic volume of the freeway was equal to or less than 750 vphpl.

Table 49: Rear end cMFs on a 3,000 ft freeway segment of Type II junction-(2R+1AR+2G +1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	3.00	1.00	1.50	0.50
	750	1.20	14.00	1.11	0.50
	1000	3.00	1.81	1.68	1.23
	1250	1.08	1.25	1.21	1.25
	1500	1.01	1.01	1.00	1.02
	1750	1.02	1.03	1.02	1.02
	2000	0.98	1.02	1.02	1.02

Table 50 shows the results of the average values of the rear end cMFs under the signal timing scenario of (5R+1AR+5G+1AR). The results of Table 50 show that the ramp meters decreased the rear end conflicts in two traffic volume scenarios as shown in the highlighted cells. As mentioned before, it is not practical to use ramp meters only in two specific traffic volume scenarios; therefore, it is not recommended to use ramp metering in the geometric configuration of Type II ramp-freeway junction.

Table 50: Rear end cMFs on a 3,000 ft freeway segment of Type II junction-(5R+1AR+5G+1AR)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	2.00	1.00	2.00	1.00
	750	2.20	8.00	0.33	1.00
	1000	2.23	0.92	1.07	1.10
	1250	1.05	1.26	1.24	1.21
	1500	1.01	1.01	1.01	1.02
	1750	1.02	1.01	1.02	1.01
	2000	1.00	1.02	1.01	1.02

5.3.2.3 The cMFs According to Conflict Severity of Freeway of Type II Ramp-Freeway Junctions

Tables E.35, E.36, and E.37 in Appendix E show the potential, slight, and serious conflicts that occurred on the freeway segment of a Type II ramp-freeway junction. The slight and serious conflicts corresponded to potential conflicts are expressed as the EPC values. Tables E.38 through E.46 show the results of the EPC values for the base case, and the signal timing scenarios of (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR) using Kansas, Massachusetts, and Virginia EPC models. cMFs of the EPC were obtained by diving the values of the EPC numbers without using ramp metering to the values of the EPC numbers with using ramp metering. The Kansas model provided the lowest cMF values of the EPC, while the Virginia model provided the highest values. The values of the cMFs obtained in the Massachusetts model was taken as the criteria to evaluate safety regarding the severity of the conflicts, because the Massachusetts model provided the median values of the cMFs among the three models. Tables E.47 through E.50 in Appendix E show the cMF values of the EPC for both (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR) signal timing scenarios using the Kansas and Virginia models, respectively. Table 51 shows the results of the average values of the cMFs of the EPC under using the (2R+1AR+2G+1AR) signal timing scenario by using the Massachusetts model. The results in the table show only one traffic volume scenario in which the ramp meters decreased the EPC value. The results of other traffic volume scenarios showed that the presence of ramp meters did not decrease the EPC values in the freeway segment on the Type II ramp-freeway segment using the signal timing scenario of (2R+1AR+2G+1AR).

Table 51: cMFs for EPC in the freeway of Type II junction (2R+1AR+2G+1AR) - Massachusetts model = 10F + 5I + 1PDO

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	1.15	1.04	1.00	0.73
	750	1.33	1.12	0.96	0.95
	1000	1.54	1.33	1.57	1.01
	1250	1.09	1.21	1.25	1.25
	1500	1.02	1.01	0.98	1.01
	1750	1.04	1.04	1.00	1.02
	2000	1.02	1.05	1.02	1.02

Table 52 shows the results of the (5R+1AR+5G+1AR) signal timing scenario in which the ramp meters decreased the EPC values in the traffic volume scenarios as indicated in the highlighted cells. The highlighted traffic volume scenarios did not cluster around the specific traffic volume limits of the freeway or the ramp, which means that using of ramp metering cannot be considered as a potential safety improvement for ramp-freeway junctions of this geometric configuration. Based on the results that were obtained by using the ramp metering and both

(2R+1AR+2G+1AR) and (5R+1AR+5G+1AR) signal timing scenarios, the ramp meters did not provide enough improvements for the safety regarding the severity of conflicts; in other words, the ramp meters did not decrease the EPC values in most of the designed traffic volume scenarios. Therefore, it is not recommended to use ramp metering in the geometric configuration of Type II ramp-freeway junction.

Table 52: cMFs for EPC in the freeway of Type II junction (5R+1AR+5G+1AR) - Massachusetts model = 10F + 5I + 1PDO

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	1.03	1.17	1.21	0.81
	750	1.21	0.95	0.94	0.89
	1000	1.42	0.93	1.08	0.98
	1250	1.08	1.26	1.23	1.21
	1500	1.02	1.00	1.00	1.03
	1750	1.04	1.03	1.01	1.02
	2000	1.03	1.03	0.99	1.03

5.3.3 Effects of Ramp Metering on Freeway Safety of Type III Ramp-Freeway Junctions

The evaluation of the effectiveness of ramp metering on freeway safety of Type III ramp-freeway junction are explained in this section based on overall, types, and severity of the cMFs. Three different signal timing scenarios were used in the ramp meters: 2 seconds red with 2 seconds green (2R+2G), 4 seconds red with 4 seconds green (4R+4G), and 4 seconds red with 2 seconds green (4R+2G). The all-red timing intervals were not used in the designed signal scenarios because the ramp in this geometric configuration had only one lane.

5.3.3.1 The Overall cMFs at the Freeway of Type III Ramp-Freeway Junctions

Tables E.51, E.52, E.53, and E.54 in Appendix E show the SSAM output results of the overall conflicts for the base case, and the signal timing scenarios of (2R+2G), (4R+4G), and (2R+4G), respectively. The average numbers of conflicts, which occurred on a freeway segment of Type III ramp-freeway junction, were used to calculate the EPC values. Table 53 shows the average values of the overall cMFs for the (2R+2G) signal timing scenario, which indicate that ramp meters decreased the number of overall conflicts when the ramp traffic volume was equal or less than 800 vphpl and the freeway traffic volume was equal or less than 1000 vphpl, simultaneously. This result shows that the ramp metering is able to improve traffic safety only in low-volume condition of the ramp and the freeway. However, the ramp meters with (2R+2G) signal timing and low traffic volume scenarios did not increase efficiency on the freeway, but they appear to improve the safety of the freeway under certain condition. Therefore, it is recommended to use ramp metering with the signal timing scenario of (2R+2G) only if a high traffic crash ratio was recorded during low traffic volume of the freeway and the ramp.

Table 53: Overall cMFs on a 3,000 ft freeway segment of Type III junction (2R+2G)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	0.89	0.96	0.84	1.14
	750	0.92	0.83	0.66	1.02
	1000	0.88	0.87	0.82	1.00
	1250	1.00	1.01	1.06	1.12
	1500	0.99	0.96	1.00	1.01
	1750	1.00	1.00	0.98	0.98
	2000	0.99	0.99	0.97	1.02

Table 54 shows the results of the average values of the overall cMFs when using the signal timing scenario of (4R+4G). The table indicates that ramp meters decreased the number of overall conflicts as shown in the highlighted cells. Most of the highlighted cells are located in the column in which traffic volume of the ramp was 800 vphpl. It is not appropriate to use ramp metering when the traffic volume of the ramp is only equal to 800 vphpl and the freeway is equal or less than 1000 vphpl because typically ramp meters are deactivated during low-volume situations. Moreover, the ramp meters in this signal scenario did not provide any positive effects regarding the efficiency and Level of Service; therefore, it is not recommended to use ramp metering with the signal timing scenario of (4R+4G) for this geometric configuration.

Table 54: Overall cMFs on a 3,000 ft freeway segment of Type III junction (4R+4G)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	0.59	0.98	0.73	1.16
	750	1.00	0.89	0.91	1.09
	1000	1.00	1.32	0.88	0.86
	1250	0.95	0.98	1.06	1.07
	1500	0.97	0.99	1.02	1.02
	1750	1.00	0.99	0.99	0.98
	2000	0.96	0.98	0.98	1.01

Table 55 shows the results of the average values of cMFs when using the signal timing scenario of (4R+2G), which indicates that ramp meters improved freeway safety when the ramp traffic volume was equal or greater than 800 vphpl and the freeway traffic volume was equal to or greater than 750 vphpl.

Table 55: Overall cMFs on a 3,000 ft freeway segment of Type III junction (4R+2G)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	0.84	1.12	1.14	1.46
	750	1.12	1.06	0.74	0.78
	1000	0.98	1.30	0.60	0.66
	1250	1.13	0.89	0.45	0.42
	1500	1.01	1.02	0.94	0.92
	1750	1.02	1.04	0.90	0.88
	2000	1.00	1.06	0.89	0.93

In the light of the cMF results under the circumstances of using of the (2R+2G) and (4R+2G) signal timing scenarios, ramp meters can be beneficial for improving freeway safety with different signal timing scenarios for different ramp and freeway volumes. The (2R+2G) signal timing scenario is able to improve freeway safety during low traffic volume of the freeway (≤1000 vphpl) and the ramp (≤800 vphpl); while the (4R+2G) signal timing scenario is able to improve freeway safety in medium to high traffic volume of the freeway (≥750 ramp) and high traffic volume of the ramp (≥800 vphpl). As a result, it is recommended that ramp metering be used at the geometric configuration of Type III ramp-freeway junction with two signal timing scenarios of (2R+2G), and (4R+2G) depending on the traffic volume of the ramp and the freeway.

5.3.3.2 The cMFs According to Conflict Type for the Freeway of Type III Junctions

This section includes the results of the cMFs based on the type of conflicts that occurred on a freeway segment of Type III ramp-freeway junction. The conflicts were divided into lane change and rear end type conflicts. Cross conflicts were not considered because the numbers of the cross conflicts were almost equal to zero; therefore, the cMFs were classified into lane change and rear end types of cMFs.

5.3.3.2.1 The Lane Change cMFs for the Freeway of Type III Ramp-Freeway Junctions

Tables E.54 through E.58 in Appendix E show the SSAM output results of the lane change conflict numbers under the base case, and the signal timing scenarios of (2R+2G), (4R+4G), and (4R+2G). Table 56 and Table 57 show the average values of the lane change cMFs using ramp meters with the signal timing scenarios of (2R+2G) and (4R+4G), respectively. The tables indicate that ramp meters decreased the number of lane change conflicts in some scattered traffic volume scenarios of the ramp and the freeway as indicated in the highlighted cells. Because the highlighted cells are spread throughout the table and did not cluster in any specific traffic volumes of the freeway or the ramp, it is not recommended that ramp meters could be used with the signal timing scenarios of (2R+2G) and (4R+4G) for the locations where a high ratio of lane change conflicts were recorded. In addition, due to the low traffic volume of the freeway, the overall modelled conflict numbers are small. For example, in case of the scenario of (2R+2G) signal time and (F750 + R600) traffic volume, the average number of lane change conflicts is one for the base case and it is 0.2 for the (2R+2G) as shown in Tables E.54 and E.55. Therefore, it is not recommended to use ramp metering for either of the signal timing scenarios of (2R+2G) and (4R+4G) in the geometric configuration of Type III ramp-freeway junction.

Table 56: Lane change cMFs on a 3,000 ft freeway segment of Type III junction (2R+2G)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / lane hour)	500	2.00	1.00	3.50	2.00
	750	1.67	0.20	1.25	1.00
	1000	1.24	1.11	1.65	0.79
	1250	0.99	0.94	1.04	1.19
	1500	0.97	0.91	1.02	1.01
	1750	1.12	0.98	0.93	1.01
	2000	0.86	0.95	0.89	1.03

Table 57: Lane change cMFs on a 3,000 ft freeway segment of Type III junction (4R+4G)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	1.00	5.00	1.50	3.00
	750	0.33	0.80	1.50	2.00
	1000	0.94	1.63	1.47	0.91
	1250	1.21	0.89	1.09	1.08
	1500	1.07	0.94	0.99	0.98
	1750	1.02	0.95	0.99	0.97
	2000	0.86	0.87	0.94	1.02

Table 58 shows the results of the average values of the lane change cMFs under the circumstance of using the signal timing scenario of (4R+2G). The ramp meters with (4R+2G) signal timing scenario provided better results of the lane change cMFs than the (2R+2G) and (4R+4G) signal timing scenarios. The highlighted cells in Table 58 indicate that the ramp meters decreased the numbers of the lane change conflicts when the traffic volume of the ramp was equal or greater than 800 vphpl regardless of the traffic volume of the freeway.

Table 58: Lane change cMFs on a 3,000 ft freeway segment of Type III junction-(4R+2G)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	1.00	3.00	0.50	1.00
	750	1.00	1.20	0.75	1.00
	1000	1.24	1.00	0.94	0.65
	1250	1.05	0.91	0.37	0.37
	1500	1.17	1.14	0.88	0.84
	1750	1.19	1.15	0.83	0.83
	2000	1.12	1.18	0.83	0.91

Conclusively, for the freeway segments with a high ratio of lane change collisions, it is recommended to use ramp meters with the signal timing scenario of (4R+2G) when the traffic volume of the ramp is equal or greater than 800 vphpl.

5.3.3.2.2 The Rear End cMFs for the Freeway Type III Ramp-Freeway Junctions

Tables E.58 through E.61 in Appendix E show the SSAM output results of the rear end conflicts, which occurred on the freeway segment in the Type III ramp-freeway junction under the circumstance of using the signal timing scenario of (2R+2G). Table 59 shows the result of the average values of the cMFs under using signal timing scenario of (2R+2G). The table shows that ramp meters decreased the number of the rear end conflicts when the traffic volume of the freeway was low (1,000 vphpl or less) and the traffic volume on the ramp was equal or less than 800 vphpl. As a result, it is recommended to use ramp metering with the signal scenario of (2R+2G) for those freeway segments that have high rate of rear end collision in the low freeway traffic volume and medium to high ratio of ramp traffic volume.

Table 59: Rear end cMFs on a 3,000 ft freeway in Type III ramp-freeway junction (2R+2G)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	0.86	0.96	0.76	1.12
	750	0.87	0.87	0.63	1.02
	1000	0.79	0.82	0.74	1.03
	1250	1.00	1.01	1.06	1.11
	1500	0.99	0.96	1.00	1.01
	1750	1.00	1.00	0.98	0.98
	2000	0.99	0.99	0.98	1.01

Table 60 shows the average values of the cMFs under the circumstance of using the signal timing scenario of (4R+4G), which illustrates that ramp metering could decrease the rear end conflicts in some of the assumed traffic volumes scenarios as shown in the highlighted cells. Because the values of cMFs in the table do not cluster in specific limits of the traffic volumes of the ramp and the freeway, using ramp metering with this signal timing scenario does not appear to be a practical and or reliably way to reduce rear end crashes. As a result, it is not recommended to use ramp metering with signal timing scenario of (4R+4G).

Table 60: Rear end cMFs on a 3,000 ft freeway in Type III ramp-freeway junction (4R+4G)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	0.58	0.90	0.71	1.12
	750	1.04	0.90	0.88	1.04
	1000	1.01	1.26	0.82	0.86
	1250	0.93	0.98	1.06	1.07
	1500	0.97	1.00	1.02	1.03
	1750	1.00	0.99	0.99	0.98
	2000	0.96	0.98	0.98	1.01

Table 61 shows the average values of the cMFs under the signal timing scenario of (4R+2G). The results show that ramp meters decreased the rear end conflicts when the traffic volume of the ramp was high (800 vphpl and more) and the traffic volume of the freeway was equal or greater than 750 vphpl. As a result, it is recommended to use ramp meters with a signal scenario of (4R+2G) for those freeway segments that have high ratio of rear end collisions and a high ramp traffic volume (800 vphpl and more).

Table 61: Rear end cMFs on a 3,000 ft in Type III ramp-freeway junction (4R + 2G)

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	0.84	1.08	1.16	1.47
	750	1.13	1.05	0.74	0.77
	1000	0.92	1.37	0.57	0.66
	1250	1.14	0.88	0.45	0.42
	1500	1.00	1.02	0.94	0.92
	1750	1.02	1.03	0.91	0.89
	2000	0.99	1.05	0.89	0.93

5.3.3.3 The cMFs According to Conflict Severity of Freeway of Type III Ramp-Freeway Junctions

Tables E.61 through E.65 in Appendix E show the results of the potential, slight, and serious conflicts that were modeled for the freeway segment of Type III ramp-freeway junction by using the base case, and the signal timing scenarios of (2R+2G), (4R+4G), and (4R+2G). Tables E.66 through E.77 in Appendix E show the EPC values for the designed signal timing and traffic volumes scenarios by using the Kansas, Massachusetts, and Virginia EPC models. Tables E.78, E.79, and E.80 show the results of the average values of the cMFs that were calculated from signal timing scenarios of (2R+2G), (4R+4G), and (4R+2G), respectively, by using the Kansas models. Tables E.81, E.82, and E.83 show the results of the average values of the cMF for the same previous signal timing scenarios but by using the Virginia model. The average values of the cMFs based on the Massachusetts model was taken for the safety evaluation because the Massachusetts model provided median values of the cMFs among the three model. Table 62 and

Table 63 show the results of the average values of the cMFs for the EPC by using the Massachusetts model and the signal timing scenarios of (2R+2G) and (4R+4G). The results of the cMFs in the tables show that the ramp meters reduced the EPC values on the freeway segment for some traffic volume scenarios as indicated in the highlighted cells. The highlighted cells are scattered through the traffic volume scenarios of the ramp and the freeway, which indicates that the use of ramp metering with these signal scenarios would be impractical as a crash-reduction tool. Based on the results of Table 62 and

Table 63, it is not recommended to use ramp metering with the signal scenarios of (2R+2G) and (4R+4G) at Type III ramp-freeway junction geometric configuration solely to reduce crashes.

Table 62: cMFs for EPC in the freeway of Type III ramp-freeway junction (2R+2G) - Massachusetts model = 10F + 5I + 1PDO

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	0.96	1.06	1.09	1.16
	750	0.98	0.93	0.81	1.11
	1000	0.83	0.97	0.95	0.95
	1250	1.01	1.00	1.03	1.12
	1500	0.99	0.95	1.00	1.00
	1750	1.01	0.98	0.45	0.96
	2000	0.98	0.98	0.96	1.00

Table 63: cMFs for EPC in the freeway of Type III junction (4R+4G)-Massachusetts model = 10F + 5I + 1PDO

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	0.62	0.92	0.90	1.10
	750	1.01	0.89	0.98	1.19
	1000	1.05	1.25	0.95	0.91
	1250	0.97	0.97	1.04	1.07
	1500	0.98	0.98	1.02	1.01
	1750	0.99	0.98	0.46	0.97
	2000	0.97	0.97	0.96	1.00

Table 64 shows the results of the average values of the cMF for EPC under the circumstance of using the signal timing scenario (4R+2G) and the Massachusetts model. The ramp meters decreased the EPC values when traffic volume of the ramp was high (800 vphpl and more) and the traffic volume of the freeway was equal or greater than 750 vphpl.

Table 64: cMFs for EPC in the freeway of Type III junction (4R+2G)-Massachusetts model = 10F + 5I + 1PDO

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	0.84	1.13	1.22	1.33
	750	1.06	1.13	0.95	0.98
	1000	0.95	1.44	0.80	0.80
	1250	1.16	0.89	0.45	0.42
	1500	1.01	1.02	0.92	0.88
	1750	1.02	1.03	0.42	0.86
	2000	1.01	1.05	0.87	0.91

Based on the cMF values for the EPCs in the three previous signal timing scenarios, the signal timing scenario (4R+2G) provided the best result for improving safety or for decreasing the EPC values. Therefore, it is recommended to use ramp metering on the geometric configuration of Type III ramp-freeway junction with the signal timing scenario of (4R+2G) when the traffic volume of the ramp is high (≥800 vphpl) and the traffic volume of the freeway is medium to high (≥750 vphpl).

5.4 Evaluation of the Effectiveness of Ramp Metering on Local Streets Upstream of the Ramp

In this study, the signal timing scenarios of the ramp meters were designed based on the average queue length that formed in the right and left lanes of the ramp, assuming the ramp is a multi-lane ramp. The average value of the maximum queue lengths that formed during every minute during the peak hour period were modeled and evaluated. The maximum queue of the vehicles were measured by using VISSIM from the stop line in front of the ramp meters to the local road intersection upstream from the ramp. The predicted effectiveness of the ramp meters on the local streets was used to evaluate the signal timing scenarios in the three geometric configurations. Average values of the maximum queues for each of the assumed ramp volume scenarios were taken for the five different seeds of model runs in VISSIM. Table 65 shows the results of the average maximum queue lengths at the ramp of Type I junction. The average of the maximum queue lengths was compared to the length of the ramp behind the ramp meters that were modeled to be 715 ft in length. The queue was assumed to reach the local street if the average of the maximum queues was greater than 715 ft. The average of the maximum queue lengths in all traffic volume scenarios was less than 715 ft except for the traffic volume scenario of (F2000+R1000) in the signal timing scenario of (5R+1AR+5G+1AR) which was 722.1 ft. Based on the results, the two designed signal timing scenarios for the Type I ramp-freeway junction is acceptable with respect to the effects of the ramp meters on the local streets. Increasing the red time intervals to be more than 5 seconds is not recommended because if the red time interval is increased, the average of the maximum queue lengths in the ramp affects the traffic flow of the local streets. In addition, compliance will most likely be reduced.

Table 65: Average of maximum queue (ft) beyond the ramp meters of Type I junction

Signal design	Seed	Ramp volume (vehicles / hour lane)			
		400	600	800	1000
2R + 1AR + 2G + 1AR	19	3.6	255.2	704.0	713.7
	47	5.5	455.6	717.0	727.6
	75	7.1	247.7	697.2	702.7
	103	7.3	199.5	723.6	700.6
	131	4.6	216.0	402.6	722.3
	Average	5.6	274.8	648.9	713.4
5R + 1AR + 5G + 1AR	19	9.5	272.6	720.5	725.5
	47	15.3	443.1	710.5	726.0
	75	10.9	261.1	664.4	714.7
	103	12.1	200.7	730.6	720.7
	131	11.3	237.2	687.4	723.5
	Average	11.8	282.9	702.7	722.1

Table 66 shows the average values of the maximum queue lengths formed behind the ramp meters of Type II ramp-freeway junctions. The distance from the stop line in front of the ramp meters to the local street was modeled to be 740 ft. The table indicates that the average of the maximum queue lengths reached the local street for both the signal timing scenarios of (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR) when the traffic volume on the ramp was equal to or greater than 600 vphpl. In addition, the previous efficiency, Level of Service, and safety results showed that ramp meters are not useful for this ramp-freeway junction geometric configuration. Therefore, it is not recommended to use ramp metering because of the negative efficiency and safety effects in the freeway, as well as the adverse effects of the ramp meters on the local street network.

Table 66: Average of maximum queue (ft) beyond the ramp meters of Type II junction

Signal design	Seed	Ramp Volume (vehicles / hour lane)			
		400	600	800	1000
2R+1AR+2G+1AR	19	330.8	758.1	758.3	757.9
	47	601.5	757.4	757.8	757.9
	75	383.9	757.2	757.8	757.5
	103	532.5	752.5	757.5	758.4
	131	223.9	743.4	756.7	757.4
	Average	414.5	753.7	757.6	757.8
5R+1AR+5G+1AR	19	445.7	758.6	758.3	757.8
	47	548.5	758.1	757.7	757.6
	75	481	757.5	757.9	758.4
	103	520.5	754.8	757.6	758
	131	233.4	756.3	758.6	759.2
	Average	445.82	757.06	758.02	758.2

Table 67 shows the result of the average value of the maximum queue lengths for the designed signal timing scenarios of (2R+2G), (4R+4G), and (4R+2G) for the Type III junction. The length of the Holmes Road ramp from the stop line in front of the ramp meters to the upstream of the local street was modeled to be 385 ft. Therefore, the queue of the vehicles reached the local streets if the average value of the maximum queues is greater than 385 ft. The ramp meters in the signal scenarios of (2R+2G) and (4R+4G) did not affect the local street network negatively because all of the average values of the maximum queues were less than 385 ft. When the signal timing scenario (4R+2G) was used, the queue lengths were 393.6, and 395 ft for the ramp traffic volumes of 800, and 1000 vphpl, respectively. Despite both of the average values of the queues being greater than 385 ft, they are close to 385 ft. Because the (4R+2G) signal timing scenario provided the best efficiency and safety positive effects on the freeway among the designed signal timing scenarios, this scenario can be used by eliminating the adverse effects of the ramp meters on the local street network. To eliminate the adverse effects of the ramp meters on the local street network, the distance between the ramp meters and the upstream of the local streets should be increased to 400 ft or more.

Table 67: Average of maximum queue (ft) beyond the ramp meters of Type III junction

Signal design	Seed	Ramp volume (vehicles / hour lane)			
		400	600	800	1000
2R + 2G	19	0.0	0.0	45.9	85.4
	47	0.0	0.0	8.8	74.4
	75	0.0	0.0	5.7	74.5
	103	0.0	0.0	32.2	44.5
	131	0.0	0.0	3.7	62.4
	Average	0.0	0.0	19.3	68.2
4R + 4G	19	4.1	12.7	62.3	106.4
	47	6.4	11.3	54.5	112.3
	75	6.8	14.7	49.3	119.6
	103	5.5	13.5	58.9	116.1
	131	8.2	14.9	66.1	130.4
	Average	6.2	13.4	58.2	117.0
4R + 2G	19	27.6	307.3	398.6	391.3
	47	29.9	269.3	387.1	392.9
	75	19.2	354.1	392.2	397.9
	103	36.7	286.5	394	397.6
	131	32.1	296.3	396.1	395.3
	Average	29.1	302.7	393.6	395

5.5 Sensitivity Analysis

To examine the effects of change in some of the assumptions on the results of the efficiency and safety of the ramp metering, a sensitivity analysis was conducted. Two of the assumptions were altered and used at the freeway of Type III ramp-freeway junction. The effects of changing the two assumptions were evaluated in the base case and in the signal timing scenario of (4R+2G). Traffic volume on the freeway was fixed as 1,750 vphpl representing a freeway traffic volume during peak hour period; in addition, the traffic volume on the ramp varied by using 400, 600, 800, and 1,000 vphpl. The car following headway of the vehicles in the ramp influence area and the traffic composition of the vehicles in the freeway segment were the two assumptions that were tested. The Minitab statistical program was used to test the effects of the assumptions' changes on the sensitivity analysis. Five percent was used as the level of significance ($\alpha = 0.05$) in the statistical F-test to assess the assumed null hypotheses.

5.5.1 Effects of Headway Change at the Ramp Influence Area on the Effectiveness of Ramp Metering on Efficiency and Safety of Freeway

In order to evaluate the effects of car-following headway on the effectiveness of ramp metering on efficiency and safety of freeways, five different headways at the ramp influence area were examined in the sensitivity analysis. The headways, which were used as indicators of the effects of the driver behavior on the efficiency and safety of the freeway, were 0.9, 1.0, 1.1, 1.2, and 1.3 sec. The average speeds (mph) in the ramp influence area and traffic conflicts on the 3,000 ft freeway segment were obtained before and after using ramp metering for the specified freeway and ramp traffic volumes. The percentage change of the average speed in the ramp influence area after using the ramp meters was used for the efficiency evaluation. Table E.84 and E.85 show the VISSIM output results of the average speeds at the ramp influence area after using different headways in the base case and signal timing scenario of (4R+2G). Table 68 shows the results of the percentage of average speed change at the ramp influence area after using ramp metering in different headway scenarios. The following null hypothesis was used to test the effects of the headway change on the efficiency of the freeway before and after using ramp metering. μ represents the percentage of average speed change in the ramp influence area after using ramp metering with the signal timing scenario of (4R+2G).

$$H_o: \mu_{0.9} = \mu_{1.0} = \mu_{1.1} = \mu_{1.2} = \mu_{1.3}$$

$$H_a: H_o \text{ is not correct}$$

Table 68: Percentages of average speed change at the ramp influence area of Type III ramp-freeway junction after using different headways (Freeway traffic volume 1750 vphpl) - (4R+2G)

Ramp influence area headway (sec.)	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
0.9	19	3.5	7.3	23.1	16.2
	47	2.7	0.0	15.5	22.6
	75	5.1	-1.0	21.0	16.9
	103	0.0	6.2	18.7	19.9
	131	1.1	0.5	20.0	20.8
	average	2.5	2.6	19.7	19.3
1	19	0.7	4.4	16.7	22.4
	47	-0.7	2.9	19.7	20.2
	75	1.9	4.5	4.0	18.2
	103	0.4	22.2	15.5	18.4
	131	3.0	6.0	19.5	19.8
	average	1.1	8.0	15.1	19.8
1.1	19	2.5	3.8	18.5	20.5
	47	2.1	5.9	14.8	21.8
	75	1.5	4.0	18.2	22.0
	103	-1.1	5.5	19.8	21.0
	131	0.4	4.9	22.6	18.4
	average	1.1	4.8	18.8	20.7
1.2	19	-2.8	1.9	20.1	19.4
	47	-1.7	0.9	16.7	13.4
	75	1.8	4.2	16.5	19.1
	103	1.7	3.8	18.7	18.7
	131	2.5	2.8	18.6	16.5
	average	0.3	2.7	18.1	17.4
1.3	19	3.1	8.5	18.6	17.3
	47	3.1	7.6	21.7	18.3
	75	0.7	-0.5	21.1	19.1
	103	-0.3	3.7	19.0	16.6
	131	1.4	1.8	19.6	21.1
	average	1.6	4.2	20.0	18.5
p-value		0.43	0.318	0.222	0.161

The results of the Table 68 show that all of the p-values, which were obtained in the ramp traffic volume scenarios after using different headway values, are greater than 0.05; therefore the null hypotheses is not rejected for all the ramp traffic volume scenarios. In the light of the statistical F-test results, it can be stated that there is no statistically significant difference between the percentages of the average speed change in the ramp influence area after using different car-following headways. As a result, the modeled driver behavior of the vehicles at the ramp influence area did not affect the results of ramp effectiveness on the freeway efficiency.

The same values of the car following headways in the ramp influence area were used to test the effects of the driver behavior on the ramp metering effectiveness on the safety of the freeway. Tables E.86 and E.87 in Appendix E show the SSAM output results of the conflict numbers that occurred in the freeway segment of Type III junction by using the base case and signal timing scenarios of (4R+2G). The tables show that when the car-following headway in the ramp influence area increased, the average number of conflicts in the freeway segment decreased. For example in Table E.86, under the circumstance of using the base case and the ramp traffic volume of 400 vphpl, the average number of conflicts were 4299, 2970, 1810, 975, and 499 for the headways of 0.9, 1, 1.1, 1.2, and 1.3, respectively. Table 69 shows the cMFs that were obtained after altering car-following headways by using the signal timing scenario of (4R+2G). The F- tests were done for the scenarios of the different headways at different ramp traffic volumes with a 95 percent level of significance. When the traffic volume of the ramp was 400 vphpl, the p-value was 0.54 that resulted in not rejecting the null hypothesis. When the traffic volume of the ramp was equal or greater than 600 vphpl, the p-values were smaller than 0.05; therefore, they resulted in rejecting the null hypotheses. In the light of the statistical results, driver behavior or car-following headway values in the ramp influence area affects the effectiveness of ramp metering on freeway safety when the traffic volume of the ramp is equal or greater than 600 vphpl. As an illustration, the Minitab output results of the statistical F-test and the cMFs boxplot, in which the ramp traffic volume was equal to 400 vphpl after using the ramp metering with signal timing scenario of (4R+2G), are shown in the following output and in the Figure 30.

