TRANSPORTATION SAFETY MODELING AND EVALUATION: ALTERNATIVE GEOMETRIC DESIGNS, ENFORCEMENT, AND AIRFIELD APPLICATIONS

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The undersigned, appointed by the Dean of the Graduate School, have examined the thesis entitled

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

A top priority of transportation agencies in the United States is to improve safety of transportation facilities through the use of latest technology, innovative designs, procedural methods, and training practices to decrease fatalities, injuries, and property damage.

In order to continue improving roadway safety, different approaches such as alternative designs have been considered. Alternative designs for roadway facilities include J-turn for minor roads and high-speed expressway intersections, the Diverging Diamond Interchange (DDI) for freeway interchanges, or red light cameras for signalized intersections. There is limited research evaluating the safety effectiveness of recently implemented alternative designs and enforcement strategies. This dissertation focused on developing jurisdiction specific crash prediction models, calibrating existing models, and applying rigorous statistical methods to study the safety effectiveness of these new alternative treatments.

This dissertation found that the DDI design replacing a conventional diamond decreased crash frequency for all severities. Fatal and injury (FI) crashes experienced a 62.6% reduction. Property damage only (PDO) crashes reduced by 35.1% and total (TOT) crashes decreased by 47.9%. The collision diagram analysis of the DDI showed that the top two crash types were: 1) rear end collisions between right turning movements on the exit ramp at the intersection, and 2) rear end collisions on the outside crossroad approach leg to the ramp terminal. The DDI design traded a severe crash type, right angle left turn crash, with less severe rear end, sideswipe, and loss of control crash types. Wrong way crashes inside the crossroad between ramp terminals accounted for 4.8% of the FI crashes occurring at the DDI. This dissertation also examined the DDI safety effect on two adjacent facilities: speed change lanes and major signalized intersections. There is

no strong evidence that DDIs impacted the safety of adjacent roadway facilities, either positively or negatively.

Another alternative intersection design studied in this dissertation was the J-turn intersection. The safety evaluation of the implementation of the J-turn replacing two-way stop-controlled intersections was effective at decreasing FI crashes by 63.8% and TOT crashes by 31.2%. The collision diagram analysis showed that the most recurrent crashes were sideswipe with 31.6% and rear end with 28.1% on the main road.

Red light running was also evaluated in this dissertation. The implementation of red light running cameras in Missouri resulted in a reduction of FI crashes by 7.4% and increase in PDO crashes by 3.8%. Additionally, right angle crashes were reduced across all severities, including 14.5% for FI crashes. Rear end crashes increased by 16.5% overall. The crash cost benefit results showed a positive net economic benefit of $35,269 per site per year in 2001 dollars (approximately $47,000 in 2015 dollars). It translated into an overall 5.0% economic crash cost benefit.

In addition to roadway safety, this dissertation also evaluated airfield safety. In the field of aviation, runway incursions are the incorrect presence of and aircraft, vehicle or person on an active runway designated for takeoff and landing. Runway incursions can result in property damage or loss of life through incidents leading to aircraft collisions or avoidance maneuvers. Efforts are on the rise to reduce the risk of runway incursions at airports. However, guidance is mostly qualitative and does not provide specific quantitative measures to predict runway incursion frequency and evaluate the effectiveness of treatments. This dissertation adapted statistical roadway safety modeling to airport airfield operations. The transferability of roadway safety modeling theory was possible because airfield operations share similar measures of exposure and hazard concepts. Thus, models were developed to estimate runway incursion frequency for hub airports in the United States. Assessing runway incursion frequency and treatment effectiveness with quantitative measures enables a more straightforward comparison of different facilities, alternatives, and treatments. The models developed in this dissertation contribute to decision making and the implementation of cost effective countermeasures to mitigate runway incursions.

1. INTRODUCTION

The introduction chapter is divided in two: 1) roadway and 2) airfield applications. Roadway applications cover an overview of the alternative geometric and enforcement designs. Detailed explanation of crash reporting and its important in safety analysis is covered. The second section focuses on the airfield applications providing a thorough literature review of existent runway incursion practices and airfield safety modeling.

1.1. Roadway Applications

1.1.1. Diverging Diamond Interchange Recently in the US, the DDI has become a popular alternative to other forms of interchange designs. Since the first DDI installation in Springfield, Missouri, in 2009, there have been more than 50 locations across the US where DDIs have been installed. Figure 1.1. shows an aerial image of the DDI located in I-270 and Dorsett Rd., Maryland Heights, Missouri.Operations are based on two-phased signals that contribute to lower delays, increased capacity compared to standard diamonds, and left turns may be free flow (Bared et al., 2006; Edara et al. 2005).

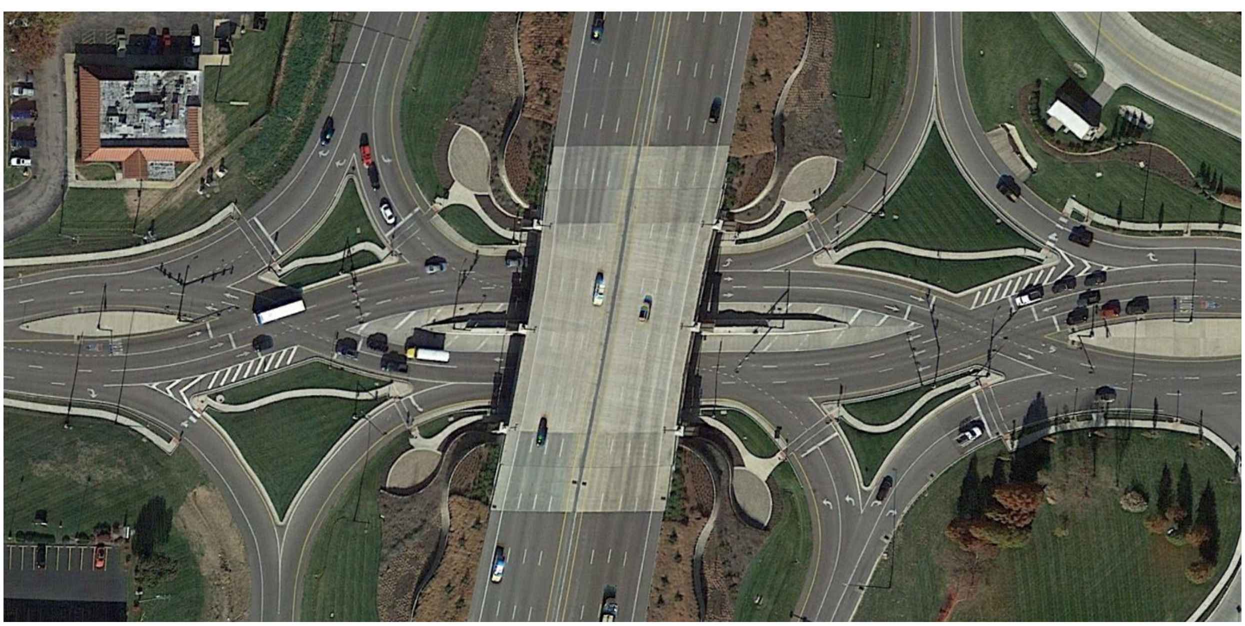


Figure 1.1. I-270 and Dorsett Rd., Maryland Heights, Missouri
(Image Lansat/Copernicus, Google 2016)

Speeds through the interchange are reduced with the crossover geometry and reverse curves. The DDI has 18 conflict points (2 crossing, 8 merging, and 8 diverging) while the conventional diamond interchange has 30 conflict points (10 crossing, 10 merging, and 10 diverging). Figure 1.2. illustrates a comparison of the conflict points between a conventional diamond interchange and the DDI.

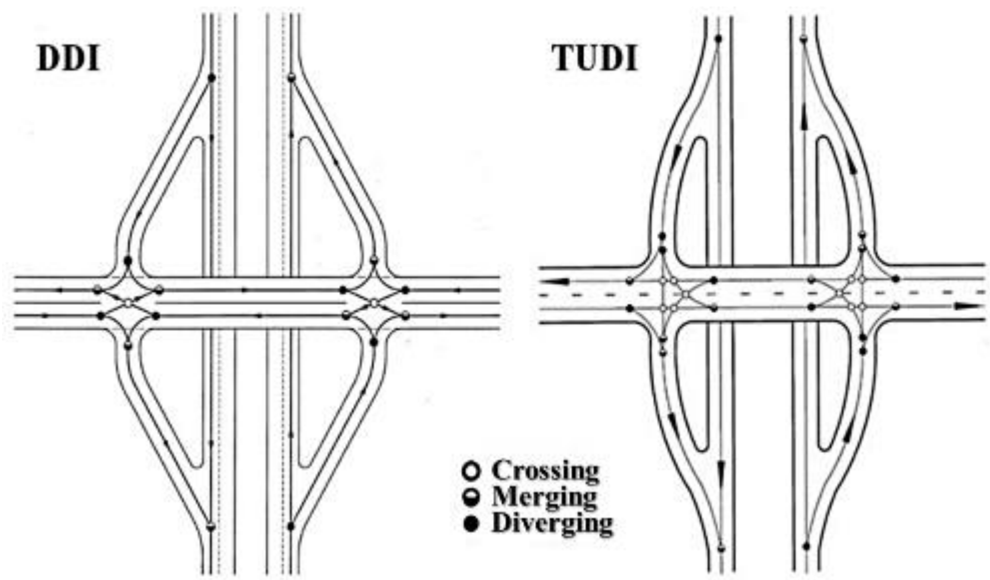


Figure 1.2. Conflict Points at DDI and TUDI Interchanges (FHWA, 2004)

Fewer conflict points across all conflict types reduce the exposure of traffic to crashes. The configuration of the DDI improves safety by reducing the number of conflict points over other interchange designs—8 out of 10 crossing conflicts are eliminated when compared to a conventional diamond (Schroeder et al., 2014). Crossing conflicts typically result in right angle collisions that have a higher potential for injuries (Hughes et al., 2010). Unfamiliarity has been a major concern due to the potential for wrong way maneuvers at crossovers. As part of a field evaluation performed by the FHWA (Vaughan et al., 2015), five DDIs were monitored over a period of six months with video detection software. The results of the study showed that wrong way maneuvers were common when vehicles first entered the DDI and during nighttime. Although the study recorded a significant number of wrong way maneuvers, no wrong way crashes occurred during the period of study (Vaughan et al., 2015).

Other benefits of DDIs include reduced cost and improved constructability. Limited funding for transportation facilities has forced agencies to consider cost effective designs. Construction cost estimates are associated with retrofitting, additional structural elements, or new designs construction. In projects in which the DDI can be adjusted to the footprint and existing right of way, costs could be as low as $2.9-$4.5 million. In cases in which the retrofit design requires the addition of other structural elements, costs may be around $8 million. A newly constructed DDI could cost approximately $14-$22 million based on the magnitude of the project (Schroeder et al., 2014; FHWA, 2014). In addition to the reduced cost, constructability is significantly improved in basic retrofitting conditions. A feasibility analysis conducted by Missouri DOT evaluated the construction season of different forms of interchanges compared to a DDI at Interstate 435 and Front Street in Kansas City, Missouri. The evaluation concluded that the DDI construction would take a single season compared to two seasons with a diamond interchange (MoDOT, n.d.; MoDOT, 2014). For instance, the retrofitting construction of the first DDI at I-44 and MO-13 in Springfield, Missouri cost $3.2 million and took six months to implement (MoDOT, 2014).

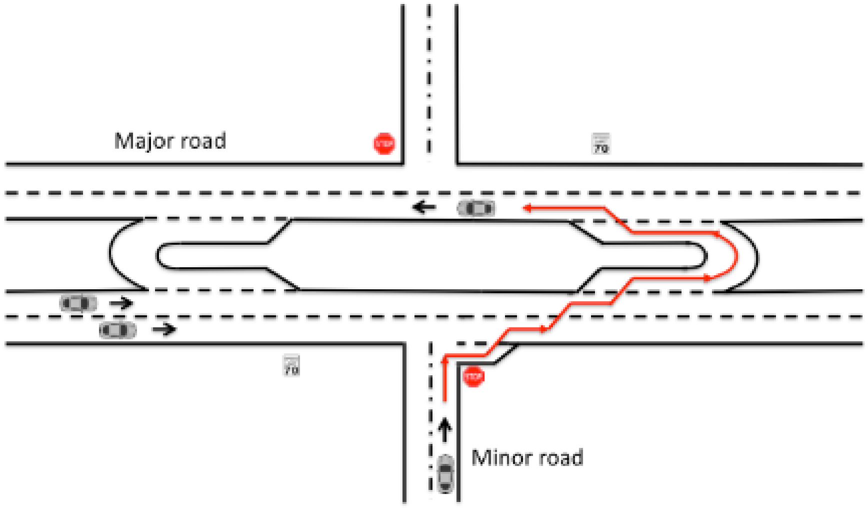
The main impetus behind the initial research on the DDI was to evaluate its operational benefits as compared to other designs. While the seminal study of Chlewicki (2003) illustrated the delay savings resulting from a DDI, the follow-up studies by Bared et al. (2006) and Edara et al. (2005) further confirmed its operational benefits, specifically the doubling of left turn movement capacity. Several subsequent studies have agreed with these early studies on the operational benefits of DDIs (Chlewicki, 2013; Chilukuri et al., 2011). Because the motivation behind the initial research into the DDI was improving operational benefits, there has been a gap in the existing knowledge pertaining to the safety performance of the DDI. Typically, empirical safety evaluations of new alternative designs are not possible until a few years after they are introduced into practice due to the lack of sufficient crash data. One study (Chilukuri et al., 2011) reviewed crash data for a one-year period after the first DDI was constructed in Springfield, Missouri. The study concluded that the DDI was operating safely based on a comparison of before and after crash frequencies. But the small sample size did not allow for a rigorous statistical safety evaluation.

The current study performed in this dissertation aims to fill the knowledge gap in the safety of the DDI. Data from Missouri were used to conduct a before-after evaluation of the DDI. Missouri was the first state to have built a DDI and has the largest number of DDIs built or under construction. Thus, Missouri offers a rich dataset for conducting a safety evaluation of DDIs. This study makes key contributions to the body of literature on DDI performance. First, this is the first study to conduct a system-wide safety evaluation using multiple DDI sites. Second, this study offers the first extensive safety evaluation of DDI using three before-after analysis methods. Third, crash modification factors (CMF) for total, fatal and injury, and property damage only crashes for a DDI were developed for the first time in this study. The CMF values provide the expected reduction in crashes achieved by a DDI as compared to a conventional diamond interchange. Fourth, an extensive review of the collision diagrams was conducted to derive trends in the types of crashes before and after a DDI was installed at the study sites.

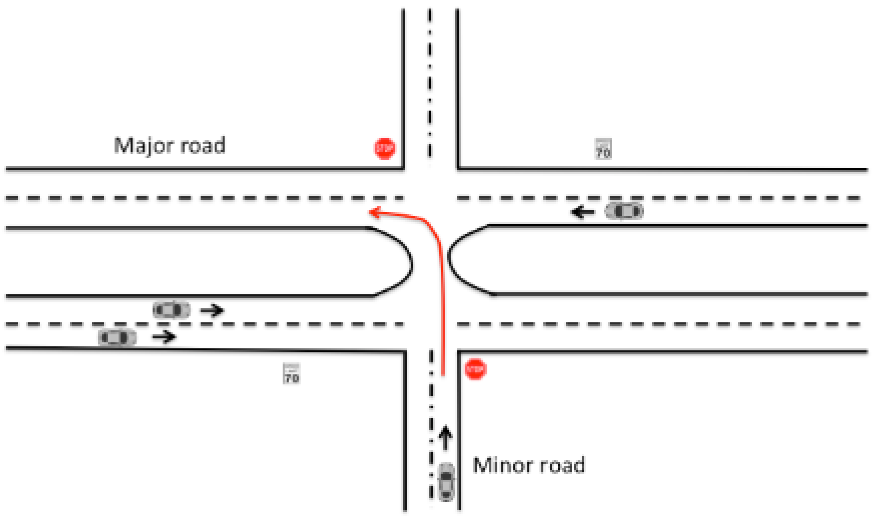
1.1.2. J-turn The majority of crashes occurring at unsignalized intersections on high-speed rural expressways are right-angle crashes resulting from turning movements (Maze et al., 2010). For example, the proportion of right-angle crashes at rural high-speed expressways in the states of Minnesota, Utah, and Iowa are 57%, 69%, and 52%, respectively (Maze et al., 2010). The issue of right-angle crashes is of concern to many states, since this crash type exhibits an elevated percentage of fatal and serious injuries. The J-turn is an alternative design with fewer and less severe conflict points than conventional two-way stop control intersections. Figure 1.3. shows the aerial image of a J-turn intersection.

Figure 1.3. US-65 and Rochester R., Ridgedale, Missouri
(Image Lansat/Copernicus, Google 2016)

Conceptual schematics of two-way stop control and J-turn intersections are illustrated in Figure 1.4. At two-way stop controlled intersection on a four-lane divided highway, vehicles accessing the major highway from the minor road can make a left turn or through movement at the intersection by crossing major road movements. Highways with high volumes or high speeds may make these minor road movements difficult to execute, and cause long delays. In contrast, in a J-turn design, vehicles accessing the major highway from the minor road make a right turning movement and then use a U-turn at a downstream location. The major road vehicles accessing the minor road via a left turning movement may or may not have to use the U-turn for their movements. One variation of the J-turn design allows for major road turning movements to occur at the intersection, but still requires the minor road movements to use the U-turn. Figure 1.4.a depicts the left-turning movement from the minor road at the two-way stop controlled intersection. Figure 1.4.b depicts the left-turning movement from the minor road at the J-turn intersection.



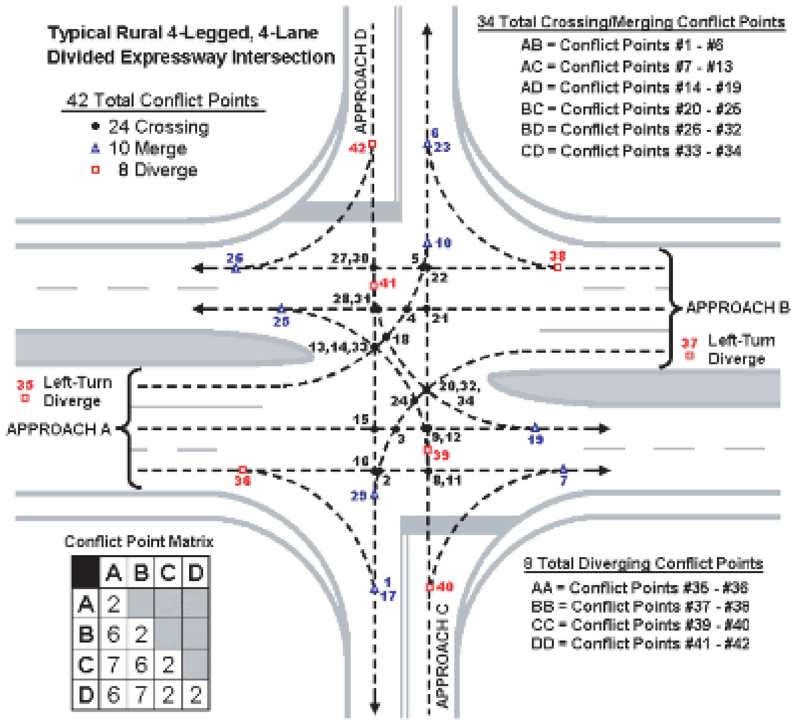
a) Two-Way Stop Controlled Intersection



b) J-turn Intersection

Figure 1.4. Left Turn Movement Diagrams

The safety of the J-turn design stems from the elimination of severe high-risk conflict points. A two-way stop controlled intersection has 42 conflict points, while a J-turn intersection has 24 conflict points (Maze et al., 2010). Figure 1.5. illustrates the comparison of conflict points. Not only does the J-turn have fewer total conflict points, but it eliminates the most severe forms of conflict, i.e., crossing conflicts that result in right-angle crashes.



a) Two-way Stop Controlled Intersection

b) J-Turn

Figure 1.5. Comparison of Conflict Points (Maze et al., 2010)

Empirical research documenting the safety effectiveness of J-turn design is limited. An evaluation was conducted of a restricted crossing U-turn (RCUT) design in Maryland (Inman and Haas, 2012); the RCUT and Superstreet designs are alternative names for the J-turn design. Reported results using the Empirical Bayes method revealed a 44% reduction in total crash frequency for J-turns in Maryland (Inman and Hass, 2010) and a 27.2% reduction in North

Carolina (Hummer et al., 2010). In terms of reduction in crash severity, Maryland J-turns witnessed 70% and 42% reductions in fatalities and injury crashes, respectively (Inman and Hass, 2010). In North Carolina, J-turns resulted in a 51% reduction in fatal and injury crashes (Hummer et al., 2010).

This dissertation makes a comprehensive evaluation of J-turn safety effectiveness using collision diagrams and statistical analysis. First, this study is unique in that it applies a project-level Empirical Bayes analysis to study of the safety effectiveness of the entire footprint of the J-turn treatment. The footprint includes the main intersection, the two U-turns, and the road segments between. Thus, this study contrasts with safety studies that focus on the intersection only. Second, the crash reduction percentages obtained using the Empirical Bayes method can be used as guidance for future J-turn installations. The crash reductions are in agreement with those witnessed in the Maryland and North Carolina studies. Third, for the first time, collision diagram analysis provided observable trends of crash frequency by type according to the spacing of the U-turns from the minor road.

1.1.3. Red Light Cameras Automated enforcement systems such as red light cameras (RLC) have generated heated discussions over issues of transportation safety, economics, and laws. The objective of implementing RLCs at signalized intersections is to reduce red light running violations and the resulting crashes. This study reviewed the current RLC literature and examined RLC programs in Missouri.

Several studies have evaluated the effect of RLC on red light running frequency in cities such as Fairfax, VA (Retting et al., 1999), Oxnard, CA (Retting et al., 1999a) in the United States, and in other countries including Singapore, Great Britain, Australia, Netherlands, and Canada (Retting et al., 2003). The results of these studies indicate that the benefits of automated enforcement reduce the number of red-light violations between 40-50% and some positive spillover effects to non-RLC equipped intersections (Retting et al., 2003). Similarly, there has been a significant reduction in crash frequency. Many studies reported exaggerated and statistically biased estimates of the effectiveness of RLC. However, some of these studies lacked methodological rigor and statistical significance, which fueled the counter-argument against benefits of RLCs. It was not until 2005 when a federally-funded study, conducted by nationally recognized safety experts, produced better information on RLC effectiveness (Council et al. 2005; Council et al., 2005a; Persaud et al., 2005). The study included significant data from seven jurisdictions and rigorous statistical methods. The implementation of RLC was found to have an overall positive effect on safety. Furthermore, recent studies in North Carolina, Virginia, and Arizona supported the findings of the national study (Miller et al., 2006; Shin and Washington, 2007; Pulugurtha and Otturu, 2014). Pulugurtha and Otturu (2014) found RLC had beneficial safety effects at intersections over a period of time after the automated enforcement was terminated.

Hu et al. (2011) examined aggregated per capita fatal crashes in 99 large US cities, and found RLCs were associated with statistically significant reductions in city-wide rates of fatal red-light running crashes. A meta-analysis of present studies of RLC also found favorable safety effects of the system (Høye, 2013). When individual intersection performance is analyzed in addition to aggregate performance, it is possible to examine the appropriateness of RLC use at an individual site. Despite general guidance on site selection for RLC treatment (Council et al., 2005), there is not a specific quantitative measure or methodological procedure to determine best site locations. Therefore, the cumulative experience of RLC programs from different jurisdictions provides a good historical database of RLC site characteristics. With more recent data in Missouri and accurate statistical methods, this study contributes updated safety estimates to the existing knowledge for furthering the study of RLC.

A commonly used argument against the use of RLC is that rear end crashes are increased while reducing right angle crashes. However, the severity of angle crashes and rear ends are very different, thus there is a need for an overall crash cost analysis (Council et al., 2005). In terms of economics, RLC revenue generation has been a concern in the eyes of the public. Some accuse cities of developing automated enforcement programs in order to generate revenue rather than to promote public safety (Sun, 2011). In general, this is not the case since many automated enforcement programs in the US generate little revenue, are revenue neutral, or require subsidy. In terms of economic benefits of RLC, crash costs can be quantified with aggregated economic costs across crash types and severity levels, including material and life losses. RLC were found to have a net economic benefit of approximately $38,000 per site per year—in 2001 dollars (Council et al., 2005; Council et al., 2005a). In addition to the economic aspect of RLC, several legal issues had restricted or terminated the implementation of automated enforcement programs.

Some legal issues involving RLC programs are: procedural, evidentiary, substantive, and state law preemption (Sun, 2011). Under procedural issues, a violator who is charged should receive all the due processes granted by the Fourteenth Amendment. The administrative program should provide notice, a hearing, the opportunity to present evidence, and the additional opportunity to appeal. Evidentiary issues include the process of authentication of the photographs. Since it is an automated process and done by a machine, some jurisdictions required that the photographic evidence had to include the officer's own observations and comments before the violator could be charged. Other legal substantive issues involve the burden of the fine falling under the owner of the vehicle or the driver. Many jurisdictions considered that it was rational to presume that the owner is responsible for the conduct of the vehicle operator. Some issues pertain to the case of a violator who is a lessee and has no ownership of the vehicle. In some cases, this issue was solved by reissuing the fine to the violator (lessee). In a few cases, the violator's failure to submit a certificate of innocence did not violate the right against self-incrimination, because the violator is not exposed to any criminal liability. Jurisdictional issues may arise when programs are administered by municipalities in which some administrative processes may be omitted, so state law can overrule the programs since they conflict with statutory requirements. Many of these issues have led to the termination of several RLC programs in different jurisdictions.

In the state of Missouri, the implementation of RLC has not been studied. Public perception in Missouri has been based mainly upon media coverage and court decisions. Also, only a few studies in other states have applied techniques that accounted for sampling bias and regression to the mean. This dissertation presents the first RLC evaluation that used the Highway Safety Manual (HSM) (AASHTO, 2010) methodology including a comprehensive safety evaluation and crash cost-benefit analysis. This study presents the use of more recent data which captures the technological and driver behavioral changes in the past decade including the recent proliferation of mobile devices and the associated distracted driving problems. A review of court decisions that terminated several RLC programs in Missouri is also presented. Although the RLC legal challenges in Missouri differ from other states, it is illustrative of the legal and public policy hurdles that go beyond the issue of safety effectiveness.

1.1.4. Crash Reporting Issues The availability of consistent and reliable crash data supports the development of effective safety countermeasures. However, significant issues exist with the crash reporting practices which needed to be addressed to prepare accurate crash data. To that end, thousands of crash reports were reviewed on a case by case basis. In this dissertation, the most significant findings of crash reporting problems are explained. Also, the consequences of using inconsistent and possibly inconsistent crash data in safety evaluations are described. To fix these inconsistencies in the data, a tutorial was developed to establish a methodology to review and correct crash data. The tutorial was used to train researchers to consistently review crash reports and validate the data.

1.1.4.1. Crash Data Entry Crash report forms provide different sections to document detailed information about the location, people, vehicles, and circumstances of crashes. In addition to the crash report form, there are training manuals explaining in detail every field, section, notation, and criteria to fill out the form (ACTAR, 2015). For some time until 2012, most states allowed crash reports to be filed manually making it even more complicated to tabulate and process the data. Figure 1.6., shows an example of a section of an actual crash report filed in 2006 in Missouri.