Table 69: The cMFs on the 3000 ft freeway segment of the Type III ramp-freeway junction using different headways (Freeway traffic volume 1750 vphpl) - (4R+2G)

Ramp influence area headway (sec.)	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
0.9	19	0.97	1.00	0.95	0.96
	47	0.98	1.02	0.94	0.90
	75	0.96	1.05	0.93	0.94
	103	0.99	1.01	0.92	0.91
	131	1.00	1.03	0.92	0.96
	average	0.98	1.02	0.93	0.93
1	19	0.99	0.98	0.96	0.90
	47	0.99	1.03	0.90	0.89
	75	1.01	0.96	0.93	0.93
	103	1.05	0.96	0.95	0.95
	131	0.94	0.95	0.91	0.94
	average	1.00	0.98	0.93	0.92
1.1	19	0.92	1.03	0.95	0.85
	47	1.01	1.08	0.90	0.89
	75	1.06	1.04	0.94	0.89
	103	1.11	1.00	0.91	0.87
	131	1.01	1.04	0.83	0.91
	average	1.02	1.04	0.90	0.88
1.2	19	1.10	0.97	0.87	0.81
	47	1.10	1.05	0.96	0.90
	75	0.95	0.97	0.89	0.89
	103	0.97	0.99	0.85	0.85
	131	0.93	1.06	0.85	0.94
	average	1.01	1.01	0.88	0.88
1.3	19	0.87	1.05	0.80	0.84
	47	0.96	1.07	0.79	0.81
	75	1.12	1.01	0.90	0.85
	103	0.94	1.02	0.84	0.90
	131	0.87	1.13	0.87	0.89
	average	0.95	1.06	0.84	0.86
p-value		0.54	**0.022**	**0.006**	**0.016**

One-way ANOVA: H=0.9, H=1.0, H=1.1, H=1.2, H=1.3

```
Source  DF       SS       MS     F      P
Factor   4  0.01593  0.00398  0.80  0.540
Error   20  0.09970  0.00499
Total   24  0.11563

S = 0.07061   R-Sq = 13.78%   R-Sq(adj) = 0.00%


                          Individual 95% CIs For Mean Based on
                          Pooled StDev
Level   N    Mean   StDev  ---+---------+---------+---------+------
H=0.9   5  0.9784  0.0173       (----------*----------)
H=1.0   5  0.9951  0.0414          (----------*----------)
H=1.1   5  1.0222  0.0733              (----------*----------)
H=1.2   5  1.0105  0.0821            (----------*----------)
H=1.3   5  0.9503  0.1039  (----------*----------)
                           ---+---------+---------+---------+------
                            0.900     0.960     1.020     1.080

Pooled StDev = 0.0706
```

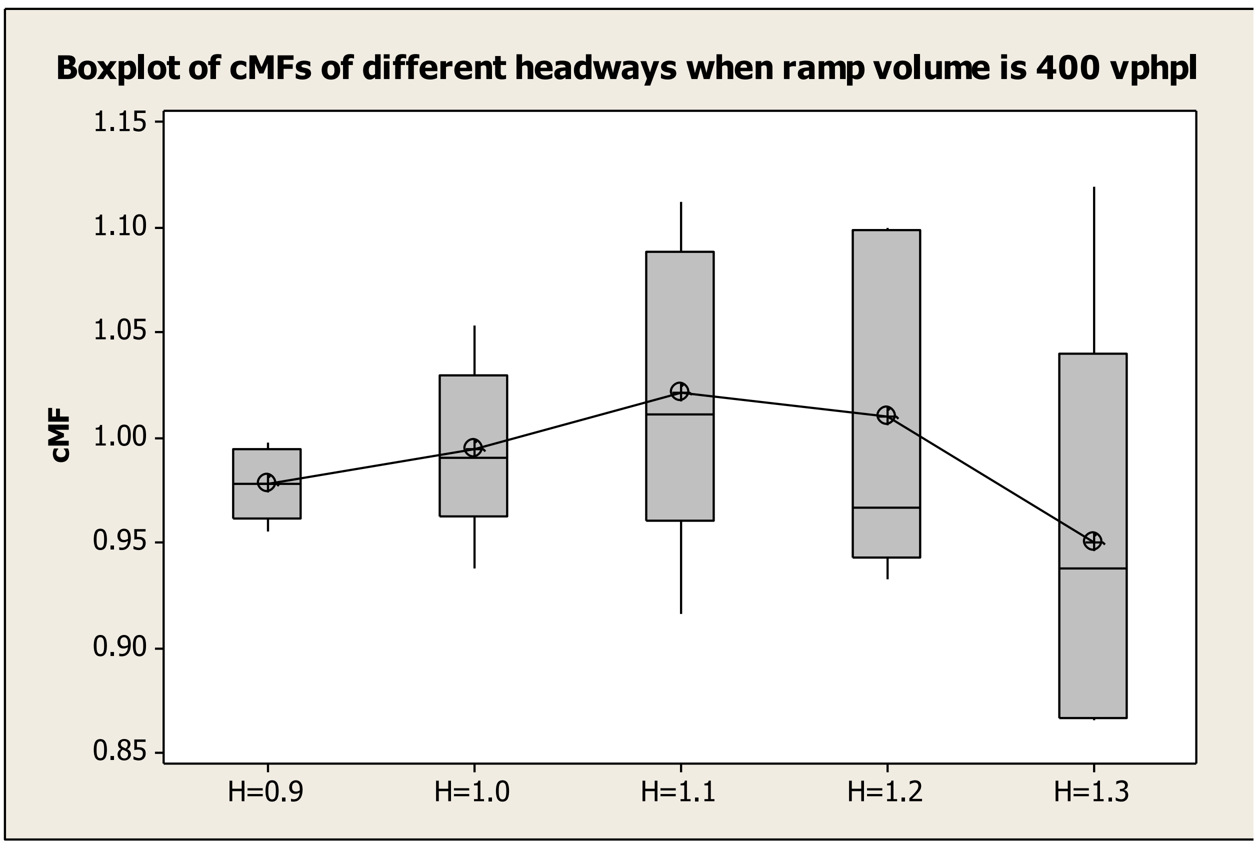


Figure 30: Boxplot diagram of the average values of the cMF using different headways at the influence area of Type III junction

5.5.2 Effects of Traffic Composition Change on the Effectiveness of Ramp Metering on the Efficiency and Safety of a Freeway

Based on the results that were obtained for the model of the freeway of Type III ramp-freeway junction, the ramp meters affected the efficiency and safety of the freeway when the traffic volume of the freeway and the ramp was high (i.e., during the peak-hour period). Although the percentage of buses and trucks during the peak hour period was small, in this study, the effects of changes to the traffic composition on the effectiveness of ramp metering on the freeway efficiency and safety were evaluated. Five different percentages of buses and trucks were examined representing different traffic composition in different daytime periods. The percentages of buses and trucks that were examined in the sensitivity analysis were 3, 5, 7, 9, and 11. Traffic volume scenarios, in which the traffic volume of the freeway was 1,750 vphpl and the traffic volumes of the ramp were 400, 600, 800, and 1,000 vphpl, were evaluated for the assumed percentages of the buses and trucks. The average values of the speeds in the ramp influence area and the average traffic conflicts in the 3000 feet of the freeway segment of a Type III junction were used to evaluate the effects of the traffic composition on the ramp metering effectiveness on the freeway efficiency and safety. The following statistical null hypothesis was assumed to test the evaluation.

$$\mathrm{H_o}: \mu_3 = \mu_5 = \mu_7 = \mu_9 = \mu_{11}$$

$$\mathrm{H_a}: \mathrm{H_o}\ \text{is not correct}$$

Tables E.88 and E.89 show the VISSIM output results of the average speed in the ramp influence area of a Type III junction after using different traffic compositions under the circumstances of the base case and the signal timing scenario of (4R+2G). The tables show that when the percentage of the buses and the trucks increased, the average speeds in the ramp influence area decreased. Table 70 shows the results of the percentage of average speed change in the ramp influence area after using the signal scenario of (4R+2G). The results of the p-values in the table show that the null hypotheses were rejected when the traffic volume of the ramp was equal to or greater than 600 vphpl, because their p-values were smaller than 0.05, as indicated by bold letters. Accordingly, the traffic composition affects the effectiveness of the ramp metering on the freeway efficiency when the traffic ramp volume is equal to or greater than 600 vphpl. In other words, ramp metering can be beneficial for traffic efficiency of the freeway only under the circumstance of having a small percentage of buses and trucks, such as 3 percent or less.

Tables E.90 and E.91 show the SSAM output results of the average number of conflicts that occurred near the 3000 ft section of the freeway segment of a Type III ramp-freeway junction for the base case and signal timing scenario of (4R+2G). According to the tables' results, when the percentage of the buses and trucks increased, the numbers of the average traffic conflicts increased. Table 71 shows the results of the cMF values that were obtained from dividing the average conflict numbers that occurred in the signal timing scenario of (4R+2G) to the average conflict numbers that occurred in the base case. The table shows that the p-values are smaller than 0.05 when the traffic volume of the ramp was equal to or greater than 800 vphpl. Therefore, the null hypotheses were rejected when of the traffic volume of the ramp is equal to or greater than 800 vphpl. Accordingly, ramp metering provides positive safety effectiveness to the freeway only when the percentage of the buses and trucks is small.

Table 70: Percentages of average speed change at the ramp influence area of Type III ramp-freeway using different traffic composition (Freeway traffic volume 1750 vphpl-(4R+2G)

Percentage of trucks and buses	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
3	19	2.5	3.8	18.5	20.5
	47	2.1	5.9	14.8	21.8
	75	1.5	4.0	18.2	22.0
	103	-1.1	5.5	19.8	21.0
	131	0.4	4.9	22.6	18.4
	average	1.1	4.8	18.8	20.7
5	19	-1.9	3.7	-0.6	-2.9
	47	1.9	0.0	0.6	1.7
	75	0.0	0.5	4.1	1.7
	103	4.3	-2.5	0.6	-1.7
	131	2.4	-1.0	-3.4	-2.8
	average	1.3	0.1	0.3	-0.8
7	19	-2.8	2.1	-5.2	-1.2
	47	-2.0	-0.5	-0.6	0.6
	75	-0.8	1.5	1.7	0.0
	103	-1.6	-0.5	-0.6	-1.2
	131	-0.4	2.1	0.0	-1.7
	average	-1.5	0.9	-0.9	-0.7
9	19	0.4	-2.6	-0.6	-3.5
	47	-2.6	-1.6	1.2	1.2
	75	0.4	1.1	-2.4	1.2
	103	2.2	1.1	0.6	0.0
	131	-0.4	2.1	1.2	0.0
	average	0.0	0.0	0.0	-0.2
11	19	-1.3	3.9	-0.6	-2.4
	47	-0.4	0.5	1.2	-3.0
	75	1.8	0.0	4.9	-1.2
	103	2.7	2.2	1.2	-1.2
	131	1.3	-1.6	1.8	-0.6
	average	0.8	1.0	1.7	-1.7
p-value		0.09	**0.003**	**0.000001**	**0.000001**

Table 71: cMFs on a 3,000 ft freeway segment of Type III junction using different traffic composition (Freeway traffic volume 1750 vphpl) - (4R+2G)

Percentage of trucks and buses	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
3	19	0.92	1.03	0.95	0.85
	47	1.01	1.08	0.90	0.89
	75	1.06	1.04	0.94	0.89
	103	1.11	1.00	0.91	0.87
	131	1.01	1.04	0.83	0.91
	average	1.02	1.04	0.90	0.88
5	19	1.03	1.00	1.03	1.03
	47	0.96	0.97	1.01	1.01
	75	1.01	1.01	1.03	0.97
	103	0.84	1.00	1.08	0.93
	131	0.95	1.01	0.99	0.97
	average	0.96	1.00	1.03	0.98
7	19	1.08	1.00	1.07	1.00
	47	1.17	1.01	1.06	0.92
	75	1.06	0.96	1.02	1.00
	103	1.01	0.94	0.95	0.99
	131	0.95	0.98	0.98	1.05
	average	1.05	0.98	1.02	0.99
9	19	0.99	1.03	1.01	0.95
	47	1.05	1.04	0.91	0.96
	75	1.05	0.99	1.10	0.97
	103	0.92	1.04	0.95	0.98
	131	0.95	1.00	0.96	0.96
	average	0.99	1.02	0.98	0.97
11	19	0.99	0.95	1.03	1.01
	47	1.05	0.95	0.95	1.02
	75	0.95	1.03	0.97	1.00
	103	0.98	0.98	1.03	0.99
	131	0.98	1.06	0.92	1.00
	average	0.99	0.99	0.98	1.00
p-value		0.238	0.052	**0.01**	**0.00001**

In conclusion, the modeled freeway traffic composition affected the ramp metering effectiveness in terms of efficiency and safety of the freeway. In other words, ramp metering provides positive effectives to the efficiency and safety of freeway only when the percentage of buses and trucks is small, or the traffic volume of the ramp is high. On the other hand, it does not have sufficient positive effects when the percentage of the buses and trucks is high, for example 7 percent, or when the traffic volume of the ramp is equal to or greater than 800 vphpl.

The evaluation results of the effectiveness of ramp metering on the efficiency, Level of Service, and safety of freeway and the sensitivity analysis that were obtained in this study are summarized in Chapter 6. Depending on the evaluation results, several points related to using ramp metering are also recommended in Chapter 6.

CHAPTER 6: FINDINGS AND CONCLUSIONS

This chapter summarizes the findings that were obtained related to the effectiveness of ramp metering on the efficiency, Level of Service, and safety of several modeled ramp-freeway junctions. It also summarizes the results of the analysis of the modeled signal timing scenarios that were designed based on the negative effects of ramp metering on the adjacent local road networks. It also summarizes the effects of the modelling's assumptions and how they change the ramp metering results. In addition, it includes several recommendations that are offered to initiate a new ramp metering algorithm and to conduct future studies related to ramp metering.

6.1 Effects of Ramp Metering on Efficiency, Level of Service, and Safety of Ramp-Freeway Junctions

The results of ramp metering effectiveness on efficiency, Level of Service, and safety of the ramp-freeway junction were summarized under the classification of the ramp-freeway junction geometric configurations.

6.1.1 Findings Related to the Type I Ramp-Freeway Junction

The effects of the ramp metering on the efficiency, Level of Service, and safety for a freeway with a Type I ramp-freeway junction are summarized in the following points:

- Ramp metering increased the freeway efficiency and raised its LOS to a higher level by using the signal timings of (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR) when the traffic volume of the freeway was equal to or greater than 1,250 vphpl and the traffic volume of the ramp was equal to or greater than 800 vphpl.
- Ramp metering improved safety of the freeway regarding the overall number of conflicts by using the designed signal timings when the traffic volume of the freeway was equal to

or greater than 1,000 vphpl and the traffic volume of the ramp was equal to or greater than 800 vphpl.

- Ramp metering decreased the number of lane change conflicts on the freeway by using the two designed signal timings when the traffic volume of the freeway was equal to or greater than 1,000 vphpl and the traffic volume of the ramp was equal to or greater than 800 vphpl. The ramp metering using designed signal timings is recommended for those freeways in which a high ratio of lane change collisions were recorded in the specified traffic volume condition.
- The ramp metering decreased the number of rear end conflicts on the freeway by using the two designed signal timings when the traffic volume of the ramp was equal to or greater than 800 vphpl, regardless the traffic volume of the freeway. Ramp metering using the designed signal timings is recommended for those freeways in which high ratio of rear end collisions were recorded in the vicinity of the ramp-freeway junction.
- Regarding the severity of conflicts, ramp metering was shown to improve safety on the freeway by using the two designed signal timings when the traffic volume of the freeway was equal to or greater than 1,000 vphpl and the traffic volume of the ramp was equal to or greater than 800 vphpl.
- If both speed and overall conflict numbers are considered as measures of effectiveness for efficiency and safety, ramp metering is recommended when the traffic volume of the freeway is equal to or greater than 1,250 vphpl, and the traffic volume of the ramp is equal to or greater than 800 vphpl. The highlighted and hatched area in Table 72 and Table 73 show the limits that ramp metering was useful for efficiency and safety.

Table 72: The effects of ramp metering on efficiency and safety using (2R+1AR+2G+1A) and ramp-freeway junction Type I

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicle / hour. lane)	500				
	750				
	1000				
	1250				
	1500				
	1750				
	2000				

Table 73: The effects of ramp metering on efficiency and safety using (5R+1AR+5G+1A) and ramp-freeway junction Type I

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicle / hour. lane)	500				
	750				
	1000				
	1250				
	1500				
	1750				
	2000				

6.1.2 Findings Related to the Type II Ramp-Freeway Junction

The effects of the ramp metering on the efficiency, Level of Service, and safety for a Type II ramp-freeway junction are summarized in the following points:

- Ramp metering provides negative effects to the freeway with a Type II junction based on the modelled results of the efficiency, Level of Service, and safety analysis in almost all of the designed signal timing and assumed traffic volume scenarios.

- However, when a Type II ramp-freeway junction is lane-balanced, use of ramp metering is not recommended. This result indicates that not only does lane balance of the ramp-freeway junction affect the ramp metering performance, but also other factors affect the ramp metering performance such as the ramp-freeway junction geometric configuration.
- Table 74 and Table 75 show the effectiveness of ramp metering on both efficiency and safety by taking into account speed and overall conflict numbers as measures of effectiveness. The hatched areas indicate the positive effects of ramp metering on safety while it does not provide any benefit to efficiency. According to the tables' results, ramp metering is not recommended because it did not provide positive effects for efficiency and safety.

Table 74: The effects of ramp metering on efficiency and safety using (2R+1AR+2G+1A) and ramp-freeway junction Type II

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicle / hour. lane)	500				
	750				
	1000				
	1250				
	1500				
	1750				
	2000				

Table 75: The effects of ramp metering on efficiency and safety using (2R+1AR+2G+1A) and ramp-freeway junction Type II

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicle / hour. lane)	500				
	750				
	1000				
	1250				
	1500				
	1750				
	2000				

6.1.3 Findings Related to the Type III Ramp-Freeway Junction

The effects of the ramp metering on the efficiency, Level of Service, and safety for a Type III ramp-freeway junction are summarized in the following points:

- Ramp metering increased the freeway efficiency by using the signal timings of (4R+2G) when the traffic volume of the freeway was equal to or greater than 1,250 vphpl and the traffic volume of the ramp was equal to or greater than 800 vphpl.
- Ramp metering with the signal timings of (2R+2G) and (4R+4G) did not increase the efficiency of the freeway.
- The results of the freeway Level of Service changes did not coincide with the results that were obtained from the two factors of speed and travel time. In some scenarios, the analyses of speed and travel times indicated that using ramp metering changes the efficiency of the freeway but the freeway's LOS did not change because the densities fell in the same ranges of the appropriate HCM's table.
- Ramp metering improved safety of the freeway regarding the overall number of conflicts by using the (2R+2G) and (4R+2G) signal timings. The ramp metering which used the

signal timing of (2R+2G), decreased the overall number of conflicts when the traffic volume of the freeway was equal to or less than 1,000 vphpl and the traffic volume of the ramp was equal to or less than 800 vphpl. It also decreased the overall number of conflicts when the signal timing of (4R+2G) was used and the traffic volume of the freeway was equal to or greater than 750 vphpl and the traffic volume of the ramp was equal to or greater than 800 vphpl. In addition, ramp metering with a signal timing of (4R+4G) is not recommended because it provides negative effects to the freeway's efficiency and capacity.

- Ramp metering decreased the number of lane change conflicts by using the signal timing (4R+2G) when the traffic volume of the ramp was equal to or greater than 800 vphpl, regardless of the traffic volume of the freeway. Only the ramp metering signal timing (4R+2G) is recommended for those freeways in which high ratio of lane change collisions.
- Ramp metering decreased the number of rear end conflicts by using the signal timings (2R+2G) and (4R+2G). Ramp metering with the signal timing (2R+2G) decreased the number of rear end conflicts when the traffic volume of the freeway is equal to or less than 1,000 vphpl and the traffic volume of the ramp was equal to or less than 800 vphpl. It also decreased the number of rear end conflicts when the signal timing (4R+2G) is used while the traffic volume of the freeway is equal to or greater than 750 vphpl and the traffic volume of the ramp was equal to or greater than 800 vphpl. Ramp metering with a signal timing (4R+4G) is not recommended because it did not provide specific limits of the ramp and freeway volumes in which safety is improved.
- Ramp metering improved the safety of the freeway regarding the severity of the conflicts by using the signal timing (4R+2G) when the traffic volume of the freeway was equal to or greater than 750 vphpl, and the traffic volume of the ramp was equal to or greater than

800 vphpl. When both measures of effectiveness, speed and overall conflict numbers, are considered for evaluating the effectiveness of ramp metering on the efficiency and safety of freeways, ramp metering is recommended when the traffic volume of the freeway is equal to or greater than 1,250 vphpl and the traffic volume of the ramp is equal to or greater than 800 vphpl and only in the traffic scenario of (4R+2G). As represented in highlighted and hatched areas in Table 76, Table 77, and Table 78 indicate the benefits of ramp metering for efficiency and safety of the freeway.

Table 76: The effects of ramp metering on efficiency and safety using (2R+2G) and ramp-freeway junction Type III

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicle / hour. lane)	500				
	750				
	1000				
	1250				
	1500				
	1750				
	2000				

Table 77: The effects of ramp metering on efficiency and safety using (4R+4G) and ramp-freeway junction Type III

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicle / hour. lane)	500				
	750				
	1000				
	1250				
	1500				
	1750				
	2000				

Table 78: The effects of ramp metering on efficiency and safety using (4R+2G) and ramp-freeway junction Type III

		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicle / hour. lane)	500				
	750				
	1000				
	1250				
	1500				
	1750				
	2000				

6.2 The Findings Related to the Design of the Signal Timings Based on the Effects of the Ramp Metering on Local Streets

In this section, the designed signal timing scenarios based on the effects of the ramp metering on the local streets are summarized. The effects of the ramp metering on the local streets were assessed based on the average value of the maximum queue lengths that occur in the right and/or left lanes of the ramp during the peak hour period. The location where the ramp meters were installed was determined based on the queue storage space on the ramp. The queue storage space was measured based on the length of the average value of the maximum queue lengths that occurred. The following points are the summary of the signal timing designs, effects of the queue, and the location of the ramp meters.

- Both of the signal timings of the (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR) were acceptable on the Type I ramp-freeway junction when the queue storage space was equal to or greater than 725 feet.

- Although use of ramp metering on the geometric configuration of a Type II ramp-freeway junction is not recommended, the queue storage space should be 760 feet or more when the signal timings (2R+1AR+2G+1AR) and (5R+1AR+5G+1AR) are used. If smaller lengths are used for the queue storage space, the ramp meters were found to adversely affect the local street network.
- For the geometric configuration of a Type III ramp-freeway junction, the signal timings of (2R+2G) and (4R+4G) need queue storage spaces of 70 and 120 feet, respectively. To avoid the adverse effects of the ramp metering on the local streets with using the signal timing of (4R+2G), 400 feet length is needed as the queue storage space.
- Regarding all of the designed signal timings, increasing red-time periods in the ramp metering signal timings provided more efficiency, capacity, and safety to the freeway because the vehicles' delay time was transferred from the freeway to the ramp.
- When the red-time period of the signal timing is increased, the adverse effects of the ramp metering on the local streets could be alleviated by using longer distances for the queue storage space.

6.3 Recommendations for Future Algorithm Development

The results were obtained from this study can be used as a first step to initiate a new algorithm that will have the following characteristics:

- The algorithm stresses traffic volume of the ramp and the freeway as quantitative information to determine the periods of ramp metering operation.
- The signal timing design in the algorithm is chosen based on the traffic volume of the freeway and the ramp and the geometric configuration of the ramp-freeway junction.

- The algorithm can be programmed based on the traffic volume data, geometric configuration of the ramp-freeway junction, signal timing designs, traffic composition, and driver behavior of the freeway. For example, if the geometric configuration of the ramp-freeway junction is as Type II, the traffic volume of the ramp and the freeway are equal to or greater than 600 vphpl and 1,250 vphpl, respectively; then ramp metering is used with signal timing (4R+2G).
- The ramp metering algorithm needs four detectors as follows:
    - Freeway upstream detectors to collect traffic volumes upstream on the freeway;
    - Ramp upstream detector to collect traffic volume in the upstream of the ramp;
    - Ramp influence area detector to collect speed data as outputs; and
    - Ramp detector to measure the queue length on the ramp
- The entire segment of the I-435 freeway with 16 ramp meters can be tested based on the algorithm that is recommended in this study to know the effects of the ramp meters on the freeway system. Different traffic volumes on the ramps and freeway segments for different time periods can used for the test.
- The study results also can be used as criteria for using ramp metering systems for those freeways in which a high ratio of overall, lane change, and/or rear end collisions were recorded. The signal timing for the ramp meters can be chosen based on the historical data of the types of collisions that occurred during the previous years and the traffic volume of the ramp and the freeway, or from modelling traffic using VISSIM and SSAM as was performed in this study.

6.4 Other Areas for Proposed Future Study

In addition to the Algorithm-focused studies discussed above, several other studies could be conducted:

- The effects of the driver behavior on ramp metering performance;
- The effects of the traffic composition of the vehicles in the freeway and the on ramp metering performance;
- Evaluation of the effectiveness of the ramp metering using CARMA algorithm on safety and efficiency on the I-435 freeway;
- Evaluation of the ramp metering violation study for both fixed-time and actuated-time signal timing designs;
- The effects of the lane balance in the ramp-freeway junction on the ramp metering performance; and
- The effects of ramp metering on efficiency and safety of freeways by modelling different land-uses in the vicinity of the freeway-ramp junctions.

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APPENDIX A

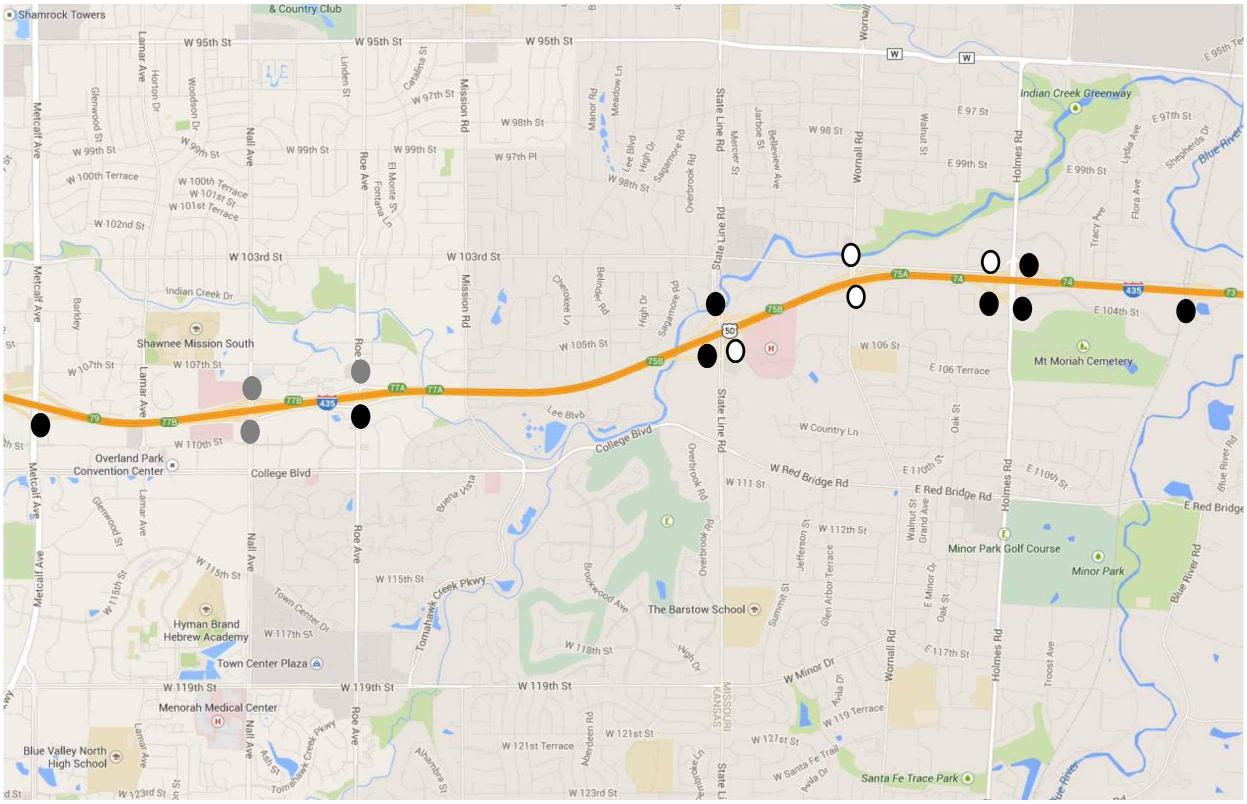

Figure A. 1: The sites of ramp meter at I-435 freeway in Kansas City

● Selected, ○ weaving (unselected), ● principally unselected

Figure A. 2: an image of I-435 freeway connected to the Metcalf Avenue [google map]

Figure A. 3: an image of I-435 freeway connected to the Roe Avenue [google map]

Figure A. 4: an image of I-435 freeway connected to the Holmes Road [google map]

Table (A.1): Upstream freeway traffic flow and composition at PM peak hour (Holmes Road)															
Lane 4 (24.35%)				Lane 3 (24.85%)				Lane 2 (23.80%)				Lane 1 (27.0%)			
Car	Bus	Truck	Total	Car	Bus	Truck	Total	Car	Bus	Truck	Total	Car	Bus	Truck	Total
1559	4	58	1621	1589	2	63	1654	1513	0	71	1584	1780	3	15	1798
96.2	0.2	3.6	%	96.1	0.1	3.8	%	95.5	0	4.5	%	99	0.15	0.85	%
Overall percentages: Car = 96.76 % Bus = 0.14% Truck = 3.1%															

Table (A.2): Proportions of freeway lanes at PM peak hour (Holmes Road)					
Lane number	4	3	2	1	Total
Traffic flow (vehicle per hour)	1621	1654	1584	1798	6657
Proportion	24.35 %	24.85%	23.8%	27.0%	100%

Table (A.3): Kansas City Scout detector and Camera data on upstream of the freeway (Holmes Road), PM peak hour					
Lanes	4	3	2	1	Total
Kansas Scout data	1461	1676	1617	1663	6417
Field data	1621	1654	1584	1798	6657
Difference	-160	+22	+33	-135	-240
% of difference	-9.9%	+0.3%	+2.1%	-7.5%	-3.6%

Table (A.4): Traffic composition of the ramp in Holmes Road at PM peak hour				
Vehicle type	Car	Bus	Truck	Total
Traffic flow (vehicle per hour)	293	4	0	297
Proportion	98.6 %	1.4 %	0%	100%

Table (A.5): Queue length in the right and left lanes of the Metcalf Avenue ramp					
Time	Right lane queue (Number of car)	Right lane queue length (ft)	Left lane queue (Number of car)	Left lane queue length (ft)	Average queue length (ft)
0:00:30	7	175	11	286	230.5
0:01:00	5	125	9	234	179.5
0:01:30	6	150	10	260	205
0:02:00	13	325	10	260	292.5
0:02:30	7	175	11	286	230.5
0:03:00	3	75	9	234	154.5
0:03:30	10	250	10	260	255
0:04:00	11	275	12	312	293.5
0:04:30	12	300	9	234	267
0:05:00	10	250	8	208	229
0:05:30	8	200	10	260	230
0:06:00	5	125	4	104	114.5
0:06:30	1	25	1	26	25.5
0:07:00	1	25	1	26	25.5
0:07:30	1	25	0	0	12.5
0:08:00	4	100	5	130	115
0:08:30	2	50	8	208	129
0:09:00	2	50	5	130	90
0:09:30	0	0	2	52	26
0:10:00	0	0	0	0	0
0:10:30	0	0	8	208	104
0:11:00	5	125	7	182	153.5
0:11:30	6	150	9	234	192
0:12:00	5	125	5	130	127.5
0:12:30	4	100	3	78	89
0:13:00	6	150	9	234	192
0:13:30	7	175	3	78	126.5
0:14:00	3	75	3	78	76.5
0:14:30	1	25	2	52	38.5
0:15:00	4	100	2	52	76
0:15:30	10	250	9	234	242
0:16:00	5	125	6	156	140.5
0:16:30	0	0	2	52	26
0:17:00	0	0	0	0	0
0:17:30	2	50	1	26	38
0:18:00	0	0	1	26	13
0:18:30	2	50	4	104	77
0:19:00	1	25	2	52	38.5

0:19:30	0	0	1	26	13
0:20:00	0	0	5	130	65
0:20:30	4	100	5	130	115
0:21:00	3	75	3	78	76.5
0:21:30	0	0	0	0	0
0:22:00	1	25	4	104	64.5
0:22:30	7	175	12	312	243.5
0:23:00	5	125	7	182	153.5
0:23:30	2	50	3	78	64
0:24:00	2	50	1	26	38
0:24:30	3	75	8	208	141.5
0:25:00	1	25	6	156	90.5
0:25:30	3	75	3	78	76.5
0:26:00	1	25	0	0	12.5
0:26:30	0	0	4	104	52
0:27:00	6	150	6	156	153
0:27:30	2	50	6	156	103
0:28:00	4	100	9	234	167
0:28:30	2	50	5	130	90
0:29:00	6	150	9	234	192
0:29:30	8	200	12	312	256
0:30:00	9	225	9	234	229.5
0:30:30	5	125	7	182	153.5
0:31:00	0	0	5	130	65
0:31:30	2	50	10	260	155
0:32:00	2	50	3	78	64
0:32:30	6	150	5	130	140
0:33:00	7	175	7	182	178.5
0:33:30	4	100	5	130	115
0:34:00	4	100	11	286	193
0:34:30	9	225	10	260	242.5
0:35:00	4	100	9	234	167
0:35:30	4	100	8	208	154
0:36:00	9	225	8	208	216.5
0:36:30	11	275	13	338	306.5
0:37:00	9	225	11	286	255.5
0:37:30	6	150	14	364	257
0:38:00	2	50	10	260	155
0:38:30	1	25	11	286	155.5
0:39:00	0	0	10	260	130
0:39:30	0	0	3	78	39
0:40:00	0	0	0	0	0

0:40:30	1	25	0	0	12.5
0:41:00	0	0	1	26	13
0:41:30	0	0	1	26	13
0:42:00	2	50	2	52	51
0:42:30	0	0	5	130	65
0:43:00	1	25	2	52	38.5
0:43:30	2	50	3	78	64
0:44:00	0	0	2	52	26
0:44:30	4	100	3	78	89
0:45:00	0	0	3	78	39
0:45:30	0	0	3	78	39
0:46:00	2	50	6	156	103
0:46:30	1	25	2	52	38.5
0:47:00	0	0	7	182	91
0:47:30	0	0	6	156	78
0:48:00	2	50	3	78	64
0:48:30	0	0	0	0	0
0:49:00	0	0	0	0	0
0:49:30	0	0	5	130	65
0:50:00	0	0	0	0	0
0:50:30	0	0	0	0	0
0:51:00	1	25	1	26	25.5
0:51:30	1	25	6	156	90.5
0:52:00	0	0	0	0	0
0:52:30	0	0	0	0	0
0:53:00	0	0	0	0	0
0:53:30	0	0	0	0	0
0:54:00	5	125	7	182	153.5
0:54:30	3	75	12	312	193.5
0:55:00	2	50	8	208	129
0:55:30	2	50	2	52	51
0:56:00	0	0	0	0	0
0:56:30	0	0	0	0	0
0:57:00	1	25	10	260	142.5
0:57:30	5	125	6	156	140.5
0:58:00	4	100	6	156	128
0:58:30	2	50	2	52	51
0:59:00	0	0	3	78	39
0:59:30	0	0	4	104	52
1:00:00	0	0	0	0	0
	Average	75.83		132.17	104
	Standard deviation	80.76		99.85	82.97

Table (A.6): Traffic signal green-time intervals for right lane of the Metcalf Avenue ramp during PM peak hour							
Cycle No.	Green time (seconds) right lane	Cycle No.	Green time (seconds) right lane	Cycle No.	Green time (seconds) right lane	Cycle No.	Green time (seconds) right lane
1	4.9	42	4.5	83	4.8	124	4.4
2	4.6	43	4.9	84	4.9	125	3.9
3	4.9	44	4.8	85	4.9	126	4.9
4	3.9	45	3.2	86	4.1	127	3.4
5	4.9	46	4.3	87	5	128	3.6
6	4.9	47	4.8	88	4.9	129	4.9
7	3.8	48	4.9	89	4.9	130	3.2
8	4.4	49	4.9	90	4.9	131	4.4
9	4.8	50	4.9	91	4.9	132	4.1
10	4	51	4.8	92	2.8	133	4.9
11	3.8	52	3.2	93	4.8	134	4.9
12	4.9	53	4.9	94	4.9	135	4.3
13	4.8	54	4.8	95	4.9	136	4.1
14	4.9	55	4.9	96	4.6	137	4.3
15	4.5	56	4.9	97	4.8	138	2.9
16	4.9	57	2.7	98	4.8	139	4.3
17	3.8	58	4.8	99	2.6	140	4.9
18	4.9	59	4.9	100	4.9	141	4.3
19	4.4	60	4.9	101	3.8	142	4.6
20	4.8	61	4.8	102	4.4	143	4.8
21	3	62	2.8	103	4.4	144	3.8
22	4.8	63	4.2	104	4.8	145	4.7
23	4.9	64	4.6	105	4.8	146	4.8
24	3.7	65	4.4	106	4.1	147	4.9
25	4.6	66	4.9	107	4.8	148	4.4
26	3.7	67	4.9	108	4.9	149	2.9
27	3.7	68	4.9	109	4.8	150	4.8
28	3.9	69	4.9	110	4.5	151	4.9
29	4.9	70	4.5	111	4.4	152	4.9
30	4.8	71	3.8	112	4.5	153	3.8
31	4.6	72	4.1	113	4.9	154	4.4
32	4.3	73	4.9	114	4.2	155	4.9
33	4.4	74	3.6	115	3.1	156	4.8
34	4.9	75	4.9	116	4.8	157	4.7
35	4.9	76	3.8	117	4.9	158	4.3
36	3.4	77	4.9	118	3	159	4.5
37	4.8	78	4.8	119	4.9	160	4.7
38	4.9	79	3.7	120	4.3	161	4.8
39	4.8	80	3.8	121	4.1	162	3.1
40	2.9	81	4.7	122	4.6	163	3.8
41	4.9	82	4.7	123	4	164	4.2

165	4.5	194	4.7	223	4.9	252	4.6
166	4.4	195	4.2	224	4.9	253	4.4
167	4.8	196	3.4	225	4.4	254	4.9
168	3.8	197	4.9	226	4.9	255	4.6
169	4.8	198	2.5	227	2.6	256	3.7
170	4.8	199	4.9	228	4.4	257	4.9
171	4.8	200	3.9	229	4.9	258	4.8
172	3.3	201	4.3	230	4.8	259	4.8
173	4.9	202	4.9	231	4.8	260	4.9
174	4.1	203	4.3	232	2.8	261	3
175	4.6	204	3.5	233	4.9	262	4.6
176	4.9	205	3.4	234	3.1	263	2.8
177	4.8	206	4.4	235	4.9	264	4.9
178	3	207	4.8	236	3.8	265	4.9
179	4.6	208	4.7	237	4.9	266	2.7
180	4.6	209	4.4	238	3.4	267	4.2
181	4.7	210	4.4	239	4.4	268	3.4
182	4.7	211	4.8	240	4.3	269	4.8
183	4.8	212	3.8	241	4.8	270	4.4
184	4.9	213	4.4	242	4.3	271	3.2
185	4.1	214	4.8	243	3.1	272	4.1
186	3.9	215	3.9	244	4.5	273	4.8
187	4.9	216	4.6	245	4.4	274	4.9
188	4.9	217	4.8	246	4.1	275	4.6
189	4.8	218	4.4	247	4.9	276	4.3
190	4.8	219	4.8	248	2.8	277	4.3
191	4.9	220	4.5	249	4.8	278	4.9
192	4.1	221	4.4	250	4.4	**Total**	**1221.3**
193	4.4	222	4.7	251	4.8	**Average**	**4.39**

Table (A.7): Traffic signal green-Time intervals for left lane of the Metcalf Avenue ramp during PM-peak hour							
Cycle No.	Green time (seconds) right lane	Cycle No.	Green time (seconds) right lane	Cycle No.	Green time (seconds) right lane	Cycle No.	Green time (seconds) right lane
1	4.3	43	4.8	85	4.4	127	4.8
2	4.7	44	4.4	86	4.8	128	4.7
3	3.3	45	4.8	87	3.8	129	4.3
4	2.9	46	4.8	88	4.9	130	4.7
5	4.2	47	2.8	89	4.9	131	4.8
6	4.9	48	4.9	90	4.6	132	3.2
7	4.4	49	4.8	91	4.3	133	4.4
8	4.9	50	4.3	92	4.7	134	4.8
9	4.4	51	3.4	93	4.7	135	4.9
10	4.9	52	4.9	94	4.9	136	3.9
11	3	53	4.5	95	4.8	137	4.8
12	4.3	54	4.8	96	4.8	138	4.9
13	4.4	55	4.6	97	3.2	139	4.9
14	4.9	56	4.1	98	4.8	140	4.9
15	4.6	57	4.8	99	2.5	141	3.2
16	4.2	58	4.5	100	4.6	142	4.8
17	4.8	59	3.9	101	4.4	143	4.8
18	4.8	60	4.5	102	4.9	144	4.3
19	4.4	61	4.9	103	4.9	145	3.4
20	4.1	62	4.9	104	4.8	146	4.8
21	4.8	63	4.9	105	4.9	147	4.1
22	4.8	64	3.1	106	4.8	148	4.7
23	4.8	65	4.9	107	4	149	3.9
24	4.2	66	3.4	108	4.2	150	4.1
25	4.8	67	4.9	109	4.9	151	4.4
26	4.4	68	4.9	110	4.9	152	4.9
27	4.5	69	4.9	111	3.7	153	4.4
28	4.8	70	4.9	112	4.8	154	2.8
29	4.5	71	3.4	113	4.2	155	4.9
30	3.1	72	4.4	114	4.9	156	4.9
31	4.9	73	4.9	115	4.4	157	4.8
32	4.9	74	4.9	116	4.5	158	4.3
33	3.4	75	4.2	117	4.8	159	4.2
34	4.9	76	4.9	118	3.4	160	2.8
35	4.8	77	2.8	119	4.6	161	4.3
36	4.8	78	4.8	120	4.6	162	4.8
37	4.6	79	4.4	121	4.8	163	4.9
38	4.8	80	4.4	122	2.8	164	4.9
39	4.9	81	3.8	123	4.8	165	4.1
40	4.9	82	2.9	124	4.4	166	4.9
41	4.2	83	4.9	125	4.7	167	4.9
42	4.4	84	4.8	126	4.7	168	4.9

169	4.4	205	4.7	241	4.8	277	4.4
170	4.9	206	2.9	242	4.9	278	4.4
171	4.9	207	4.8	243	4.5	279	4.5
172	4.9	208	3.8	244	3.9	280	4.4
173	4.7	209	4.4	245	4.9	281	4.8
174	3	210	4.4	246	4.4	282	4.3
175	4.9	211	4.5	247	4.2	283	2.9
176	4.9	212	4.9	248	4.9	284	4.5
177	2.9	213	2.7	249	4.8	285	3.3
178	4.9	214	4.8	250	4.9	286	2.9
179	4.9	215	4.9	251	4.5	287	4.4
180	3.2	216	3	252	4.9	288	3.8
181	4.9	217	4.4	253	4.8	289	2.8
182	4.9	218	4.8	254	4.8	290	4.2
183	4.1	219	4.1	255	4.5	291	4.5
184	4.9	220	4.3	256	4.3	292	4.8
185	4.8	221	4.9	257	4.9	293	2.9
186	4.8	222	4.9	258	4.6	294	4.4
187	4.8	223	4.9	259	3	295	4.7
188	4.8	224	4.1	260	4.8	296	4.5
189	2.9	225	4.9	261	4.8	297	4.9
190	4.6	226	4.7	262	4.2	298	4.4
191	4.8	227	3.4	263	4.8	299	4.4
192	3.6	228	4.8	264	2.8	300	4.9
193	4.9	229	4.4	265	4.8	301	4.7
194	4.9	230	4.8	266	4.5	302	4.9
195	4.9	231	4.3	267	4.2	303	4
196	4.2	232	4.9	268	4.9	304	3.3
197	4.6	233	4.5	269	3.4	305	4.9
198	2.7	234	4.9	270	4.8	306	4.9
199	4.9	235	4.5	271	2.3	307	3.9
200	4.8	236	3.9	272	4.1	**Total**	**1354.4**
201	4.9	237	4.8	273	4.9	**Average**	**4.41**
202	4.9	238	4.2	274	4.9		
203	4.4	239	3.2	275	4.7		
204	4.8	240	4.9	276	4.7		

APPENDIX B

Table (B.1): Wiedemann 99 parameters [Woody, 2006]			
Category	VISSIM Code	Description	Default value
Thresholds for Dx	CC0	Standstill distance: desired distance between lead and following vehicle at v = 0 mph	4.92 ft
	CC1	Headway time: desired time in seconds between lead and following vehicle	0.90 sec
	CC2	Following variation: additional distance over safety distance that a vehicle requires	13.12 ft
	CC3	Threshold for entering ‘following’ state: time in seconds before a vehicle starts to decelerate to reach safety distance (negative)	-8.00 sec
Thresholds for Dv	CC4	Negative ‘following’ threshold: specifies variation in speed between lead and following vehicle	0.35 ft/s
	CC5	Positive ‘following threshold’: specifies variation in speed between lead and following vehicle	0.35 ft/s
	CC6	Speed dependency of oscillation: influence of distance on speed oscillation	11.44
Acceleration rates	CC7	Oscillation acceleration: acceleration during the oscillation process	0.82 ft/s^2
	CC8	Standstill acceleration: desired acceleration starting from standstill	11.48 ft/s^2
	CC9	Acceleration at 50 mph: desired acceleration at 50 mph	4.92 ft/s^2

Table (B.2): Queue lengths (ft) for every 30 seconds during one hour for different seeds						
Time	Queue, ft seed 19	Queue , ft seed 47	Queue, ft seed 75	Queue, ft seed 103	Queue, ft seed 131	VISSIM average queue, ft
0:00:30	6	28.5	33	0.5	241	61.8
0:01:00	8	112.5	91	3	235	89.9
0:01:30	22	153	52	2.5	338.5	113.6
0:02:00	46	206.5	28	6	288.5	115
0:02:30	33.5	143	10.5	41.5	271.5	100
0:03:00	45.5	234	18.5	106	412	163.2
0:03:30	49.5	281	17.5	149	411	181.6
0:04:00	45.5	285.5	29	153.5	426.5	188
0:04:30	36.5	310.5	36	203	535.5	224.3
0:05:00	32	256.5	19	250.5	505.5	212.7
0:05:30	30	132.5	31	231	315	147.9
0:06:00	56	223	58.5	148	171.5	131.4
0:06:30	83	291.5	80.5	171.5	251.5	175.6
0:07:00	53	162.5	136.5	201.5	183.5	147.4
0:07:30	10	199	76.5	150.5	101	107.4
0:08:00	12	240	110.5	60.5	35.5	91.7
0:08:30	23	392	181	40	9.5	129.1
0:09:00	57.5	353	134.5	35	17.5	119.5
0:09:30	119.5	268	118.5	26	18.5	110.1
0:10:00	170	187.5	133.5	27.5	48	113.3
0:10:30	164.5	332	189.5	19.5	81	157.3
0:11:00	188.5	221.5	223	1	79	142.6
0:11:30	149.5	244.5	173.5	25.5	36	125.8
0:12:00	207.5	168	120	31	22	109.7
0:12:30	164	121	163.5	33.5	30.5	102.5
0:13:00	157	188	110	22.5	47	104.9
0:13:30	103	104	73.5	12.5	74.5	73.5
0:14:00	73	346.5	91	1.5	43	111
0:14:30	22	186.5	114	21.5	53	79.4
0:15:00	22.5	111.5	156.5	40.5	51.5	76.5
0:15:30	49	136	225.5	49	66.5	105.2
0:16:00	53.5	249.5	206	17.5	55	116.3
0:16:30	21	255	145	82	89.5	118.5
0:17:00	37	171.5	205	147.5	122	136.6
0:17:30	105.5	166.5	181.5	202.5	64	144
0:18:00	93	153.5	236.5	169	46	139.6
0:18:30	88.5	101.5	248	91	22	110.2
0:19:00	39	62.5	235.5	64.5	16	83.5
0:19:30	32	26.5	229.5	59.5	7	70.9

0:20:00	8.5	53.5	146	50	0.5	51.7
0:20:30	31	124	100.5	85	1.5	68.4
0:21:00	7	182	83.5	71	10.5	70.8
0:21:30	2.5	121.5	70	67	22	56.6
0:22:00	14	211.5	105	126.5	53	102
0:22:30	6.5	273	64.5	209	101	130.8
0:23:00	11.5	175	59	140.5	80	93.2
0:23:30	31	100.5	62.5	125	117.5	87.3
0:24:00	60.5	48.5	75.5	72	176.5	86.6
0:24:30	45	31.5	23	61	183	68.7
0:25:00	17.5	9	2.5	27	176.5	46.5
0:25:30	2.5	6.5	2.5	9.5	168	37.8
0:26:00	7.5	16	12	17.5	52.5	21.1
0:26:30	25	55.5	13.5	16.5	95.5	41.2
0:27:00	28	42.5	2	21	235	65.7
0:27:30	31	45	2	18	234	66
0:28:00	45.5	34.5	10.5	8.5	178.5	55.5
0:28:30	31	24.5	28	10.5	207	60.2
0:29:00	10	4	18.5	9.5	169.5	42.3
0:29:30	6.5	12	6.5	13	191.5	45.9
0:30:00	8.5	6.5	18	16	121	34
0:30:30	7.5	21.5	14	14	47	20.8
0:31:00	16	6	11	1	37	14.2
0:31:30	47	13	15.5	1.5	43.5	24.1
0:32:00	29.5	19	17.5	3	67	27.2
0:32:30	39	9	15.5	8	95	33.3
0:33:00	42	9.5	16	12	170.5	50
0:33:30	29.5	17.5	41	71.5	156	63.1
0:34:00	16	13.5	85.5	129.5	163	81.5
0:34:30	30	11.5	115	124	220.5	100.2
0:35:00	16.5	12	108.5	99.5	227.5	92.8
0:35:30	7.5	14.5	63.5	97	106.5	57.8
0:36:00	18.5	30	68.5	117	186.5	84.1
0:36:30	57	38	208.5	155.5	129.5	117.7
0:37:00	67.5	80.5	240.5	81	48.5	103.6
0:37:30	52	143	229.5	44	9.5	95.6
0:38:00	79.5	99	167.5	47	10	80.6
0:38:30	45	95.5	176.5	25	18	72
0:39:00	20	140.5	206	13	10.5	78
0:39:30	24.5	121.5	216.5	3	8	74.7
0:40:00	4	94	161	29	0.5	57.7
0:40:30	9.5	46.5	164.5	45	7	54.5

0:41:00	38.5	11.5	131	17	7.5	41.1
0:41:30	57	7	84	10.5	24.5	36.6
0:42:00	40.5	13	86.5	40	36.5	43.3
0:42:30	20.5	35	116	94	21	57.3
0:43:00	16.5	3	232.5	160.5	26	87.7
0:43:30	10	16	213	151.5	43	86.7
0:44:00	14.5	39	173	126	114	93.3
0:44:30	24.5	90.5	239.5	134	181	133.9
0:45:00	7	97	234.5	149.5	234.5	144.5
0:45:30	18.5	47.5	188.5	83	197.5	107
0:46:00	73	5.5	184	79	186.5	105.6
0:46:30	124.5	6	230.5	121.5	151.5	126.8
0:47:00	123	12	281.5	68.5	219	140.8
0:47:30	148.5	43.5	300	25	165.5	136.5
0:48:00	298.5	48.5	263	92.5	203.5	181.2
0:48:30	285.5	34	183.5	113.5	226.5	168.6
0:49:00	328.5	35	203.5	261	246	214.8
0:49:30	355	14.5	263.5	313.5	233.5	236
0:50:00	470.5	25.5	197	413	147	250.6
0:50:30	424	28.5	152.5	413	56	214.8
0:51:00	430.5	14	256.5	325	68	218.8
0:51:30	215	14.5	232	154	48	132.7
0:52:00	219.5	14.5	155	245.5	60	138.9
0:52:30	296.5	29.5	78	584.5	166	230.9
0:53:00	214.5	66	94	627	110	222.3
0:53:30	187.5	92.5	29.5	498	193.5	200.2
0:54:00	252.5	71	57	586	120	217.3
0:54:30	277.5	27	58.5	605	77	209
0:55:00	234	42	26.5	542	87	186.3
0:55:30	141.5	104	7	474	171.5	179.6
0:56:00	113.5	120.5	13.5	536	203	197.3
0:56:30	98.5	178	10	589	389.5	253
0:57:00	30.5	141	17	527.5	211.5	185.5
0:57:30	66	197	13	496	183.5	191.1
0:58:00	111.5	300	11.5	489.5	138.5	210.2
0:58:30	102.5	222	14	474	89.5	180.4
0:59:00	58	277.5	10	482	45.5	174.6
0:59:30	150	480	17.5	480	8	227.1
1:00:00	238	538.5	44	434.5	4	251.8
					average	**116.3**

APPENDIX C

Table (C.1): Average speed (mph) at the influence area of Type I junction-No ramp metering										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			S	Avg. S	S	Avg. S	S	Avg. S	S	Avg. S
Freeway volume (vehicles / hour lane)	500	19	60.4	60.4	59.5	59.6	59.0	59.0	59.0	59.0
		47	60.7		59.8		59.0		59.0	
		75	60.3		59.6		58.9		59.0	
		103	60.3		59.4		59.0		58.8	
		131	60.5		59.6		59.0		59.1	
	750	19	60.2	60.1	59.1	59.1	58.5	58.6	58.5	58.5
		47	60.1		59.3		58.6		58.5	
		75	60.0		59.0		58.6		58.5	
		103	60.1		59.2		58.8		58.5	
		131	60.1		59.1		58.6		58.4	
	1000	19	59.5	59.4	58.3	58.5	57.3	57.2	57.2	56.9
		47	59.5		58.6		57.2		57.0	
		75	59.3		58.1		57.3		56.9	
		103	59.5		58.8		56.8		55.4	
		131	59.3		58.5		57.6		57.8	
	1250	19	56.5	56.9	54.7	54.8	47.6	48.5	46.8	47.7
		47	57.1		55.0		48.4		47.9	
		75	56.7		53.5		48.3		47.3	
		103	56.8		54.5		48.2		45.7	
		131	57.5		56.3		49.8		50.7	
	1500	19	54.8	54.6	49.2	49.9	28.5	36.1	27.8	23.9
		47	53.9		49.9		34.5		22.6	
		75	55.1		50.4		42.3		22.8	
		103	54.3		49.6		32.8		20.4	
		131	54.8		50.4		42.6		25.7	
	1750	19	54.3	54.4	49.4	49.9	28.7	36.0	21.8	23.9
		47	54.3		50.3		35.6		29.0	
		75	54.1		50.3		41.2		28.2	
		103	55.6		49.7		32.8		20.1	
		131	53.9		50.0		41.5		20.3	
	2000	19	53.9	54.4	50.0	49.9	28.1	36.0	21.4	21.3
		47	54.2		49.5		32.6		24.7	
		75	54.7		50.1		41.9		22.4	
		103	55.0		49.8		36.0		18.6	
		131	54.0		50.2		41.2		19.6	

S = Average speed at different seeds
Avg. S = Average of average speeds at different seeds

Table (C.2): Average speed (mph) at the ramp influence area of Type I junction- (2R+1AR+2G+1AR)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			S	Avg. S	S	Avg. S	S	Avg. S	S	Avg. S
Freeway volume (vehicles / hour lane)	500	19	60.6	60.4	59.7	59.7	59.5	59.6	59.7	59.6
		47	60.6		59.8		59.7		59.7	
		75	60.3		59.5		59.8		59.5	
		103	60.3		59.5		59.5		59.6	
		131	60.3		59.8		59.7		59.7	
	750	19	60.2	60.2	59.3	59.3	59.2	59.3	59.2	59.3
		47	60.1		59.2		59.2		59.3	
		75	60.1		59.4		59.4		59.4	
		103	60.3		59.4		59.3		59.3	
		131	60.1		59.2		59.2		59.2	
	1000	19	59.5	59.4	58.7	58.5	58.4	58.5	58.4	58.5
		47	59.3		58.3		58.5		58.5	
		75	59.2		58.5		58.4		58.4	
		103	59.5		58.6		58.5		58.5	
		131	59.4		58.5		58.5		58.5	
	1250	19	57.3	57.1	53.9	54.7	54.6	54.1	54.0	54.0
		47	56.7		55.1		53.6		54.2	
		75	56.6		53.6		53.3		53.5	
		103	57.7		55.7		54.0		54.3	
		131	57.4		55.3		54.8		54.1	
	1500	19	54.5	54.2	49.6	49.6	49.6	49.6	49.2	49.6
		47	53.7		49.2		49.8		49.5	
		75	53.8		49.6		49.4		49.9	
		103	55.3		49.9		49.0		49.6	
		131	53.7		49.6		50.2		49.9	
	1750	19	54.0	53.9	49.9	49.6	49.6	49.6	49.0	49.2
		47	53.8		49.1		49.1		49.3	
		75	54.6		49.3		50.0		49.4	
		103	53.4		50.3		49.6		49.0	
		131	53.6		49.3		49.9		49.1	
	2000	19	54.4	53.8	49.7	49.7	48.8	49.4	48.2	49.1
		47	53.7		49.6		49.5		49.0	
		75	53.7		49.9		48.9		49.4	
		103	54.2		49.7		49.8		49.3	
		131	53.2		49.8		49.9		49.4	

S = Average speed at different seeds
Avg. S = Average of average speeds at different seeds

Table (C.3): Average speed (mph) at the influence area of Type I junction- (5R+1AR+5G+1AR)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			S	Avg. S	S	Avg. S	S	Avg. S	S	Avg. S
Freeway volume (vehicles / hour lane)	500	19	60.5	60.5	59.6	59.6	59.7	59.6	59.5	59.6
		47	60.7		59.8		59.6		59.8	
		75	60.4		59.4		59.6		59.3	
		103	60.5		59.5		59.5		59.5	
		131	60.3		59.8		59.7		59.7	
	750	19	60.2	60.1	59.3	59.3	59.2	59.2	59.2	59.2
		47	60.2		59.4		59.4		59.2	
		75	60.0		59.3		59.3		59.2	
		103	60.0		59.5		59.1		59.4	
		131	60.3		59.1		59.2		59.2	
	1000	19	59.6	59.4	58.2	58.4	58.6	58.4	58.4	58.4
		47	59.3		58.1		58.3		58.1	
		75	59.2		58.3		58.3		58.4	
		103	59.5		58.6		58.5		58.5	
		131	59.4		58.6		58.4		58.6	
	1250	19	57.1	56.9	54.3	54.6	54.4	54.6	53.5	54.1
		47	56.6		55.4		54.7		53.9	
		75	56.4		54.0		54.8		52.9	
		103	57.2		55.2		54.8		54.5	
		131	57.1		54.2		54.5		55.9	
	1500	19	54.3	54.3	49.5	49.9	49.5	49.7	49.9	49.7
		47	54.4		49.4		49.9		50.1	
		75	55.7		50.4		49.3		48.9	
		103	53.6		50.1		49.7		49.3	
		131	53.7		50.2		49.9		50.2	
	1750	19	54.6	53.6	49.9	50.2	49.8	50.2	49.6	49.7
		47	53.4		50.9		50.5		49.8	
		75	53.3		49.6		50.1		49.7	
		103	53.9		50.0		50.0		49.6	
		131	53.0		50.4		50.5		49.6	
	2000	19	53.3	53.6	49.7	49.8	49.5	49.7	49.1	49.7
		47	52.7		50.3		50.1		49.9	
		75	54.6		49.2		49.8		50.0	
		103	53.8		50.2		49.2		50.0	
		131	53.8		49.4		49.8		49.6	

S = Average speed at different seeds

Avg. S = Average of average speeds at different seeds

Table (C.4): Travel time (sec.) per vehicle on a 3000 ft freeway segment of Type I junction No ramp metering										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			T	Avg. T	T	Avg. T	T	Avg. T	T	Avg. T
Freeway volume (vehicles / hour lane)	500	19	31.5	31.5	31.7	31.6	31.7	31.7	31.8	31.7
		47	31.5		31.5		31.7		31.6	
		75	31.5		31.6		31.7		31.6	
		103	31.5		31.6		31.7		31.9	
		131	31.5		31.6		31.6		31.6	
	750	19	32.0	32.0	32.2	32.2	32.3	32.3	32.3	32.3
		47	32.1		32.2		32.2		32.2	
		75	32.1		32.2		32.3		32.3	
		103	32.0		32.1		32.2		32.4	
		131	32.0		32.2		32.3		32.2	
	1000	19	33.1	33.1	33.5	33.4	34.1	34.1	34.2	34.3
		47	33.3		33.5		34.7		34.5	
		75	33.1		33.6		34.2		34.6	
		103	33.1		33.2		33.9		34.8	
		131	33.0		33.1		33.8		33.3	
	1250	19	39.9	39.5	41.3	40.9	46.1	45.6	47.1	46.4
		47	39.2		41.0		45.7		46.5	
		75	40.0		42.5		46.5		47.3	
		103	39.6		40.6		44.9		47.2	
		131	38.9		39.3		44.9		43.7	
	1500	19	43.2	43.7	47.6	47.4	62.9	58.3	69.3	73.2
		47	44.3		47.2		62.2		73.9	
		75	43.1		47.4		53.8		74.4	
		103	44.1		48.1		58.9		78.6	
		131	43.7		46.8		53.9		70.0	
	1750	19	44.0	43.5	47.8	47.2	62.5	58.6	74.0	73.2
		47	43.9		46.7		61.4		68.3	
		75	43.8		46.6		54.3		68.5	
		103	42.6		47.7		60.6		79.1	
		131	43.2		47.1		54.3		76.2	
	2000	19	43.7	43.7	46.5	47.2	62.0	58.5	73.8	75.2
		47	44.3		48.0		63.6		71.0	
		75	43.8		47.4		54.3		74.4	
		103	43.2		46.7		58.1		79.1	
		131	43.7		47.2		54.7		77.7	

T = Average travel time per vehicle at different seeds
Avg. T = Average of average travel times at different seeds

Table (C.5): Travel time (sec.) per vehicle on a 3000 ft freeway segment of Type I junction (2R+1AR+2G+1AR)										
			Ramp volume (vehicles / hour lane)							
			400		600		800		1000	
		Seed No.	T	Avg. T	T	Avg. T	T	Avg. T	T	Avg. T
Freeway volume (vehicles / hour lane)	500	19	31.7	31.6	31.9	31.8	31.9	31.9	31.9	31.9
		47	31.6		31.7		31.8		31.8	
		75	31.6		31.9		31.8		31.9	
		103	31.6		31.8		31.9		31.9	
		131	31.6		31.8		31.9		31.8	
	750	19	32.1	32.1	32.4	32.3	32.4	32.4	32.4	32.4
		47	32.1		32.3		32.4		32.3	
		75	32.1		32.3		32.4		32.4	
		103	32.1		32.3		32.3		32.3	
		131	32.1		32.4		32.3		32.4	
	1000	19	33.2	33.2	33.5	33.6	33.7	33.6	33.7	33.6
		47	33.6		34.0		33.7		33.7	
		75	33.1		33.5		33.5		33.5	
		103	33.1		33.4		33.6		33.6	
		131	33.2		33.5		33.3		33.3	
	1250	19	39.0	39.4	41.8	41.2	41.5	41.9	42.0	42.0
		47	39.8		41.2		42.4		42.1	
		75	40.3		42.5		42.8		42.6	
		103	39.0		40.1		42.1		41.3	
		131	38.8		40.6		40.8		42.0	
	1500	19	43.4	43.8	47.3	47.8	48.7	48.0	47.9	47.9
		47	44.3		48.0		47.6		48.1	
		75	44.3		47.8		48.1		47.9	
		103	42.7		47.9		48.4		48.2	
		131	44.1		48.0		47.1		47.6	
	1750	19	43.8	43.9	47.5	48.2	47.4	47.7	48.3	48.1
		47	43.8		48.6		48.1		48.0	
		75	43.8		48.3		47.6		47.7	
		103	44.3		47.9		48.3		48.2	
		131	43.7		48.5		46.9		48.3	
	2000	19	44.3	44.3	48.1	48.0	48.5	48.4	48.7	48.1
		47	44.4		48.1		48.4		48.0	
		75	43.9		47.9		48.7		47.8	
		103	44.1		48.0		47.9		48.3	
		131	44.7		47.9		48.3		47.6	

T = Average travel time per vehicle at different seeds
Avg. T = Average of average travel times at different seeds

Table (C.6): Travel time (sec.) per vehicle on a 3000 ft freeway segment of Type I junction (5R+1AR+5G+1AR)										
			Ramp volume (vehicles / hour lane)							
			400		600		800		1000	
		Seed No.	T	Avg. T	T	Avg. T	T	Avg. T	T	Avg. T
Freeway volume (vehicles / hour lane)	500	19	31.7	31.6	31.9	31.8	31.8	31.8	31.9	31.8
		47	31.6		31.7		31.8		31.7	
		75	31.7		31.9		31.8		31.8	
		103	31.6		31.8		31.8		31.8	
		131	31.6		31.8		31.7		31.7	
	750	19	32.2	32.2	32.4	32.3	32.3	32.3	32.3	32.3
		47	32.2		32.3		32.3		32.3	
		75	32.1		32.3		32.3		32.3	
		103	32.1		32.3		32.3		32.3	
		131	32.2		32.3		32.3		32.3	
	1000	19	33.3	33.2	33.8	33.6	33.4	33.5	33.7	33.5
		47	33.6		34.0		34.0		34.0	
		75	33.2		33.7		33.6		33.4	
		103	33.0		33.4		33.4		33.3	
		131	33.1		33.3		33.3		33.3	
	1250	19	39.4	39.8	41.3	41.1	41.4	41.1	42.5	41.8
		47	40.1		40.4		41.1		42.3	
		75	40.7		41.9		41.4		43.1	
		103	39.3		40.2		40.6		40.9	
		131	39.5		41.7		41.1		40.0	
	1500	19	43.9	43.5	48.4	47.6	47.6	47.8	47.7	47.8
		47	44.0		47.9		47.8		47.0	
		75	42.1		47.0		48.1		47.7	
		103	44.0		47.3		48.2		48.5	
		131	43.6		47.6		47.4		47.9	
	1750	19	43.6	44.3	47.2	47.3	47.4	47.3	48.1	47.9
		47	44.4		46.4		47.3		47.9	
		75	44.7		48.3		47.5		48.0	
		103	44.3		47.9		47.5		48.1	
		131	44.7		46.8		46.9		47.6	
	2000	19	44.4	44.3	47.4	47.7	47.9	47.7	48.7	47.7
		47	45.0		47.6		47.0		47.4	
		75	43.8		48.5		47.6		47.4	
		103	44.2		47.5		48.4		47.4	
		131	43.9		47.7		47.8		47.8	

T = Average travel time per vehicle at different seeds
Avg. T = Average of average travel times at different seeds

Table (C.7): Average speed (mph) at the influence area of Type II junction No ramp metering										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			S	Avg. S	S	Avg. S	S	Avg. S	S	Avg. S
Freeway volume (vehicles / hour lane)	500	19	59.3	59.3	59.2	59.0	59.3	59.2	58.9	59.0
		47	59.4		59.1		59.1		59.3	
		75	59.3		59.0		59.1		59.1	
		103	59.0		59.1		59.3		59.1	
		131	59.4		58.7		59.0		58.6	
	750	19	58.9	59.0	58.7	58.7	58.6	58.6	58.5	58.6
		47	59.1		58.7		58.7		58.7	
		75	59.1		59.0		58.6		58.6	
		103	59.0		58.4		58.5		58.5	
		131	59.0		58.5		58.4		58.6	
	1000	19	57.7	57.9	57.3	57.5	56.8	57.5	56.6	57.5
		47	58.1		57.3		57.8		57.8	
		75	58.1		57.8		57.6		57.4	
		103	57.7		57.6		57.4		57.9	
		131	57.9		57.7		57.7		57.8	
	1250	19	22.1	25.6	20.8	22.8	18.7	22.3	19.6	22.4
		47	22.0		21.5		22.4		20.9	
		75	23.8		21.1		21.1		22.3	
		103	25.6		24.9		24.1		23.9	
		131	34.6		25.9		25.2		25.4	
	1500	19	14.1	14.2	13.9	14.0	13.8	14.1	14.0	14.0
		47	14.3		14.1		14.1		14.0	
		75	14.0		13.9		14.1		13.8	
		103	14.4		14.0		14.4		14.2	
		131	14.2		13.9		14.1		13.8	
	1750	19	14.2	14.2	14.1	14.1	14.0	14.1	14.0	14.0
		47	14.2		14.1		14.1		14.0	
		75	14.4		14.1		14.0		13.9	
		103	14.2		14.2		14.2		14.1	
		131	14.1		14.0		14.1		14.2	
	2000	19	14.0	14.1	13.8	14.0	14.0	14.1	13.9	13.9
		47	14.0		14.1		14.3		13.8	
		75	14.0		14.1		14.0		14.0	
		103	14.4		14.1		13.8		14.1	
		131	14.2		14.0		14.3		13.9	

S = Average speed at different seeds
Avg. S = Average of average speeds at different seeds

Table (C.8): Average speed (mph) at the influence area of Type II junction (2R+1AR+2G+1AR)									
		Ramp volume (vehicles / hour lane)							
	Seed No.	400		600		800		1000	
Freeway volume (vehicles / hour lane)		S	Avg. S	S	Avg. S	S	Avg. S	S	Avg. S
500	19	58.8	58.8	58.8	58.8	58.7	58.7	58.9	58.8
	47	58.8		58.8		58.7		58.9	
	75	59.0		59.0		58.7		58.9	
	103	58.8		58.8		58.9		58.7	
	131	58.4		58.4		58.6		58.4	
750	19	58.9	59.0	58.4	58.3	58.2	58.2	58.4	58.4
	47	59.1		58.3		58.3		58.4	
	75	59.2		58.6		58.3		58.4	
	103	58.9		58.1		58.2		58.5	
	131	59.1		58.3		58.2		58.1	
1000	19	56.2	57.6	56.3	57.0	55.5	56.9	56.4	56.9
	47	57.5		57.2		57.1		56.9	
	75	58.2		57.6		57.6		57.3	
	103	57.8		57.2		57.3		56.8	
	131	58.4		56.6		57.1		57.2	
1250	19	21.0	24.4	17.8	18.4	17.7	18.3	17.6	17.9
	47	19.7		17.7		17.5		17.3	
	75	21.7		16.4		16.0		16.5	
	103	23.3		19.4		20.1		19.5	
	131	36.1		20.6		20.3		18.6	
1500	19	14.0	13.9	13.8	13.9	13.8	13.7	13.6	13.8
	47	14.0		14.1		13.7		13.8	
	75	13.5		14.2		13.6		13.8	
	103	14.1		13.8		13.7		14.0	
	131	13.9		13.8		13.7		13.9	
1750	19	13.8	13.9	13.7	13.8	13.9	13.8	13.4	13.9
	47	13.7		13.7		13.8		13.9	
	75	14.0		13.9		13.5		14.0	
	103	14.0		13.7		13.8		13.9	
	131	14.1		13.9		13.9		14.1	
2000	19	14.1	14.0	13.8	13.7	13.6	13.8	13.7	13.7
	47	14.1		13.7		13.9		13.8	
	75	13.7		13.6		13.9		13.5	
	103	14.0		13.7		13.7		13.9	
	131	14.2		13.8		13.8		13.7	

S = Average speed at different seeds

Avg. S = Average of average speeds at different seeds

Table (C.9): Average speed (mph) at the influence area of Type II junction (5R+1AR+5G+1AR)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			S	Avg. S	S	Avg. S	S	Avg. S	S	Avg. S
Freeway volume (vehicles / hour lane)	500	19	59.2	59.4	58.6	58.6	58.7	58.7	58.5	58.7
		47	59.4		58.8		58.9		58.7	
		75	59.5		58.5		58.8		58.2	
		103	59.3		58.7		58.8		59.0	
		131	59.5		58.6		58.5		58.9	
	750	19	59.1	59.0	58.1	58.3	58.3	58.3	58.2	58.4
		47	58.9		58.4		58.4		58.4	
		75	59.1		58.5		58.3		58.5	
		103	58.8		58.2		58.1		58.5	
		131	59.1		58.1		58.3		58.3	
	1000	19	57.8	57.8	56.5	57.1	57.1	57.0	56.2	57.1
		47	57.7		57.5		56.5		57.6	
		75	58.1		57.3		57.1		57.3	
		103	57.5		57.0		57.0		57.0	
		131	57.8		57.1		57.3		57.4	
	1250	19	21.6	24.5	18.1	17.9	17.7	18.1	18.0	18.6
		47	19.4		17.1		17.2		17.4	
		75	21.8		16.7		17.2		17.1	
		103	24.0		18.6		19.1		19.0	
		131	35.9		19.1		19.1		21.4	
	1500	19	14.0	13.9	13.9	13.8	13.6	13.7	13.7	13.7
		47	13.8		13.8		13.7		13.7	
		75	13.8		13.6		13.7		13.6	
		103	13.9		13.8		14.0		13.9	
		131	14.1		13.7		13.6		13.6	
	1750	19	13.8	13.9	13.8	13.8	13.7	13.8	13.8	13.7
		47	13.9		13.8		13.8		13.8	
		75	13.9		13.7		13.7		13.6	
		103	14.1		13.7		14.1		13.7	
		131	14.0		14.0		13.9		13.8	
	2000	19	13.5	13.8	13.8	13.9	13.8	13.8	13.9	13.8
		47	14.1		13.6		14.1		13.8	
		75	13.7		13.9		13.6		13.7	
		103	13.9		14.2		13.7		13.6	
		131	14.0		14.0		13.8		13.8	