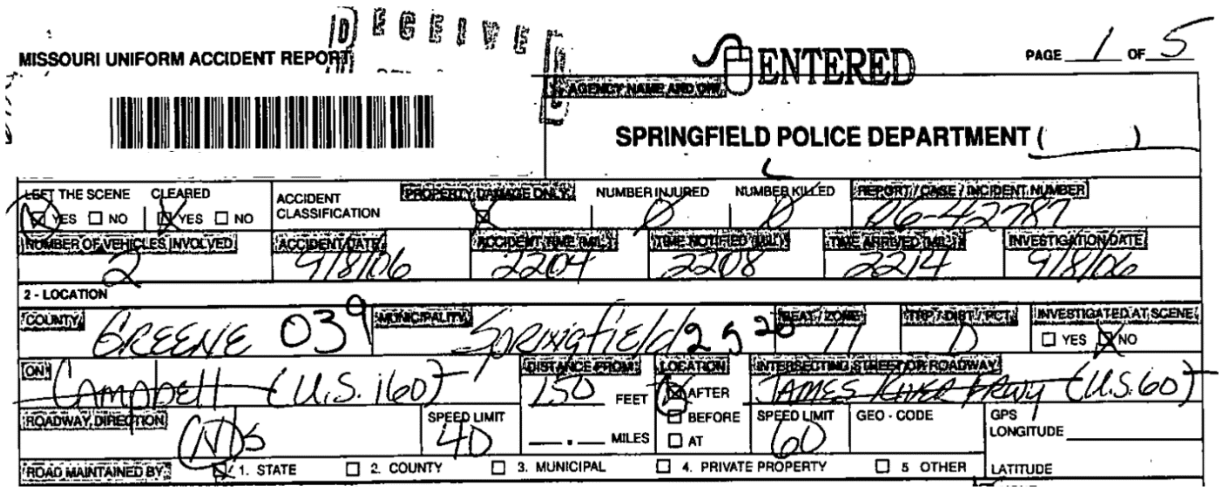

MISSOURI UNIFORM ACCIDENT REPORT RECEIVED ENTERED PAGE 1 OF 5
AGENCY NAME AND ORI: SPRINGFIELD POLICE DEPARTMENT ( )
LEFT THE SCENE ☒ YES ☐ NO | CLEARED ☒ YES ☐ NO | ACCIDENT CLASSIFICATION | PROPERTY DAMAGE ONLY ☒ | NUMBER INJURED 0 | NUMBER KILLED 0 | REPORT / CASE / INCIDENT NUMBER 06-42787
NUMBER OF VEHICLES INVOLVED 2 | ACCIDENT DATE 9/8/06 | ACCIDENT TIME (MIL.) 2204 | TIME NOTIFIED (MIL.) 2208 | TIME ARRIVED (MIL.) 2214 | INVESTIGATION DATE 9/8/06
2 - LOCATION
COUNTY GREENE 039 | MUNICIPALITY Springfield 2520 | BEAT / ZONE 17 | TRP / DIST / PCT D | INVESTIGATED AT SCENE ☐ YES ☒ NO
ON Campbell (U.S. 160) | DISTANCE FROM 150 FEET ☒ AFTER ☐ BEFORE ☐ AT ___.___ MILES | LOCATION | INTERSECTING STREET OR ROADWAY JAMES River Fwy (U.S. 60)
ROADWAY DIRECTION (N) B | SPEED LIMIT 40 | SPEED LIMIT 60 | GEO - CODE | GPS LONGITUDE
ROAD MAINTAINED BY ☒ 1. STATE ☐ 2. COUNTY ☐ 3. MUNICIPAL ☐ 4. PRIVATE PROPERTY ☐ 5 OTHER | LATITUDE

Figure 1.6. Hand Written Crash Report

Despite technological improvements, police officers still have significant challenges completing crash report forms. Due to these challenges, the crash databases may contain errors in orientation, time of day, location, collision diagrams, or even missing information. For instance, in Figure 1.6., in the right bottom corner of the image, the crash report form provides longitude and latitude GPS coordinates as an alternative to accurately locate crashes. As observed in the figure, this field was omitted. Some studies have been conducted regarding GPS use, and the studies found that 80% of the crashes were located within reasonable levels of accuracy (Sarasua et al., 2008). GPS crash location is being used 48% to 54% of the time. The primary method of crash location is still through traditional route and link methods (Delucia and Scopatz, 2005; Ogle, 2007).

Regarding unavailable or missing information when the police officer arrived at the crash, 27% of the time, persons were removed from the scene (Popkin et al., 1991). Some crashes are not assessed at the scene by the police, and persons involved are required to visit the police station to provide the recollection of events and file the crash report. There are six main types of information: crash, roadway, vehicle, driver, citation/adjudication, and injury control information. Figure 1.7. shows a data flow schematic for highway safety information systems dealing with the six types of information (Ogle, 2007; NHTSA, 2003).

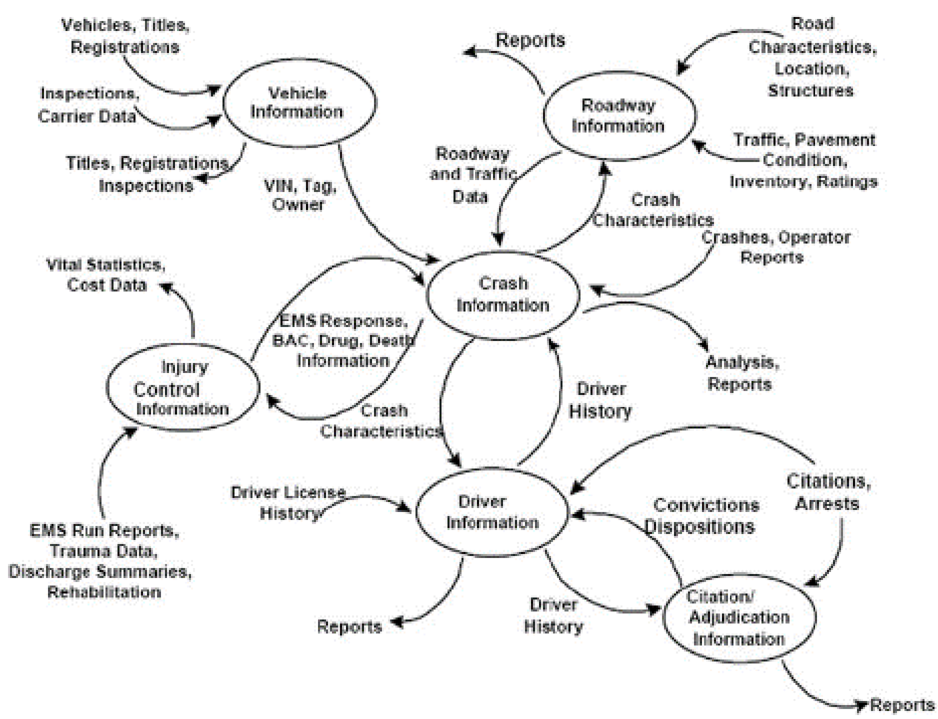
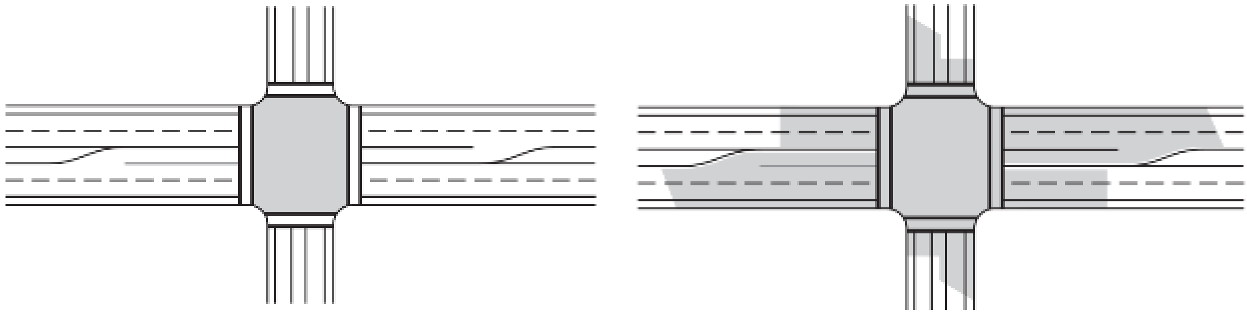


Figure 1.7. Crash Information Flow Schematic (Ogle, 2007; NHTSA, 2003)

1.1.4.2. Crash Assignment Issues Crash reports are usually reviewed and processed to correct errors and validate information. A reviewer is usually assigned the task to review all the sections of the crash reports, introduce the information in the database, and geocode the location of the crash. However, there are differences on crash reporting threshold (i.e. the influence area in the vicinity of facility) and crash assignment according to the jurisdiction.

For instance, at intersections, crashes are commonly assigned using certain spatial threshold value. In the states of Michigan and California, where there is no clear guidance to assign crashes, for research purposes, any crash occurring at an intersection or within 250 ft. of the intersection major road or 100 ft. in the minor road should be considered an intersection related crash (Vogt, 1999; Bonneson et al., 2012). There might be differences in criteria among agencies in the same state. For instance, in Missouri, two state agencies differ in the way they assign crashes. The Statewide Traffic Accident System (STARS) administers the crash report form and preparation manual, which recommends the use of physical area of an intersection (STARS, 2012, ANSI, 2007). Whereas, Missouri DOT uses a threshold of 132 ft. for intersection related crashes for major and minor roads. Figure 1.8.a shows the limits of the physical area of a conventional intersection.

Other approach focuses on the functional area of intersections. For instance, Washington State includes a field in the crash database which indicates whether a crash was related to the intersection based on the evaluation of functional area of intersection vicinity, geometry, control, or driver behavior (Ogle, 2007). Figure 1.8.b shows an example of the physical area of a conventional intersection.



a) Physical Area b) Functional Area

Figure 1.8. Crash Assignment Criteria for Intersections (AASHTO, 2010)

1.1.4.3. Crash Reporting Severity Levels In the latest edition of the Highway Safety Manual (HSM) (AASHTO, 2010); the different levels of severity are referred as “crash severity”. Crashes are classified by the level of injury or property damage. Injury is defined as “bodily harm to a person”. Property damage represents damage generated to vehicles, private or public property (e.g. light posts, sidewalk, or signal) (ANSI, 2007).

Crash severity is often divided into categories according to the KABCO scale, which provides five levels of injury severity (K, Fatal injury; A, Incapacitating injury; B, Non-incapacitating injury; C, Possible injury, and O, No injury/Property damage only). However, even if the KABCO scale is used, jurisdictions have different definitions for each category. The severity levels are very inconsistent among jurisdictions because of time allocated for investigation, medical follow up, and costs assessments (AAA, 2015). Table 1.1. provides some examples of practices of crash reporting and severities by state.

Table 1.1. Crash Reporting by Severity (AAA, 2015)

State	Crash Severity			
	Fatal	Injuries	Property Damage[1]	Issue Report Deadline
Alabama	✓[2]	✓	> $250	30 days
Alaska	✓	✓	> $2,000	10 days
Arizona	✓	✓	> $300	Damage not settled in 6 months
Colorado	✓	✓	✓	10 days
Connecticut			> $1000	5 days
DC			> $250	5 days
Idaho	✓	✓	> $1,500	Immediately
Illinois	✓	✓	> $1,500 or > $500 (uninsured)	10 days
Idaho	✓	✓	> $1,500	3 days
Pennsylvania	✓	✓	✓	5 days
Vermont			> $3,000	3 days
Virginia			> $1,500	1 day
Puerto Rico			> $100	Within 4 hours

Notes: [1] Some states may equate fatalities and injuries as costs
[2] Check mark indicates all categories of corresponding severity level

1.2. Airfield Application

1.2.1. Runway Incursions In the US the Federal Aviation Administration (FAA) Office of Runway Safety is responsible for determining whether an occurrence at an aerodrome is a runway incursion and/or surface conflict. Only surface events at airports with an operating Airport Traffic Control Tower are recorded and classified as runway incursions and surface incidents. Runway incursions are assessed by the Office of Runway Safety and classified by the severity of the event. In the 2008 fiscal year, the FAA adopted the following International Civil Aviation Organization definition for a runway incursion as well as severity categories (FAA, 2013). A runway incursion is "[a]ny occurrence at an aerodrome involving the incorrect presence of an aircraft, vehicle, or person on the protected area of a surface designated for the landing and take-off of aircraft" (FAA, 2013). The severity levels are:

Accident An incursion that results in a collision. For the purposes of tracking incursion performance, an accident and a Category A runway incursion are considered to be similar.

Category A A serious incident in which a collision was narrowly avoided.

Category B An incident in which separation decreases and there is a significant potential for collision, which may result in a time critical corrective/evasive response to avoid a collision.

Category C An incident characterized by ample time and/or distance to avoid a collision.

Category D An incident that meets the definition of a runway incursion, such as incorrect presence of a single vehicle/person/aircraft on the protected area of a surface that is designated for the landing and take-off of aircraft, but with no immediate safety consequences.

Category E An incident in which insufficient or conflicting evidence of the event precludes assigning another category.

1.2.2. Airfield Incursion Models Statistical safety models in transportation use count data to express the safety of facilities (e.g. runways, road segments, intersections, grade crossing, etc.) as a function of traits (e.g. demand, geometrics, operation). Initially, such models represented a relationship that estimated the average number of accidents for various amounts of exposure. Over the years, safety modeling has broadened. Currently, transportation safety modeling uses not only exposure but also other traits, and provides estimates not only of the average number of accidents but also of the diversity of group means on a population (Hauer, 2015). In this study, airfield incursion frequency models were developed to predict runway incursions as a function of runway exposure and hazards.

1.2.2.1. Exposure There are significant milestones regarding the definition of exposure (Hauer 1982; Hauer, 1995; Keall and Frith, 1999; Hakkert et al., 2002). The measure of exposure is generally defined as some form of the total number of operations in a system (e.g. aircraft take offs and landings). The associated exposure can be calculated using the amount of travel for certain activities or users of the system, and the number of incidents (e.g. runway incursions). Assessments can be used to improve transportation safety and determine public health priorities (i.e. risk of death or injury). There are also methods to identify potential system failures, which are defined as hazards.

1.2.2.2. Hazard According to the FAA (2007), a hazard is defined as "any existing or potential condition that may lead to injury, illness, or death of people; damage to or loss of a system, equipment, or property; or damage to the environment. A hazard is a condition that is a prerequisite to an incident." The hazard system identification considers all the possible sources of system failure. Examples of sources include equipment, weather, human error, maintenance procedures, or external services.

1.2.3. Background Airfield Safety Modeling In contrast to the important but reactive approach to aviation safety (i.e. analyzing past accidents), there has been an emerging emphasis on the proactive approach involving statistical modeling of aviation safety (Oster et al., 2013). The proactive approach complements the reactive approach since accidents are rare occurrences, and there are safety issues even in the absence of accidents (Lofquist, 2010).

Netjasov and Janic (2008) reviewed research on risk and safety modeling in civil aviation and categorized the models into the following categories: causal models for aircraft and air traffic control/management operations, collision risk, human factor error, and third-party risk. Select literature from these four categories is reviewed here; the reader is referred to Netjasov and Janic (2008) for additional references and details of other models.