S = Average speed at different seeds

Avg. S = Average of average speeds at different seeds

Table (C.10): Travel time (sec.) per vehicle on a 3000 ft freeway segment of Type II junction No ramp metering										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			T	Avg. T	T	Avg. T	T	Avg. T	T	Avg. T
Freeway volume (vehicles / hour lane)	500	19	31.7	31.8	31.7	31.7	31.8	31.7	31.8	31.8
		47	31.7		31.6		31.6		31.7	
		75	31.8		31.8		31.7		31.7	
		103	31.8		31.8		31.8		31.8	
		131	31.8		31.8		31.8		31.9	
	750	19	32.2	32.3	32.3	32.3	32.3	32.3	32.3	32.3
		47	32.3		32.2		32.2		32.2	
		75	32.2		32.1		32.2		32.2	
		103	32.3		32.4		32.4		32.4	
		131	32.3		32.3		32.2		32.3	
	1000	19	33.5	33.4	33.8	33.6	34.3	33.7	34.2	33.6
		47	33.2		33.4		33.2		33.4	
		75	33.3		33.4		33.6		33.6	
		103	33.7		33.6		33.9		33.5	
		131	33.4		33.6		33.4		33.4	
	1250	19	73.3	68.1	76.6	71.9	81.2	73.5	78.5	72.7
		47	73.8		74.9		74.1		76.9	
		75	70.4		75.4		76.0		72.9	
		103	64.8		66.1		68.4		68.1	
		131	58.3		66.7		67.9		67.3	
	1500	19	106.0	106.0	106.8	106.2	107.3	106.5	106.1	106.8
		47	106.2		105.2		106.6		106.9	
		75	107.9		105.9		107.1		107.9	
		103	104.6		106.5		105.0		105.7	
		131	105.3		106.8		106.5		107.5	
	1750	19	105.9	105.9	106.3	106.3	107.6	106.1	106.7	106.1
		47	106.5		106.5		106.6		106.4	
		75	106.1		106.3		106.3		105.4	
		103	105.8		106.2		104.7		105.2	
		131	105.1		106.2		105.1		106.8	
	2000	19	107.2	106.0	107.6	106.6	107.4	106.5	104.4	106.2
		47	106.1		106.2		105.7		106.8	
		75	107.4		106.7		106.8		107.2	
		103	104.5		106.5		107.6		105.8	
		131	105.0		105.8		104.9		106.8	

T = Average travel time per vehicle at different seeds

Avg. T = Average of average travel times at different seeds

Table (C.11): Travel time (sec.) per vehicle on a 3000 ft freeway segment of Type II junction (2R+1AR+2G+1AR)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			T	Avg. T	T	Avg. T	T	Avg. T	T	Avg. T
Freeway volume (vehicles / hour lane)	500	19	31.8	31.8	31.8	31.8	31.8	31.8	31.8	31.8
		47	31.7		31.7		31.7		31.7	
		75	31.8		31.8		31.8		31.8	
		103	31.9		31.9		31.8		31.8	
		131	31.8		31.8		31.9		31.8	
	750	19	32.3	32.3	32.4	32.3	32.3	32.3	32.3	32.3
		47	32.2		32.4		32.3		32.2	
		75	32.2		32.1		32.2		32.3	
		103	32.3		32.3		32.4		32.3	
		131	32.3		32.4		32.3		32.2	
	1000	19	34.8	33.7	34.4	33.8	34.8	33.9	34.2	33.8
		47	34.0		33.4		33.6		33.9	
		75	33.2		33.4		33.5		33.4	
		103	33.5		33.8		33.9		34.2	
		131	33.2		34.2		33.7		33.5	
	1250	19	75.1	70.3	84.5	80.9	83.5	81.2	83.9	82.0
		47	77.4		83.2		84.0		85.1	
		75	72.4		85.5		86.6		84.5	
		103	69.3		77.0		75.4		76.3	
		131	57.1		74.4		76.3		80.2	
	1500	19	107.1	107.2	107.8	107.0	107.6	107.7	108.6	107.5
		47	106.3		106.1		108.3		105.9	
		75	109.0		106.3		108.8		107.1	
		103	105.9		107.0		106.4		107.4	
		131	107.6		107.8		107.3		108.5	
	1750	19	107.8	107.0	108.0	107.9	107.8	107.9	109.4	107.8
		47	107.9		108.2		108.2		108.1	
		75	106.7		107.3		108.8		106.1	
		103	107.0		107.8		107.7		107.9	
		131	105.6		108.1		106.8		107.4	
	2000	19	106.6	106.8	108.5	108.2	108.8	107.8	108.1	107.9
		47	106.2		108.5		106.8		107.4	
		75	108.1		108.7		107.5		108.3	
		103	107.0		108.3		107.9		107.8	
		131	105.9		107.2		108.0		108.1	

T = Average travel time per vehicle at different seeds

Avg. T = Average of average travel times at different seeds

Table (C.12): Travel time (sec.) per vehicle on a 3000 ft freeway segment of Type II junction (5R+1AR+5G+1AR)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			T	Avg. T	T	Avg. T	T	Avg. T	T	Avg. T
Freeway volume (vehicles / hour lane)	500	19	31.8	31.8	31.8	31.8	31.7	31.7	31.8	31.8
		47	31.7		31.7		31.7		31.7	
		75	31.7		31.7		31.7		31.8	
		103	31.8		31.8		31.8		31.8	
		131	31.8		31.8		31.8		31.8	
	750	19	32.3	32.3	32.4	32.6	32.3	32.3	32.4	32.3
		47	32.4		32.2		32.2		32.2	
		75	32.1		33.6		32.2		32.2	
		103	32.3		32.3		32.4		32.4	
		131	32.3		32.3		32.3		32.2	
	1000	19	33.8	33.7	34.0	33.7	33.7	33.7	34.5	33.8
		47	33.7		33.6		34.0		33.5	
		75	33.4		33.6		33.4		33.5	
		103	34.1		33.7		33.8		33.8	
		131	33.7		33.5		33.6		33.5	
	1250	19	72.4	69.6	82.6	82.3	84.6	81.8	82.6	79.9
		47	78.8		86.3		84.0		83.0	
		75	72.2		84.8		82.5		83.0	
		103	67.2		78.8		78.4		77.8	
		131	57.2		79.1		79.4		73.1	
	1500	19	106.8	106.8	107.6	108.0	108.9	108.5	108.1	108.1
		47	107.3		108.4		109.2		107.3	
		75	107.3		108.0		109.2		109.2	
		103	106.7		107.4		107.2		107.8	
		131	106.0		108.7		108.2		108.1	
	1750	19	106.3	106.3	107.7	107.6	108.6	107.5	107.9	107.6
		47	105.9		107.7		107.2		107.0	
		75	108.1		107.8		107.9		108.4	
		103	106.7		107.9		106.5		107.1	
		131	104.6		106.8		107.5		107.8	
	2000	19	108.6	107.2	107.3	107.1	108.6	107.6	106.5	107.9
		47	105.8		107.5		105.1		106.5	
		75	108.2		108.7		109.6		109.1	
		103	106.6		106.0		107.1		109.0	
		131	106.6		105.8		107.6		108.3	

T = Average travel time per vehicle at different seeds

Avg. T = Average of average travel times at different seeds

Table (C.13): Average speed (mph) at the ramp influence area of Type III junction No ramp metering										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			S	Avg. S	S	Avg. S	S	Avg. S	S	Avg. S
Freeway volume (vehicles / hour lane)	500	19	58.9	58.6	57.9	57.8	57.3	57.4	57.4	57.5
		47	58.4		57.8		57.6		57.5	
		75	58.9		57.8		57.3		57.6	
		103	58.8		57.6		57.3		57.6	
		131	58.0		57.9		57.7		57.4	
	750	19	58.6	58.2	57.4	57.7	56.9	57.0	56.6	56.9
		47	58.0		57.9		57.2		56.9	
		75	57.7		58.0		57.0		57.2	
		103	58.1		57.8		57.2		57.0	
		131	58.5		57.3		56.9		56.9	
	1000	19	57.3	57.7	56.8	56.9	55.8	56.2	55.5	55.8
		47	57.8		56.7		56.3		55.8	
		75	57.8		57.4		56.4		56.0	
		103	57.7		56.8		56.4		55.7	
		131	57.8		56.9		56.3		55.9	
	1250	19	53.7	52.1	45.2	36.7	19.5	23.2	18.4	21.5
		47	50.2		34.0		18.4		18.0	
		75	52.7		35.0		26.1		22.9	
		103	49.0		33.4		28.9		27.6	
		131	54.9		36.0		23.3		20.7	
	1500	19	28.2	28.0	20.3	20.6	17.5	17.7	18.0	17.8
		47	28.3		20.8		17.6		17.5	
		75	27.0		20.8		18.0		18.1	
		103	28.0		20.7		17.8		17.5	
		131	28.7		20.6		17.7		17.7	
	1750	19	27.8	27.9	20.9	20.4	17.8	17.8	17.6	17.6
		47	28.2		20.5		18.2		17.4	
		75	27.3		20.1		17.6		17.3	
		103	28.4		20.0		17.7		17.6	
		131	27.8		20.5		17.7		17.9	
	2000	19	28.2	28.0	20.4	20.6	17.9	17.7	18.2	17.8
		47	27.9		20.9		17.7		17.7	
		75	27.4		20.5		17.6		17.7	
		103	28.6		20.6		17.8		17.6	
		131	27.9		20.8		17.6		17.8	

S = Average speed at different seeds
Avg. S = Average of average speeds at different seeds

Table (C.14): Average speed (mph) at the ramp influence area of Type III junction (2R + 2G)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			S	Avg. S	S	Avg. S	S	Avg. S	S	Avg. S
Freeway volume (vehicles / hour lane)	500	19	58.3	58.6	58.0	58.0	57.3	57.4	57.3	57.2
		47	59.0		58.2		57.6		56.6	
		75	58.5		58.1		57.8		57.5	
		103	58.9		57.7		56.8		57.4	
		131	58.3		58.1		57.5		57.4	
	750	19	58.1	58.3	57.5	57.7	56.8	57.1	57.0	57.0
		47	58.5		57.5		57.2		57.0	
		75	58.5		57.9		57.3		57.1	
		103	58.2		57.7		57.0		57.0	
		131	58.4		57.7		57.2		57.0	
	1000	19	57.7	57.7	56.6	56.8	55.9	56.1	55.9	55.9
		47	57.6		56.7		56.1		55.9	
		75	57.8		57.0		56.2		56.1	
		103	57.7		56.9		56.3		55.7	
		131	57.7		57.0		55.9		55.9	
	1250	19	55.0	52.2	45.0	37.5	18.9	22.3	17.7	19.1
		47	48.4		34.4		18.0		18.2	
		75	52.3		34.6		24.5		19.0	
		103	50.8		33.9		28.3		22.5	
		131	54.5		39.6		22.0		18.1	
	1500	19	28.0	27.8	20.3	20.6	17.8	17.8	17.8	17.6
		47	28.7		21.0		18.1		17.7	
		75	27.3		20.7		17.9		17.4	
		103	28.0		20.3		17.5		17.5	
		131	27.2		20.9		17.7		17.6	
	1750	19	28.5	28.0	20.5	20.5	17.8	17.9	17.6	17.6
		47	27.7		20.8		17.8		17.7	
		75	27.3		20.3		18.1		17.5	
		103	28.0		20.5		18.2		18.1	
		131	28.5		20.6		17.7		17.3	
	2000	19	27.8	27.8	20.5	20.7	17.3	17.6	17.9	17.6
		47	28.3		21.3		17.7		17.9	
		75	27.5		20.7		17.8		17.4	
		103	27.7		20.3		17.4		17.3	
		131	27.9		20.6		17.9		17.4	

S = Average speed at different seeds
Avg. S = Average of average speeds at different seeds

Table (C.15): Average speed (mph) at the influence area of Type III junction (4R+4G)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			S	Avg. S	S	Avg. S	S	Avg. S	S	Avg. S
Freeway volume (vehicles / hour lane)	500	19	59.0	58.5	57.6	57.9	57.4	57.6	59.0	58.5
		47	58.4		58.1		57.7		58.4	
		75	58.1		58.3		57.8		58.1	
		103	58.5		58.0		57.5		58.5	
		131	58.6		57.7		57.4		58.6	
	750	19	58.4	58.4	57.5	57.6	57.0	57.1	56.8	57.0
		47	58.0		57.8		57.2		56.8	
		75	58.4		57.5		57.6		57.2	
		103	58.8		57.7		56.9		57.2	
		131	58.6		57.6		56.9		57.0	
	1000	19	57.7	57.6	56.6	56.8	56.1	56.2	55.6	56.0
		47	57.5		56.7		56.0		55.9	
		75	57.7		57.0		56.6		56.3	
		103	57.4		56.7		56.0		56.1	
		131	57.9		57.1		56.4		56.1	
	1250	19	54.0	53.1	46.0	37.3	19.1	22.2	18.1	19.9
		47	52.3		33.2		17.9		18.0	
		75	50.5		34.2		23.8		20.9	
		103	54.1		33.0		27.0		23.8	
		131	54.8		40.0		23.0		18.9	
	1500	19	27.5	27.8	20.4	20.5	17.5	17.8	17.1	17.4
		47	28.6		21.0		17.7		17.6	
		75	27.2		20.4		17.9		17.4	
		103	28.2		20.0		18.1		17.6	
		131	27.7		20.9		17.8		17.5	
	1750	19	27.7	27.8	20.4	20.6	17.4	17.8	17.7	17.5
		47	28.2		21.4		17.9		17.6	
		75	27.7		20.5		17.9		17.2	
		103	27.6		20.4		17.8		17.5	
		131	27.8		20.3		17.9		17.6	
	2000	19	28.0	28.3	20.4	20.7	17.6	17.8	17.6	17.6
		47	29.3		20.9		17.9		17.5	
		75	27.4		20.8		18.0		17.5	
		103	28.4		20.5		18.0		17.9	
		131	28.4		20.7		17.6		17.7	

S = Average speed at different seeds

Avg. S = Average of average speeds at different seeds

Table (C.16): Average speed (mph) at the ramp influence area of Type III junction (4R+2G)										
			Ramp volume (vehicles / hour lane)							
			400		600		800		1000	
		Seed No.	S	Avg. S	S	Avg. S	S	Avg. S	S	Avg. S
Freeway volume (vehicles / hour lane)	500	19	58.4	58.4	57.2	57.9	57.9	57.9	58.2	58.0
		47	58.5		57.9		58.0		58.1	
		75	58.5		58.1		57.6		57.7	
		103	58.8		58.0		58.1		58.1	
		131	57.9		58.1		58.1		57.8	
	750	19	58.4	58.3	57.1	57.6	56.7	57.5	56.6	57.5
		47	58.7		57.6		57.5		57.5	
		75	58.1		57.8		57.7		57.8	
		103	58.3		58.1		57.9		57.9	
		131	58.2		57.4		57.7		57.6	
	1000	19	57.5	57.3	56.6	56.7	56.7	56.8	56.8	56.8
		47	57.1		56.5		56.6		56.9	
		75	57.4		56.9		57.1		56.5	
		103	57.5		56.8		56.7		56.6	
		131	56.8		56.6		56.9		57.0	
	1250	19	54.2	51.7	48.8	40.9	48.5	38.0	46.1	40.9
		47	48.6		33.6		30.4		47.1	
		75	53.2		36.1		34.5		34.3	
		103	48.3		38.4		36.9		39.2	
		131	54.2		47.6		39.8		37.6	
	1500	19	27.7	28.2	21.5	21.5	20.7	21.0	21.0	21.3
		47	28.2		21.6		21.2		20.9	
		75	27.7		21.0		21.0		21.7	
		103	28.4		21.5		21.0		21.2	
		131	29.2		21.8		21.3		21.5	
	1750	19	28.5	28.2	21.7	21.4	21.1	21.1	21.2	21.2
		47	28.8		21.7		20.9		21.2	
		75	27.7		20.9		20.8		21.1	
		103	28.1		21.1		21.2		21.3	
		131	27.9		21.5		21.7		21.2	
	2000	19	28.6	28.3	21.3	21.6	21.0	21.2	20.9	21.1
		47	28.4		21.9		21.0		21.4	
		75	27.6		21.3		21.4		21.2	
		103	28.2		21.7		21.4		21.1	
		131	28.6		21.6		21.3		20.9	

S = Average speed at different seeds

Avg. S = Average of average speeds at different seeds

Table (C.17): Travel time (sec.) per vehicle on a 3000 ft freeway segment of Type III junction No ramp metering										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			T	Avg. T	T	Avg. T	T	Avg. T	T	Avg. T
Freeway volume (vehicles / hour lane)	500	19	31.3	31.3	31.5	31.4	31.6	31.5	31.6	31.6
		47	31.3		31.3		31.5		31.5	
		75	31.2		31.4		31.5		31.5	
		103	31.3		31.4		31.5		31.6	
		131	31.3		31.5		31.6		31.6	
	750	19	31.8	31.7	32.0	31.9	32.1	32.1	32.2	32.1
		47	31.8		32.0		32.1		32.2	
		75	31.6		31.8		32.0		31.9	
		103	31.7		31.9		32.1		32.0	
		131	31.8		31.9		32.1		32.2	
	1000	19	32.8	32.7	33.3	33.0	34.0	33.7	34.3	33.9
		47	32.8		33.1		34.0		34.0	
		75	32.8		32.8		33.3		33.8	
		103	32.6		33.0		33.7		33.9	
		131	32.6		33.0		33.3		33.7	
	1250	19	38.7	40.3	46.4	52.0	80.5	76.1	86.7	80.8
		47	42.7		55.9		93.3		95.3	
		75	40.9		53.1		69.4		74.3	
		103	42.2		52.4		62.7		65.2	
		131	37.2		52.4		74.6		82.5	
	1500	19	78.9	79.0	92.4	91.7	100.4	99.9	99.9	100.4
		47	78.4		90.5		99.6		100.6	
		75	80.5		92.2		99.7		100.0	
		103	78.9		91.4		99.2		100.6	
		131	78.4		92.2		100.4		100.7	
	1750	19	79.6	79.3	91.9	92.0	100.0	100.0	100.3	100.7
		47	78.8		90.9		99.8		100.9	
		75	80.5		92.7		100.6		101.0	
		103	78.3		92.2		99.7		101.1	
		131	79.4		92.1		99.9		100.4	
	2000	19	79.2	79.2	92.1	91.6	99.7	100.1	100.4	100.3
		47	79.5		90.8		99.9		100.0	
		75	80.1		92.0		100.4		100.5	
		103	77.9		91.7		100.0		100.4	
		131	79.2		91.5		100.3		100.0	

T = Average travel time per vehicle at different seeds
Avg. T = Average of average travel times at different seeds

Table (C.18): Travel time (sec.) per vehicle on a 3000 ft freeway segment of Type III junction (2G+2R)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			T	Avg. T	T	Avg. T	T	Avg. T	T	Avg. T
Freeway volume (vehicles / hour lane)	500	19	31.3	31.3	31.5	31.4	31.6	31.5	31.6	31.5
		47	31.2		31.4		31.5		31.5	
		75	31.2		31.4		31.5		31.5	
		103	31.3		31.4		31.5		31.5	
		131	31.4		31.5		31.6		31.6	
	750	19	31.8	31.7	32.0	31.9	32.1	32.1	32.2	32.1
		47	31.8		31.9		32.2		32.2	
		75	31.7		31.8		31.9		32.0	
		103	31.7		31.9		32.0		32.0	
		131	31.7		31.9		32.1		32.0	
	1000	19	32.8	32.8	33.3	33.2	34.4	33.7	34.5	34.1
		47	32.6		33.4		33.8		34.2	
		75	32.7		33.0		33.2		34.0	
		103	32.9		33.1		33.8		34.4	
		131	32.8		33.0		33.4		33.6	
	1250	19	38.9	40.1	47.6	52.4	84.5	78.5	96.1	89.3
		47	41.9		56.7		95.0		97.9	
		75	40.5		54.0		71.4		85.4	
		103	41.1		53.8		63.5		74.7	
		131	38.1		49.8		78.2		92.3	
	1500	19	79.1	79.2	92.5	91.5	100.1	100.1	100.9	101.1
		47	77.9		89.8		99.5		100.5	
		75	80.4		91.9		99.8		101.4	
		103	79.0		91.9		100.8		101.4	
		131	79.5		91.2		100.4		101.5	
	1750	19	78.3	78.9	91.9	91.7	100.5	100.4	101.9	101.3
		47	78.9		90.9		100.1		100.8	
		75	80.1		92.4		100.4		101.7	
		103	79.0		91.7		99.9		100.5	
		131	78.0		91.4		101.0		101.7	
	2000	19	79.6	79.1	92.3	91.4	100.9	100.4	101.2	101.2
		47	78.4		89.8		100.0		100.6	
		75	79.8		91.4		100.3		101.3	
		103	78.6		91.5		100.7		101.5	
		131	79.0		91.8		100.0		101.6	

T = Average travel time per vehicle at different seeds
Avg. T = Average of average travel times at different seeds

Table (C.19): Travel time (sec.) per vehicle on a 3000 ft freeway segment of Type III junction (4G+4R)

Freeway volume (vehicles / hour lane)		Seed No.	Ramp volume (vehicles / hour lane) 400 T	400 Avg. T	600 T	600 Avg. T	800 T	800 Avg. T	1000 T	1000 Avg. T
	500	19	31.3	31.3	31.5	31.4	31.6	31.5	31.3	31.3
		47	31.3		31.4		31.5		31.3	
		75	31.2		31.4		31.5		31.2	
		103	31.3		31.4		31.5		31.3	
		131	31.3		31.5		31.6		31.3	
	750	19	31.8	31.7	32.0	31.9	32.1	32.0	32.2	32.1
		47	31.8		31.9		32.1		32.2	
		75	31.7		31.9		31.9		32.0	
		103	31.7		31.9		32.0		32.0	
		131	31.7		31.9		32.1		32.1	
	1000	19	32.8	32.7	33.5	33.2	34.3	33.7	34.1	33.8
		47	32.6		33.1		33.8		33.8	
		75	32.8		33.2		33.5		34.0	
		103	32.7		33.2		33.8		33.6	
		131	32.6		32.9		33.3		33.3	
	1250	19	39.9	40.0	47.5	52.6	84.7	78.6	94.7	86.5
		47	40.9		57.7		95.6		98.1	
		75	42.3		55.4		72.1		79.1	
		103	38.7		52.5		65.3		71.7	
		131	38.4		49.8		75.4		89.1	
	1500	19	79.8	79.2	92.3	91.6	100.6	100.2	102.1	101.6
		47	77.9		90.0		100.0		101.2	
		75	80.6		92.2		100.5		101.8	
		103	78.4		92.0		99.6		101.3	
		131	79.5		91.4		100.5		101.4	
	1750	19	79.5	79.2	92.5	91.8	100.7	100.2	101.5	101.3
		47	78.3		90.1		99.8		101.0	
		75	79.8		92.2		100.6		101.7	
		103	78.9		92.2		99.6		101.1	
		131	79.4		91.8		100.5		101.3	
	2000	19	79.0	78.7	92.3	91.5	101.0	100.1	101.1	101.4
		47	77.4		90.4		100.1		101.9	
		75	80.3		91.4		100.2		101.5	
		103	78.1		91.8		99.3		101.2	
		131	78.6		91.6		99.8		101.1	

T = Average travel time per vehicle at different seeds

Avg. T = Average of average travel times at different seeds

<table>
<tr><th colspan="11">Table (C.20): Travel time (sec.) per vehicle on a 3000 ft freeway of Type III junction (2G+4R)</th></tr>
<tr><th colspan="2" rowspan="3"></th><th rowspan="3">Seed No.</th><th colspan="8">Ramp volume (vehicles / hour lane)</th></tr>
<tr><th colspan="2">400</th><th colspan="2">600</th><th colspan="2">800</th><th colspan="2">1000</th></tr>
<tr><th>T</th><th>Avg. T</th><th>T</th><th>Avg. T</th><th>T</th><th>Avg. T</th><th>T</th><th>Avg. T</th></tr>
<tr><td rowspan="35">Freeway volume (vehicles / hour lane)</td><td rowspan="5">500</td><td>19</td><td>31.4</td><td rowspan="5">31.4</td><td>31.5</td><td rowspan="5">31.5</td><td>31.6</td><td rowspan="5">31.5</td><td>31.6</td><td rowspan="5">31.5</td></tr>
<tr><td>47</td><td>31.3</td><td>31.5</td><td>31.5</td><td>31.5</td></tr>
<tr><td>75</td><td>31.3</td><td>31.5</td><td>31.5</td><td>31.5</td></tr>
<tr><td>103</td><td>31.4</td><td>31.5</td><td>31.5</td><td>31.5</td></tr>
<tr><td>131</td><td>31.4</td><td>31.5</td><td>31.5</td><td>31.5</td></tr>
<tr><td rowspan="5">750</td><td>19</td><td>31.8</td><td rowspan="5">31.8</td><td>32.1</td><td rowspan="5">32.0</td><td>32.1</td><td rowspan="5">32.0</td><td>32.1</td><td rowspan="5">32.0</td></tr>
<tr><td>47</td><td>31.8</td><td>32.0</td><td>32.0</td><td>32.0</td></tr>
<tr><td>75</td><td>31.7</td><td>31.9</td><td>31.9</td><td>31.9</td></tr>
<tr><td>103</td><td>31.7</td><td>32.0</td><td>31.9</td><td>31.9</td></tr>
<tr><td>131</td><td>31.8</td><td>32.0</td><td>32.0</td><td>32.0</td></tr>
<tr><td rowspan="5">1000</td><td>19</td><td>32.9</td><td rowspan="5">32.8</td><td>33.4</td><td rowspan="5">33.1</td><td>33.2</td><td rowspan="5">33.1</td><td>33.4</td><td rowspan="5">33.3</td></tr>
<tr><td>47</td><td>32.7</td><td>33.2</td><td>33.1</td><td>33.3</td></tr>
<tr><td>75</td><td>32.9</td><td>33.2</td><td>33.2</td><td>33.5</td></tr>
<tr><td>103</td><td>32.8</td><td>32.9</td><td>33.1</td><td>33.4</td></tr>
<tr><td>131</td><td>32.6</td><td>32.9</td><td>33.1</td><td>33.1</td></tr>
<tr><td rowspan="5">1250</td><td>19</td><td>39.1</td><td rowspan="5">40.5</td><td>45.0</td><td rowspan="5">50.0</td><td>45.8</td><td rowspan="5">52.6</td><td>46.6</td><td rowspan="5">53.0</td></tr>
<tr><td>47</td><td>43.0</td><td>57.9</td><td>61.0</td><td>63.0</td></tr>
<tr><td>75</td><td>40.6</td><td>52.4</td><td>54.1</td><td>54.7</td></tr>
<tr><td>103</td><td>41.8</td><td>49.4</td><td>51.4</td><td>49.0</td></tr>
<tr><td>131</td><td>37.9</td><td>45.5</td><td>50.8</td><td>51.6</td></tr>
<tr><td rowspan="5">1500</td><td>19</td><td>79.7</td><td rowspan="5">79.1</td><td>91.9</td><td rowspan="5">91.6</td><td>92.1</td><td rowspan="5">92.1</td><td>92.7</td><td rowspan="5">92.3</td></tr>
<tr><td>47</td><td>79.0</td><td>91.2</td><td>92.2</td><td>92.3</td></tr>
<tr><td>75</td><td>80.3</td><td>92.4</td><td>92.3</td><td>92.3</td></tr>
<tr><td>103</td><td>78.4</td><td>90.7</td><td>92.1</td><td>92.0</td></tr>
<tr><td>131</td><td>78.0</td><td>91.9</td><td>91.9</td><td>92.0</td></tr>
<tr><td rowspan="5">1750</td><td>19</td><td>78.9</td><td rowspan="5">79.2</td><td>91.3</td><td rowspan="5">91.5</td><td>92.8</td><td rowspan="5">92.3</td><td>92.3</td><td rowspan="5">92.1</td></tr>
<tr><td>47</td><td>78.3</td><td>91.2</td><td>92.5</td><td>91.5</td></tr>
<tr><td>75</td><td>80.2</td><td>92.4</td><td>92.6</td><td>92.6</td></tr>
<tr><td>103</td><td>79.1</td><td>91.2</td><td>92.3</td><td>92.6</td></tr>
<tr><td>131</td><td>79.4</td><td>91.6</td><td>91.3</td><td>91.7</td></tr>
<tr><td rowspan="5">2000</td><td>19</td><td>78.9</td><td rowspan="5">79.1</td><td>91.1</td><td rowspan="5">91.6</td><td>92.8</td><td rowspan="5">92.4</td><td>91.9</td><td rowspan="5">92.3</td></tr>
<tr><td>47</td><td>78.8</td><td>91.8</td><td>92.6</td><td>92.0</td></tr>
<tr><td>75</td><td>80.4</td><td>92.4</td><td>92.3</td><td>92.9</td></tr>
<tr><td>103</td><td>79.0</td><td>91.1</td><td>92.6</td><td>92.1</td></tr>
<tr><td>131</td><td>78.6</td><td>91.6</td><td>91.7</td><td>92.8</td></tr>
<tr><td colspan="11">T = Average travel time per vehicle at different seeds
Avg. T = Average of average travel times at different seeds</td></tr>
</table>

APPENDIX D

Table (D.1): Average density (vehicles per mile per lane) at the ramp influence area of Type I junction - No ramp metering

			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			D	Avg. D	D	Avg. D	D	Avg. D	D	Avg. D
Freeway volume (vehicles / hour lane)	500	19	8.0	8.1	9.7	9.7	11.3	11.2	11.5	11.5
		47	8.2		9.8		11.3		11.6	
		75	8.2		9.7		11.2		11.6	
		103	8.0		9.7		11.2		11.7	
		131	7.9		9.4		10.8		11.3	
	750	19	10.5	10.6	12.3	12.3	13.8	13.8	14.1	14.1
		47	10.8		12.4		14.0		14.2	
		75	10.9		12.4		13.8		14.3	
		103	10.5		12.2		13.7		14.2	
		131	10.5		12.0		13.5		13.9	
	1000	19	13.3	13.4	15.1	15.0	16.9	16.8	17.1	17.3
		47	13.5		15.1		16.9		17.2	
		75	13.6		15.2		16.8		17.4	
		103	13.2		14.9		16.9		18.4	
		131	13.3		14.8		16.3		16.6	
	1250	19	16.7	16.5	18.8	18.9	28.2	26.7	29.2	28.4
		47	16.4		18.7		26.4		28.4	
		75	16.7		19.9		27.4		28.9	
		103	16.3		19.1		26.3		32.3	
		131	16.2		17.8		25.1		23.4	
	1500	19	18.5	18.8	25.9	24.6	62.2	49.7	72.1	83.6
		47	19.5		24.9		56.3		86.5	
		75	18.6		23.8		39.0		84.4	
		103	19.1		24.9		51.9		98.5	
		131	18.5		23.3		39.0		76.6	
	1750	19	19.3	19.0	25.6	24.4	63.7	51.0	86.1	82.7
		47	19.2		23.6		54.5		68.4	
		75	19.1		23.6		41.5		69.6	
		103	18.1		24.7		54.5		96.6	
		131	19.4		24.4		40.8		92.6	
	2000	19	19.6	19.0	24.3	24.5	62.1	50.3	89.9	90.4
		47	19.2		25.4		59.0		78.3	
		75	18.6		24.3		39.6		86.8	
		103	18.5		24.1		49.6		100.9	
		131	19.3		24.4		41.1		96.3	

D = Average density at different seeds
Avg. D = Average of average densities at different seeds

Table (D.2): Average density (vehicles per mile per lane) at the ramp influence area of Type I junction - (2R+1AR+2G+1AR)

Freeway volume (vehicles / hour lane)		Seed No.	Ramp volume (vehicles / hour lane): 400		600		800		1000	
			D	Avg. D	D	Avg. D	D	Avg. D	D	Avg. D
	500	19	8.0		9.6		9.6		9.6	
		47	8.2		9.8		9.8		9.8	
		75	8.2	8.1	9.8	9.6	9.8	9.7	9.8	9.7
		103	8.0		9.6		9.7		9.7	
		131	7.9		9.4		9.5		9.5	
	750	19	10.5		12.2		12.2		12.2	
		47	10.9		12.5		12.5		12.5	
		75	10.9	10.7	12.5	12.3	12.5	12.3	12.5	12.3
		103	10.6		12.1		12.3		12.3	
		131	10.6		12.1		12.2		12.2	
	1000	19	13.3		15.0		15.1		15.1	
		47	13.6		15.3		15.2		15.2	
		75	13.7	13.4	15.2	15.0	15.3	15.1	15.3	15.1
		103	13.2		14.8		15.0		15.0	
		131	13.3		14.9		14.9		14.9	
	1250	19	16.4		19.4		18.1		19.3	
		47	16.5		18.7		20.1		19.2	
		75	16.7	16.4	19.7	18.9	19.7	19.2	19.6	19.3
		103	16.1		18.2		19.4		19.3	
		131	16.3		18.3		18.8		19.2	
	1500	19	19.1		25.1		26.6		26.0	
		47	19.2		25.8		24.9		25.1	
		75	19.5	19.2	25.9	25.4	25.2	25.5	24.3	25.0
		103	18.5		24.8		26.3		24.9	
		131	19.5		25.2		24.4		24.5	
	1750	19	19.5		24.5		25.2		26.8	
		47	19.4		26.0		25.2		25.8	
		75	18.6	19.4	26.0	25.2	24.6	24.8	25.7	26.1
		103	19.8		23.9		24.5		26.6	
		131	19.9		25.6		24.3		25.6	
	2000	19	18.9		25.0		27.5		27.4	
		47	19.7		24.8		25.9		26.5	
		75	19.4	19.3	24.7	24.9	26.5	25.9	24.9	25.6
		103	18.9		25.3		25.0		24.3	
		131	19.8		24.7		24.4		25.0	

D = Average density at different seeds

Avg. D = Average of average densities at different seeds

Table (D.3): Average density (vehicles per mile per lane) at the ramp influence area of Type I junction - (5R+1AR+5G+1AR)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			D	Avg. D	D	Avg. D	D	Avg. D	D	Avg. D
Freeway volume (vehicles / hour lane)	500	19	8.0	8.1	9.5	9.6	9.3	9.6	9.6	9.7
		47	8.2		9.8		9.8		9.8	
		75	8.2		9.8		9.7		9.7	
		103	8.0		9.6		9.7		9.7	
		131	8.0		9.4		9.5		9.5	
	750	19	10.5	10.7	12.1	12.2	12.2	12.3	12.2	12.3
		47	10.9		12.4		12.4		12.4	
		75	10.9		12.5		12.5		12.5	
		103	10.6		12.1		12.3		12.3	
		131	10.6		12.1		12.1		12.1	
	1000	19	13.3	13.4	15.1	15.1	15.0	15.1	15.1	15.1
		47	13.6		15.3		15.3		15.3	
		75	13.6		15.3		15.2		15.2	
		103	13.2		14.8		14.9		14.9	
		131	13.3		14.8		14.9		14.9	
	1250	19	16.4	16.5	19.4	18.9	19.0	18.9	20.0	19.3
		47	16.7		18.4		19.0		19.4	
		75	16.8		19.3		18.8		20.0	
		103	16.1		18.5		18.8		18.8	
		131	16.3		18.7		18.7		18.1	
	1500	19	18.9	18.9	25.0	24.7	24.7	24.5	24.6	24.5
		47	18.8		25.3		24.1		24.2	
		75	18.2		24.0		25.2		24.5	
		103	19.5		24.9		24.0		25.8	
		131	19.3		24.1		24.7		23.3	
	1750	19	18.9	19.4	25.9	24.4	25.9	24.4	25.3	25.2
		47	19.1		23.2		23.4		24.8	
		75	19.5		25.0		24.5		25.2	
		103	19.2		24.2		25.0		25.8	
		131	20.4		23.9		23.4		24.7	
	2000	19	19.8	19.4	25.8	24.9	26.5	25.1	25.9	25.1
		47	20.2		23.8		24.3		25.3	
		75	18.3		25.4		24.5		24.2	
		103	19.3		24.1		25.9		25.1	
		131	19.2		25.2		24.5		24.8	