Causal models consider risk assessment as the deviation from operational rules due to contributing factors or events. These factors can be evaluated to determine the effects on the occurrence of aircraft collisions that may contribute to a serious accident (Netjasov and Janic, 2008; Ale et al., 2009). One causal model was based on fault tree analysis (Kumamoto and Henley, 1996); the top of the tree is the hazard and the tree paths are represented by different independent or combined events. A variation of causal models identifies sequences of events that lead to an aircraft accident. This approach could include dividing the components of the system into zones or through event tree analyses (Bedford and Cooke, 2001). The National Aerospace Laboratory in the Netherlands developed a model using Monte Carlo simulation to assess risk and safety of aircraft operations by identifying the hazard and building scenarios. The method was applied to operations such as crossing or diverging runways, parallel runways, wake-vortex hazard, and aircraft approach (Blom, et al., 2001). Based on probability theory, the Bayesian Belief Networks method was used to evaluate causes of risk (Luxhøj and Coit, 2006). Netjasov and Janic (2008) explained that because causal models involve calculations of probabilities where causal factor dependencies are complex, it is often difficult for such models to appear transparent and comprehensible. Also, many of the models are only qualitative and meant only for identifying the sequence of events leading to an accident. In contrast, the models presented in this document are quantitative in nature, and are designed to be easily interpreted by practitioners.

Collision risk models focus on aircraft conflicts in the air or on the ground. Aircraft collisions in the air are very rare but have the most severe consequences. Aircraft spacing is evaluated with respect to the associated risk of airport capacity and standards. Existing models predict vertical, horizontal, and longitudinal collision risks with probability estimates. The models described in this dissertation address many incursion severities and not just those that result in a collision. This has a twofold benefit. One, more significant data becomes available if serious but non-collision incidents are also considered. Two, non-collision incidents have the potential for severe consequences, thus the fact that a catastrophe was averted does not diminish the underlying problem.

Human error is considered one of the most frequent contributing factors to aviation accidents (FAA, 2009) and is defined as, “an incorrect execution of a particular task, which then triggers a series of subsequent reactions in the execution of other tasks, resulting in a serious aircraft accident” (Netjasov and Janic, 2008). Human error models focus on predicting errors based on communication, environmental condition, human mental and physical state, and reaction to particular scenarios. Developing models involving human behavior is complex and relies on the modeler’s input expertise. And the application of these models is limited. The models presented in this dissertation do not model human error explicitly, but considers it via variables that represent different conditions that affect human behavior.

A study performed at Amsterdam’s Schiphol International Airport exemplifies models considering third party risk (Blom, et al., 2001). Three types of risks were defined—individual risk, societal risk, and potential of loss of life (Ale, 2002). The study developed different models. For example, a model was developed to calculate the probability of an aircraft accident near airports as a function of the probability of an accident per aircraft movement and annual airport traffic volume. Netjasov and Janic (2008) noted one shortcoming of such models as lacking generality, thus each airport needs to develop a specific model. One way of addressing this shortcoming is to use a general frequency models for any airport within the same jurisdiction and administration specified as base conditions—the models presented in this dissertation assumed base conditions of the U.S. aviation industry, and the models can be applied at any U.S. airport classified as hub.

Much of the literature specific to runway incursions focus on active prevention systems. Schönefeld and Möller (2012) classified prevention systems into the categories of surveillance, traffic information, tracking, situational assessment, and human machine interfaces and signals.

Some of these prevention systems provide some level of modeling or forecasting, but the emphasis is on technological solutions for operations. Beyond prevention system, there are only a few existing models that are oriented towards long term planning and design. Green (2014) presented preliminary results from a system-level Bayesian Belief Network model for runway incursion and excursion events. He considered multiple system-level factors such as airport issues (e.g. layout, markings and lighting, closures) and air traffic control issues (e.g. certification, training, operations). Shi et al. (2013) used log-based reasoning for predicting and detecting runway incursions, and for supplying instructions and suggestions to pilots and drivers to avoid such incursions. Xu and Huang (2011) developed a runway incursion forecasting model on an annual basis using least square support vector regression (LS-SVR), and compared it against a generalized regression neural network. Biernbaum and Hagemann (2012) produced cross tabulations that revealed interesting relationships among variables in terms of runway incident severity and incident type. They encouraged future research to involve the actual estimation of models of runway incursion frequency.

An informative discussion on modeling runway incursion severity was presented by Wilke et al. (2015). The authors found that there is a surprising lack of literature on an analytical framework for runway incursion severity assessment. They presented a structured framework for modeling incursion causal factors and included a multi-process approach composed of a description of airport surface system architecture, definition of terms, data requirements, airport characteristics, and statistical analysis. They applied the Mann-Whitney U and Kruskal-Wallis tests along with logistic regression to estimate the impact of airport characteristics on incursion severity. The number of runways and taxiway segments and the number of conflict points were found to be the most significant characteristics impacting runway incursion severity. This dissertation uses similar airfield characteristics such as the number of runways, total runway length, number of runway intersections in modeling incursions and follows the recommendation made by Wilke et al. (2015) to examine alternative model specifications in future research.

Aviation safety modeling has focused on approaches with impressive modeling results. However, most of these models are of limited application because of complexity and site-specific characteristics. In terms of runway incursion modeling, some previous models were high level (e.g. system-level) or focused on real-time operations. Practitioner-friendly models for runway incursion frequency prediction for planning and design are lacking.

2. METHODOLOGY

The methodology chapter describes the characteristics of count data and modeling techniques. In cases in which modeling is not feasible, calibration of exiting models is an alternative. Also in this chapter, different techniques to evaluate the safety effectiveness of treatment used in this dissertation are described.

2.1. Transportation Safety Modeling

Several assumptions and model considerations are made to develop safety prediction models. This section describes some of these considerations: count data, probability distribution, parameter estimation, and model diagnostics.

2.1.1. Count Data Cameron and Trivedi (2013) in their book defined count data as follows: "An event count refers to the number of times an event occurs." For instance, the number of airline accidents or earthquakes would be considered as count data. Count data is the realization of a nonnegative integer that is valued as a random discrete variable. A statistical model of event counts usually specifies a probability distribution of the number of occurrences of the event that are known up to some parameters. Estimation and inference in such models are concerned with the unknown parameters given the probability distribution and the count data. Such specification involves no other variables, and the number of events is assumed to be independently and identically distributed.

2.1.2. Probability Distribution In the early development of statistical safety modeling of road crashes, the Poisson distribution was used. After several studies found that count data could be overdispersed (Abdel-Aty and Radwan, 2000; Kononov et al., 2011), the Negative Binomial distribution replaced the Poisson distribution. Thus the Negative Binomial has been widely used to develop safety modeling on roadways (AASHTO, 2010). However, safety models can also use the Negative Multinomial distribution which is the most suitable probability distribution for modeling rare events such as accidents or incidents (Ulfarsson and Shankar, 2003; Hauer et al., 2004; Hauer, 2015).

The Negative Binomial distribution can only be used when data pertain to a single period of time (average of data from all years in the period), whereas the Multinomial variation accounts for variation in the data from year to year. For instance, in the case of the Negative Binomial, 5 years of accident data from 1994-1998 is condensed and considered as one single period of analysis. The accident counts are considered as the sum of accidents for that five-year period, and the measures of exposure of all five years are averaged. There is no variation represented in the measures of exposure from year to year when averaged, so significant information is lost. In contrast, the Negative Multinomial distribution allows the use of all available data for each year. It can describe how safety units change over time. Using the same example of 5 years of data, the Negative Multinomial captures the fluctuations within each year with a scale parameter. For certain variables, such as yearly operations and weather measurements, it is crucial to capture the effect of yearly variations on runway incursions.

2.1.3. Exploratory Data Analysis (EDA) All predictor variables were analyzed in terms of trends, distributions, and combinations. The analysis established an overview of the predictor variables for model development. Hauer (2015) describes the exploratory data analysis as helping the modeler to answer two important questions: 1) whether a variable is safety related and 2) what function form should be used.

2.1.3.1. Safety Related Variable Exploratory Data Analysis (EDA) was used to examine the characteristics of the data and to find if an orderly relationship exists, which is a promising indicator that a variable is a safety related trait (Hauer, 2015). Bins are generated for a range of values for each prediction variable. Observations in each bin are represented by an average, and the results are laid out in a chart to observe any orderly distribution between predictor and response variables.

***2.1.3.2.* Variable Functional Form** Variable functional form specification is a complex task that requires intuition which is acquired by the connection and understanding of the modeler with the data. In addition to the shape of the function, the modeler also determines the variables that should be included. Some variable selection methods include forward, backward, and stepwise selection. Such selection processes are based on confidence intervals or other test statistics. Hauer (2004, 2015) recommends a method in which the predicted values (base model) and the observed data are compared. The comparison is evaluated through a ratio which represents graphically the trend captured by the new variable for improving prediction. A predictor variable for a model should have practical or inherent information that captures its influence on runway incursion events. After analyzing several functional forms (e.g. linear, exponential, polynomial, logistic, Hoerl, or composites) the final functional form that met all goodness-of-fit measures was included in the model.

2.1.4. Parameter Estimation Model parameter estimation was performed through maximum likelihood estimation. The Negative Binomial and Negative Multinomial were used to represent the data according to the application. The likelihood functions (Hauer, 2015) based on the both distributions used in model parameter estimation are illustrated in Equation 2.1 and 2.2.

Negative Binomial Likelihood Function:

$$ln\left[\mathcal{L}_i^*\left(\beta_0,\, \beta_1,\, \ldots,\, b\right)\right] = \sum_{i=1}^{n} \left\{ \begin{array}{c} ln\Gamma(k_i + b) - ln\Gamma(b) + b\,ln(b) + k_i ln\left[y_i \hat{E}(\mu_i)\right] \\ -(b + k_i) ln\left[b + y_i \hat{E}(\mu_i)\right] \end{array} \right\} \tag{2.1}$$

Negative Multinomial Likelihood Function:

$$ln\left[\mathcal{L}_i^*(\beta_0,\ \beta_1,\ \ldots,\ b)\right] = b\,ln(b) + \left[\sum_{j=1}^{m_i} k_{ij}ln\left(\hat{E}\{u_{ij}\}\right)\right] + ln\Gamma\left(\sum_{j=1}^{m_i} k_{ij} + b\right) - ln\Gamma(b) - \left(\sum_{j=1}^{m_i} k_{ij} + b\right) ln\left[\left(\sum_{j=1}^{m_i} \hat{E}\{u_{ij}\}\right) + b\right] \quad (2.2)$$

The letter i denotes units and j denotes time periods. The mean incident count for unit i in time period j is u_{ij}. The traits of i and j define population of units that are assumed to be Gamma distributed with mean $E(u_{ij})$ and variance $E(u_{ij})^2/b$. The value $1/b$ is called the overdispersion parameter which is also commonly denoted by the letter k. The parameter estimates of the model coefficients are $\beta_0,\ \beta_1,\ \ldots,\ b$. The likelihood function that maximizes the estimates are those that maximize the sum of $ln\left[\mathcal{L}_i^*(\beta_0,\ \beta_1,\ \ldots,\ b)\right]$.

2.1.5. Model Diagnostics Regression modeling is an attempt to find a numerical expression to represent natural events—crashes. The objective is to develop a safety prediction model that fits the data well. There are several goodness of fit measures to determine how well the model fits the data. However, fit is usually judged by residuals—difference of observed crashes and model predicted crashes. Residuals are commonly judged by how small and close to zero the values are. This approach focuses on an unbiased overall fit which has no practical importance on safety modeling because it deals with count data and its relationship with safety traits at different levels. A safety prediction model that is successfully unbiased overall in a range of the most significant measure of exposure may be completely biased elsewhere (ranges where model under or overpredicts). Safety prediction models should perform well across all ranges of its traits (Hauer, 2015).

Common statistical modeling focuses on parameter estimates rather than the prediction itself and bias is overlooked. When a single number is used to measure the goodness of fit for the overall model, the measure is too general and does not provide useful information for safety prediction models (Hauer, 2015). Therefore, goodness of fit for the prediction model of this study was evaluated throughout the evolution following measures for all traits such as: variation of parameter coefficients, Log-likelihood, Cumulative Residuals (CURE) plots, and Overdispersion. These measures are considered appropriate for safety prediction modeling and current approach. (Srinivasan et al., 2013; Hauer, 2015).

2.2. Calibrating Existing Models to Local Conditions

The HSM (AASHTO, 2010) established as a national standard a prediction methodology using Safety Performance Functions (SPF), Crash Modification Factors (CMF), and Calibration factors (C) by facility and severity type. Equation 2.3 shows the general form of the HSM prediction methodology.

$$N_{pred} = N_{spf} \times C_i \times \left(CMF_1 \times CMF_2 \times \ldots \times CMF_j\right) \qquad (2.3)$$

Where,

N_{pred} = predicted crash frequency (crashes/year);

N_{spf} = predicted crash frequency for site type SPF (crashes/year)

C_i = calibration factor according to local conditions;

CMF_j = crash modification factor specific to a site type characteristic *j*.

The base model SPF and CMFs are usually developed using combined data from a couple of states. For the application to other states different than the ones of the model, the calibration factor C is used to adjust the prediction to local conditions. The HSM provides additional guidance to perform the calibration. However, the main implicit assumption of the methodology is that SPFs and CMFs are universal, transferable to all states, and not a function of specific site characteristics (Srinivasan et al., 2013). In a study involving urban intersections in Toronto, Persaud et al. (2002) compared models developed with specific data from Toronto and the calibrated model from other jurisdictions (Vancouver and California). The results of the transferability were mixed suggesting that a single calibration may be inappropriate. With a similar approach, Sacchi et al. (2012) evaluated transferability of the HSM methodology to Italian two-lane undivided rural roads by comparing the calibrated predictions with the local data estimates. The results showed that the base SPFs differ significantly with increasing exposure, and the CMFs revealed some bias to the site characteristics of the modeled data.

Additionally, common base SPF models follow a negative binomial regression with log-linear relationship (functional form) between crash frequency and site type characteristics (AASHTO, 2010; Srinivasan et al., 2013). Although it may be used for convenience and consistency purposes, it may not be the most appropriate general functional form for every site type and severity. Equation 2.4 and 2.5 show general log-linear (power family) functional forms commonly used.

$$N = f(X_1, \dots, X_n, \beta_0, \dots, \beta_n) = e^{\beta_0 + \beta_1 X_1 + \beta_2 X_2 + \dots} \qquad (2.4)$$

$$N = f(X_1, \dots, X_n, \beta_0, \dots, \beta_n) = X_1 e^{\beta_0 + \beta_1 X_2 + \beta_2 X_3 + \dots} \qquad (2.5)$$

Where *N* denotes the expected number of crashes per unit of time as a function of predictor variables $X_1, \dots, X_n$ and corresponding parameters $\beta_0, \dots, \beta_n$ estimated based on the functional form of each variable. The choice of the model regression equation is usually not explained or documented. The power family function may be commonly used because generalized linear modeling (GLM) statistical software can easily specify the power function. As a result, statistical modeling may seem disconnected from the observed data that is trying to describe. (Hauer, 2010; Kononov et al., 2011). Several publications (Hauer et al., 1997; Hauer, 2004; Hauer et al., 2004; Hauer, 2015) provide evidence of the importance of custom functional forms for each predictor variable and classified variables as multiplicative or additive terms based on what it is being represented. The choice of function should be such that it resembles the form suggested in the data including peaks, valleys, and points of inflection (Hauer, 2004). Kononov et al. (2011) compared SPFs for urban freeways developed with sigmoid and exponential functions with Colorado and California data. The study related flow, speed, and density of the urban freeways crash frequency to find the adequate functional form (sigmoid). The results showed that SPFs with sigmoid functional form had better quality fit compared with power function family SPFs developed with the GLM framework.

Developing a jurisdiction specific model instead of calibrating existing models are likely to increase the accuracy and reliability of the predictions (AASHTO, 2010; Garber et al., 2010; Srinivasan and Carter, 2011; Sacchi et al. 2012; Srinivasan et al., 2013; Lu et al., 2014). However, there are cases in which developing models is not possible and the calibration of existing models is the only alternative. The main factor that may lead a jurisdiction to choose to perform a calibration is the lack of data. The HSM recommends at least 100 to 200 sites with over 300 crashes per year for the total group during a period of at least 3 years (AASHTO, 2010). These recommendations are difficult to meet in cases in which a state does not have the minimum recommended number of facilities or crashes for a specific facility type (e.g. six lane ramp terminals). Therefore, the only alternative to predict crashes for local conditions is to calibrate existing models developed with data from other states.

The calibration methodology is simple overall comparison between the predicted crashes using the models and the observed data with local conditions. The HSM recommends between 30 and 50 sites with at least 100 crashes per year for the whole group (AASHTO, 2010). The calibration is less demanding in terms of data requirements. Equation 2.6 illustrates the equation to obtain the calibration factor.

$$C_i = \frac{\sum_{all\ sites} Observed\ Crashes}{\sum_{all\ sites} Predicted\ Crashes} \qquad (2.6)$$

2.3. Safety Effectiveness

The HSM defines safety effectiveness evaluation as the process of estimating quantitative measures as the change in number of crashes due to a treatment, project, or group of projects. The evaluation of the effectiveness of safety treatments is crucial to provide statistically supported estimates for future decision making and policy development (AASHTO, 2010). The design of before and after observational methods consists of using the before period to estimate what would have been the expected crashes in the after period had the treatment not been implemented (Hauer, 1997). In this study, three before and after observational methods were used—Naïve, Comparison Group (CG), and Empirical Bayes (EB). The HSM recommends at least 10 to 20 sites for a safety evaluation, and for the CG method a minimum of 650 aggregated crashes at the comparable sites (AASHTO, 2010). Two approaches are considered when estimating the safety effectiveness of a treatment depending on the type and extend of the project—project level and site specific.

2.3.1. Naïve Method The naïve method is a natural starting point to study the safety effect of treatments and establish the foundation of more rigorous methods. It is simple and serves as an upper bound with significant statistical precision. The naïve method does not consider the geometric and operational features of the sites under study; it focuses on observed crash data only. Therefore, it does not account for regression to the mean bias or compensate for factors that change over time (terrain, weather, traffic, driver behavior, vehicle fleet, etc.). The method reflects the effect of change all of these factors and the corresponding effect of the treatment (Hauer, 1997). The expected number of crashes for the after period (π) is calculated using Equation 2.7.

$$\pi = \sum r_d(j) \times K(j) \qquad (2.7)$$

Where,

π = expected crashes in the after period;

$K(j)$ = observed crashes in the before period at facility j;

$r_d(j)$ = ratio of duration of after period to before period for facility j.

$$r_d(j) = \frac{Duration\ of\ after\ period\ (j)}{Duration\ of\ before\ period\ (j)} \qquad (2.8)$$

The safety effectiveness is calculated using the odds ratio (θ) which is a function of the expected crashes (π), observed crashes (λ), and the variance of expected crashes [$var(\pi)$] as,

$$Safety\ Effectiveness = 100 \times (1 - \theta) \qquad (2.9)$$

Where,

$$\theta = \frac{\frac{\lambda}{\pi}}{1 + \frac{var(\pi)}{\pi^2}} \qquad (2.10)$$

2.3.2. Comparison Group The concept of the Comparison Group (CG) method is to identify a group of untreated facilities (similar to the treated facilities before any change) to estimate the measure of how safety would have changed for the treatment group. The assumptions consider that different factors influence safety in the same manner for treatment and compassion groups during before and after periods (Hauer, 1997). Each comparison site is carefully selected to resemble traffic, geometry, and crash frequency of the treatment site before the treatment implementation. Also, sites within the same district should be considered to account for local conditions. One comparison site is usually matched to a treated site. The suitability of the comparison group is verified using the sample odds ratio test (AASHTO, 2010; Hauer, 1997). The test consists of comparing crashes of the treated sites and the comparison sites over a period of time before any treatment took place. Crashes are tracked over time to observed the fluctuation and determine if the comparison group follows the distribution of the treated sites.

The CG method uses the safety prediction models to certain extent. It uses the models to predict crashes for each treated and comparison group facility with the before period site characteristics. The prediction and duration of the before and after periods of each facility is compared with the computation of an adjustment factor. Basically, adjustment factors are obtained for all possible combinations between treated facilities and comparison facilities. This adjustment factors are used in combination with the observed data to estimate the expected crashes.

The method is a significant improvement from the naïve method because it accounts for unrecognized or unmeasured causal factors. Despite the improvement, it is not superior to the Empirical Bayes method because of methodological assumptions. The weaknesses are in the accuracy of the results which are based on the similarity of comparison sites. It does not account for regression to the mean bias. The Comparison Group method cannot consider treatment sites at which the observed crash frequency in the after period is equal to zero; thus, it may lead to underestimate the effectiveness of the treatment. Locations in which the crash frequency was zero, the effectiveness of the treatment may have been the most effective (AASHTO, 2010). The best conditions for the method are significant crash data for the after period, similar after period durations among sites, and strong resemblance to treated facilities in the before period.

2.3.3. Empirical Bayes Crashes are random events that fluctuate over time at any given site. In common safety evaluation practice, crash frequency (crashes/year) over a short period of time is used to quantify the frequency of crashes at roadway facilities. Although this is a fair estimate, it is not completely accurate. As illustrated in Figure 2.1., short term average crashes may not accurately describe expected average crash frequency. Short-term crash rates may significantly be different from the long-term estimates. This difference is magnified in locations in which a small number of crashes are observed, so variations in crash frequency represent an even larger fluctuation in relation to the expected crash frequency. Therefore, it would be difficult to identify high, average, or low crash frequencies at a site using short term crash rates (AASHTO, 2010).

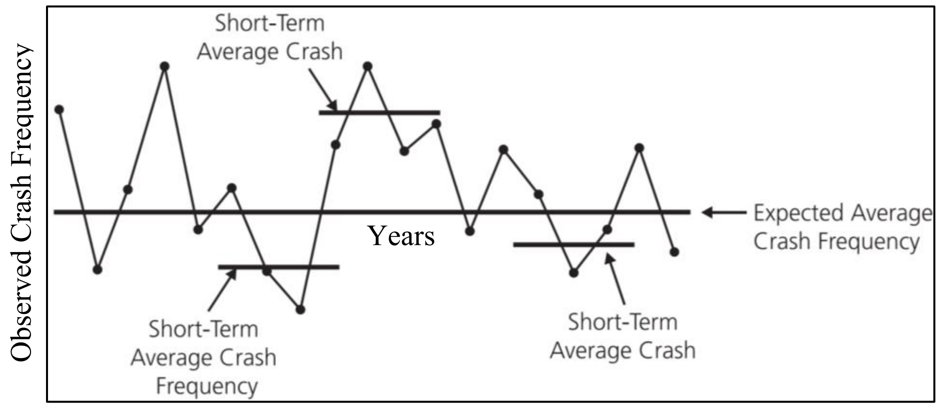


Figure 2.1. Expected and Short Term Crash Frequency (AASHTO, 2010)

In the case of countermeasure treatments, it is difficult to determine if changes in crash frequency are due to changes in site conditions or natural fluctuations. There is a tendency, called regression to the mean (RTM), which dictates that a period with comparatively high crash frequency will likely be followed by a comparatively low crash frequency or vice versa (low crash frequency followed by a high frequency period) (Hauer, 1996). Since sites are many times selected for treatments based on short term trends in the observed data, RTM bias is introduced—selection bias. The effect of this bias is significant while evaluating treatment effectiveness. When using conventional before and after studies to evaluate safety treatments, the perceived effectiveness is an overestimate of the actual treatment effectiveness. Figure 2.2. illustrates graphically the regression to the mean effect and the difference between actual and perceived effectiveness.

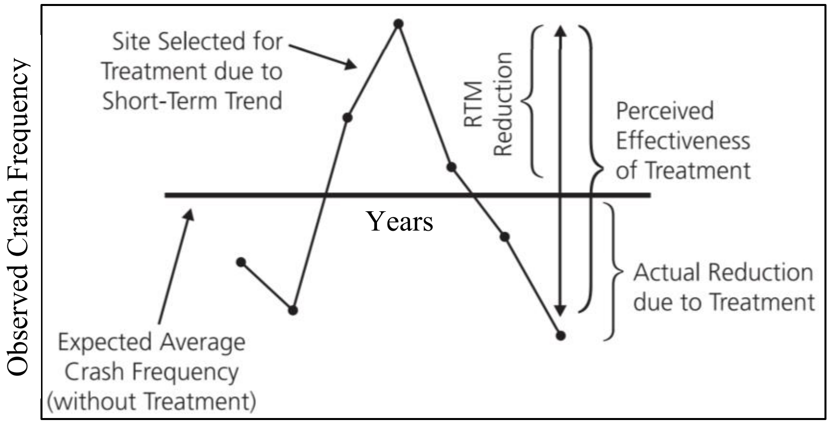


Figure 2.2. Regression to the Mean Bias (AASHTO, 2010)

Treated sites that were selected for improvement due to an unusually high number of crashes suffer from a selection bias that can result in high RTM in safety effectiveness evaluations. The Empirical Bayes method, as applied in this study, accounts for RTM and provides unbiased estimates. The main objective of the method is to determine an unbiased expected crash frequency in the after period had the treatment not been implemented (Hauer, 1997). The safety prediction models have an additional parameter called overdispersion (k) which forms the basis for the application of the Empirical Bayes method. The expected crash frequency ($N_{exp,b}$) in Equation 2.11 is then calculated as the weighted average (w) of the observed crashes ($N_{obs,b}$) and the model predicted crash frequency in the before period ($N_{pred,b}$). The weight (w) is determined using the overdispersion parameter (k) of the base SPF model.

$$N_{exp,b} = w \times N_{pred,b} + (1 - w) \times N_{obs,b} \qquad (2.11)$$

Where,

$$w = \frac{1}{1 + k \times N_{pred,b}} \qquad (2.12)$$

The expected crashes in the after period ($N_{exp,a}$) are then calculated by multiplying an adjustment factor (r) to the expected crashes in the before period ($N_{exp,b}$).

$$N_{exp,a} = r \times N_{exp,b} \qquad (2.13)$$

The adjustment factor (r) is introduced to account for variations between before and after periods. These variations include the durations of periods and traffic volume. Therefore, the factor is the ratio of the predicted crashes in the after period ($N_{pred,a}$) over predicted crashes in the before period ($N_{pred,b}$).

$$r = \frac{N_{pred,a}}{N_{pred,b}} \qquad (2.14)$$

The expected crashes in the after period ($N_{exp,a}$) is then compared with the actual observed crash frequency in the after period ($N_{obs,a}$). Equation 2.15 shows the comparison designated as *OR'*.

$$OR' = \frac{N_{obs,a}}{N_{exp,a}} \qquad (2.15)$$

Since OR' is potentially biased, it is adjusted using Equation 2.16 to remove bias and account for regression to the mean using the variance of the expected crashes in the after period.

$$OR = \frac{OR'}{1 + \frac{Var\left[N_{exp,a}\right]}{\left[N_{exp,a}\right]^2}} \qquad (2.16)$$

Where,

$$Var\left[N_{exp,a}\right] = \left[(r)^2 \times N_{exp,b} \times (1 - w)\right] \qquad (2.17)$$

The comparison (unbiased *OR*) of expected and observed crash frequency for the after period forms the basis for deriving the safety effectiveness, as shown in Equation 2.18. The safety effectiveness is the measure of the treatment effectiveness at a site or group of sites after implementation. When crash frequency decreases after a treatment, the safety effectiveness is positive. When crash frequency increases, the safety effectiveness is negative.

$$Safety\ Effectiveness(\%) = 100 \times (1 - OR) \qquad (2.18)$$

2.3.3.1. Empirical Bayes for Crash Cost Benefit The change in crash costs over all treated facilities in a jurisdiction for specific crash types was estimated. Based on the method used for the safety effectiveness described previously, the Empirical Bayes method measures the difference between net crash costs expected without treatment and observed with treatment in the after period. The cost modification factor (θ_{Cost}) is a measure quantifying the change in crash cost with the treatment (Council et al., 2005, 2005a):

$$\theta_{cost} = \frac{\frac{\Lambda_{cost,a}}{\Pi_{cost,a}}}{\left\{1 + \left[\frac{Var(\Pi_{cost,a})}{{\Pi_{cost,a}}^2}\right]\right\}} \qquad (2.19)$$

$$Var(\theta_{cost}) = \frac{{\theta_{cost}}^2 \left\{\left[\frac{Var(\Lambda_{cost,a})}{{\Lambda_{cost,a}}^2}\right] + \left[\frac{Var(\Pi_{cost,a})}{{\Pi_{cost,a}}^2}\right]\right\}}{\left\{1 + \left[\frac{Var(\Pi_{cost,a})}{{\Pi_{cost,a}}^2}\right]\right\}^2} \qquad (2.20)$$

Where,

θ_{cost} = cost modification factor;

$Var(\theta_{cost})$ = variance of crash modification factor;

$\Lambda_{cost,a}$ = cost of crashes at treated sites in the after period;

$\Pi_{cost,a}$ = expected cost of crashes in the after period over all treated sites had there been no RLC (after correcting for regression to the mean).