D = Average density at different seeds

Avg. D = Average of average densities at different seeds

Table (D.4): Average density (vehicles per mile per lane) comparison at the ramp influence area of Type I junction

		Signal design	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	No ramp meter	8.1	9.7	11.2	11.5
		2R + 1AR + 2G + 1AR	8.1	9.6	9.7	9.7
		5R + 1AR + 5G + 1AR	8.1	9.6	9.6	9.7
	750	No ramp meter	10.6	12.3	13.8	14.1
		2R + 1AR + 2G + 1AR	10.7	12.3	12.3	12.3
		5R + 1AR + 5G + 1AR	10.7	12.2	12.3	12.3
	1000	No ramp meter	13.4	15.0	16.8	17.3
		2R + 1AR + 2G + 1AR	13.4	15.0	15.1	15.1
		5R + 1AR + 5G + 1AR	13.4	15.1	15.1	15.1
	1250	No ramp meter	16.5	18.9	26.7	28.4
		2R + 1AR + 2G + 1AR	16.4	18.9	19.2	19.3
		5R + 1AR + 5G + 1AR	16.5	18.9	18.9	19.3
	1500	No ramp meter	18.8	24.6	49.7	83.6
		2R + 1AR + 2G + 1AR	19.2	25.4	25.5	25.0
		5R + 1AR + 5G + 1AR	18.9	24.7	24.5	24.5
	1750	No ramp meter	19.0	24.4	51.0	82.7
		2R + 1AR + 2G + 1AR	19.4	25.2	24.8	26.1
		5R + 1AR + 5G + 1AR	19.4	24.4	24.4	25.2
	2000	No ramp meter	19.0	24.5	50.3	90.4
		2R + 1AR + 2G + 1AR	19.3	24.9	25.9	25.6
		5R + 1AR + 5G + 1AR	19.4	24.9	25.1	25.1

Table (D.5): Average density (vehicles per mile per lane) at the ramp influence area of Type II junction - No ramp metering										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			D	Avg. D	D	Avg. D	D	Avg. D	D	Avg. D
Freeway volume (vehicles / hour lane)	500	19	9.7	9.7	10.0	10.0	10.0	10.0	10.0	10.1
		47	9.9		10.1		10.1		10.2	
		75	9.7		10.1		10.1		10.1	
		103	9.8		10.0		10.0		10.0	
		131	9.5		10.0		10.0		10.1	
	750	19	12.8	12.9	13.1	13.3	13.1	13.2	13.1	13.3
		47	13.1		13.4		13.3		13.4	
		75	12.9		13.3		13.3		13.3	
		103	13.0		13.3		13.3		13.3	
		131	12.6		13.2		13.2		13.2	
	1000	19	16.3	16.3	16.7	16.6	16.8	16.6	16.9	16.6
		47	16.2		16.6		16.5		16.5	
		75	16.2		16.6		16.6		16.7	
		103	16.3		16.6		16.6		16.5	
		131	16.3		16.4		16.4		16.4	
	1250	19	67.8	66.8	74.0	74.6	80.3	75.8	77.0	75.8
		47	81.4		82.4		81.5		83.9	
		75	72.2		82.3		82.1		79.9	
		103	63.9		67.6		68.8		69.7	
		131	48.8		66.6		66.2		68.5	
	1500	19	107.6	107.3	107.8	107.8	108.5	107.6	107.9	107.7
		47	107.3		107.3		107.0		107.5	
		75	108.1		108.1		107.6		107.9	
		103	106.8		107.9		106.6		107.4	
		131	106.8		107.7		108.1		107.9	
	1750	19	107.2	107.2	107.9	107.9	107.9	107.9	107.9	108.0
		47	107.2		108.0		107.9		108.0	
		75	107.0		107.5		107.6		108.4	
		103	106.9		107.6		107.8		108.0	
		131	107.5		108.5		108.1		107.8	
	2000	19	107.9	107.2	108.2	107.8	108.1	107.7	108.2	108.1
		47	106.9		107.4		106.5		108.6	
		75	107.8		107.9		108.0		107.6	
		103	106.3		107.6		108.3		107.8	
		131	107.3		107.9		107.6		108.3	

D = Average density at different seeds

Avg. D = Average of average densities at different seeds

Table (D.6): Average density (vehicle per mile per lane) at the ramp influence area of Type II junction - (2R+1AR+2G+1AR)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			D	Avg. D	D	Avg. D	D	Avg. D	D	Avg. D
Freeway volume (vehicles / hour lane)	500	19	10.5	10.5	10.5	10.5	10.6	10.6	10.5	10.5
		47	10.6		10.6		10.6		10.6	
		75	10.6		10.6		10.6		10.6	
		103	10.5		10.5		10.5		10.5	
		131	10.5		10.5		10.5		10.5	
	750	19	12.8	12.9	13.6	13.7	13.6	13.7	13.6	13.7
		47	13.1		13.8		13.8		13.8	
		75	12.9		13.8		13.8		13.8	
		103	13.0		13.8		13.7		13.7	
		131	12.6		13.7		13.7		13.6	
	1000	19	16.9	16.3	17.4	17.1	17.7	17.2	17.4	17.1
		47	16.4		17.0		17.0		17.1	
		75	16.2		17.1		17.1		17.1	
		103	16.3		17.1		17.1		17.2	
		131	15.8		17.1		16.9		16.9	
	1250	19	72.0	70.5	84.6	89.0	85.6	89.0	86.0	89.9
		47	86.1		94.6		95.3		96.2	
		75	77.6		97.5		98.9		95.6	
		103	70.3		84.3		82.0		83.2	
		131	46.3		83.9		83.3		88.6	
	1500	19	108.7	108.3	109.4	108.9	108.8	109.3	109.8	109.0
		47	107.5		108.4		109.1		109.3	
		75	108.5		108.4		109.3		108.7	
		103	108.5		109.0		109.7		108.9	
		131	108.4		109.4		109.5		108.5	
	1750	19	108.7	108.3	109.3	109.2	109.0	109.0	109.8	109.0
		47	108.7		109.8		108.9		108.8	
		75	108.3		108.4		109.2		109.0	
		103	108.3		109.4		108.8		108.5	
		131	107.6		109.1		109.2		108.8	
	2000	19	107.7	108.1	108.6	109.0	109.1	109.0	109.1	109.1
		47	108.3		109.1		108.3		108.7	
		75	108.1		109.4		109.1		109.4	
		103	108.4		108.9		109.2		109.0	
		131	108.0		108.8		109.3		109.2	

D = Average density at different seeds

Avg. D = Average of average densities at different seeds

Table (D.7): Average density (vehicle per mile per lane) at the ramp influence area of Type II junction - (5R+1AR+5G+1AR)										
			Ramp volumes (vehicle / hour lane)							
		Seed No.	400		600		800		1000	
			D	Avg. D	D	Avg. D	D	Avg. D	D	Avg. D
Freeway volume (vehicles / hour lane)	500	19	9.7	9.7	10.5	10.5	10.5	10.5	10.5	10.5
		47	9.9		10.6		10.6		10.6	
		75	9.7		10.6		10.6		10.6	
		103	9.8		10.5		10.5		10.5	
		131	9.5		10.5		10.5		10.5	
	750	19	12.8	12.9	13.6	13.7	13.6	13.7	13.6	13.7
		47	13.1		13.8		13.8		13.8	
		75	12.9		13.8		13.8		13.8	
		103	13.0		13.7		13.8		13.7	
		131	12.6		13.7		13.7		13.6	
	1000	19	16.4	16.3	17.3	17.1	17.2	17.1	17.4	17.1
		47	16.3		17.0		17.2		16.9	
		75	16.2		17.1		17.1		17.1	
		103	16.4		17.1		17.1		17.1	
		131	16.3		16.9		16.9		16.9	
	1250	19	68.0	70.0	83.2	89.4	84.9	89.8	82.8	87.3
		47	88.3		96.0		96.9		96.4	
		75	77.4		96.2		95.3		94.0	
		103	69.8		86.1		86.0		83.3	
		131	46.3		85.4		85.8		80.0	
	1500	19	108.2	108.1	108.9	109.1	109.2	108.9	108.9	109.0
		47	108.0		108.7		108.7		108.6	
		75	108.5		109.4		108.9		109.4	
		103	107.7		108.7		108.5		108.9	
		131	108.1		109.6		109.0		109.3	
	1750	19	108.8	108.5	109.0	108.7	109.0	108.7	109.1	109.2
		47	108.3		108.6		108.6		109.3	
		75	108.7		108.7		109.1		109.9	
		103	108.3		109.0		108.3		108.8	
		131	108.4		108.1		108.7		108.9	
	2000	19	109.8	108.8	109.1	108.9	109.0	109.0	109.5	109.0
		47	108.6		109.0		108.3		108.9	
		75	109.3		109.2		109.1		108.7	
		103	108.2		108.5		109.4		109.0	
		131	108.3		108.7		109.1		108.8	

D = Average density at different seeds

Avg. D = Average of average densities at different seeds

Table (D.8): Average density (vehicle per mile per lane) comparison at the ramp freeway influence area of Type II junction						
		Signal design	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	No ramp meter	9.7	10.0	10.0	10.1
		2R + 1AR + 2G + 1AR	10.5	10.5	10.6	10.5
		5R + 1AR + 5G + 1AR	9.7	10.5	10.5	10.5
	750	No ramp meter	12.9	13.3	13.2	13.3
		2R + 1AR + 2G + 1AR	12.9	13.7	13.7	13.7
		5R + 1AR + 5G + 1AR	12.9	13.7	13.7	13.7
	1000	No ramp meter	16.3	16.6	16.6	16.6
		2R + 1AR + 2G + 1AR	16.3	17.1	17.2	17.1
		5R + 1AR + 5G + 1AR	16.3	17.1	17.1	17.1
	1250	No ramp meter	66.8	74.6	75.8	75.8
		2R + 1AR + 2G + 1AR	70.5	89.0	89.0	89.9
		5R + 1AR + 5G + 1AR	70.0	89.4	89.8	87.3
	1500	No ramp meter	107.3	107.8	107.6	107.7
		2R + 1AR + 2G + 1AR	108.3	108.9	109.3	109.0
		5R + 1AR + 5G + 1AR	108.1	109.1	108.9	109.0
	1750	No ramp meter	107.2	107.9	107.9	108.0
		2R + 1AR + 2G + 1AR	108.3	109.2	109.0	109.0
		5R + 1AR + 5G + 1AR	108.5	108.7	108.7	109.2
	2000	No ramp meter	107.2	107.8	107.7	108.1
		2R + 1AR + 2G + 1AR	108.1	109.0	109.0	109.1
		5R + 1AR + 5G + 1AR	108.8	108.9	109.0	109.0

Table (D.9): Average density (vehicles per mile per lane) at the ramp influence area of Type III junction - No ramp metering										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			D	Avg. D	D	Avg. D	D	Avg. D	D	Avg. D
Freeway volume (vehicles / hour lane)	500	19	7.8	7.6	8.8	8.6	9.8	9.5	9.9	9.7
		47	7.8		8.7		9.6		9.9	
		75	7.5		8.5		9.3		9.6	
		103	7.5		8.5		9.4		9.6	
		131	7.6		8.5		9.4		9.7	
	750	19	10.7	10.6	11.8	11.7	12.8	12.6	12.9	12.8
		47	11.0		11.9		12.8		13.2	
		75	10.5		11.4		12.3		12.6	
		103	10.4		11.6		12.5		12.6	
		131	10.6		11.6		12.5		12.8	
	1000	19	13.9	14.0	15.0	15.0	16.1	16.0	16.3	16.3
		47	14.0		15.2		16.2		16.6	
		75	14.0		14.9		15.8		16.2	
		103	13.9		15.0		16.0		16.2	
		131	14.1		15.0		16.0		16.3	
	1250	19	18.2	19.2	24.2	32.7	73.5	62.2	79.1	72.3
		47	20.1		37.1		80.0		81.2	
		75	19.0		34.1		53.7		82.1	
		103	20.5		33.6		43.7		48.3	
		131	18.0		34.4		60.3		70.7	
	1500	19	51.9	52.6	71.8	71.1	82.4	82.0	81.4	82.2
		47	51.9		70.0		81.9		82.7	
		75	55.2		71.3		81.2		82.2	
		103	52.5		71.0		82.4		82.5	
		131	51.3		71.5		82.3		82.1	
	1750	19	53.3	52.9	71.3	71.7	82.6	82.0	82.6	82.7
		47	52.1		70.9		81.1		83.1	
		75	54.6		73.0		82.3		83.6	
		103	51.3		71.7		81.8		82.6	
		131	53.3		71.5		82.3		81.6	
	2000	19	52.5	52.7	72.0	71.0	82.6	82.3	81.6	82.3
		47	52.9		69.7		81.6		82.6	
		75	53.7		71.6		82.3		82.4	
		103	50.8		71.1		82.0		82.8	
		131	53.5		70.8		83.1		82.3	

D = Average density at different seeds

Avg. D = Average of average densities at different seeds

Table (D.10): Average density (vehicle per mile per lane) at the ramp influence area of Type III junction - (2R+2G)										
		Seed No.	Ramp volume (vehicles / hour lane)							
			400		600		800		1000	
			D	Avg. D	D	Avg. D	D	Avg. D	D	Avg. D
Freeway volume (vehicles / hour lane)	500	19	7.8	7.6	8.8	8.6	10.1	9.6	10.1	9.9
		47	7.7		8.7		9.5		10.1	
		75	7.5		8.5		9.3		9.8	
		103	7.5		8.5		9.5		9.8	
		131	7.6		8.5		9.4		9.9	
	750	19	10.7	10.6	11.8	11.7	12.8	12.6	13.1	13.0
		47	10.9		11.9		12.9		13.4	
		75	10.5		11.4		12.3		12.8	
		103	10.4		11.6		12.6		12.8	
		131	10.6		11.6		12.5		13.0	
	1000	19	13.9	14.0	15.0	15.0	16.2	16.1	16.5	16.6
		47	14.2		15.2		16.3		16.8	
		75	13.9		14.9		15.8		16.5	
		103	13.9		15.0		16.1		16.5	
		131	14.0		15.0		15.9		16.6	
	1250	19	17.9	19.1	24.4	32.0	74.1	64.3	80.4	75.4
		47	21.0		36.9		79.4		79.4	
		75	19.0		35.2		57.1		74.9	
		103	19.5		33.5		45.4		64.0	
		131	18.1		29.9		65.6		78.2	
	1500	19	52.3	52.9	72.1	70.7	80.9	80.4	80.3	80.7
		47	50.6		69.4		79.4		81.0	
		75	54.4		70.6		80.1		81.4	
		103	52.8		71.0		81.0		80.2	
		131	54.2		70.6		80.8		80.7	
	1750	19	51.2	52.2	72.2	71.1	80.4	79.7	80.4	80.9
		47	52.7		69.9		80.4		81.1	
		75	54.0		71.8		78.9		81.2	
		103	52.5		70.6		79.4		80.0	
		131	50.8		70.9		79.6		81.9	
	2000	19	53.3	52.7	71.2	70.9	82.0	81.1	79.9	81.2
		47	51.7		69.3		80.6		80.5	
		75	53.8		70.9		81.1		81.8	
		103	52.0		72.2		81.9		82.1	
		131	52.6		70.7		79.7		81.9	

D = Average density at different seeds
Avg. D = Average of average densities at different seeds

Table (D.11): Average density (vehicle per mile per lane) at the ramp influence area of Type III junction - (4R+4G)										
		Seed No.	Ramp volume (vehicles / hour lane)							
			400		600		800		1000	
			D	Avg. D	D	Avg. D	D	Avg. D	D	Avg. D
Freeway volume (vehicles / hour lane)	500	19	7.8	7.6	8.8	8.6	9.7	9.5	7.8	7.6
		47	7.7		8.7		9.6		7.7	
		75	7.5		8.5		9.3		7.5	
		103	7.5		8.5		9.4		7.5	
		131	7.6		8.5		9.4		7.6	
	750	19	10.7	10.6	11.8	11.6	12.7	12.6	13.0	12.9
		47	10.9		11.9		12.8		13.2	
		75	10.5		11.5		12.3		12.6	
		103	10.4		11.5		12.5		12.7	
		131	10.6		11.5		12.5		12.8	
	1000	19	13.9	14.0	15.0	15.0	16.1	16.0	16.4	16.4
		47	14.2		15.2		16.2		16.6	
		75	14.0		14.9		15.8		16.2	
		103	13.9		15.0		16.0		16.3	
		131	14.0		15.0		16.0		16.4	
	1250	19	18.2	18.6	23.7	32.3	73.2	63.9	79.6	72.9
		47	19.1		37.9		79.2		80.7	
		75	19.8		35.9		58.7		68.4	
		103	18.0		34.4		47.4		59.5	
		131	18.1		29.5		61.2		76.5	
	1500	19	53.6	53.0	71.7	70.8	81.1	80.7	81.9	81.4
		47	50.7		69.5		80.9		80.6	
		75	54.5		71.1		80.6		82.1	
		103	52.2		71.6		80.3		81.2	
		131	53.8		70.2		80.4		81.2	
	1750	19	53.0	52.6	70.8	70.5	80.8	80.2	79.7	80.9
		47	51.2		68.6		79.9		80.9	
		75	53.3		71.5		80.1		81.8	
		103	52.5		70.4		79.6		80.7	
		131	53.2		71.2		80.4		81.4	
	2000	19	52.4	52.0	71.6	70.7	80.9	80.4	81.3	80.9
		47	49.5		69.2		80.0		81.7	
		75	54.8		70.7		80.4		81.5	
		103	51.3		71.4		79.4		79.4	
		131	52.1		70.8		81.3		80.6	

D = Average density at different seeds
Avg. D = Average of average densities at different seeds

Table (D.12): Average density (vehicle per mile per lane) at the ramp influence area of Type III junction - (4R+2G)										
			Ramp volume (vehicles / hour lane)							
		Seed No.	400		600		800		1000	
			D	Avg. D	D	Avg. D	D	Avg. D	D	Avg. D
Freeway volume (vehicles / hour lane)	500	19	7.8	7.6	8.7	8.5	8.7	8.6	8.7	8.6
		47	7.7		8.7		8.7		8.8	
		75	7.6		8.4		8.5		8.5	
		103	7.5		8.4		8.4		8.4	
		131	7.6		8.5		8.6		8.6	
	750	19	10.7	10.6	11.7	11.6	11.7	11.6	11.7	11.6
		47	10.9		11.9		12.0		12.0	
		75	10.5		11.4		11.4		11.4	
		103	10.5		11.4		11.4		11.4	
		131	10.6		11.5		11.6		11.6	
	1000	19	13.9	14.0	15.0	15.0	15.0	15.0	15.0	15.1
		47	14.2		15.3		15.3		15.3	
		75	14.0		15.0		14.9		15.0	
		103	13.8		14.8		14.9		14.9	
		131	14.1		15.0		15.1		15.1	
	1250	19	18.1	19.4	21.4	29.2	21.8	32.7	23.4	29.8
		47	21.0		39.4		44.7		28.9	
		75	18.7		33.2		35.3		35.1	
		103	20.8		29.5		31.1		28.9	
		131	18.2		22.6		30.5		32.9	
	1500	19	53.4	52.5	71.3	71.0	73.3	72.5	72.9	72.1
		47	52.9		70.9		72.4		72.3	
		75	54.2		72.3		72.5		71.6	
		103	51.8		70.3		72.6		72.0	
		131	50.3		70.3		71.8		71.7	
	1750	19	52.1	52.8	70.5	71.5	72.4	72.2	72.4	72.4
		47	51.3		70.7		72.7		71.7	
		75	54.2		73.2		73.3		73.3	
		103	52.8		71.7		72.3		72.1	
		131	53.5		71.2		70.4		72.5	
	2000	19	51.6	52.7	71.6	71.1	72.3	71.9	72.0	72.2
		47	52.1		70.1		72.1		72.1	
		75	55.0		72.3		72.0		71.7	
		103	52.9		70.9		72.0		71.8	
		131	51.9		70.7		71.1		73.4	

D = Average density at different seeds

Avg. D = Average of average densities at different seeds

Table (D.13): Average density (vehicle per mile per lane) comparison at the ramp influence area of Type III junction

Freeway volume (vehicles / hour lane)		Signal design	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
	500	No ramp meter	7.6	8.6	9.5	9.7
		2R + 2G	7.6	8.6	9.6	9.9
		4R + 4G	7.6	8.6	9.5	7.6
		4R + 2G	7.6	8.5	8.6	8.6
	750	No ramp meter	10.6	11.7	12.6	12.8
		2R + 2G	10.6	11.7	12.6	13.0
		4R + 4G	10.6	11.6	12.6	12.9
		4R + 2G	10.6	11.6	11.6	11.6
	1000	No ramp meter	14.0	15.0	16.0	16.3
		2R + 2G	14.0	15.0	16.1	16.6
		4R + 4G	14.0	15.0	16.0	16.4
		4R + 2G	14.0	15.0	15.0	15.1
	1250	No ramp meter	19.2	32.7	62.2	72.3
		2R + 2G	19.1	32.0	64.3	75.4
		4R + 4G	18.6	32.3	63.9	72.9
		4R + 2G	19.4	29.2	32.7	29.8
	1500	No ramp meter	52.6	71.1	82.0	82.2
		2R + 2G	52.9	70.7	80.4	80.7
		4R + 4G	53.0	70.8	80.7	81.4
		4R + 2G	52.5	71.0	82.5	72.1
	1750	No ramp meter	52.9	71.7	82.0	82.7
		2R + 2G	52.2	71.1	79.7	80.9
		4R + 4G	52.6	70.5	80.2	80.9
		4R + 2G	52.8	71.5	72.2	72.4
	2000	No ramp meter	52.7	71.0	82.3	82.3
		2R + 2G	52.7	70.9	81.1	81.2
		4R + 4G	52.0	70.7	80.4	80.9
		4R + 2G	52.7	71.1	71.9	72.2

APPENDIX E

Table (E.1): Overall number of conflicts on a 3000 ft freeway segment of Type I junction No ramp metering						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	5	3	7	7
		47	5	10	16	11
		75	9	6	10	8
		103	5	9	11	9
		131	6	11	1	3
		average	6.0	7.8	9.0	7.6
	750	19	6	10	17	12
		47	18	25	18	16
		75	13	17	15	13
		103	14	19	12	11
		131	16	17	18	17
		average	13.4	17.6	16.0	13.8
	1000	19	18	58	106	102
		47	27	38	98	121
		75	27	63	109	153
		103	36	22	144	327
		131	14	23	51	33
		average	24.4	40.8	101.6	147.2
	1250	19	188	321	1864	1991
		47	154	375	1558	2077
		75	116	692	1762	2088
		103	195	470	1511	2338
		131	99	126	1541	1143
		average	150.4	396.8	1647.2	1927.4
	1500	19	350	1614	4898	5663
		47	689	1493	4401	6914
		75	411	1304	2948	6714
		103	568	1462	4008	7807
		131	381	1232	2920	6150
		average	479.8	1421.0	3835.0	6649.6
	1750	19	659	1731	4942	6708
		47	559	1279	4248	5388
		75	592	1250	3163	5529
		103	336	1476	4247	7689
		131	463	1461	3007	7342
		average	521.8	1439.4	3921.4	6531.2
	2000	19	525	1249	4657	6757
		47	555	1597	4676	6168
		75	399	1420	2924	6657
		103	423	1319	3968	7988
		131	614	1389	3066	7615
		average	503.2	1394.8	3858.2	7037

Table (E.2): Overall number of conflicts on a 3000 ft freeway segment of Type I junction (2R+1AR+2G+1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	10	13	19	10
		47	3	7	18	12
		75	5	7	8	17
		103	10	12	15	18
		131	11	9	19	15
		average	7.8	9.6	15.8	14.4
	750	19	12	18	15	16
		47	15	23	17	17
		75	13	20	17	18
		103	13	10	13	14
		131	19	13	14	17
		average	14.4	16.8	15.2	16.4
	1000	19	26	30	47	47
		47	27	63	31	31
		75	28	32	41	41
		103	28	29	53	53
		131	20	46	23	23
		average	25.8	40.0	39.0	39.0
	1250	19	105	481	563	772
		47	118	361	672	490
		75	116	575	494	494
		103	81	242	558	479
		131	112	295	360	500
		average	106.4	390.8	529.4	547.0
	1500	19	547	1497	1780	1612
		47	513	1666	1416	1593
		75	584	1540	1481	1302
		103	374	1361	1609	1444
		131	701	1666	1347	1485
		average	543.8	1546.0	1526.6	1487.2
	1750	19	564	1567	1417	1792
		47	511	1710	1549	1583
		75	394	1608	1405	1530
		103	724	1356	1382	1729
		131	686	1593	1331	1658
		average	575.8	1566.8	1416.8	1658.4
	2000	19	463	1524	1881	1918
		47	728	1484	1718	1812
		75	526	1473	1699	1463
		103	497	1599	1458	1400
		131	669	1484	1508	1416
		average	576.6	1512.8	1652.8	1601.8

Table (E.3): Overall number of conflicts on a 3000 ft freeway segment of Type I junction (5R+1AR+5G+1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	9	6	14	12
		47	8	14	19	18
		75	9	14	11	7
		103	12	15	12	13
		131	10	13	18	10
		average	9.6	12.4	14.8	12.0
	750	19	14	22	20	16
		47	20	15	18	13
		75	25	20	17	15
		103	17	9	14	17
		131	6	14	13	17
		average	16.4	16.0	16.4	15.6
	1000	19	26	69	23	35
		47	36	44	49	51
		75	24	55	43	24
		103	20	29	38	29
		131	21	21	38	32
		average	25.4	43.6	38.2	34.2
	1250	19	81	535	404	625
		47	187	206	412	494
		75	147	442	324	571
		103	85	354	380	332
		131	122	424	363	181
		average	124.4	392.2	376.6	440.6
	1500	19	421	1602	1489	1412
		47	439	1634	1472	1354
		75	272	1347	1543	1399
		103	535	1422	1411	1598
		131	492	1375	1542	1151
		average	431.8	1476.0	1491.4	1382.8
	1750	19	486	1624	1575	1509
		47	479	1150	1311	1552
		75	586	1554	1399	1582
		103	542	1445	1483	1630
		131	850	1293	1218	1448
		average	588.6	1413.2	1397.2	1544.2
	2000	19	626	1600	1710	1718
		47	785	1351	1305	1545
		75	275	1596	1493	1224
		103	623	1366	1667	1491
		131	561	1470	1423	1393
		average	574.0	1476.6	1519.6	1474.2

Table (E.4): Number of lane change conflicts on a 3000 ft freeway segment of Type I junction No ramp metering						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	3	3	5	5
		47	5	9	15	9
		75	7	3	5	5
		103	4	6	6	5
		131	6	8	0	2
		average	5.0	5.8	6.2	5.2
	750	19	5	8	8	10
		47	15	20	12	10
		75	9	10	12	8
		103	13	13	7	6
		131	13	8	14	15
		average	11.0	11.8	10.6	9.8
	1000	19	12	21	22	19
		47	15	22	23	23
		75	15	17	28	33
		103	23	16	17	51
		131	10	12	18	12
		average	15.0	17.6	21.6	27.6
	1250	19	37	44	166	90
		47	34	60	145	151
		75	47	73	130	178
		103	53	58	114	197
		131	42	36	135	99
		average	42.6	54.2	138.0	143.0
	1500	19	52	120	228	296
		47	62	95	246	237
		75	46	92	212	202
		103	71	107	210	250
		131	49	68	165	262
		average	56.0	96.4	212.2	249.4
	1750	19	57	103	222	218
		47	56	80	268	277
		75	50	85	195	284
		103	37	99	231	244
		131	47	102	218	233
		average	49.4	93.8	226.8	251.2
	2000	19	60	89	193	226
		47	69	112	259	238
		75	42	95	167	253
		103	46	94	194	253
		131	50	92	205	315
		average	53.4	96.4	203.6	257

Table (E.5): Number of lane change conflicts on a 3000 ft freeway segment of Type I junction (2R + 1AR + 2G +1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	10	8	16	9
		47	3	7	17	9
		75	5	7	7	16
		103	9	12	14	18
		131	11	8	15	14
		average	7.6	8.4	13.8	13.2
	750	19	9	17	10	15
		47	13	18	14	14
		75	11	19	15	14
		103	12	8	9	9
		131	15	11	12	15
		average	12.0	14.6	12.0	13.4
	1000	19	18	19	20	20
		47	18	24	21	21
		75	15	16	24	24
		103	22	19	16	16
		131	11	17	17	17
		average	16.8	19.0	19.6	19.6
	1250	19	43	80	54	78
		47	34	65	75	54
		75	46	72	64	71
		103	30	41	59	50
		131	49	63	51	59
		average	40.4	64.2	60.6	62.4
	1500	19	54	113	151	155
		47	44	122	117	113
		75	52	139	133	121
		103	45	98	125	114
		131	67	123	108	109
		average	52.4	119.0	126.8	122.4
	1750	19	50	110	110	131
		47	44	113	113	116
		75	50	126	126	104
		103	46	96	96	139
		131	86	102	102	130
		average	55.2	109.4	109.4	124.0
	2000	19	56	99	136	166
		47	62	111	131	138
		75	54	108	127	125
		103	51	127	125	100
		131	57	101	113	106
		average	56.0	109.2	126.4	127

Table (E.6): Number of lane change conflicts on a 3000 ft freeway segment of Type I junction (5R+1AR+2G+1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	9	4	13	11
		47	8	12	17	17
		75	8	13	10	6
		103	11	14	8	8
		131	9	13	16	9
		average	9.0	11.2	12.8	10.2
	750	19	13	16	16	15
		47	14	13	12	11
		75	20	16	13	13
		103	16	8	13	17
		131	6	10	7	14
		average	13.8	12.6	12.2	14.0
	1000	19	16	19	13	15
		47	17	19	20	18
		75	11	22	25	15
		103	16	18	21	14
		131	17	17	23	21
		average	15.4	19.0	20.4	16.6
	1250	19	39	70	50	63
		47	41	48	56	49
		75	48	53	59	73
		103	25	51	48	51
		131	56	51	64	42
		average	41.8	54.6	55.4	55.6
	1500	19	62	124	112	113
		47	51	116	101	121
		75	41	109	110	122
		103	64	102	119	118
		131	44	94	119	70
		average	52.4	109.0	112.2	108.8
	1750	19	46	124	115	120
		47	48	76	93	129
		75	69	131	85	129
		103	55	104	122	123
		131	68	112	101	122
		average	57.2	109.4	103.2	124.6
	2000	19	56	128	131	116
		47	73	105	135	102
		75	48	110	86	85
		103	58	107	144	103
		131	43	118	87	108
		average	55.6	113.6	116.6	102.8

Table (E.7): Number of rear end conflicts on a 3000 ft freeway segment of Type I junction No ramp metering						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	2	0	2	2
		47	0	1	1	2
		75	2	3	5	3
		103	1	3	5	4
		131	0	3	1	1
		average	1.0	2.0	2.8	2.4
	750	19	1	2	9	2
		47	3	5	6	6
		75	4	7	3	5
		103	1	6	5	5
		131	3	9	4	2
		average	2.4	5.8	5.4	4.0
	1000	19	6	37	84	83
		47	12	16	75	98
		75	12	46	81	120
		103	13	6	127	276
		131	4	11	33	21
		average	9.4	23.2	80.0	119.6
	1250	19	151	277	1698	1901
		47	120	315	1413	1926
		75	69	619	1632	1910
		103	142	412	1397	2141
		131	57	90	1406	1044
		average	107.8	342.6	1509.2	1784.4
	1500	19	298	1494	4670	5367
		47	627	1398	4155	6677
		75	365	1212	2736	6512
		103	497	1355	3798	7557
		131	332	1164	2755	5888
		average	423.8	1324.6	3622.8	6400.2
	1750	19	602	1628	4720	6490
		47	503	1199	3980	5111
		75	542	1165	2968	5245
		103	299	1377	4016	7445
		131	416	1359	2789	7109
		average	472.4	1345.6	3694.6	6280.0
	2000	19	465	1160	4464	6531
		47	486	1485	4417	5930
		75	357	1325	2757	6404
		103	377	1225	3774	7735
		131	564	1297	2861	7300
		average	449.8	1298.4	3654.6	6780

Table (E.8): Number of rear end conflicts on a 3000 ft freeway segment of Type I junction (2R+1AR+2G+1AR)

		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	0	5	3	1
		47	0	0	1	3
		75	0	0	1	1
		103	1	0	1	0
		131	0	1	4	1
		average	0.2	1.2	2.0	1.2
	750	19	3	1	5	1
		47	2	5	3	3
		75	2	1	2	4
		103	1	2	4	5
		131	4	2	2	2
		average	2.4	2.2	3.2	3.0
	1000	19	8	11	27	27
		47	9	39	10	10
		75	13	16	17	17
		103	6	10	37	37
		131	9	29	6	6
		average	9.0	21.0	19.4	19.4
	1250	19	62	401	509	694
		47	84	296	597	436
		75	70	503	430	423
		103	51	201	499	429
		131	63	232	309	441
		average	66.0	326.6	468.8	484.6
	1500	19	493	1384	1629	1457
		47	467	1544	1299	1480
		75	532	1401	1348	1181
		103	329	1263	1484	1330
		131	634	1543	1239	1376
		average	491.0	1427.0	1399.8	1364.8
	1750	19	514	1447	1307	1661
		47	467	1566	1436	1467
		75	344	1483	1279	1426
		103	678	1241	1286	1590
		131	600	1496	1229	1528
		average	520.6	1446.6	1307.4	1534.4
	2000	19	407	1425	1745	1752
		47	666	1373	1587	1674
		75	472	1365	1572	1338
		103	446	1472	1333	1300
		131	612	1383	1395	1310
		average	520.6	1403.6	1526.4	1474.8

Table (E.9): Number of rear end conflicts on a 3000 ft freeway segment of Type I junction (5R+1AR+5G+1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	0	2	1	1
		47	0	2	2	1
		75	1	1	1	1
		103	1	1	4	5
		131	1	0	2	1
		average	0.6	1.2	2.0	1.8
	750	19	1	6	4	1
		47	6	2	6	2
		75	5	4	4	2
		103	1	1	1	0
		131	0	4	6	3
		average	2.6	3.4	4.2	1.6
	1000	19	10	50	10	20
		47	19	25	29	33
		75	13	33	18	9
		103	4	11	17	15
		131	4	4	15	11
		average	10.0	24.6	17.8	17.6
	1250	19	42	465	354	562
		47	146	158	356	445
		75	99	389	265	498
		103	60	303	332	281
		131	66	373	299	139
		average	82.6	337.6	321.2	385.0
	1500	19	359	1478	1377	1299
		47	388	1518	1371	1233
		75	231	1238	1433	1277
		103	471	1320	1292	1480
		131	448	1281	1423	1081
		average	379.4	1367.0	1379.2	1274.0
	1750	19	440	1500	1460	1389
		47	431	1074	1218	1423
		75	517	1423	1314	1453
		103	487	1341	1361	1507
		131	782	1181	1117	1326
		average	531.4	1303.8	1294.0	1419.6
	2000	19	570	1472	1579	1602
		47	712	1246	1170	1443
		75	227	1486	1407	1139
		103	565	1259	1523	1388
		131	518	1352	1336	1285
		average	518.4	1363.0	1403.0	1371.4

Table (E.10): Number of conflicts according to severity types on a 3000 ft freeway segment of Type I junction - No ramp metering

			Ramp volume (vehicles / hour lane)											
			400			600			800			1000		
		Severity	PO	SL	SE	PO	SL	SE	PO	SL	SE	PO	SL	SE
Freeway volume (vehicles / hour lane)	500	19	1	4	0	0	2	1	3	3	1	1	4	2
		47	0	5	0	3	7	0	3	11	2	1	9	1
		75	1	7	1	1	4	1	3	7	0	1	6	1
		103	0	4	1	2	6	1	4	6	1	1	7	1
		131	0	5	1	2	9	0	1	0	0	0	2	1
		average	0.4	5.0	0.6	1.6	5.6	0.6	2.8	5.4	0.8	0.8	5.6	1.2
	750	19	1	5	0	2	8	0	3	12	2	3	7	2
		47	5	10	3	5	18	2	2	16	0	3	13	0
		75	1	11	1	3	14	0	1	11	3	1	11	1
		103	4	9	1	3	15	1	5	5	2	2	9	0
		131	4	12	0	3	14	0	6	12	0	4	12	1
		average	3.0	9.4	1.0	3.2	13.8	0.6	3.4	11.2	1.4	2.6	10.4	0.8
	1000	19	2	14	2	21	36	1	55	50	1	52	49	1
		47	5	20	2	8	27	3	45	52	1	66	54	1
		75	5	20	2	27	36	0	50	58	1	72	78	3
		103	12	23	1	2	19	1	81	59	4	189	124	14
		131	1	12	1	6	15	2	19	32	0	9	23	1
		average	5.0	17.8	1.6	12.8	26.6	1.4	50.0	50.2	1.4	77.6	65.6	4.0
	1250	19	92	93	3	182	136	3	1295	763	57	1244	712	35
		47	69	83	2	192	178	5	963	573	22	1233	804	40
		75	39	76	1	384	290	18	1094	638	30	1289	757	42
		103	82	110	3	248	215	7	882	603	26	1422	867	49
		131	33	66	0	51	71	4	953	564	24	650	471	22
		average	63.0	85.6	1.8	211.4	178.0	7.4	1037.4	628.2	31.8	1167.6	722.2	37.6
	1500	19	175	169	6	979	597	38	3145	1723	30	3630	1977	56
		47	395	282	12	911	559	23	2845	1510	46	4453	2417	44
		75	223	181	7	746	538	20	1854	1063	31	4338	2340	36
		103	311	247	10	850	592	20	2540	1421	47	4980	2768	59
		131	216	159	6	730	481	21	1761	1138	21	4031	2063	56
		average	264.0	207.6	8.2	843.2	553.4	24.4	2429.0	1371.0	35.0	4286.4	2313.0	50.2
	1750	19	394	257	8	1025	690	16	3172	1738	32	4266	2415	27
		47	316	229	14	770	494	15	2719	1487	42	3495	1858	35
		75	336	250	6	740	489	21	1973	1163	27	3620	1884	25
		103	179	149	8	833	606	37	2678	1517	52	4951	2703	35
		131	273	183	7	861	573	27	1844	1136	27	4651	2628	63
		average	299.6	213.6	8.6	845.8	570.4	23.2	2477.2	1408.2	36.0	4196.6	2297.6	37.0
	2000	19	306	206	13	770	460	19	2864	1755	38	4264	2460	33
		47	302	244	9	937	627	33	3005	1628	43	4095	2036	37
		75	215	177	7	827	566	27	1813	1086	25	4194	2425	38
		103	223	195	5	792	511	16	2448	1470	50	5036	2910	42
		131	354	250	10	844	511	34	1886	1158	22	4781	2759	75
		average	280	214.4	8.8	834.0	535.0	25.8	2403.2	1419.4	35.6	4474	2518.0	45.0

Note: PO = Potential conflict severity type; SL = Slight conflict severity type ; SE = Serious conflict severity type

Table (E.11): Number of conflicts according to severity types on a 3000 ft freeway segment of Type I junction - (2R+1AR+2G+1AR)

			Ramp volume (vehicles / hour lane)											
			400			600			800			1000		
		Severity	PO	SL	SE	PO	SL	SE	PO	SL	SE	PO	SL	SE
Freeway volume (vehicles / hour lane)	500	19	1	7	2	2	11	0	7	11	1	3	6	1
		47	0	2	1	1	6	0	3	14	1	2	9	1
		75	1	4	0	0	4	3	3	5	0	3	13	1
		103	2	7	1	1	10	1	0	12	3	3	11	4
		131	1	10	0	2	5	2	4	14	1	1	12	2
		average	1.0	6.0	0.8	1.2	7.2	1.2	3.4	11.2	1.2	2.4	10.2	1.8
	750	19	3	9	0	5	12	1	6	9	0	2	13	1
		47	1	13	1	2	19	2	3	13	1	3	12	17
		75	1	11	1	3	15	2	5	10	2	4	14	0
		103	3	8	2	3	6	1	5	8	0	3	11	0
		131	6	13	0	4	9	0	3	9	2	3	11	3
		average	2.8	10.8	0.8	3.4	12.2	1.2	4.4	9.8	1.0	3.0	12.2	4.2
	1000	19	5	19	2	2	24	4	16	31	0	16	31	0
		47	3	23	1	24	39	0	7	23	1	7	23	1
		75	7	19	2	7	25	0	11	28	2	11	28	2
		103	4	22	2	6	22	1	23	29	1	25	27	1
		131	6	13	1	12	34	0	3	18	2	3	18	2
		average	5.0	19.2	1.6	10.2	28.8	1.0	12.0	25.8	1.2	12.4	25.4	1.2
	1250	19	34	69	2	259	214	8	323	228	12	449	308	15
		47	45	73	0	178	174	9	373	288	11	264	216	10
		75	43	72	1	316	246	13	272	216	6	273	214	7
		103	27	52	2	128	107	7	311	229	18	269	201	9
		131	34	77	1	150	142	3	188	165	7	259	232	9
		average	36.6	68.6	1.2	206.2	176.6	8.0	293.4	225.2	10.8	302.8	234.2	10.0
	1500	19	311	223	13	889	574	34	1060	683	37	965	606	41
		47	275	226	12	1015	624	27	877	514	25	996	568	29
		75	338	234	12	922	589	29	789	649	43	753	524	25
		103	206	165	3	804	526	31	947	631	31	871	543	30
		131	399	291	11	1003	635	28	796	522	29	878	566	41
		average	305.8	227.8	10.2	926.6	589.6	29.8	893.8	599.8	33.0	892.6	561.4	33.2
	1750	19	337	212	15	931	602	34	857	535	25	1098	653	41
		47	312	189	10	1012	662	36	944	575	30	963	588	32
		75	218	170	6	977	599	32	824	553	28	900	605	25
		103	439	277	8	784	540	32	851	514	17	1024	669	36
		131	390	277	19	949	607	37	808	499	24	979	642	37
		average	339.2	225.0	11.6	930.6	602.0	34.2	856.8	535.2	24.8	992.8	631.4	34.2
	2000	19	250	194	19	912	576	36	1153	698	30	1131	730	57
		47	385	329	14	841	605	38	1012	665	41	1070	693	49
		75	306	207	13	895	536	42	1042	599	58	859	573	31
		103	277	209	11	950	610	39	844	582	32	797	573	30
		131	375	273	21	886	574	24	912	560	36	837	536	43
		average	318.6	242.4	15.6	896.8	580.2	35.8	992.6	620.8	39.4	938.8	621.0	42.0

Note: PO = Potential conflict severity type; SL = Slight conflict severity type ; SE = Serious conflict severity type

Table (E.12): Number of conflicts according to severity types on a 3000 ft freeway segment of Type I junction - (5R+1AR+5G+1AR)

			Ramp Volume (vehicles / hour lane)											
			400			600			800			1000		
		Severity	PO	SL	SE	PO	SL	SE	PO	SL	SE	PO	SL	SE
Freeway volume (vehicles / hour lane)	500	19	3	5	1	1	5	0	3	10	1	2	9	1
		47	1	6	1	3	10	1	4	15	0	4	11	3
		75	0	8	1	3	9	2	3	8	0	1	5	1
		103	1	10	1	3	9	3	3	8	1	2	10	1
		131	2	7	1	2	11	0	4	11	3	1	8	1
		average	1.4	7.2	1.0	2.4	8.8	1.2	3.4	10.4	1.0	2.0	8.6	1.4
	750	19	3	9	2	4	16	2	5	14	1	1	13	2
		47	4	15	1	0	13	2	4	13	1	1	12	0
		75	5	17	3	3	15	2	4	11	2	2	13	0
		103	0	17	0	1	7	1	1	11	2	3	12	2
		131	1	4	1	2	10	2	4	8	1	3	13	1
		average	2.6	12.4	1.4	2.0	12.2	1.8	3.6	11.4	1.4	2.0	12.6	1.0
	1000	19	4	22	0	30	38	1	5	17	1	13	22	0
		47	13	20	3	16	27	1	15	34	0	18	29	4
		75	5	18	1	20	35	0	12	31	0	6	15	3
		103	5	12	3	9	20	0	15	23	0	8	21	0
		131	2	18	1	3	16	2	9	29	0	10	22	0
		average	5.8	18.0	1.6	15.6	27.2	0.8	11.2	26.8	0.2	11.0	21.8	1.4
	1250	19	19	62	0	306	219	10	211	188	5	365	249	11
		47	96	88	3	89	110	7	224	178	10	271	214	9
		75	56	89	2	251	186	5	166	154	4	324	236	11
		103	31	53	1	186	158	10	214	162	4	166	164	2
		131	34	84	4	229	189	6	171	190	2	75	104	2
		average	47.2	75.2	2.0	212.2	172.4	7.6	197.2	174.4	5.0	240.2	193.4	7.0
	1500	19	230	185	6	959	603	40	901	569	19	832	547	33
		47	248	182	9	963	649	22	852	588	32	779	553	22
		75	150	119	3	831	493	23	948	562	33	816	540	43
		103	300	225	10	860	529	33	836	532	43	923	639	36
		131	285	197	10	801	547	27	926	567	49	681	439	31
		average	242.6	181.6	7.6	882.8	564.2	29.0	892.6	563.6	35.2	806.2	543.6	33.0
	1750	19	283	195	8	967	621	36	950	585	40	892	588	29
		47	274	195	10	720	414	16	768	514	29	948	568	36
		75	342	234	10	938	584	32	847	532	20	868	674	40
		103	295	235	12	866	557	22	879	573	31	992	610	28
		131	500	334	16	699	550	44	722	479	17	855	552	41
		average	338.8	238.6	11.2	838.0	545.2	30.0	833.2	536.6	27.4	911.0	598.4	34.8
	2000	19	366	248	12	961	613	26	1024	657	29	1054	616	48
		47	457	317	11	783	543	25	773	517	15	961	560	24
		75	134	136	5	956	599	41	892	561	40	738	458	28
		103	353	258	12	802	534	30	932	706	29	866	594	31
		131	314	236	11	883	545	42	829	560	34	825	544	24
		average	324.8	239	10.2	877.0	566.8	32.8	890.0	600.2	29.4	888.8	554.4	31.0

Note: PO = Potential conflict severity type; SL = Slight conflict severity type ; SE = Serious conflict severity type

Table (E.13): EPC on 3000 feet freeway of Type I junction - No ramp metering Kansas model = 6(F+I) + 1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	34.0	38.8	40.0	41.6
	750	65.4	89.6	79.0	69.8
	1000	121.4	180.8	359.6	495.2
	1250	587.4	1323.8	4997.4	5726.4
	1500	1558.8	4310.0	10865.0	18465.6
	1750	1632.8	4407.4	11142.4	18204.2
	2000	1619.2	4198.8	11133.2	19852.0

Table (E.14): EPC on 3000 feet freeway of Type I junction - (2R+ 1AR+2G+1AR) Kansas model = 6(F+I)+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	41.8	51.6	77.8	74.4
	750	72.4	83.8	69.2	101.4
	1000	129.8	189.0	174.0	172.0
	1250	455.4	1313.8	1709.4	1768.0
	1500	1733.8	4643.0	4690.6	4460.2
	1750	1758.8	4747.8	4216.8	4986.4
	2000	1866.6	4592.8	4953.8	4916.8

Table (E.15): EPC on 3000 feet freeway of Type I junction - (5R+1AR+5G+1AR) Kansas model = 6(F+I)+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	50.6	62.4	71.8	62.0
	750	85.4	86.0	80.4	83.6
	1000	123.4	183.6	173.2	150.2
	1250	510.4	1292.2	1273.6	1442.6
	1500	1377.8	4442.0	4485.4	4265.8
	1750	1837.6	4289.2	4217.2	4710.2
	2000	1820.0	4474.6	4667.6	4401.2

Table (E.16): EPC on a 3000 ft freeway segment of Type I junction - No ramp metering Massachusetts model = 10F+5I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	31.4	35.6	37.8	40.8
	750	60.0	78.2	73.4	62.6
	1000	110.0	159.8	315.0	445.6
	1250	509.0	1175.4	4496.4	5154.6
	1500	1384.0	3854.2	9634.0	16353.4
	1750	1453.6	3929.8	9878.2	16054.6
	2000	1440.0	3767.0	9856.2	17514.0

Table (E.17): EPC on a 3000 ft freeway segment of Type I junction - (2R+1AR+2G+1AR) Massachusetts model = 10F+5I+1PDO					
		Rampv (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	39.0	49.2	71.4	71.4
	750	64.8	76.4	63.4	106.0
	1000	117.0	164.2	153.0	151.4
	1250	391.6	1169.2	1527.4	1573.8
	1500	1546.8	4172.6	4222.8	4031.6
	1750	1580.2	4282.6	3780.8	4491.8
	2000	1686.6	4155.8	4490.6	4463.8

Table (E.18): EPC on a 3000 ft freeway segment of Type I junction - (5R+1AR+5G+1AR) Massachusetts model = 10F+5I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	47.4	58.4	65.4	59.0
	750	78.6	81.0	74.6	75.0
	1000	111.8	159.6	147.2	134.0
	1250	443.2	1150.2	1119.2	1277.2
	1500	1226.6	3993.8	4062.6	3854.2
	1750	1643.8	3864.0	3790.2	4251.0
	2000	1621.8	4039.0	4185.0	3970.8

Table (E.19): EPC on a 3000 ft freeway segment of Type I junction - No ramp metering Virginia model = 12F+6I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	22.6	25.6	28.6	32.0
	750	43.2	51.8	53.8	43.4
	1000	77.6	109.4	217.4	322.4
	1250	341.4	834.2	3303.6	3785.4
	1500	985.2	2796.2	6962.0	11827.8
	1750	1043.6	2835.4	7133.8	11533.4
	2000	1028.8	2748.6	7088.6	12568.0

Table (E.20): EPC on a 3000 ft freeway segment of Type I junction - (2R+1AR+2G+1AR) Virginia model = 12F+6I +1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	28.6	37.2	51.4	54.6
	750	44.8	54.4	45.8	90.0
	1000	81.8	108.6	103.8	103.0
	1250	256.8	832.0	1098.6	1125.4
	1500	1111.6	3053.0	3089.2	2975.2
	1750	1153.4	3147.0	2760.0	3297.4
	2000	1233.0	3067.0	3327.8	3305.8

Table (E.21): EPC on a 3000 ft freeway segment of Type I junction - (5R+1AR+5G+1AR) Virginia model = 12F + 6I + 1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	35.0	43.2	46.6	44.6
	750	56.6	60.2	54.6	51.8
	1000	79.0	106.8	94.0	93.2
	1250	296.8	820.6	780.4	904.4
	1500	878.6	2923.4	3005.8	2833.0
	1750	1189.0	2833.6	2771.8	3123.8
	2000	1164.2	2971.0	3043.4	2924.0

Table (E.22): cMFs for EPC on freeway of Type I junction - (2R+1AR+2G+1AR) Kansas model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vphpl)	500	1.23	1.33	1.95	1.79
	750	1.11	0.94	0.88	1.45
	1000	1.07	1.05	0.48	0.35
	1250	0.78	0.99	0.34	0.31
	1500	1.11	1.08	0.43	0.24
	1750	1.08	1.08	0.38	0.27
	2000	1.15	1.09	0.44	0.25

Table (E.23): cMFs for EPC on freeway of Type I junction - (5R+1AR+5G+1AR) Kansas model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vphpl)	500	1.49	1.61	1.80	1.49
	750	1.31	0.96	1.02	1.20
	1000	1.02	1.02	0.48	0.30
	1250	0.87	0.98	0.25	0.25
	1500	0.88	1.03	0.41	0.23
	1750	1.13	0.97	0.38	0.26
	2000	1.12	1.07	0.42	0.22

Table (E.24): cMFs for EPC on freeway of Type I junction - (2R+1AR+2G+1AR) Virginia model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vphpl)	500	1.27	1.45	1.80	1.71
	750	1.04	1.05	0.85	2.07
	1000	1.05	0.99	0.48	0.32
	1250	0.75	1.00	0.33	0.30
	1500	1.13	1.09	0.44	0.25
	1750	1.11	1.11	0.39	0.29
	2000	1.20	1.12	0.47	0.26

Table (E.25): cMFs for EPC on freeway of Type I junction - (5R+1AR+5G+1AR) Virginia model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vphpl)	500	1.55	1.69	1.63	1.39
	750	1.31	1.16	1.01	1.19
	1000	1.02	0.98	0.43	0.29
	1250	0.87	0.98	0.24	0.24
	1500	0.89	1.05	0.43	0.24
	1750	1.14	1.00	0.39	0.27
	2000	1.13	1.08	0.43	0.23

Table (E.26): Overall number of conflicts on a 3000 ft freeway segment of Type II junction No ramp metering						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	3	7	7	14
		47	3	11	11	13
		75	10	11	9	6
		103	3	9	9	7
		131	10	8	10	15
		average	5.8	9.2	9.2	11.0
	750	19	9	10	7	8
		47	12	8	13	9
		75	10	10	18	11
		103	8	10	11	12
		131	14	6	9	10
		average	10.6	8.8	11.6	10.0
	1000	19	32	33	68	62
		47	17	41	24	32
		75	16	17	30	43
		103	26	22	35	23
		131	23	30	15	19
		average	22.8	28.6	34.4	35.8
	1250	19	2948	3131	3586	3495
		47	3479	3605	3308	3726
		75	3051	3761	3675	3286
		103	2613	2937	2942	2945
		131	1980	2789	2903	2849
		average	2814.2	3244.6	3282.8	3260.2
	1500	19	4930	4968	5086	4997
		47	5076	5183	5009	5097
		75	5256	5113	5296	5152
		103	4979	5217	5048	5143
		131	4910	5225	5170	5068
		average	5030.2	5141.2	5121.8	5091.4
	1750	19	5059	5052	5170	5208
		47	5168	4988	5086	5082
		75	5116	5053	5060	5004
		103	4978	5082	5048	5210
		131	4976	5088	5092	5026
		average	5059.4	5052.6	5091.2	5106.0
	2000	19	5118	5094	5122	5212
		47	5221	5031	5059	5188
		75	5062	5115	5019	5082
		103	5036	5156	5264	5040
		131	5097	4965	4994	5027
		average	5106.8	5072.2	5091.6	5109.8

Table (E.27): Overall number of conflicts on a 3000 ft freeway segment of Type II junction (2R+1AR+2G+1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	9	8	10	14
		47	6	12	2	7
		75	9	8	9	9
		103	6	16	12	5
		131	7	8	17	10
		average	7.4	10.4	10.0	9.0
	750	19	11	19	10	12
		47	19	10	12	7
		75	16	13	12	12
		103	14	10	13	10
		131	8	13	11	9
		average	13.6	13.0	11.6	10.0
	1000	19	93	69	123	55
		47	59	34	54	50
		75	18	17	26	10
		103	23	17	33	51
		131	15	82	38	28
		average	41.6	43.8	54.8	38.8
	1250	19	3219	3978	3887	3958
		47	3866	4336	4339	4358
		75	3279	4450	4439	4462
		103	3078	3747	3576	3657
		131	1821	3654	3566	3888
		average	3052.6	4033.0	3961.4	4064.6
	1500	19	5020	5231	5094	5233
		47	5013	5077	5065	5043
		75	5012	5047	5046	5228
		103	5204	5191	5100	5159
		131	5129	5279	5215	5234
		average	5075.6	5165.0	5104.0	5179.4
	1750	19	5113	5133	5158	5373
		47	5208	5293	5232	5174
		75	5143	5086	5241	4984
		103	5144	5216	5195	5236
		131	5142	5314	5027	5174
		average	5150.0	5208.4	5170.6	5188.2
	2000	19	5035	5107	5109	5118
		47	5104	5250	5272	5314
		75	5092	5224	5116	5333
		103	5096	5330	5283	5166
		131	4860	5073	5283	5233
		average	5037.4	5196.8	5212.6	5232.8

Table (E.28): Overall number of conflicts on a 3000 ft freeway segment of Type II junction (5R+1AR+5G+1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	9	11	7	8
		47	7	12	16	13
		75	7	12	9	12
		103	5	11	12	5
		131	6	8	9	12
		average	6.8	10.8	10.6	10.0
	750	19	20	12	10	12
		47	19	8	7	11
		75	6	7	15	11
		103	10	12	12	7
		131	14	11	13	5
		average	13.8	10.0	11.4	9.2
	1000	19	24	40	21	77
		47	61	24	74	32
		75	27	24	28	25
		103	33	24	24	28
		131	36	21	30	24
		average	36.2	26.6	35.4	37.2
	1250	19	2913	3850	4059	3739
		47	3748	4337	4515	4485
		75	3355	4403	4379	4271
		103	2955	3968	3574	3770
		131	1881	3755	3865	3461
		average	2970.4	4062.6	4078.4	3945.2
	1500	19	5168	5180	5292	5079
		47	5090	5218	5180	5235
		75	5271	5050	5293	5274
		103	5070	5163	5117	5202
		131	4993	5196	5129	5290
		average	5118.4	5161.4	5202.2	5216.0
	1750	19	5121	5303	5135	5155
		47	5195	5007	5340	5110
		75	5232	5069	5423	5144
		103	5055	5277	4954	5143
		131	5012	4943	5151	5127
		average	5123.0	5119.8	5200.6	5135.8
	2000	19	5148	5186	5160	5178
		47	5098	5290	5072	5157
		75	5088	5275	5177	5167
		103	5156	5013	5135	5267
		131	5087	5173	5063	5271
		average	5115.4	5187.4	5121.4	5208.0

Table (E.29): Number of lane change conflicts on a 3000 ft freeway segment of Type II junction - No ramp metering						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	3	6	7	14
		47	3	10	10	12
		75	9	11	8	6
		103	3	8	9	7
		131	10	8	10	14
		average	5.6	8.6	8.8	10.6
	750	19	9	10	7	8
		47	11	8	11	9
		75	8	9	15	9
		103	8	10	8	9
		131	12	6	8	9
		average	9.6	8.6	9.8	8.8
	1000	19	17	11	15	13
		47	13	10	14	14
		75	10	11	10	15
		103	13	9	9	17
		131	13	14	7	15
		average	13.2	11.0	11.0	14.8
	1250	19	187	212	202	186
		47	222	248	211	220
		75	190	274	220	230
		103	190	203	199	194
		131	153	185	226	203
		average	188.4	224.4	211.6	206.6
	1500	19	269	242	239	250
		47	229	252	267	268
		75	251	282	287	258
		103	245	270	256	262
		131	249	291	262	256
		average	248.6	267.4	262.2	258.8
	1750	19	268	242	272	264
		47	282	252	250	266
		75	249	248	244	256
		103	262	248	268	295
		131	278	221	259	254
		average	267.8	242.2	258.6	267.0
	2000	19	257	272	274	245
		47	225	275	289	266
		75	261	240	254	265
		103	256	264	234	246
		131	262	252	251	234
		average	252.2	260.6	260.4	251.2

Table (E.30): Number of lane change conflicts on a 3000 ft freeway segment of Type II junction - (2R+1AR+2G+1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	8	8	9	13
		47	6	12	2	7
		75	7	7	9	9
		103	6	15	11	5
		131	7	7	16	10
		average	6.8	9.8	9.4	8.8
	750	19	11	15	9	12
		47	19	6	9	5
		75	13	12	11	11
		103	12	8	10	10
		131	7	10	9	9
		average	12.4	10.2	9.6	9.4
	1000	19	17	13	14	14
		47	13	16	22	18
		75	13	8	13	7
		103	10	7	12	14
		131	11	16	16	12
		average	12.8	12.0	15.4	13.0
	1250	19	216	269	227	231
		47	258	230	258	243
		75	220	257	257	270
		103	200	244	256	242
		131	131	248	243	276
		average	205.0	249.6	248.2	252.4
	1500	19	262	282	246	264
		47	274	237	247	264
		75	254	260	276	252
		103	258	267	286	258
		131	256	245	231	272
		average	260.8	258.2	257.2	262.0
	1750	19	242	258	267	267
		47	274	249	263	278
		75	243	295	274	247
		103	258	281	237	254
		131	271	256	243	260
		average	257.6	267.8	256.8	261.2
	2000	19	255	292	273	265
		47	256	282	257	272
		75	276	258	267	303
		103	260	333	246	289
		131	257	269	292	294
		average	260.8	286.8	267.0	284.6

Table (E.31): Number of lane change conflicts on a 3000 ft freeway segment of Type II junction - (5R+1AR+5G+1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	8	11	7	8
		47	7	11	15	11
		75	7	12	8	12
		103	4	10	11	5
		131	6	7	8	12
		average	6.4	10.2	9.8	9.6
	750	19	19	10	10	11
		47	14	8	5	7
		75	5	5	14	10
		103	9	10	12	7
		131	11	9	13	5
		average	11.6	8.4	10.8	8.0
	1000	19	10	13	5	12
		47	14	8	15	21
		75	13	10	6	13
		103	22	11	12	10
		131	15	10	14	14
		average	14.8	10.4	10.4	14.0
	1250	19	202	250	251	204
		47	267	266	276	264
		75	227	269	270	262
		103	226	283	244	235
		131	167	286	264	222
		average	217.8	270.8	261.0	237.4
	1500	19	292	242	271	240
		47	299	236	262	284
		75	291	265	278	267
		103	296	272	279	281
		131	258	295	262	293
		average	287.2	262.0	270.4	273.0
	1750	19	275	244	265	224
		47	242	279	272	267
		75	257	273	283	243
		103	236	295	237	269
		131	256	268	265	264
		average	253.2	271.8	264.4	253.4
	2000	19	250	276	250	258
		47	244	294	266	280
		75	253	255	258	247
		103	259	238	265	272
		131	271	240	259	287
		average	255.4	260.6	259.6	268.8

Table (E.32): Number of rear end conflicts on a 3000 ft freeway segment of Type II junction No ramp metering						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	0	1	0	0
		47	0	1	1	1
		75	1	0	1	0
		103	0	1	0	0
		131	0	0	0	1
		average	0.2	0.6	0.4	0.4
	750	19	0	0	0	0
		47	1	0	2	0
		75	2	1	3	2
		103	0	0	3	3
		131	2	0	1	1
		average	1.0	0.2	1.8	1.2
	1000	19	15	22	53	49
		47	4	31	10	18
		75	6	6	20	28
		103	13	13	26	6
		131	10	16	8	4
		average	9.6	17.6	23.4	21.0
	1250	19	2761	2919	3384	3309
		47	3257	3357	3097	3506
		75	2861	3487	3455	3056
		103	2423	2734	2743	2751
		131	1827	2604	2677	2646
		average	2625.8	3020.2	3071.2	3053.6
	1500	19	4661	4726	4847	4747
		47	4847	4931	4742	4829
		75	5005	4831	5009	4894
		103	4734	4947	4792	4881
		131	4661	4934	4908	4812
		average	4781.6	4873.8	4859.6	4832.6
	1750	19	4791	4810	4898	4944
		47	4886	4736	4836	4816
		75	4867	4805	4816	4748
		103	4716	4834	4780	4915
		131	4698	4867	4833	4772
		average	4791.6	4810.4	4832.6	4839.0
	2000	19	4861	4822	4848	4967
		47	4996	4756	4770	4922
		75	4801	4875	4765	4817
		103	4780	4892	5030	4794
		131	4835	4713	4743	4793
		average	4854.6	4811.6	4831.2	4858.6

Table (E.33): Number of rear end conflicts on a 3000 ft freeway segment of Type II junction (2R+1AR+2G+1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	1	0	1	1
		47	0	0	0	0
		75	2	1	0	0
		103	0	1	1	0
		131	0	1	1	0
		average	0.6	0.6	0.6	0.2
	750	19	0	4	1	0
		47	0	4	3	2
		75	3	1	1	1
		103	2	2	3	0
		131	1	3	2	0
		average	1.2	2.8	2.0	0.6
	1000	19	76	56	109	41
		47	46	18	32	32
		75	5	9	13	3
		103	13	10	21	37
		131	4	66	22	16
		average	28.8	31.8	39.4	25.8
	1250	19	3003	3709	3660	3727
		47	3608	4106	4081	4115
		75	3059	4193	4182	4192
		103	2878	3503	3320	3415
		131	1690	3406	3323	3612
		average	2847.6	3783.4	3713.2	3812.2
	1500	19	4758	4949	4848	4969
		47	4739	4840	4818	4779
		75	4758	4787	4770	4976
		103	4946	4924	4814	4901
		131	4873	5034	4984	4962
		average	4814.8	4906.8	4846.8	4917.4
	1750	19	4871	4875	4891	5106
		47	4934	5044	4969	4896
		75	4900	4791	4967	4737
		103	4886	4935	4958	4982
		131	4871	5058	4784	4914
		average	4892.4	4940.6	4913.8	4927.0
	2000	19	4780	4815	4836	4853
		47	4848	4968	5015	5042
		75	4816	4966	4849	5030
		103	4836	4997	5037	4877
		131	4603	4804	4991	4939
		average	4776.6	4910.0	4945.6	4948.2

Table (E.34): Number of rear end conflicts on a 3000 ft freeway segment of Type II junction (5R+1AR+5G+1AR)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	1	0	0	0
		47	0	1	1	2
		75	0	0	1	0
		103	1	1	1	0
		131	0	1	1	0
		average	0.4	0.6	0.8	0.4
	750	19	1	2	0	1
		47	5	0	2	4
		75	1	2	1	1
		103	1	2	0	0
		131	3	2	0	0
		average	2.2	1.6	0.6	1.2
	1000	19	14	27	16	65
		47	47	16	59	11
		75	14	14	22	12
		103	11	13	12	18
		131	21	11	16	10
		average	21.4	16.2	25.0	23.2
	1250	19	2711	3600	3808	3535
		47	3481	4071	4239	4221
		75	3128	4134	4109	4009
		103	2729	3685	3330	3535
		131	1714	3469	3601	3239
		average	2752.6	3791.8	3817.4	3707.8
	1500	19	4876	4938	5021	4839
		47	4791	4982	4918	4951
		75	4980	4785	5015	5007
		103	4774	4891	4838	4921
		131	4735	4901	4867	4997
		average	4831.2	4899.4	4931.8	4943.0
	1750	19	4846	5059	4870	4931
		47	4953	4728	5068	4843
		75	4975	4796	5140	4901
		103	4819	4982	4717	4874
		131	4756	4675	4886	4863
		average	4869.8	4848.0	4936.2	4882.4
	2000	19	4898	4910	4910	4920
		47	4854	4996	4806	4877
		75	4835	5020	4919	4920
		103	4897	4775	4870	4995
		131	4816	4933	4804	4984
		average	4860.0	4926.8	4861.8	4939.2

Table (E.35): Number of conflicts according to severity types on a 3000 ft freeway segment of Type II junction - (No ramp metering)														
			Ramp volume (vehicles / hour lane)											
			400			600			800			1000		
		severity	PO	SL	SE	PO	SL	SE	PO	SL	SE	PO	SL	SE
Freeway volume (vehicles / hour lane)	500	19	0	2	1	0	5	2	1	6	0	2	9	3
		47	1	2	0	1	8	2	5	6	0	2	9	2
		75	1	8	1	1	9	1	0	7	2	0	5	1
		103	0	2	1	3	6	0	2	5	2	0	5	2
		131	1	7	2	1	6	1	0	8	2	1	11	3
		average	0.6	4.2	1.0	1.2	6.8	1.2	1.6	6.4	1.2	1.0	7.8	2.2
	750	19	1	7	1	2	7	1	1	5	1	0	7	1
		47	1	10	1	0	7	1	1	11	1	0	7	2
		75	1	7	2	0	9	1	3	13	2	2	9	0
		103	3	4	1	1	8	1	3	7	1	3	7	2
		131	2	11	1	0	5	1	0	7	2	1	8	1
		average	1.6	7.8	1.2	0.6	7.2	1.0	1.6	8.6	1.4	1.2	7.6	1.2
	1000	19	12	20	0	13	20	0	33	35	0	33	27	2
		47	4	13	0	17	23	1	8	16	0	10	21	1
		75	5	10	1	5	12	0	13	16	1	17	25	1
		103	7	19	0	5	17	0	19	16	0	2	21	0
		131	7	16	0	11	19	0	4	11	0	2	16	1
		average	7.0	15.6	0.2	10.2	18.2	0.2	15.4	18.8	0.2	12.8	22.0	1.0
	1250	19	1893	1020	35	2025	1072	34	2307	1238	41	2191	1260	44
		47	2189	1248	42	2236	1316	53	2116	1168	24	2408	1291	27
		75	1960	1059	32	2323	1356	82	2358	1276	41	2084	1161	41
		103	1657	916	40	1879	1017	41	1887	1018	37	1848	1058	39
		131	1245	696	39	1768	988	33	1822	1032	49	1833	960	56
		average	1788.8	987.8	37.6	2046.2	1149.8	48.6	2098.0	1146.4	38.4	2072.8	1146.0	41.4
	1500	19	3203	1689	38	3123	1809	36	3204	1838	44	3148	1817	32
		47	3243	1787	46	3241	1876	66	3149	1813	47	3191	1862	44
		75	3321	1897	38	3264	1794	55	3297	1949	50	3181	1920	51
		103	3140	1796	43	3272	1893	52	3142	1862	44	3222	1896	25
		131	3077	1787	46	3412	1768	45	3262	1870	38	3191	1837	40
		average	3196.8	1791.2	42.2	3262.4	1828.0	50.8	3210.8	1866.4	44.6	3186.6	1866.4	38.4
	1750	19	3295	1710	54	3256	1752	44	3225	1903	42	3412	1772	24
		47	3334	1781	53	3310	1643	35	3165	1886	35	3188	1860	34
		75	3269	1784	63	3247	1748	58	3198	1829	33	3281	1667	56
		103	3247	1683	48	3292	1748	42	3269	1733	46	3254	1921	35
		131	3330	1593	53	3177	1868	43	2856	2193	43	3327	1676	23
		average	3295.0	1710.2	54.2	3256.4	1751.8	44.4	3142.6	1908.8	39.8	3292.4	1779.2	34.4
	2000	19	3370	1716	32	3324	1727	43	3236	1855	31	3288	1896	28
		47	3422	1763	36	3146	1829	56	3165	1829	65	3274	1862	52
		75	3278	1757	27	3297	1785	33	3136	1846	37	3227	1812	43
		103	3168	1823	45	3243	1877	36	3302	1927	35	3294	1692	54
		131	3228	1836	33	3221	1675	69	3138	1815	41	3173	1814	40
		average	3293.2	1779	34.6	3246.2	1778.6	47.4	3195.4	1854.4	41.8	3251.2	1815.2	43.4
Note: PO = Potential conflict severity type; SL = Slight conflict severity type ; SE = Serious conflict severity type														