Additionally, the change in crash cost (Φ_{cost}) and variance can be estimated in dollar costs:

$$\Phi_{cost} = \Pi_{cost,a} - \Lambda_{cost,a} \qquad (2.21)$$

$$Var(\Phi_{cost}) = Var(\Pi_{cost,a}) + Var(\Lambda_{cost,a}) \qquad (2.22)$$

2.3.4. Collision Diagram Analysis Collision diagrams describe the location, circumstances, and contributing factors of a crashes. For instance, a comprehensive collision diagram analysis was performed for roundabouts by Rodegerdts et al. (2010). The diagram provided significant information regarding the effect of the roundabout geometry and operations in the type of crashes observed. Although collision diagrams provide significant information, they are not commonly used because of data limitations and time-consuming process needed for their developement. Figure 2.3. illustrates the collision diagram analyses performed by Rodegerdts et al. (2010).

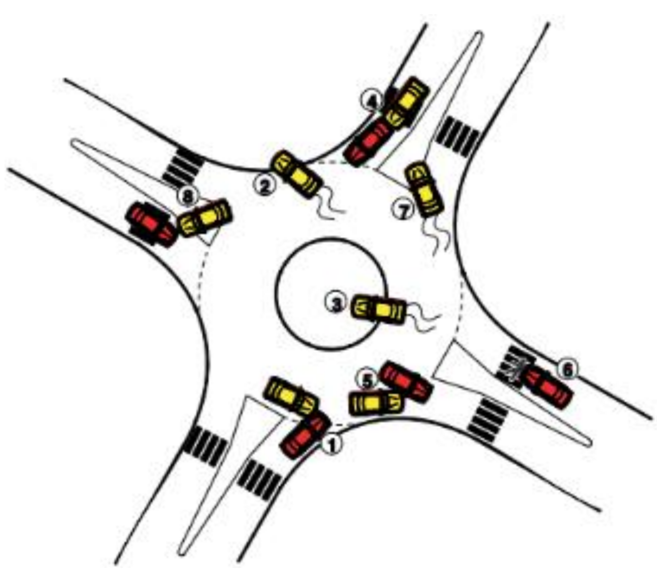


Figure 2.3. Roundabout Collision Diagram Analysis (Rodegerdts et al., 2010)

For the research performed in his dissertation, a methodology was developed to generate collision diagrams based on crash type, location, and distribution. First, crash reports were reviewed on a case by case basis and the information of the crash was summarized in a diagram. Figure 2.4. shows an example of crash landing notation used in creating the collision diagrams.

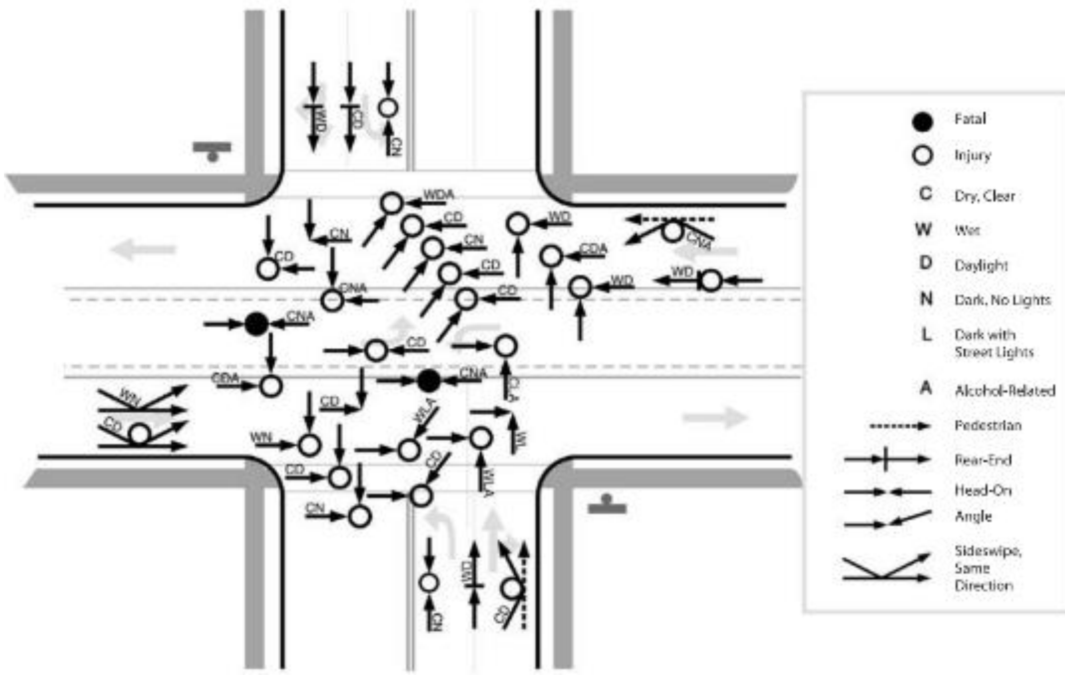


Figure 2.4. Collision Diagram (AASHTO, 2010)

Second, after all crashes are coded and landed in the collision diagram, site related crashes are filtered. In other words, crashes which occur within the boundaries of the functional area of the facility but were not influenced by any geometric or operation condition at the sites were removed. For instance, a rural roadway segment in which most crashes were loss of control and under severe weather conditions—there is not direct influence of roadway geometry over a straight segment under severe conditions.

Third, the filtered crashes were analyzed and classified according to type and location. Also, the frequency of each category is determined to provide priority indicators. The final analysis is collision diagram with illustrative figures over the geometry of the facility similar to Figure 2.3.

2.3.5. Project Level and Site Specific Safety Evaluation Safety evaluations are performed according to the extent of the projects and scope of analysis. The project level approach consists of evaluation all the facilities that are within the footprint or are part of a coordinated system. On the other hand, the site specific approach focuses on an individual facility component. Therefore, the project level analysis of interchange aggregates different facilities that are within the footprint and operations—ramp terminals, freeway segment, ramps, and speed change lanes. For the site specific approach, if a treatment was implemented at one of the ramp terminals, the safety evaluation focuses on the ramp terminal facility only and does not considered the rest of the facilities of the interchange. Thus, project level focuses on a set of facilities, and site specific focuses on one facility type.

3. CRASH REPORT REVIEW AT DIAMOND INTERCHANGES: A TUTORIAL

The tutorial is divided in two phases. The first phase covers the assignment of crashes to ramp terminals, and the second phase covers the assignment of crashes to the freeway segment, speed change lanes, and ramps.

3.1. Phase 1: Ramp Terminal

3.1.1. Introduction Crash reporting is a process of compiling information regarding the circumstances of a roadway accident and its participants. A police officer is in charge of documenting all relevant information on a crash report form. This officer is typically from a local police jurisdiction such as a city or county but can also be from the Missouri State Highway Patrol (MSHP). In the state of Missouri, the format of the crash report is the Missouri Uniform Accident Report MUAR (2002-2011) (STARS, 2002) and Missouri Uniform Crash Report MUCR (2012-present) (STARS, 2012). Both formats provide detailed instructions on how to complete the form. MSHP is the state depository for traffic crash reports with the responsibility of training officers to complete the reports following the Statewide Traffic Accident Records System (STARS) standards. Unfortunately, it is hard to establish consistent crash reporting because of different factors. These factors could include the experience of the police officers, supervisor, resources, necessary training, and crash report processing. While working on different research projects, some reporting inconsistencies were found in the data. One type of inconsistency is the inaccurate reporting of crash locations on freeway interchanges, the so-called crash landing problem. This is a significant problem for safety analysis of freeway interchanges, because it is important to locate crashes on the appropriate facility of a freeway such as the mainline, ramps, or terminals. This tutorial presents a methodology to review crash reports at interchanges and assign these crashes to the correct facility within the interchange.

3.1.2. Methodology The methodology developed in this tutorial starts with a description of a conventional diamond interchange and its facility types. A description of the crash report formats is covered along with the fields of the reports that are used to facilitate the identification of the location and the circumstances of the crashes. The criteria for assigning crashes to the ramp terminals are described in detail. The consistent application of the correction procedure is important since a reviewer of a crash report has discretion over how to interpret a crash report. Therefore, the most common scenarios in crash reports are described and explained to establish a standard. Lastly, a test involving a set of different crash reports is provided to evaluate a reviewer's familiarity with the established standards. This test provides valuable feedback to a reviewer in order to bring about greater consistency among separate reviewers of crash reports.

3.1.3. Description of Conventional Diamond Interchange A conventional diamond interchange is a grade separated intersection of a freeway and a crossroad. In order to link both roads, the design contains ramp terminals on each side of the freeway to distribute traffic with exit and entrance ramps from and to the freeway. Figure 3.1. shows in detail the components of the interchange. Figure 3.1. shows speed-change (S-C) lanes in magenta, ramps in yellow, and terminals in blue. S-C lanes encompass the area between the ramp and the mainline from the gore point to the taper. Mainline freeway lanes adjacent to the S-C lanes are considered part of the interchange area since crashes could be caused by movements to or from the ramps to the mainline. Ramp terminals are intersections involving the crossroad and ramps and could be signalized or stop-controlled.

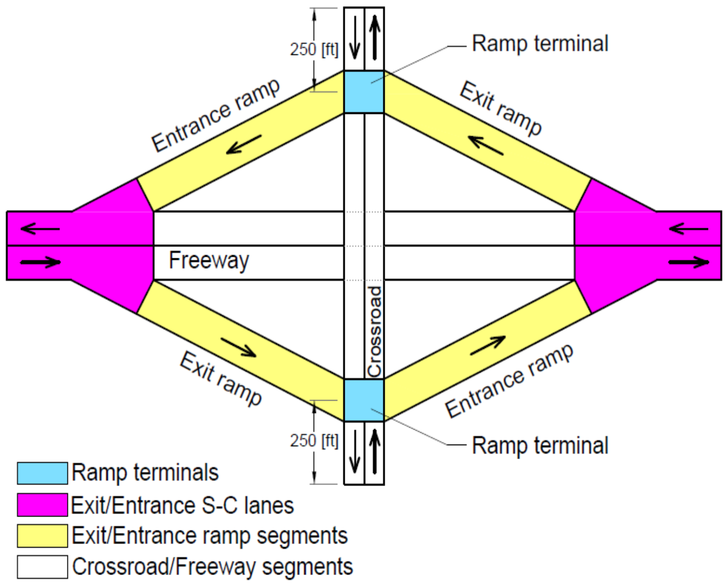
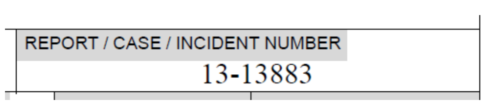


Figure 3.1. Facilities at a Conventional Diamond Interchange

3.1.4. Description of Crash Reports The crash report sections used for location correction consist of the image number, collision diagram, and narrative/statements of the crash.

3.1.4.1. Image Number The crash report has an identification number, but it is not used here because the electronic crash data available is not linked to that identification number. An example of an identification number is shown in Figure 3.2. The image number is a unique number assigned by the Missouri DOT to identify a crash report. The ***image number*** is compatible with the electronic crash report. The crash data format along with highlighted examples of image numbers are shown in Table 3.1. The file containing a crash report is in a .pdf or a .tif format. Each crash report filename includes the image number identification (e.g. 40073302.pdf).


REPORT / CASE / INCIDENT NUMBER

13-13883

Figure 3.2. Crash Identification Number (not used)

Table 3.1. Crash Data Format

County	Desg	Travelway	Dir	Cont Log	Accident Class	Accident Date	Severity Rating	Image #	Intersection #	Log Unit	Intrsc	Intrchg	Grpd	Light Cond	Road Surf Cond	Weather Cond	Tway Id	Property Damage	Day of Week	Time
GREENE	US	60	W	260.879	REAR END	8/23/2004	PROPERTY DAMAGE ONLY	1040034802	0	17.526		Y		DAYLIGHT	DRY	CLEAR	7783	NONE	MON	1630
GREENE	US	60	W	260.879	REAR END	8/24/2004	PROPERTY DAMAGE ONLY	1040034914	0	17.526		Y		DAYLIGHT	DRY	CLEAR	7783	NONE	TUE	1815
GREENE	US	60	E	79.732	LEFT TURN	8/27/2004	PROPERTY DAMAGE ONLY	1040035269	0	9.874		Y		DARK	DRY	CLEAR	7782	NONE	FRI	555
GREENE	US	60	W	260.879	REAR END	9/1/2004	MINOR INJURY	1040036166	0	17.526		Y		DAYLIGHT	DRY	CLOUDY	7783	NONE	WED	1655
GREENE	US	160	E	95.619	PASSING	9/3/2004	PROPERTY DAMAGE ONLY	1040036368	0	25.1		Y		DAYLIGHT	DRY	CLEAR	7806	NONE	FRI	1540

3.1.4.2. Location The second section of the crash report presents a specific description of the location of the crash. An example of this section is shown in Figure 3.3. In this section, the focus is on the road in which the crash was assigned (ON), the roadway direction section (RDWY. DIR.), distance from (N/A, ft., Miles), location (N/A, before, after, at), intersecting road (INTERSECTING), and intersecting road direction (INT. DIR.). These fields identify the road on which the crash occurred and the distance from the intersecting road. For example, Figure 3.3. shows that a crash occurred on Eastbound Interstate 44 at the Kansas Expressway. Note that the accuracy of the distances and the reference point varies according to the person who filled out the form. The location information should be used in conjunction with the collision diagram and statement/narrative.

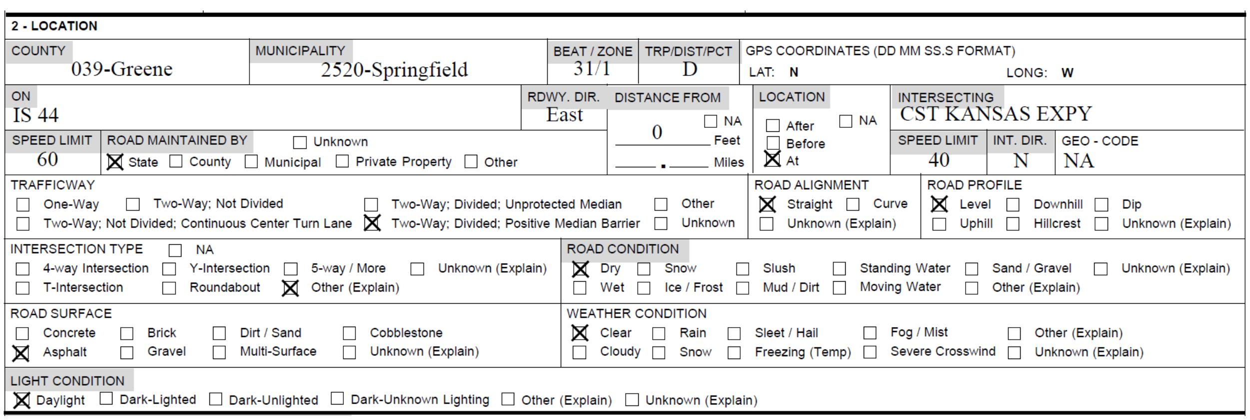

2 - LOCATION
COUNTY 039-Greene | MUNICIPALITY 2520-Springfield | BEAT / ZONE 31/1 | TRP/DIST/PCT D | GPS COORDINATES (DD MM SS.S FORMAT) LAT: N LONG: W
ON IS 44 | RDWY. DIR. East | DISTANCE FROM 0 ☐ NA Feet . Miles | LOCATION ☐ After ☐ Before ☒ At ☐ NA | INTERSECTING CST KANSAS EXPY
SPEED LIMIT 60 | ROAD MAINTAINED BY ☐ Unknown ☒ State ☐ County ☐ Municipal ☐ Private Property ☐ Other | SPEED LIMIT 40 | INT. DIR. N | GEO - CODE NA
TRAFFICWAY ☐ One-Way ☐ Two-Way; Not Divided ☐ Two-Way; Divided; Unprotected Median ☐ Other ☐ Two-Way; Not Divided; Continuous Center Turn Lane ☒ Two-Way; Divided; Positive Median Barrier ☐ Unknown
ROAD ALIGNMENT ☒ Straight ☐ Curve ☐ Unknown (Explain)
ROAD PROFILE ☒ Level ☐ Downhill ☐ Dip ☐ Uphill ☐ Hillcrest ☐ Unknown (Explain)
INTERSECTION TYPE ☐ NA ☐ 4-way Intersection ☐ Y-Intersection ☐ 5-way / More ☐ Unknown (Explain) ☐ T-Intersection ☐ Roundabout ☒ Other (Explain)
ROAD CONDITION ☒ Dry ☐ Snow ☐ Slush ☐ Standing Water ☐ Sand / Gravel ☐ Unknown (Explain) ☐ Wet ☐ Ice / Frost ☐ Mud / Dirt ☐ Moving Water ☐ Other (Explain)
ROAD SURFACE ☐ Concrete ☐ Brick ☐ Dirt / Sand ☐ Cobblestone ☒ Asphalt ☐ Gravel ☐ Multi-Surface ☐ Unknown (Explain)
WEATHER CONDITION ☒ Clear ☐ Rain ☐ Sleet / Hail ☐ Fog / Mist ☐ Other (Explain) ☐ Cloudy ☐ Snow ☐ Freezing (Temp) ☐ Severe Crosswind ☐ Unknown (Explain)
LIGHT CONDITION ☒ Daylight ☐ Dark-Lighted ☐ Dark-Unlighted ☐ Dark-Unknown Lighting ☐ Other (Explain) ☐ Unknown (Explain)

Figure 3.3. Location of the Crash

3.1.4.3. Collision Diagram A collision diagram shows the circumstances and location of the crash. Figure 3.4. shows an example of a collision diagram involving a multi-vehicle collision. The legend of the collision diagram is located on the header of the page. The legend provides crucial information for interpreting the direction of travel of each vehicle involved in the crash. As seen in Figure 3.4., the north arrow is clearly marked for orientating the diagram.

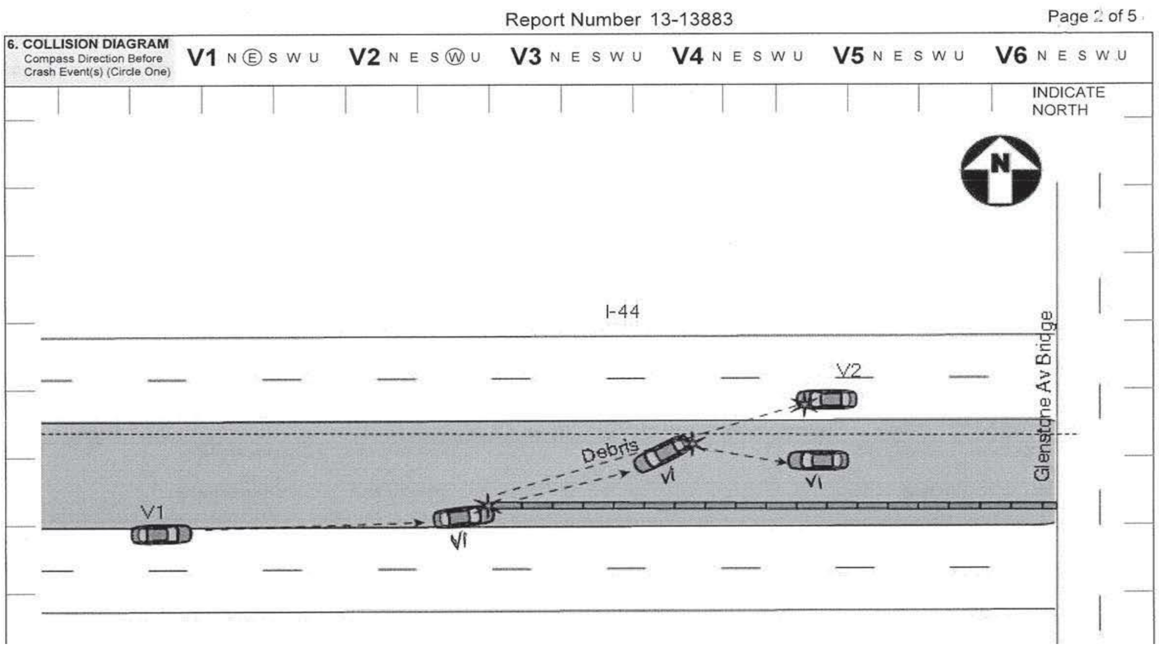


Figure 3.4. Collision Diagram

The amount of detail contained in the collision diagram is dependent upon the reporting agency and personnel. If the crash was reported at the police station after the incident, then the crash report might not have a collision diagram. Therefore, the collision diagram might have limited or no information. If that is the case, then other resources, such as the narrative and statements of the crash, need to be used to locate the crash.

3.1.4.4. Narrative/Statements The narrative contains a written description of the crash and statements collected from witnesses and/or people involved in a crash. The details in this section are also subject to the experience and expertise of the reporting personnel. Figure 3.5. shows an example of a narrative for the same crash shown in Figure 3.4. This example contains statements by both drivers (e.g. D1) and witnesses (e.g. W1).

9. NARRATIVE / STATEMENTS (If additional room is necessary, use Section 11 - Narrative / Statements Continuation)

V1 WAS E/B ON I-44 IN THE #1 LANE IN THE AREA OF MO 13 (KANSAS EX.). V2 WAS W/B ON I-44 IN THE #1 LANE IN THE AREA OF MO 13. V1 SWERVED OFF THE ROADWAY LEFT COLLIDING WITH A METAL GUARDRAIL. V1 SPUN CLOCK-WISE AND WENT PARTIALLY AIRBORNE AS IT SPUN. IT CAME TO REST AND COLLIDED WITH THE MEDIAN CABLE BARRIER. DURING V1'S COLLISION WITH THE GUARDRAIL DEBRIS WAS THROWN AT THE WINDSHIELD OF V2 AS SHE WAS PASSING THE AREA OF IMPACT. THE DEBRIS CRACKED HER WINDSHIELD.

D1 SAID HE WAS E/ ON I-44 WHEN ANOTHER VEHICLE SWERVED TOWARDS HIS LANE. HE SAID HE SWERVED LEFT AND STRUCK THE GUARDRAIL. HE COULD NOT DESCRIBE ANYTHING ABOUT THE OTHER VEHICLE TO INCLUDE MAKE, MODEL OR COLOR. D1 SAID HE WAS NOT INJURED BUT DID APPEAR UNCOMFORTABLE AND LIMPED. HE DECLINED EMS.

W1 LEFT HIS INFORMATION WITH ANOTHER ON SCENE OFFICER THEN LEFT BEFORE I COULD INTERVIEW HIM. I WAS NOT ABLE TO CONTACT HIM LATER VIA THE PHONE NUMBER HE LEFT. HE DID TELL THE OTHER ON SCENE OFFICER THAT HE WAS BEHIND V1 AND SAW IT SWERVE LEFT FROM THE #1 LANE AND COLLIDE WITH THE GUARD RAIL.

Figure 3.5. Narrative and Statements Section

3.1.5. Review and Assignment Procedure Each crash report should be reviewed by following the standards developed in this tutorial. The goal is to locate the crash on the appropriate freeway interchange facility.

3.1.5.1. STEP 1: Crash Location Review The first step in reviewing a crash report is to determine the specific location of the crash. Initially, the travelway name, orientation, and direction of travel of the vehicle or vehicles involved need to be determined. The different fields of the crash report described in the previous section should be used to find the specific location of the crash with respect to the interchange orientation. Additionally, Google Earth or Google Maps may be used to locate and visualize the facilities of the interchange. It is strongly recommended that the location be found on a map before making any decisions to assign the crash. The information provided in the location, collision diagram, and statement/narrative sections could be inconsistent within the same report. Therefore, as a general rule, at least ***2 out of the 3*** sections should be in agreement.

3.1.5.2. STEP 2: Crash Circumstances Review The second step of the review consists of understanding the scenario of the crash events with respect to the location. The statements provided by the witnesses and peopled involved in the crash should be carefully interpreted because they are personal opinions, interpretations, and claims. Such statements might have been made to protect their own interests and to prevent negative consequences of their actions. A driver made claim should be confirmed by the officer narrative. The narrative of the officer is not only intended to describe the crash events but to state the results of the investigation. Understanding the different factors in the scenario of the crash helps the reviewer to correctly assign the crash to the corresponding facility (ramp terminal) or discard the crash if it is not ramp terminal related.

3.1.5.3. STEP 3: Assignment of Crashes to Ramp Terminals This is a crucial step of the entire review process, and the reviewer should be careful in understanding the concepts in this section to avoid locating or classifying crashes to the wrong ramp terminal facility. Crashes that occurred in the crossroad approaches, exit ramp, and are ***ramp terminal related,*** should be assigned to one of the two ramp terminals of the interchange. In the vicinity of a ramp terminal, crashes in the crossroad exiting direction and entrance ramp should also be assigned to the ramp terminal that contributed to the crash.

The ramp terminal for the crash location should be designated based on compass direction relative to the freeway direction: North (N), South (S), East (E), or West (W). If the freeway runs in the north-south direction, the crash location should be coded as (E) if the crash is being assigned to the ramp terminal located on the east side of the freeway and as (W) if the crash is being assigned to the ramp terminal located on the west side of the freeway. If the freeway runs in the east-west direction, the crash location should be coded as (N) if the crash is being assigned to the ramp terminal located on the north side of the freeway and as (S) if the crash is being assigned to the ramp terminal located on the south side of the freeway. If the freeway runs in a diagonal direction, the reviewer should estimate visually if the freeway runs closer to the north-south direction or east-west direction to make the crash location assignment. The use of Google Earth or Google Maps is recommended to determine the location and orientation of the ramp terminals with the freeway. ***This is critical*** because an incorrect assignment could alter the safety analysis of an interchange significantly. Some examples are provided in Figure 3.6.

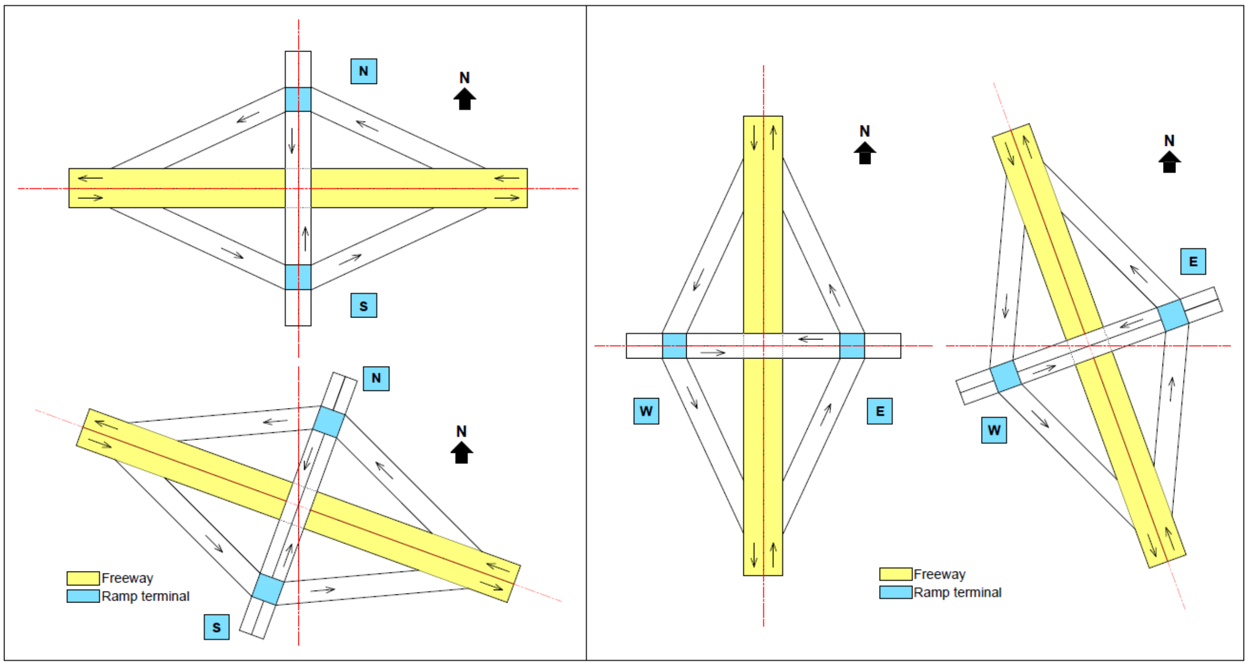


Figure 3.6. Examples Ramp Terminal Assignment

3.1.6. Ramp Terminal Related Crashes Throughout the tutorial it has been mentioned that the objective in the first phase of review of the crash reports is to determine and/or verify if the crashes actually occurred at one of the ramp terminals of an interchange. Therefore, all crashes that are ***"ramp terminal related"*** are of interest. Ramp terminal related means that a crash occurred due to the ramp terminal geometric design, operational performance, and the influence of these factors in driver behavior. According to common crash reporting practices, crashes that are within 250 ft. on the roadways away from the center of the intersection (in the approaching direction of the crossroad legs and exit ramp segment) are considered intersection-related crashes (Vogt, 1999; Bonneson et al., 2012). However, there are some specific exceptions to this threshold. For instance, a crash that occurs beyond 250 ft. in the exit ramp segment or crossroad legs that was caused by queuing at the ramp terminal is still ramp terminal related. Rear end and sideswipe crashes due to the accumulation of traffic from the ramp terminal are considered ramp terminal related crashes, because the crash circumstances were generated by the ramp terminal congestion (Bauer and Hardwood, 1998). The assignment is conducted based on the location, circumstance of the crash, and ramp terminal related crash criteria.