Table (E.36): Number of conflicts according to severity types on a 3000 ft freeway segment of Type II junction - (2R+1AR+2G+1AR)														
			Ramp volume (vehicles / hour lane)											
			400			600			800			1000		
		severity	PO	SL	SE	PO	SL	SE	PO	SL	SE	PO	SL	SE
Freeway volume (vehicles / hour lane)	500	19	1	8	0	1	6	1	3	6	1	3	9	2
		47	1	4	1	3	7	2	0	1	1	1	4	2
		75	3	6	0	1	7	0	3	5	1	0	8	1
		103	1	4	1	3	12	1	3	7	2	1	4	0
		131	0	5	2	2	5	1	4	12	1	2	7	1
		average	1.2	5.4	0.8	2.0	7.4	1.0	2.6	6.2	1.2	1.4	6.4	1.2
	750	19	1	9	1	7	11	1	0	8	2	3	8	1
		47	1	16	2	4	6	0	3	8	1	1	5	1
		75	4	9	3	1	10	2	1	9	2	3	8	1
		103	3	10	1	3	7	0	3	9	1	1	7	2
		131	1	4	3	5	8	0	3	8	0	1	7	1
		average	2.0	9.6	2.0	4.0	8.4	0.6	2.0	8.4	1.2	1.8	7.0	1.2
	1000	19	53	39	1	37	32	0	75	46	2	23	31	1
		47	28	30	1	12	22	0	19	34	1	22	27	1
		75	4	14	0	5	12	0	9	17	0	4	6	0
		103	8	15	0	7	10	0	11	20	0	25	25	1
		131	2	13	0	43	38	1	13	24	1	6	21	1
		average	19.0	22.2	0.4	20.8	22.8	0.2	25.4	28.2	0.8	16.0	22.0	0.8
	1250	19	2021	1140	58	2534	1388	56	2394	1439	54	2554	1373	31
		47	2457	1351	58	2761	1537	38	2715	1568	56	2721	1598	39
		75	2092	1156	31	2896	1511	43	2737	1645	57	2811	1614	37
		103	1917	1126	35	2414	1287	46	2207	1323	46	2343	1262	52
		131	1176	619	26	2367	1247	40	2208	1316	42	2404	1430	54
		average	1932.6	1078.4	41.6	2594.4	1394.0	44.6	2452.2	1458.2	51.0	2566.6	1455.4	42.6
	1500	19	3164	1805	51	3286	1889	56	3236	1812	46	3325	1865	43
		47	3264	1706	43	3197	1833	47	3289	1732	44	3197	1796	50
		75	3184	1772	56	3267	1733	47	3171	1811	64	3355	1812	61
		103	3248	1911	45	3253	1901	37	3331	1715	54	3238	1874	47
		131	3197	1893	39	3336	1902	41	3333	1843	39	3286	1868	80
		average	3211.4	1817.4	46.8	3267.8	1851.6	45.6	3272.0	1782.6	49.4	3280.2	1843.0	56.2
	1750	19	3209	1856	48	3356	1733	44	3228	1877	53	3530	1784	59
		47	3258	1909	41	3418	1837	38	3285	1882	65	3346	1765	63
		75	3350	1752	41	3271	1757	58	3302	1876	63	3268	1672	44
		103	3280	1838	26	3295	1868	53	3277	1885	33	3252	1933	51
		131	3242	1850	50	3339	1910	65	3213	1755	59	3300	1832	42
		average	3267.8	1841.0	41.2	3335.8	1821.0	51.6	3261.0	1855.0	54.6	3339.2	1797.2	51.8
	2000	19	3178	1800	57	3195	1859	53	3205	1873	31	3348	1734	36
		47	3250	1808	46	3276	1922	52	3363	1866	43	3372	1892	50
		75	3203	1828	61	3299	1882	43	3217	1850	49	3459	1817	57
		103	3188	1857	51	3314	1919	97	3341	1906	36	3236	1893	37
		131	3055	1765	40	3193	1827	53	3333	1902	48	3264	1917	52
		average	3174.8	1811.6	51	3255.4	1881.8	59.6	3291.8	1879.4	41.4	3335.8	1850.6	46.4
Note: PO = Potential conflict severity type; SL = Slight conflict severity type ; SE = Serious conflict severity type														

Table (E.37): Number of conflicts according to severity types on a 3000 ft freeway segment of Type II junction - (5R+1AR+5G+1AR)

			Ramp volume (vehicles / hour lane)											
			400			600			800			1000		
		severity	PO	SL	SE	PO	SL	SE	PO	SL	SE	PO	SL	SE
Freeway volume (vehicles / hour lane)	500	19	3	5	1	0	9	2	2	3	2	2	4	2
		47	1	4	2	2	10	0	2	13	1	2	10	1
		75	2	4	1	3	7	2	1	7	1	1	9	2
		103	1	4	0	1	9	1	2	8	2	1	4	0
		131	1	4	1	1	5	2	0	7	2	1	10	1
		average	1.6	4.2	1.0	1.4	8.0	1.4	1.4	7.6	1.6	1.4	7.4	1.2
	750	19	0	17	3	2	9	1	1	9	0	1	9	2
		47	8	11	0	3	3	2	0	5	2	3	7	1
		75	1	5	0	2	5	0	3	10	2	1	9	1
		103	3	7	0	4	7	1	2	9	1	1	5	1
		131	1	11	2	1	10	0	3	10	0	1	4	0
		average	2.6	10.2	1.0	2.4	6.8	0.8	1.8	8.6	1.0	1.4	6.8	1.0
	1000	19	9	14	1	17	21	2	9	11	1	42	35	0
		47	27	33	1	7	17	0	38	35	1	6	26	0
		75	10	17	0	10	14	0	13	14	1	8	17	0
		103	13	20	0	8	16	0	9	15	0	9	19	0
		131	15	21	0	7	14	0	6	24	0	7	15	2
		average	14.8	21.0	0.4	9.8	16.4	0.4	15.0	19.8	0.6	14.4	22.4	0.4
	1250	19	1802	1067	44	2481	1287	82	2583	1416	60	2353	1344	42
		47	2370	1311	67	2701	1579	57	2925	1537	53	2921	1517	47
		75	2149	1152	54	2706	1645	52	2798	1538	43	2641	1571	59
		103	1789	1130	36	2537	1388	43	2313	1214	47	2405	1316	49
		131	1141	701	39	2342	1349	64	2508	1321	36	2214	1207	40
		average	1850.2	1072.2	48.0	2553.4	1449.6	59.6	2625.4	1405.2	47.8	2506.8	1391.0	47.4
	1500	19	3388	1718	62	3418	1729	33	3403	1852	37	3205	1836	38
		47	3158	1879	53	3272	1906	40	3373	1748	59	3276	1908	51
		75	3321	1899	51	3166	1835	49	3286	1958	49	3308	1916	50
		103	3320	1692	58	3235	1880	48	3208	1857	52	3282	1855	65
		131	3147	1800	46	3258	1898	40	3298	1759	72	3256	1971	63
		average	3266.8	1797.6	54.0	3269.8	1849.6	42.0	3313.6	1834.8	53.8	3265.4	1897.2	53.4
	1750	19	3229	1855	37	3392	1870	41	3234	1846	55	3384	1735	36
		47	3303	1855	37	3137	1819	51	3348	1939	53	3215	1851	44
		75	3275	1909	48	3236	1789	44	3414	1966	43	3267	1836	41
		103	3220	1796	39	3434	1776	67	3101	1792	61	3212	1897	34
		131	3096	1865	51	3105	1799	39	3220	1877	54	3313	1769	45
		average	3224.6	1856.0	42.4	3260.8	1810.6	48.4	3263.4	1884.0	53.2	3278.2	1817.6	40.0
	2000	19	3268	1856	24	3241	1909	36	3222	1888	50	3224	1911	43
		47	3257	1799	42	3381	1850	59	3185	1830	57	3270	1851	36
		75	3156	1879	53	3301	1931	43	3266	1852	59	3254	1871	42
		103	3208	1912	36	3172	1810	31	3351	1731	53	3316	1881	70
		131	3214	1837	36	3384	1730	59	3295	1729	39	3313	1905	53
		average	3220.6	1856.6	38.2	3295.8	1846.0	45.6	3263.8	1806.0	51.6	3275.4	1883.8	48.8

Note: PO = Potential conflict severity type; SL = Slight conflict severity type ; SE = Serious conflict severity type

Table (E.38): EPC on a 3000 ft freeway segment of Type II junction - No ramp metering Kansas model = 6(F+I)+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	31.8	49.2	47.2	61.0
	750	55.6	49.8	61.6	54.0
	1000	101.8	120.6	129.4	150.8
	1250	7941.2	9236.6	9206.8	9197.2
	1500	14197.2	14535.2	14676.8	14615.4
	1750	13881.4	14033.6	14834.2	14174.0
	2000	14174.8	14202.2	14572.6	14402.8

Table (E.39): EPC on a 3000 ft freeway segment of Type II junction - (2R+1AR+2G+1AR) Kansas model = 6(F+I)+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	38.4	52.4	47.0	47.0
	750	71.6	58.0	59.6	51.0
	1000	154.6	158.8	199.4	152.8
	1250	8652.6	11226.0	11507.4	11554.6
	1500	14396.6	14651.0	14264.0	14675.4
	1750	14561.0	14571.4	14718.6	14433.2
	2000	14350.4	14903.8	14816.6	14717.8

Table (E.40): EPC on a 3000 ft freeway of Type II junction - (5R+1AR+5G+1AR) Kansas model = 6(F+I)+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	32.8	57.8	56.6	53.0
	750	69.8	48.0	59.4	48.2
	1000	143.2	110.6	137.4	151.2
	1250	8571.4	11608.6	11343.4	11137.2
	1500	14376.4	14619.4	14645.2	14969.0
	1750	14615.0	14414.8	14886.6	14423.8
	2000	14589.4	14645.4	14409.4	14871.0

Table (E.41): EPC on a 3000 ft freeway segment of Type II junction - No ramp metering Massachusetts model = 10F+5I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	31.6	47.2	45.6	62.0
	750	52.6	46.6	58.6	51.2
	1000	87.0	103.2	111.4	132.8
	1250	7103.8	8281.2	8214.0	8216.8
	1500	12574.8	12910.4	12988.8	12902.6
	1750	12388.0	12459.4	13084.6	12532.4
	2000	12534.2	12613.2	12885.4	12761.2

Table (E.42): EPC on a 3000 ft freeway segment of Type II junction - (2R+1AR+2G+1AR) Massachusetts model = 10F+5I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	36.2	49.0	45.6	45.4
	750	70.0	52.0	56.0	48.8
	1000	134.0	136.8	174.4	134.0
	1250	7740.6	10010.4	10253.2	10269.6
	1500	12766.4	12981.8	12679.0	13057.2
	1750	12884.8	12956.8	13082.0	12843.2
	2000	12742.8	13260.4	13102.8	13052.8

Table (E.43): EPC on a 3000 ft freeway segment of Type II junction - (5R+1AR+5G+1AR) Massachusetts model = 10F+5I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	32.6	55.4	55.4	50.4
	750	63.6	44.4	54.8	45.4
	1000	123.8	95.8	120.0	130.4
	1250	7691.2	10397.4	10129.4	9935.8
	1500	12794.8	12937.8	13025.6	13285.4
	1750	12928.6	12797.8	13215.4	12766.2
	2000	12885.6	12981.8	12809.8	13182.4

Table (E.44): EPC on a 3000 ft freeway segment of Type II junction-No ramp metering Virginia model = 12F+6I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	25.2	36.0	35.2	50.8
	750	39.4	34.2	44.2	38.4
	1000	56.2	67.2	74.2	90.8
	1250	5203.4	6078.8	5998.0	6007.6
	1500	9076.8	9356.0	9345.2	9246.6
	1750	9076.0	9044.6	9346.6	9042.8
	2000	9045.4	9150.8	9260.2	9217.6

Table (E.45): EPC on a 3000 ft freeway segment of Type II junction-(2R+1AR+2G+1AR) Virginia model = 12F+6I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	27.0	36.2	35.6	35.0
	750	54.8	36.4	41.6	37.2
	1000	90.4	91.6	119.6	91.6
	1250	5667.0	7311.6	7438.8	7444.0
	1500	9225.2	9369.8	9212.6	9483.6
	1750	9285.2	9418.0	9481.2	9352.4
	2000	9221.6	9616.0	9426.8	9444.4

Table (E.46): EPC on a 3000 ft freeway segment of Type II junction-(2R+1AR+2G+1AR) Virginia model = 12F+6I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles / hour lane)	500	26.2	42.2	43.4	38.0
	750	45.2	32.4	39.6	33.8
	1000	82.6	63.8	81.6	86.4
	1250	5642.8	7617.4	7414.6	7248.6
	1500	9307.6	9322.6	9463.6	9597.8
	1750	9301.4	9273.4	9553.8	9211.0
	2000	9248.8	9381.0	9301.0	9512.4

Table (E.47): cMFs for EPC on freeway of Type II junction - (2R+1AR+2G+1AR) Kansas model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vphpl)	500	1.21	1.07	1.00	0.77
	750	1.29	1.16	0.97	0.94
	1000	1.52	1.32	1.54	1.01
	1250	1.09	1.22	1.25	1.26
	1500	1.01	1.01	0.97	1.00
	1750	1.05	1.04	0.99	1.02
	2000	1.01	1.05	1.02	1.02

Table (E.48): cMFs for EPC on freeway of Type II junction - (5R+1AR+5G+1AR) Kansas model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vphpl)	500	1.03	1.17	1.20	0.87
	750	1.26	0.96	0.96	0.89
	1000	1.41	0.92	1.06	1.00
	1250	1.08	1.26	1.23	1.21
	1500	1.01	1.01	1.00	1.02
	1750	1.05	1.03	1.00	1.02
	2000	1.03	1.03	0.99	1.03

Table (E.49): cMFs for EPC on freeway of Type II junction - (2R+1AR+2G+1AR) Virginia model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vphpl)	500	1.07	1.01	1.01	0.69
	750	1.39	1.06	0.94	0.97
	1000	1.61	1.36	1.61	1.01
	1250	1.09	1.20	1.24	1.24
	1500	1.02	1.00	0.99	1.03
	1750	1.02	1.04	1.01	1.03
	2000	1.02	1.05	1.02	1.02

Table (E.50): cMFs for EPC on freeway of Type II junction - (5R+1AR+5G+1AR) Virginia model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vphpl)	500	1.04	1.17	1.23	0.75
	750	1.15	0.95	0.90	0.88
	1000	1.47	0.95	1.10	0.95
	1250	1.08	1.25	1.24	1.21
	1500	1.03	1.00	1.01	1.04
	1750	1.02	1.03	1.02	1.02
	2000	1.02	1.03	1.00	1.03

Table (E.51): Overall number of conflicts on a 3000 ft freeway segment of Type III junction (No ramp metering)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	14	14	10	8
		47	9	13	14	13
		75	8	4	15	12
		103	8	7	14	7
		131	5	11	11	10
		average	8.8	9.8	12.8	10.0
	750	19	12	11	16	18
		47	11	24	22	21
		75	11	10	24	18
		103	7	15	20	15
		131	9	22	17	15
		average	10.0	16.4	19.8	17.4
	1000	19	19	34	39	64
		47	22	19	47	38
		75	18	15	32	45
		103	14	23	42	39
		131	15	21	28	43
		average	17.6	22.4	37.6	45.8
	1250	19	123	653	2763	2825
		47	325	1224	2989	3006
		75	293	998	1906	2103
		103	286	869	1426	1675
		131	119	1076	2187	2772
		average	229.2	964.0	2254.2	2476.2
	1500	19	1793	2851	3035	3006
		47	1886	2632	3087	3130
		75	1896	2745	2969	2964
		103	1802	2555	3019	3078
		131	1832	2828	2934	2993
		average	1841.8	2722.2	3008.8	3034.2
	1750	19	1878	2658	3001	3120
		47	1777	2567	3107	3023
		75	1796	2740	3082	3189
		103	1683	2743	3112	3154
		131	1917	2678	3072	3044
		average	1810.2	2677.2	3074.8	3106.0
	2000	19	1864	2677	3112	2963
		47	1826	2672	3159	2983
		75	1829	2709	3158	3069
		103	1732	2686	3092	3051
		131	1881	2680	3100	3106
		average	1826.4	2684.8	3124.2	3034.4

Table (E.52): Overall number of conflicts on a 3000 ft freeway segment of Type III junction (2R + 2G)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	2	10	10	9
		47	11	8	14	16
		75	8	8	11	15
		103	10	12	12	11
		131	8	9	7	6
		average	7.8	9.4	10.8	11.4
	750	19	10	16	17	14
		47	10	11	15	17
		75	11	12	13	21
		103	7	17	10	19
		131	8	12	10	18
		average	9.2	13.6	13.0	17.8
	1000	19	17	11	47	61
		47	14	28	41	49
		75	9	25	13	41
		103	22	17	33	59
		131	15	16	21	18
		average	15.4	19.4	31.0	45.6
	1250	19	111	670	2853	3054
		47	397	1271	2948	2858
		75	243	1103	2119	2802
		103	237	981	1538	2311
		131	160	823	2515	2819
		average	229.6	969.6	2394.6	2768.8
	1500	19	1736	2730	2988	3055
		47	1742	2541	2965	2930
		75	1879	2618	3010	3219
		103	1827	2633	3115	3063
		131	1949	2532	2952	3105
		average	1826.6	2610.8	3006.0	3074.4
	1750	19	1815	2666	2956	3030
		47	1771	2608	3070	3004
		75	1910	2781	2936	3092
		103	1853	2704	2912	2937
		131	1738	2637	3166	3184
		average	1817.4	2679.2	3008.0	3049.4
	2000	19	1821	2719	3139	3022
		47	1740	2546	3129	3069
		75	1799	2643	2961	3021
		103	1874	2717	3036	3133
		131	1794	2647	2952	3165
		average	1805.6	2654.4	3043.4	3082.0

Table (E.53): Overall number of conflicts on a 3000 ft freeway segment of Type III junction (4R + 4G)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	4	11	10	14
		47	6	9	14	11
		75	3	9	5	9
		103	6	9	8	12
		131	7	10	10	12
		average	5.2	9.6	9.4	11.6
	750	19	13	12	22	19
		47	7	13	14	23
		75	13	23	17	17
		103	11	13	18	21
		131	6	12	19	15
		average	10.0	14.6	18.0	19.0
	1000	19	23	44	50	68
		47	10	37	26	34
		75	18	18	25	37
		103	22	22	38	34
		131	15	27	27	25
		average	17.6	29.6	33.2	39.6
	1250	19	198	639	2797	2905
		47	280	1210	2985	2911
		75	308	1095	2249	2540
		103	138	912	1672	2110
		131	166	850	2261	2813
		average	218.0	941.2	2392.8	2655.8
	1500	19	1800	2712	3217	3173
		47	1561	2544	2999	3088
		75	2023	2768	3052	3156
		103	1629	2784	2980	2912
		131	1909	2698	3060	3190
		average	1784.4	2701.2	3061.6	3103.8
	1750	19	1865	2680	3109	3090
		47	1664	2400	3079	3019
		75	1838	2699	3035	2983
		103	1822	2703	3086	3077
		131	1883	2756	2959	3079
		average	1814.4	2647.6	3053.6	3049.6
	2000	19	1768	2724	3011	3031
		47	1628	2601	2975	3219
		75	1942	2595	3006	3074
		103	1703	2650	3171	2982
		131	1698	2564	3079	3018
		average	1747.8	2626.8	3048.4	3064.8

Table (E.54): Overall number of conflicts on a 3000 ft freeway segment of Type III junction (4R + 2G)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	3	11	14	12
		47	6	10	15	16
		75	10	12	17	18
		103	12	12	16	15
		131	6	10	11	12
		average	7.4	11.0	14.6	14.6
	750	19	8	18	20	18
		47	12	19	12	13
		75	12	15	12	11
		103	12	15	12	11
		131	12	20	17	15
		average	11.2	17.4	14.6	13.6
	1000	19	23	37	33	35
		47	12	28	22	29
		75	13	36	24	35
		103	27	19	22	30
		131	11	26	12	22
		average	17.2	29.2	22.6	30.2
	1250	19	157	505	542	653
		47	378	1342	1611	1738
		75	298	1068	1043	1085
		103	289	696	849	656
		131	175	656	1012	1033
		average	259.4	853.4	1011.4	1033.0
	1500	19	1997	2827	2782	2833
		47	1900	2740	2825	2860
		75	1856	2766	2766	2774
		103	1844	2722	2837	2687
		131	1684	2873	2885	2730
		average	1856.2	2785.6	2819.0	2776.8
	1750	19	1721	2732	2849	2650
		47	1798	2785	2790	2694
		75	1912	2853	2884	2846
		103	1872	2749	2828	2751
		131	1928	2784	2539	2780
		average	1846.2	2780.6	2778.0	2744.2
	2000	19	1736	2639	2874	2834
		47	1841	2707	2796	2783
		75	1895	2829	2819	2819
		103	1864	3286	2763	2857
		131	1774	2726	2636	2829
		average	1822.0	2837.4	2777.6	2824.4

Table (E.55): Number of lane change conflicts on a 3000 ft freeway segment of Type III junction- (No ramp metering)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	0	1	0	1
		47	1	0	2	0
		75	0	0	0	0
		103	0	0	0	0
		131	0	0	0	0
		average	0.2	0.2	0.4	0.2
	750	19	1	1	1	2
		47	1	3	1	2
		75	0	0	0	0
		103	0	1	2	1
		131	1	0	0	0
		average	0.6	1.0	0.8	1.0
	1000	19	3	6	6	5
		47	5	2	2	5
		75	1	0	2	8
		103	4	8	6	6
		131	4	3	1	10
		average	3.4	3.8	3.4	6.8
	1250	19	17	15	204	194
		47	24	52	193	216
		75	25	46	91	141
		103	13	63	85	84
		131	23	47	121	180
		average	20.4	44.6	138.8	163.0
	1500	19	39	177	197	225
		47	35	163	196	217
		75	61	155	214	206
		103	34	146	200	222
		131	47	161	184	188
		average	43.2	160.4	198.2	211.6
	1750	19	49	158	241	201
		47	33	156	206	215
		75	48	161	228	239
		103	54	135	205	204
		131	49	170	200	219
		average	46.6	156.0	216.0	215.6
	2000	19	47	162	230	212
		47	49	158	234	209
		75	56	150	233	200
		103	46	142	211	205
		131	44	145	199	227
		average	48.4	151.4	221.4	210.6

Table (E.56): Number of lane change conflicts on a 3000 ft freeway segment of Type III junction - (2R + 2G)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	0	0	0	2
		47	1	0	3	0
		75	0	0	2	0
		103	1	1	2	0
		131	0	0	0	0
		average	0.4	0.2	1.4	0.4
	750	19	1	1	1	0
		47	2	0	0	3
		75	0	0	3	0
		103	1	0	0	1
		131	1	0	1	1
		average	1.0	0.2	1.0	1.0
	1000	19	5	3	7	7
		47	2	6	7	6
		75	1	4	1	3
		103	8	6	10	8
		131	5	2	3	3
		average	4.2	4.2	5.6	5.4
	1250	19	17	15	181	194
		47	26	46	189	191
		75	18	42	117	181
		103	14	71	76	173
		131	26	35	160	230
		average	20.2	41.8	144.6	193.8
	1500	19	39	157	197	220
		47	36	139	176	195
		75	53	131	191	245
		103	37	157	226	211
		131	44	148	218	194
		average	41.8	146.4	201.6	213.0
	1750	19	42	123	198	219
		47	57	160	184	210
		75	51	169	187	212
		103	52	160	196	220
		131	60	156	237	232
		average	52.4	153.6	200.4	218.6
	2000	19	32	152	206	213
		47	46	130	187	242
		75	49	139	183	191
		103	35	147	194	229
		131	46	153	218	206
		average	41.6	144.2	197.6	216.2

Table (E.57): Number of lane change conflicts on a 3000 ft freeway segment of Type III junction - (4R + 4G)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	1	2	1	1
		47	0	2	0	1
		75	0	0	1	0
		103	0	0	1	0
		131	0	1	0	1
		average	0.2	1.0	0.6	0.6
	750	19	1	2	2	3
		47	0	0	2	3
		75	0	1	2	0
		103	0	1	0	2
		131	0	0	0	2
		average	0.2	0.8	1.2	2.0
	1000	19	2	7	7	8
		47	3	8	4	6
		75	2	4	4	5
		103	5	7	4	6
		131	4	5	6	6
		average	3.2	6.2	5.0	6.2
	1250	19	23	24	184	187
		47	41	46	236	218
		75	28	36	122	163
		103	11	69	84	133
		131	20	23	133	180
		average	24.6	39.6	151.8	176.2
	1500	19	50	132	211	212
		47	36	144	180	209
		75	49	167	192	197
		103	50	163	203	204
		131	47	144	192	214
		average	46.4	150.0	195.6	207.2
	1750	19	43	155	226	204
		47	49	126	210	232
		75	46	135	221	195
		103	53	159	224	212
		131	47	168	188	208
		average	47.6	148.6	213.8	210.2
	2000	19	45	138	181	183
		47	39	121	225	216
		75	45	121	207	233
		103	41	155	225	216
		131	37	126	204	228
		average	41.4	132.2	208.4	215.2

Table (E.58): Number of lane change conflicts on a 3000 ft freeway segment of Type III junction - (4R + 2G)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	0	0	1	0
		47	0	2	0	1
		75	0	1	0	0
		103	0	0	0	0
		131	1	0	0	0
		average	0.2	0.6	0.2	0.2
	750	19	0	2	1	1
		47	2	0	0	0
		75	1	1	2	1
		103	0	2	0	0
		131	0	1	0	3
		average	0.6	1.2	0.6	1.0
	1000	19	6	5	2	2
		47	1	3	3	7
		75	3	4	3	5
		103	9	4	6	4
		131	2	3	2	4
		average	4.2	3.8	3.2	4.4
	1250	19	25	22	17	16
		47	23	53	81	134
		75	24	54	74	65
		103	14	56	65	47
		131	21	17	22	41
		average	21.4	40.4	51.8	60.6
	1500	19	54	188	158	162
		47	46	173	189	187
		75	61	177	178	178
		103	43	181	162	169
		131	49	193	183	189
		average	50.6	182.4	174.0	177.0
	1750	19	35	187	193	204
		47	64	156	174	168
		75	62	167	198	175
		103	60	177	161	152
		131	57	209	165	192
		average	55.6	179.2	178.2	178.2
	2000	19	52	168	190	189
		47	52	189	191	188
		75	70	193	178	197
		103	36	161	189	214
		131	61	185	175	166
		average	54.2	179.2	184.6	190.8

Table (E.58): Number of rear end conflicts on a 3000 ft freeway segment of Type III junction (No ramp metering)

		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	14	13	10	7
		47	8	13	12	13
		75	8	4	15	12
		103	8	7	14	7
		131	5	11	11	10
		average	8.6	9.6	12.4	9.8
	750	19	11	10	15	16
		47	10	21	21	19
		75	11	10	24	18
		103	7	14	18	14
		131	8	22	17	15
		average	9.4	15.4	19.0	16.4
	1000	19	16	28	33	59
		47	17	17	45	33
		75	17	15	30	37
		103	10	15	36	33
		131	11	18	27	33
		average	14.2	18.6	34.2	39.0
	1250	19	106	638	2559	2631
		47	301	1172	2796	2790
		75	268	952	1815	1962
		103	273	806	1341	1591
		131	96	1029	2066	2592
		average	208.8	919.4	2115.4	2313.2
	1500	19	1754	2674	2838	2781
		47	1851	2469	2891	2913
		75	1835	2590	2755	2758
		103	1768	2409	2819	2856
		131	1785	2667	2750	2805
		average	1798.6	2561.8	2810.6	2822.6
	1750	19	1829	2500	2760	2919
		47	1744	2411	2901	2808
		75	1748	2579	2854	2950
		103	1629	2608	2907	2950
		131	1868	2508	2872	2825
		average	1763.6	2521.2	2858.8	2890.4
	2000	19	1817	2515	2882	2751
		47	1777	2514	2925	2774
		75	1773	2559	2925	2869
		103	1686	2544	2881	2846
		131	1837	2535	2901	2879
		average	1778.0	2533.4	2902.8	2823.8

Table (E.59): Number of rear end conflicts on a 3000 ft freeway segment of Type III junction (2R + 2G)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	2	10	10	7
		47	10	8	11	16
		75	8	8	9	15
		103	9	11	10	11
		131	8	9	7	6
		average	7.4	9.2	9.4	11.0
	750	19	9	15	16	14
		47	8	11	15	14
		75	11	12	10	21
		103	6	17	10	18
		131	7	12	9	17
		average	8.2	13.4	12.0	16.8
	1000	19	12	8	40	54
		47	12	22	34	43
		75	8	21	12	38
		103	14	11	23	51
		131	10	14	18	15
		average	11.2	15.2	25.4	40.2
	1250	19	94	655	2672	2860
		47	371	1225	2759	2667
		75	225	1061	2002	2621
		103	223	910	1462	2138
		131	134	788	2355	2589
		average	209.4	927.8	2250.0	2575.0
	1500	19	1697	2573	2791	2835
		47	1706	2402	2789	2735
		75	1826	2487	2819	2974
		103	1790	2476	2889	2852
		131	1905	2384	2734	2911
		average	1784.8	2464.4	2804.4	2861.4
	1750	19	1773	2543	2758	2811
		47	1714	2448	2886	2794
		75	1859	2612	2749	2880
		103	1801	2544	2716	2717
		131	1678	2481	2929	2952
		average	1765.0	2525.6	2807.6	2830.8
	2000	19	1789	2567	2933	2809
		47	1694	2416	2942	2827
		75	1750	2504	2778	2830
		103	1839	2570	2842	2904
		131	1748	2494	2734	2959
		average	1764.0	2510.2	2845.8	2865.8

Table (E.60): Number of rear end conflicts on a 3000 ft freeway segment of Type III junction (4R + 4G)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	3	9	9	13
		47	6	7	14	10
		75	3	9	4	9
		103	6	9	7	12
		131	7	9	10	11
		average	5.0	8.6	8.8	11.0
	750	19	12	10	20	16
		47	7	13	12	20
		75	13	22	15	17
		103	11	12	18	19
		131	6	12	19	13
		average	9.8	13.8	16.8	17.0
	1000	19	21	37	43	60
		47	7	29	22	28
		75	16	14	21	32
		103	17	15	34	28
		131	11	22	21	19
		average	14.4	23.4	28.2	33.4
	1250	19	175	615	2613	2718
		47	239	1164	2749	2693
		75	280	1059	2127	2377
		103	127	843	1588	1977
		131	146	827	2128	2633
		average	193.4	901.6	2241.0	2479.6
	1500	19	1750	2580	3006	2961
		47	1525	2400	2819	2879
		75	1974	2601	2860	2959
		103	1579	2621	2777	2708
		131	1862	2554	2868	2976
		average	1738.0	2551.2	2866.0	2896.6
	1750	19	1822	2525	2883	2886
		47	1615	2274	2869	2787
		75	1792	2564	2814	2788
		103	1769	2544	2862	2865
		131	1836	2588	2771	2871
		average	1766.8	2499.0	2839.8	2839.4
	2000	19	1723	2586	2830	2848
		47	1589	2480	2750	3003
		75	1897	2474	2799	2841
		103	1662	2495	2946	2766
		131	1661	2438	2875	2790
		average	1706.4	2494.6	2840.0	2849.6

Table (E.61): Number of rear end conflicts on a 3000 ft freeway segment of Type III junction (4R + 2G)						
		Seed No.	Ramp volume (vehicles / hour lane)			
			400	600	800	1000
Freeway volume (vehicles / hour lane)	500	19	3	11	14	12
		47	6	10	15	16
		75	10	12	17	18
		103	12	12	16	15
		131	6	10	11	12
		average	7.4	11.0	14.6	14.6
	750	19	8	18	20	18
		47	12	19	12	13
		75	12	15	12	11
		103	12	15	12	11
		131	12	20	17	15
		average	11.2	17.4	14.6	13.6
	1000	19	23	37	33	35
		47	12	28	22	29
		75	13	36	24	35
		103	27	19	22	30
		131	11	26	12	22
		average	17.2	29.2	22.6	30.2
	1250	19	157	505	542	653
		47	378	1342	1611	1738
		75	298	1068	1043	1085
		103	289	696	849	656
		131	175	656	1012	1033
		average	259.4	853.4	1011.4	1033.0
	1500	19	1997	2827	2782	2833
		47	1900	2740	2825	2860
		75	1856	2766	2766	2774
		103	1844	2722	2837	2687
		131	1684	2873	2885	2730
		average	1856.2	2785.6	2819.0	2776.8
	1750	19	1721	2732	2849	2650
		47	1798	2785	2790	2694
		75	1912	2853	2884	2846
		103	1872	2749	2828	2751
		131	1928	2784	2539	2780
		average	1846.2	2780.6	2778.0	2744.2
	2000	19	1736	2639	2874	2834
		47	1841	2707	2796	2783
		75	1895	2829	2819	2819
		103	1864	3286	2763	2857
		131	1774	2726	2636	2829
		average	1822.0	2837.4	2777.6	2824.4

Table (E.62): Number of conflicts according to severity types on a 3000 ft freeway segment of Type III junction - (No ramp metering)

			Ramp volume (vehicles / hour lane)											
			400			600			800			1000		
		Severity	PO	SL	SE	PO	SL	SE	PO	SL	SE	PO	SL	SE
Freeway volume (vehicles / hour lane)	500	19	2	10	2	3	10	1	3	6	1	1	5	2
		47	1	6	2	2	10	1	4	10	0	1	11	1
		75	0	7	1	1	3	0	4	10	1	5	6	1
		103	2	5	1	2	4	1	5	9	0	1	5	1
		131	1	4	0	0	9	2	1	10	0	1	7	2
		average	1.2	6.4	1.2	1.6	7.2	1.0	3.4	9.0	0.4	1.8	6.8	1.4
	750	19	3	8	1	3	7	1	5	9	2	3	13	2
		47	1	9	1	4	19	1	4	18	0	6	14	1
		75	1	8	2	2	7	1	12	12	0	4	12	1
		103	1	4	2	6	8	1	6	12	2	5	8	2
		131	3	4	2	4	16	2	5	12	0	7	7	1
		average	1.8	6.6	1.6	3.8	11.4	1.2	6.4	12.6	0.8	5.0	10.8	1.4
	1000	19	4	15	0	11	22	1	16	22	1	24	39	1
		47	1	20	1	5	14	0	24	23	0	18	20	0
		75	5	13	0	4	11	0	12	20	0	18	27	0
		103	3	9	2	7	15	1	13	28	1	16	20	3
		131	1	12	2	5	13	3	7	19	2	15	28	0
		average	2.8	13.8	1.0	6.4	15.0	1.0	14.4	22.4	0.8	18.2	26.8	0.8
	1250	19	70	53	0	430	211	12	1774	956	33	1780	1007	38
		47	193	127	5	763	448	13	1877	1075	37	1902	1061	43
		75	177	110	6	625	357	16	1186	686	34	1322	759	22
		103	170	114	2	523	323	23	879	527	20	1070	583	22
		131	67	52	0	661	392	23	1342	818	27	1753	995	24
		average	135.4	91.2	2.6	600.4	346.2	17.4	1411.6	812.4	30.2	1565.4	881.0	29.8
	1500	19	1148	628	17	1798	1015	38	1919	1072	44	1871	1091	44
		47	1175	701	10	1700	904	28	1982	1075	30	1924	1147	59
		75	1257	617	22	1771	956	18	1887	1048	34	1887	1034	43
		103	1180	613	9	1598	916	41	1913	1070	36	1948	1087	43
		131	1183	629	20	1812	987	29	1892	1007	35	1913	1049	31
		average	1188.6	637.6	15.6	1735.8	955.6	30.8	1918.6	1054.4	35.8	1908.6	1081.6	44.0
	1750	19	1215	653	10	1739	895	24	1933	1021	47	1953	1127	40
		47	1123	643	11	1659	879	29	1954	1109	44	1894	1084	45
		75	1174	604	18	1742	977	21	2007	1042	33	2002	1140	47
		103	1077	588	18	1706	999	38	2001	1074	37	1995	1123	36
		131	1207	698	12	1683	958	37	1949	10088	35	1900	1098	46
		average	1159.2	637.2	13.8	1705.8	941.6	29.8	1968.8	2866.8	39.2	1948.8	1114.4	42.8
	2000	19	1164	685	15	1733	917	27	1958	1102	52	1831	1105	27
		47	1161	654	11	1723	925	24	1980	1133	46	1898	1038	47
		75	1174	639	16	1726	944	39	1969	1142	47	1922	1110	37
		103	1138	577	17	1728	934	24	1920	1125	47	1926	1082	43
		131	1229	641	11	1721	923	36	1975	1084	41	1986	1077	43
		average	1173.2	639.2	14.0	1726.2	928.6	30.0	1960.4	1117.2	46.6	1912.6	1082.4	39.4

Note: PO = Potential conflict severity type; SL = Slight conflict severity type ; SE = Serious conflict severity type

Table (E.63): Number of conflicts according to severity types on a 3000 ft freeway segment of Type III junction - (2R + 2G)														
			Ramp volume (vehicles / hour lane)											
			400			600			800			1000		
		Severity	PO	SL	SE	PO	SL	SE	PO	SL	SE	PO	SL	SE
Freeway volume (vehicles / hour lane)	500	19	0	1	1	0	8	2	2	7	1	1	7	1
		47	2	7	2	0	7	1	1	11	2	4	10	2
		75	0	6	2	0	7	1	1	9	1	2	12	1
		103	1	8	1	0	10	2	1	9	2	1	8	2
		131	0	7	1	3	6	0	0	6	1	0	5	1
		average	0.6	5.8	1.4	0.6	7.6	1.2	1.0	8.4	1.4	1.6	8.4	1.4
	750	19	0	8	2	4	11	1	3	13	1	2	12	0
		47	2	7	1	3	7	1	1	11	3	2	12	3
		75	0	9	2	3	7	2	3	9	1	9	10	2
		103	1	5	1	2	12	3	3	6	1	5	13	1
		131	1	6	1	0	10	2	2	7	1	2	14	2
		average	0.8	7.0	1.4	2.4	9.4	1.8	2.4	9.2	1.4	4.0	12.2	1.6
	1000	19	2	15	0	3	7	1	18	27	2	28	31	2
		47	5	8	1	5	23	0	18	21	2	19	30	0
		75	0	7	2	3	19	3	1	10	2	18	22	1
		103	6	16	0	2	15	0	6	25	2	27	32	0
		131	2	13	0	2	14	0	3	17	1	6	12	0
		average	3.0	11.8	0.6	3.0	15.6	0.8	9.2	20.0	1.8	19.6	25.4	0.6
	1250	19	56	55	0	411	245	14	1849	966	38	1905	1097	52
		47	250	140	7	780	478	13	1922	990	36	1872	945	41
		75	140	100	3	698	385	20	1346	753	20	1804	962	36
		103	144	87	6	596	370	15	966	554	18	1463	815	33
		131	90	68	2	529	278	16	1581	913	21	1727	1053	39
		average	136.0	90.0	3.6	602.8	351.2	15.6	1532.8	835.2	26.6	1754.2	974.4	40.2
	1500	19	1130	602	4	1756	938	36	1977	975	36	1961	1046	48
		47	1080	652	10	1592	923	26	1885	1043	37	1852	1046	32
		75	1228	635	16	1714	871	33	1929	1044	37	2044	1124	51
		103	1158	662	7	1708	896	29	1967	1100	48	1961	1060	42
		131	1263	672	14	1646	854	32	1840	1059	53	2029	1023	53
		average	1171.8	644.6	10.2	1683.2	896.4	31.2	1919.6	1044.2	42.2	1969.4	1059.8	45.2
	1750	19	1157	639	19	1743	901	22	1928	989	39	1947	1041	42
		47	1103	642	26	1703	880	25	1943	1089	38	1944	1015	45
		75	1239	658	13	1809	941	31	1884	1022	30	1976	1083	33
		103	1203	634	16	1695	977	32	1867	994	51	1888	1004	45
		131	1130	593	15	1725	882	30	2003	1116	47	2050	1089	45
		average	1166.4	633.2	17.8	1735.0	916.2	28.0	1925.0	1042.0	41.0	1961.0	1046.4	42.0
	2000	19	1185	623	13	1752	921	46	1986	1107	46	1975	1006	41
		47	1088	633	19	1662	854	30	2001	1087	41	1972	1048	49
		75	1182	603	14	1737	869	37	1905	1019	37	1940	1033	48
		103	1232	630	12	1746	937	34	1925	1062	49	1995	1082	56
		131	1142	638	14	1707	914	26	1846	1054	52	2001	1116	48
		average	1165.8	625.4	14.4	1720.8	899.0	34.6	1932.6	1065.8	45.0	1976.6	1057.0	48.4
Note: PO = Potential conflict severity type; SL = Slight conflict severity type ; SE = Serious conflict severity type														

Table (E.64): Number of conflicts according to severity types on a 3000 ft freeway segment of Type III junction - (4R + 4G)

		Ramp volume (vehicles / hour lane)											
		400			600			800			1000		
	Severity	PO	SL	SE	PO	SL	SE	PO	SL	SE	PO	SL	SE
Freeway volume (vehicles / hour lane) 500	19	1	2	1	3	7	1	2	7	1	3	9	2
	47	1	2	3	1	7	1	0	11	3	2	7	2
	75	0	2	1	1	7	1	1	4	0	3	6	0
	103	3	3	0	2	7	0	2	5	1	3	8	1
	131	0	6	1	2	8	0	1	9	0	2	8	2
	average	1.0	3.0	1.2	1.8	7.2	0.6	1.2	7.2	1.0	2.6	7.6	1.4
750	19	0	12	1	3	7	2	2	19	1	5	13	1
	47	0	6	1	1	11	1	3	10	1	5	15	3
	75	3	9	1	9	13	1	7	9	1	5	9	3
	103	2	7	2	3	9	1	7	10	1	5	14	2
	131	1	4	1	2	9	1	7	9	3	3	11	1
	average	1.2	7.6	1.2	3.6	9.8	1.2	5.2	11.4	1.4	4.6	12.4	2.0
1000	19	4	17	2	17	27	0	22	27	1	35	32	1
	47	1	7	2	13	24	0	10	15	1	12	21	1
	75	3	13	2	4	14	0	7	17	1	12	25	0
	103	5	15	2	4	18	0	14	22	2	10	21	3
	131	2	11	2	5	21	1	3	22	2	5	18	2
	average	3.0	12.6	2.0	8.6	20.8	0.2	11.2	20.6	1.4	14.8	23.4	1.4
1250	19	120	77	1	396	230	13	1794	965	38	1830	1044	31
	47	157	119	4	783	409	18	1918	1010	57	1832	1032	47
	75	179	124	5	712	368	15	1447	774	28	1649	851	40
	103	84	53	1	548	333	31	1054	597	21	1367	717	26
	131	93	71	2	525	304	21	1448	789	24	1784	989	40
	average	126.6	88.8	2.6	592.8	328.8	19.6	1532.2	827.0	33.6	1692.4	926.6	36.8
1500	19	1141	650	9	1735	954	23	1983	1189	45	2019	1114	40
	47	1007	543	11	1654	867	23	1932	1030	37	1983	1072	33
	75	1297	708	18	1792	944	32	1970	1046	36	2019	1097	40
	103	1049	567	13	1750	995	39	1940	1000	40	1860	1006	46
	131	1208	686	15	1786	893	19	1957	1063	40	2036	1098	56
	average	1140.4	630.8	13.2	1743.4	930.6	27.2	1956.4	1065.6	39.6	1983.4	1077.4	43.0
1750	19	1212	638	15	1730	925	25	1994	1071	44	1974	1088	28
	47	1076	578	10	1506	874	20	1918	1103	58	1956	1015	48
	75	1189	631	18	1752	914	33	1964	1028	43	1858	1071	54
	103	1174	629	19	1722	961	20	1934	1099	53	1912	1117	48
	131	1217	650	16	1787	940	29	1922	991	46	1975	1062	42
	average	1173.6	625.2	15.6	1699.4	922.8	25.4	1946.4	1058.4	48.8	1935.0	1070.6	44.0
2000	19	1117	635	16	1751	933	40	1912	1062	37	1893	1087	51
	47	1032	592	4	1710	860	31	1920	1010	45	2046	1113	60
	75	1182	737	23	1702	858	35	1905	1060	41	1990	1022	62
	103	1092	600	11	1728	886	36	2017	1116	38	1946	992	44
	131	1098	590	10	1667	863	34	1952	1068	59	1912	1061	45
	average	1104.2	630.8	12.8	1711.6	880.0	35.2	1941.2	1063.2	44.0	1957.4	1055.0	52.4

Note: PO = Potential conflict severity type; SL = Slight conflict severity type ; SE = Serious conflict severity type

Table (E.65): Number of conflicts according to severity types on a 3000 ft freeway segment of Type III junction - (4R + 2G)

			Ramp volume (vehicles / hour lane)											
			400			600			800			1000		
		Severity	PO	SL	SE	PO	SL	SE	PO	SL	SE	PO	SL	SE
Freeway volume (vehicles / hour lane)	500	19	0	2	1	0	10	1	3	11	0	1	11	0
		47	0	5	1	2	7	1	3	11	1	6	9	1
		75	2	6	2	2	9	1	5	11	1	3	12	3
		103	3	8	1	3	7	2	3	12	1	4	10	1
		131	0	6	0	2	7	1	1	10	0	2	9	1
		average	1.0	5.4	1.0	1.8	8.0	1.2	3.0	11.0	0.6	3.2	10.2	1.2
	750	19	2	6	0	6	12	0	2	14	4	2	13	3
		47	3	7	2	1	16	2	3	8	1	1	11	1
		75	0	10	2	3	9	3	0	10	2	1	9	1
		103	2	8	2	3	9	3	3	8	1	1	9	1
		131	3	9	0	4	15	1	3	13	1	2	10	3
		average	2.0	8.0	1.2	3.4	12.2	1.8	2.2	10.6	1.8	1.4	10.4	1.8
	1000	19	2	20	1	7	30	0	4	26	3	5	28	2
		47	2	10	0	2	24	2	4	17	1	5	24	0
		75	3	10	0	12	21	3	5	19	0	12	22	1
		103	7	18	2	2	15	2	1	19	2	10	20	0
		131	1	9	1	6	18	2	1	10	1	2	19	1
		average	3.0	13.4	0.8	5.8	21.6	1.8	3.0	18.2	1.4	6.8	22.6	0.8
	1250	19	84	72	1	304	188	13	324	208	10	400	244	9
		47	224	142	12	837	492	13	1019	574	18	1113	595	30
		75	178	111	9	659	395	14	654	378	11	677	395	13
		103	178	104	7	431	244	21	531	295	23	404	236	16
		131	104	68	3	393	257	6	625	371	16	651	369	13
		average	153.6	99.4	6.4	524.8	315.2	13.4	630.6	365.2	15.6	649.0	367.8	16.2
	1500	19	1273	712	12	1813	973	41	1783	961	38	1814	975	44
		47	1201	695	4	1763	941	36	1864	932	29	1838	996	26
		75	1173	669	14	1763	965	38	1786	955	25	1806	947	21
		103	1212	616	16	1742	935	45	1826	979	32	1715	955	17
		131	1063	611	10	1851	979	43	1840	1004	41	1785	922	23
		average	1184.4	660.6	11.2	1786.4	958.6	40.6	1819.8	966.2	33.0	1791.6	959.0	26.2
	1750	19	1127	580	14	1752	948	32	1796	1016	37	1705	908	37
		47	1145	636	17	1807	949	29	1793	978	19	1750	905	39
		75	1233	658	21	1826	991	36	1841	1022	21	1789	1025	32
		103	1176	670	26	1752	974	23	1821	957	50	1746	986	19
		131	1228	692	8	1783	978	23	1610	898	31	1785	961	34
		average	1181.8	647.2	17.2	1784.0	968.0	28.6	1772.2	974.2	31.6	1755.0	957.0	32.2
	2000	19	1118	604	14	1712	907	20	1811	1023	40	1806	996	32
		47	1157	671	13	1751	929	27	1803	963	30	1807	932	44
		75	1218	659	18	1855	945	29	1805	982	32	1804	977	38
		103	1163	689	12	2088	1152	46	1760	960	43	1828	987	42
		131	1159	597	18	1755	929	42	1693	915	28	1857	940	32
		average	1163.0	644.0	15.0	1832.2	972.4	32.8	1774.4	968.6	34.6	1820.4	966.4	37.6

Note: PO = Potential conflict severity type; SL = Slight conflict severity type ; SE = Serious conflict severity type

Table (E.66): EPC on a 3000 ft freeway segment of Type III junction - No ramp metering Kansas model = 6(F+I) + 1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	46.8	50.8	59.8	51.0
	750	51.0	79.4	86.8	78.2
	1000	91.6	102.4	153.6	183.8
	1250	698.2	2782.0	6467.2	7030.2
	1500	5107.8	7654.2	8459.8	8662.2
	1750	5065.2	7534.2	19404.8	8892.0
	2000	5092.4	7477.8	8943.2	8643.4

Table (E.67): EPC on a 3000 ft freeway segment of Type III junction - (2R + 2G) Kansas model = 6(F+I)+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	43.8	53.4	59.8	60.4
	750	51.2	69.6	66.0	86.8
	1000	77.4	101.4	140.0	175.6
	1250	697.6	2803.6	6703.6	7841.8
	1500	5100.6	7248.8	8438.0	8599.4
	1750	5072.4	7400.2	8423.0	8491.4
	2000	5004.6	7322.4	8597.4	8609.0

Table (E.68): EPC on a 3000 ft freeway segment of Type III junction - (4R + 4G) Kansas model = 6(F+I)+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	26.2	48.6	50.4	56.6
	750	54.0	69.6	82.0	91.0
	1000	90.6	134.6	143.2	163.6
	1250	675.0	2683.2	6695.8	7472.8
	1500	5004.4	7490.2	8587.6	8705.8
	1750	5018.4	7388.6	8589.6	8622.6
	2000	4965.8	7202.8	8584.4	8601.8

Table (E.69): EPC on a 3000 ft freeway segment of Type III junction - (4R + 2G) Kansas model = 6(F+I) + 1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	39.4	57.0	72.6	71.6
	750	57.2	87.4	76.6	74.6
	1000	88.2	146.2	120.6	147.2
	1250	788.4	2496.4	2915.4	2953.0
	1500	5215.2	7781.6	7815.0	7702.8
	1750	5168.2	7763.6	7807.0	7690.2
	2000	5117.0	7863.4	7793.6	7844.4

Table (E.70): EPC on a 3000 ft freeway segment of Type III junction - No ramp metering Massachusetts model = 10F+5I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	45.2	47.6	52.4	49.8
	750	50.8	72.8	77.4	73.0
	1000	81.8	91.4	134.4	160.2
	1250	617.4	2505.4	5775.6	6268.4
	1500	4532.6	6821.8	7548.6	7756.6
	1750	4483.2	6711.8	16694.8	7948.8
	2000	4509.2	6669.2	8012.4	7718.6

Table (E.71): EPC on a 3000 ft freeway segment of Type III junction - (2R+2G) Massachusetts model = 10F+5I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	43.6	50.6	57.0	57.6
	750	49.8	67.4	62.4	81.0
	1000	68.0	89.0	127.2	152.6
	1250	622.0	2514.8	5974.8	7028.2
	1500	4496.8	6477.2	7562.6	7720.4
	1750	4510.4	6596.0	7545.0	7613.0
	2000	4436.8	6561.8	7711.6	7745.6

Table (E.72): EPC on a 3000 ft freeway segment of Type III junction - (4R+4G) Massachusetts model = 10F + 5I + 1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	28.0	43.8	47.2	54.6
	750	51.2	64.6	76.2	86.6
	1000	86.0	114.6	128.2	145.8
	1250	596.6	2432.8	6003.2	6693.4
	1500	4426.4	6668.4	7680.4	7800.4
	1750	4455.6	6567.4	7726.4	7728.0
	2000	4386.2	6463.6	7697.2	7756.4

Table (E.73): EPC on a 3000 ft freeway segment of Type III junction - (4R+2G) Massachusetts model = 10F + 5I + 1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	38.0	53.8	64.0	66.2
	750	54.0	82.4	73.2	71.4
	1000	78.0	131.8	108.0	127.8
	1250	714.6	2234.8	2612.6	2650.0
	1500	4599.4	6985.4	6980.8	6848.6
	1750	4589.8	6910.0	6959.2	6862.0
	2000	4533.0	7022.2	6963.4	7028.4

Table (E.74): EPC on a 3000 ft freeway segment of Type III junction - No ramp metering Virginia model = 12F + 6I + 1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	34.8	35.2	35.2	39.0
	750	40.8	52.4	53.8	54.2
	1000	56.2	63.4	91.2	108.2
	1250	440.2	1847.8	4211.2	4566.0
	1500	3288.6	4972.2	5511.4	5681.4
	1750	3236.4	4888.2	11039.6	5805.6
	2000	3258.8	4872.0	5871.2	5632.6

Table (E.75): EPC on a 3000 ft freeway segment of Type III junction - (2R+2G) Virginia model = 12F+6I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	34.8	37.8	43.0	43.6
	750	38.6	52.2	46.8	59.8
	1000	45.6	59.4	90.8	103.0
	1250	449.2	1843.6	4357.6	5159.8
	1500	3228.0	4746.8	5558.6	5691.2
	1750	3279.6	4819.6	5543.0	5604.2
	2000	3214.8	4833.0	5670.0	5728.4

Table (E.76): EPC on a 3000 ft freeway segment of Type III junction - (4R+4G) Virginia model = 12F+6I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	24.4	30.6	34.8	42.2
	750	38.4	47.4	56.2	65.8
	1000	64.8	73.4	89.8	101.8
	1250	424.2	1814.4	4416.4	4913.8
	1500	3191.2	4861.6	5628.4	5731.6
	1750	3236.4	4772.6	5707.2	5674.8
	2000	3150.2	4774.0	5658.8	5751.2

Table (E.77): EPC on a 3000 ft freeway segment of Type III junction - (4R+2G) Virginia model = 12F+6I+1PDO					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	29.2	40.2	43.2	48.2
	750	40.4	61.6	55.6	54.2
	1000	52.8	92.2	74.4	84.2
	1250	528.6	1631.2	1913.4	1946.8
	1500	3300.6	5149.4	5114.4	4983.0
	1750	3329.8	5031.2	5074.0	5012.4
	2000	3275.0	5143.0	5095.4	5170.8

Table (E.78): cMFs for EPC on freeway of Type III junction - (2R+2G) - Kansas model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	0.94	1.05	1.00	1.18
	750	1.00	0.88	0.76	1.11
	1000	0.84	0.99	0.91	0.96
	1250	1.00	1.01	1.04	1.12
	1500	1.00	0.95	1.00	0.99
	1750	1.00	0.98	0.43	0.95
	2000	0.98	0.98	0.96	1.00

Table (E.79): cMFs for EPC on freeway of Type III junction - (4R+4G)- Kansas model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	0.56	0.96	0.84	1.11
	750	1.06	0.88	0.94	1.16
	1000	0.99	1.31	0.93	0.89
	1250	0.97	0.96	1.04	1.06
	1500	0.98	0.98	1.02	1.01
	1750	0.99	0.98	0.44	0.97
	2000	0.98	0.96	0.96	1.00

Table (E.80): cMFs for EPC on freeway of Type III junction - (4R+2G) - Kansas model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	0.84	1.12	1.21	1.40
	750	1.12	1.10	0.88	0.95
	1000	0.96	1.43	0.79	0.80
	1250	1.13	0.90	0.45	0.42
	1500	1.02	1.02	0.92	0.89
	1750	1.02	1.03	0.40	0.86
	2000	1.00	1.05	0.87	0.91

Table (E.81): cMFs for EPC on freeway of Type III junction - (2R+2G) - Virginia model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	1.00	1.07	1.22	1.12
	750	0.95	1.00	0.87	1.10
	1000	0.81	0.94	1.00	0.95
	1250	1.02	1.00	1.03	1.13
	1500	0.98	0.95	1.01	1.00
	1750	1.01	0.99	0.50	0.97
	2000	0.99	0.99	0.97	1.02

Table (E.82): cMFs for EPC on freeway of Type III junction - (4R+4G) - Virginia model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	0.70	0.87	0.99	1.08
	750	0.94	0.90	1.04	1.21
	1000	1.15	1.16	0.98	0.94
	1250	0.96	0.98	1.05	1.08
	1500	0.97	0.98	1.02	1.01
	1750	1.00	0.98	0.52	0.98
	2000	0.97	0.98	0.96	1.02

Table (E.83): cMFs for EPC on freeway of Type III junction - (4R+2G) - Virginia model					
		Ramp volume (vehicles / hour lane)			
		400	600	800	1000
Freeway volume (vehicles/hour lane)	500	0.84	1.14	1.23	1.24
	750	0.99	1.18	1.03	1.00
	1000	0.94	1.45	0.82	0.78
	1250	1.20	0.88	0.45	0.43
	1500	1.00	1.04	0.93	0.88
	1750	1.03	1.03	0.46	0.86
	2000	1.00	1.06	0.87	0.92

Table (E.84): Average speed (mph) at the ramp influence area of Type III junction - Using different headways - (Freeway traffic volume 1750 vphpl) - Base Case					
Influenced area headway (sec.)	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
0.9	19	26	19.2	16.9	17.3
	47	26.3	20.5	17.4	16.8
	75	25.6	20.2	16.7	17.2
	103	26.6	19.5	17.1	17.1
	131	26.2	20.1	17	16.8
	average	26.1	19.9	17.0	17.0
1	19	27.1	20.3	17.4	17.0
	47	27.4	20.5	17.3	17.3
	75	26.4	19.8	19.8	17.6
	103	27.5	17.6	17.4	17.4
	131	26.8	20.0	17.4	17.2
	average	27.0	19.6	17.9	17.3
1.1	19	27.8	20.9	17.8	17.6
	47	28.2	20.5	18.2	17.4
	75	27.3	20.1	17.6	17.3
	103	28.4	20.0	17.7	17.6
	131	27.8	20.5	17.7	17.9
	average	27.9	20.4	17.8	17.6
1.2	19	28.9	21.0	17.9	18.0
	47	29.0	21.6	18.6	18.6
	75	27.7	21.2	18.2	18.3
	103	29.3	20.9	18.2	18.2
	131	28.2	21.5	18.3	18.2
	average	28.6	21.2	18.2	18.3
1.3	19	29.3	21.1	18.8	19.1
	47	29.5	21.1	18.4	18.6
	75	28.3	22.0	18.5	18.8
	103	29.7	21.8	18.4	18.7
	131	29.5	21.7	18.4	18.5
	average	29.3	21.5	18.5	18.7

Table (E.85): Average speed (mph) at the ramp influence area of Type III junction - Using different headways - (Freeway traffic volume 1750 vphpl) - (4R+2G)					
Influenced area headway (sec.)	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
0.9	19	26.9	20.6	20.8	20.1
	47	27.0	20.5	20.1	20.6
	75	26.9	20.0	20.2	20.1
	103	26.6	20.7	20.3	20.5
	131	26.5	20.2	20.4	20.3
	average	26.8	20.4	20.4	20.3
1	19	27.3	21.2	20.3	20.8
	47	27.2	21.1	20.7	20.8
	75	26.9	20.7	20.6	20.8
	103	27.6	21.5	20.1	20.6
	131	27.6	21.2	20.8	20.6
	average	27.3	21.1	20.5	20.7
1.1	19	28.5	21.7	21.1	21.2
	47	28.8	21.7	20.9	21.2
	75	27.7	20.9	20.8	21.1
	103	28.1	21.1	21.2	21.3
	131	27.9	21.5	21.7	21.2
	average	28.2	21.4	21.1	21.2
1.2	19	28.1	21.4	21.5	21.5
	47	28.5	21.8	21.7	21.1
	75	28.2	22.1	21.2	21.8
	103	29.8	21.7	21.6	21.6
	131	28.9	22.1	21.7	21.2
	average	28.7	21.8	21.5	21.4
1.3	19	30.2	22.9	22.3	22.4
	47	30.4	22.7	22.4	22.0
	75	28.5	21.9	22.4	22.4
	103	29.6	22.6	21.9	21.8
	131	29.9	22.1	22.0	22.4
	average	29.7	22.4	22.2	22.2

Table (E.86): Traffic conflict number on 3000 feet freeway segment of Type III junction - Using different headways - (Freeway Traffic Volume 1750 vphpl) - Base Case					
Influenced area headway (Sec.)	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
0.9	19	4262	4965	5266	5284
	47	4226	4884	5306	5458
	75	4382	4800	5423	5280
	103	4247	4962	5313	5419
	131	4380	4907	5343	5241
	average	4299.4	4903.6	5330.2	5336.4
1	19	2877	3922	4209	4340
	47	2922	3811	4324	4319
	75	3009	3931	4270	4237
	103	2883	3873	4204	4295
	131	3162	3947	4343	4254
	average	2970.6	3896.8	4270	4289
1.1	19	1878	2658	3001	3120
	47	1777	2567	3107	3023
	75	1796	2740	3082	3189
	103	1683	2743	3112	3154
	131	1917	2678	3072	3044
	average	1810.2	2677.2	3074.8	3106.0
1.2	19	961	1675	2103	2175
	47	908	1600	1865	1976
	75	1099	1592	2040	2052
	103	909	1693	2073	2131
	131	1000	1653	1957	1990
	average	975.4	1642.6	2007.6	2064.8
1.3	19	545	1000	1348	1360
	47	480	961	1338	1372
	75	524	1146	1296	1320
	103	466	1020	1372	1327
	131	482	1011	1293	1331
	average	499.4	1027.6	1329.4	1342.0

Table (E.87): Traffic conflict number on 3000 feet freeway segment of Type III junction - Using different headways - (Freeway Traffic Volume 1750 vphpl) - (4R+2G)					
Influenced area headway (Sec.)	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
0.9	19	4129	4953	4978	5073
	47	4134	5004	4980	4920
	75	4185	5060	5033	4950
	103	4215	5023	4894	4912
	131	4369	5055	4932	5034
	average	4206.4	5019.0	4963.4	4977.8
1	19	2842	3831	4051	3901
	47	2894	3916	3878	3844
	75	3025	3792	3977	3961
	103	3038	3736	3993	4097
	131	2967	3767	3957	3994
	average	2953.2	3808.4	3971.2	3959.4
1.1	19	1721	2732	2849	2650
	47	1798	2785	2790	2694
	75	1912	2853	2884	2846
	103	1872	2749	2828	2751
	131	1928	2784	2539	2780
	average	1846.2	2780.6	2778.0	2744.2
1.2	19	1056	1627	1823	1758
	47	999	1681	1788	1779
	75	1048	1547	1814	1836
	103	879	1676	1755	1813
	131	933	1755	1656	1879
	average	983.0	1657.2	1767.2	1813.0
1.3	19	473	1051	1084	1136
	47	461	1025	1061	1107
	75	587	1159	1162	1124
	103	437	1044	1146	1195
	131	417	1141	1121	1188
	average	475.0	1084.0	1114.8	1150.0

Table (E.88): Average speed (mph) at the ramp influence area of Type III junction - Using different traffic composition-(Freeway traffic volume 1750 vphpl) - Base Case					
Percentage of trucks and buses	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
3	19	27.8	20.9	17.8	17.6
	47	28.2	20.5	18.2	17.4
	75	27.3	20.1	17.6	17.3
	103	28.4	20	17.7	17.6
	131	27.8	20.5	17.7	17.9
	average	27.9	20.4	17.8	17.6
5	19	26.1	19.1	17.4	17.5
	47	25.7	20.5	17.5	17.2
	75	25.5	19.9	17.2	17.3
	103	25.5	20.0	17.2	17.2
	131	25.4	20.2	17.7	17.7
	average	25.6	19.9	17.4	17.4
7	19	25.0	18.8	17.2	17.1
	47	25.0	19.4	17.1	16.8
	75	24.4	19.5	17.2	17.2
	103	24.9	19.1	17.2	17.0
	131	25.1	19.3	17.4	17.2
	average	24.9	19.2	17.2	17.1
9	19	23.8	19.2	16.8	17.0
	47	23.4	18.8	16.6	16.6
	75	23.5	18.6	17.0	16.8
	103	23.0	18.5	16.7	16.6
	131	23.5	18.7	16.8	17.0
	average	23.4	18.8	16.8	16.8
11	19	22.7	17.9	16.4	16.9
	47	22.6	18.4	16.3	16.5
	75	22.3	18.7	16.3	16.6
	103	22.2	17.8	16.7	16.3
	131	22.6	18.6	16.5	16.8
	average	22.5	18.3	16.4	16.6

Table (E.89): Average speed (mph) at the ramp influence area of Type III junction -Using different traffic composition-(Freeway traffic volume 1750 vphpl) - (4R+2G)					
Percentage of trucks and buses	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
3	19	28.5	21.7	21.1	21.2
	47	28.8	21.7	20.9	21.2
	75	27.7	20.9	20.8	21.1
	103	28.1	21.1	21.2	21.3
	131	27.9	21.5	21.7	21.2
	average	28.2	21.4	21.1	21.2
5	19	25.6	19.8	17.3	17.0
	47	26.2	20.5	17.6	17.5
	75	25.5	20.0	17.9	17.6
	103	26.6	19.5	17.3	16.9
	131	26.0	20.0	17.1	17.2
	average	26.0	20.0	17.4	17.2
7	19	24.3	19.2	16.3	16.9
	47	24.5	19.3	17.0	16.9
	75	24.2	19.8	17.5	17.2
	103	24.5	19.0	17.1	16.8
	131	25.0	19.7	17.4	16.9
	average	24.5	19.4	17.1	16.9
9	19	23.9	18.7	16.7	16.4
	47	22.8	18.5	16.8	16.8
	75	23.6	18.8	16.6	17.0
	103	23.5	18.7	16.8	16.6
	131	23.4	19.1	17.0	17.0
	average	23.4	18.8	16.8	16.8
11	19	22.4	18.6	16.3	16.5
	47	22.5	18.5	16.5	16.0
	75	22.7	18.7	17.1	16.4
	103	22.8	18.2	16.9	16.1
	131	22.9	18.3	16.8	16.7
	average	22.7	18.5	16.7	16.3

Table (E.90): Traffic conflict number on a 3000 ft freeway segment of of Type III junction - Using different traffic composition - (Freeway traffic volume 1750 vphpl) - Base Case					
Percentage of trucks and buses	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
3	19	1878	2658	3001	3120
	47	1777	2567	3107	3023
	75	1796	2740	3082	3189
	103	1683	2743	3112	3154
	131	1917	2678	3072	3044
	average	1810.2	2677.2	3074.8	3106.0
5	19	1986	2716	3094	3111
	47	1986	2683	3023	3071
	75	2115	2787	3094	3196
	103	2168	2728	3154	3299
	131	2126	2712	3144	3084
	average	2076.2	2725.2	3101.8	3152.2
7	19	2088	2888	3083	3151
	47	1941	2782	3074	3336
	75	2166	2968	3054	3091
	103	2068	2941	3276	3240
	131	2262	2917	3229	2994
	average	2105.0	2899.2	3143.2	3162.4
9	19	2213	2855	3200	3345
	47	2391	2853	3369	3203
	75	2310	2878	3075	3332
	103	2447	2874	3306	3300
	131	2452	3000	3318	3304
	average	2362.6	2892.0	3253.6	3296.8
11	19	2474	3086	3318	3125
	47	2407	3077	3208	3151
	75	2494	2910	3111	3276
	103	2480	3051	3235	3234
	131	2394	2928	3313	3221
	average	2449.8	3010.4	3237.0	3201.4

Table (E.91): Traffic conflict number on a 3000 ft freeway segment of Type III junction - Using different traffic composition - (Freeway Traffic Volume 1750 vphpl) - (4R+2G)					
Percentage of trucks and buses	Seed	Ramp traffic volume (vehicles / hour lane)			
		400	600	800	1000
3	19	1721	2732	2849	2650
	47	1798	2785	2790	2694
	75	1912	2853	2884	2846
	103	1872	2749	2828	2751
	131	1928	2784	2539	2780
	average	1846.2	2780.6	2778.0	2744.2
5	19	2042	2711	3195	3213
	47	1897	2610	3044	3098
	75	2145	2823	3188	3101
	103	1829	2740	3398	3054
	131	2025	2751	3110	2980
	average	1987.6	2727.0	3187.0	3089.2
7	19	2247	2881	3287	3152
	47	2269	2819	3262	3054
	75	2296	2860	3100	3098
	103	2093	2765	3126	3206
	131	2160	2868	3176	3154
	average	2213.0	2838.6	3190.2	3132.8
9	19	2197	2949	3235	3178
	47	2506	2981	3065	3074
	75	2418	2861	3380	3241
	103	2251	2993	3129	3249
	131	2334	3006	3179	3183
	average	2341.2	2958.0	3197.6	3185.0
11	19	2437	2926	3405	3170
	47	2537	2937	3063	3208
	75	2377	3004	3014	3264
	103	2438	2976	3322	3188
	131	2354	3098	3058	3219
	average	2428.6	2988.2	3172.4	3209.8