Figure 3.7. describes the possible locations of crashes that are of interest. These crashes involve the ramp terminal itself, the crossroad approach legs, the exit ramps, part of the entrance ramps, and a small section of the freeway adjacent to the exit ramps. *Crashes that are reported at the ramp terminal, approach crossroad legs, exit ramp, and are within 250 ft. and ramp terminal related, should be assigned to one of the two ramp terminals. Also, crashes in the crossroad exiting direction and entrance ramp, in the vicinity of a ramp terminal, should be assigned to the ramp terminal that contributed to the crash. The assignment should be made according to the location of the ramp terminal with respect to the freeway [North (N), South (S), East (E), and West (W)], as described in the previous section.*

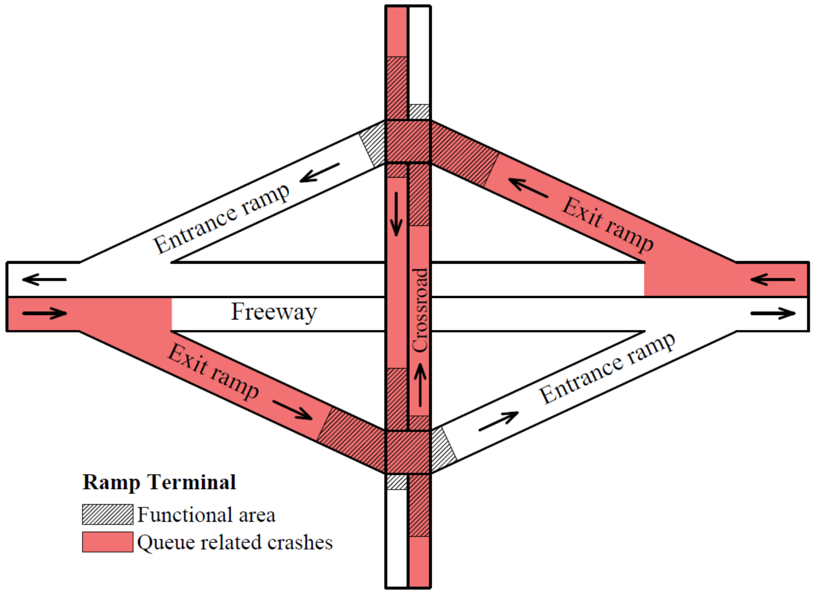


Figure 3.7. Area of Interest for Ramp Terminal Related Crashes

The exit ramp and some parts of the freeway are highlighted. Recalling the ***ramp terminal related criteria***, some crashes that occur on the exit ramp or part of the freeway due to queuing generated from the ramp terminal should be assigned to the corresponding ramp terminal (N, S, E, or W).

There are cases in which a crash occurred between two ramp terminals, and it might be difficult to determine which of the ramp terminals the crash belongs. Figure 3.8. shows an example where one of the ramp terminals was so congested that a queue reached the other ramp terminal. This crash should be assigned to the ramp terminal which generated the queue instead of the ramp terminal that the queue reached and generated the crash.

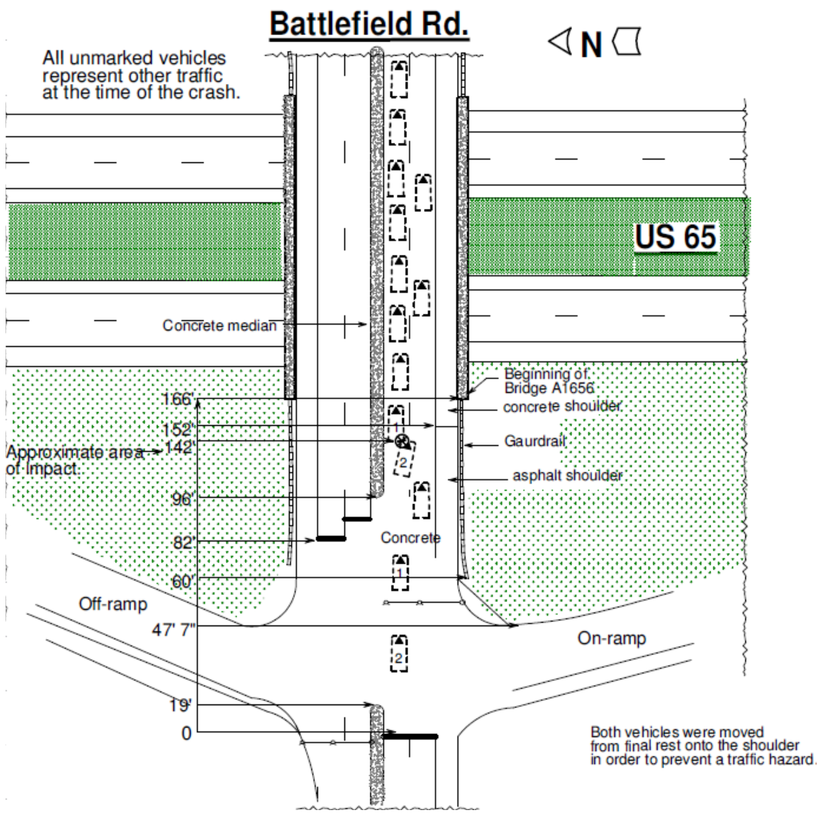


Figure 3.8. Illustration of Queue between Ramp Terminals

In Figure 3.9., the highlighted areas involve exit ramps, entrance ramps, and freeway segments. Crashes that occurred on these facilities are not relevant in this stage of the review. These types of crashes should be coded with the letter X.

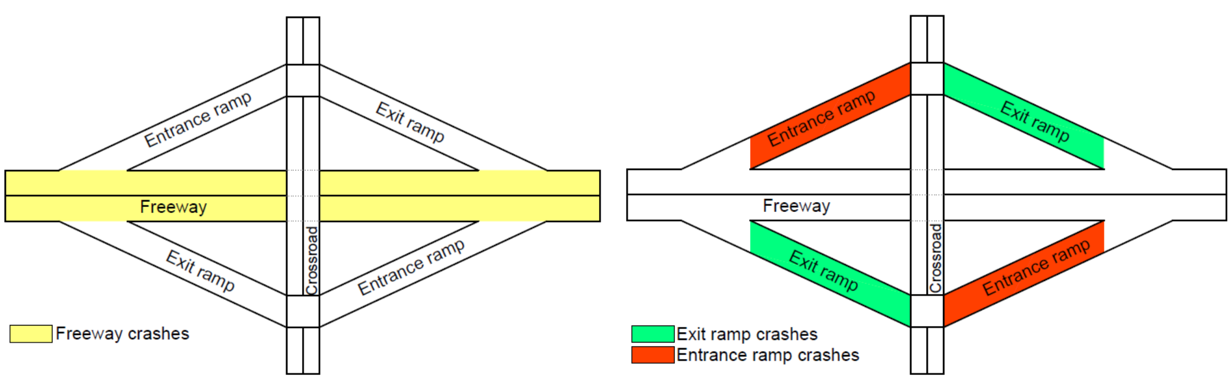


Figure 3.9. Distribution of Crashes According to the Facility

If the crash report describes an event that meets all the criteria mentioned before, but it is a circumstance that is a rare event in which the ramp terminal design might not have been a contributing factor, it should also be assigned as none (X). For instance, a crash due to extreme congestion in which queuing vehicles were invading the opposing lane traffic and caused a crash with another vehicle following the same behavior or the opposing traffic. This situation is generated because drivers might decide to quit attempting to enter the ramp terminal because of the congestion, and they decide to turn around invading the median or opposing lane traffic to make the maneuver. This example can be observed in Figure 3.10. This crash should be coded as X.

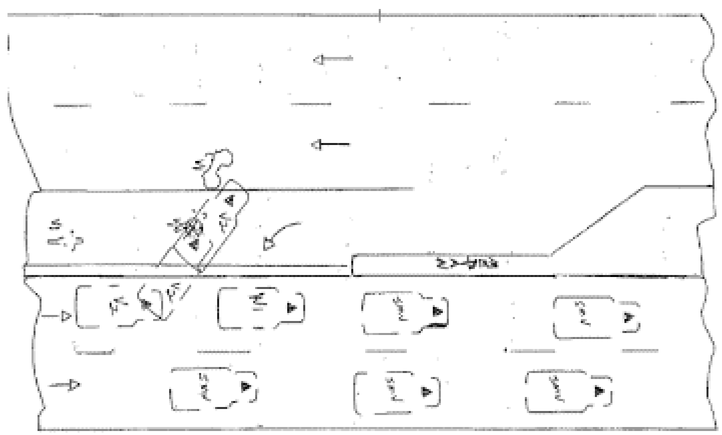


Figure 3.10. Rare Event Crash

There are several cases with particular circumstances or rare events that generated a crash. Again, this type of crashes should be assigned as X because they are not of interest. Therefore, the following list provides examples of rare events to be considered while conducting the review of the crash reports:

- A crash generated by a vehicle avoiding or hitting an object near the ramp terminal
- A crash generated by a vehicle avoiding or hitting wild animals near the ramp terminal

- A crash generated by vehicles pulling over because of an emergency vehicle
- A crash generated due to police pursuit
- A run off the road crash due to a driver falling asleep
- A crash generated by a vehicle malfunctioning or a tire exploding
- Property damage by object coming or blowing out from one vehicle damaging other vehicles on the road (ex. windshield brakeage)
- Fatal or injured driver due to shooting
- Crashes due to a work zone and not the operation of the interchange
- A crash generated by congested traffic due to another crash, i.e. a secondary crash

Cases in which a driver was distracted by a secondary task should not be considered a rare event, for instance, drivers lighting up a cigarette, drinking water, putting sunglasses on, or picking up something from the passenger seat. Any type of distraction while driving is considered part of driving behavior. And drivers could try to cover up the fact that they were recklessly using their cellphones while driving.

Table 3.2. is an example of the result of the review and assignment of crashes. In the crash data output, a column was added to include the coding of the assignment according to the ramp terminal or none (N, S, E, W, or X).

Table 3.2. Coding of Reviewed Crashes

County	Desg	Travelway	Dir	Cont Log	Accident Class	Accident Date	Severity Rating	Image #	Intersection #	Log Unit	Intrsc	Intrchg	Grpd	Light Cond	Road Surf Cond	Weather Cond	Tway Id	Property Damage	Day of Week	Time	Interchange	Ramp terminal
GREENE	US	60	W	260.879	REAR END	8/23/2004	PROPERTY DAMAGE ONLY	1040034802	0	17.526		Y		DAYLIGHT	DRY	CLEAR	7783	NONE	MON	1630	1	N
GREENE	US	60	W	260.879	REAR END	8/24/2004	PROPERTY DAMAGE ONLY	1040034914	0	17.526		Y		DAYLIGHT	DRY	CLEAR	7783	NONE	TUE	1815	1	N
GREENE	US	60	E	79.732	LEFT TURN	8/27/2004	PROPERTY DAMAGE ONLY	1040035269	0	9.874		Y		DARK	DRY	CLEAR	7782	NONE	FRI	555	1	S
GREENE	US	60	W	260.879	REAR END	9/1/2004	MINOR INJURY	1040036166	0	17.526		Y		DAYLIGHT	DRY	CLOUDY	7783	NONE	WED	1655	1	N
GREENE	US	160	E	95.619	PASSING	9/3/2004	PROPERTY DAMAGE ONLY	1040036368	0	25.1		Y		DAYLIGHT	DRY	CLEAR	7806	NONE	FRI	1540	1	N

3.1.7. Examples A set of examples was developed to describe the procedure and methodology of this tutorial. The purpose of the examples is to illustrate in detail the most common scenarios, interpretation of the crash reports, and the different tools used in the procedure.

3.1.7.1. Example 1 Table 3.3. contains the crash data necessary to start the review of the crash.

Table 3.3. Crash Data Example 1

County	Desg	Travelway	Dir	Cont Log	Accident Class	Accident Date	Severity Rating	Image #	Intersection #	Log Unit	Intrsc	Intrchg	Grpd	Light Cond	Road Surf Cond	Weather Cond	Tway Id	Property Damage	Day of Week	Time	Interchange	Ramp terminal
GREENE	US	160	E	95.675	LEFT TURN RIGHT ANGLE COLLISION	7/4/2011	MINOR INJURY	3110016727	652131	25.156	Y	Y		DAYLIGHT	DRY	CLEAR	7806	NONE	MON	1605	1	S

3.1.7.1.1. Step 1 Locate the interchange in Google Earth file. It can be observed in Table 3.3., in column "Interchange" that is coded as 1. Figure 3.11. shows the location of the interchange in Google Earth.

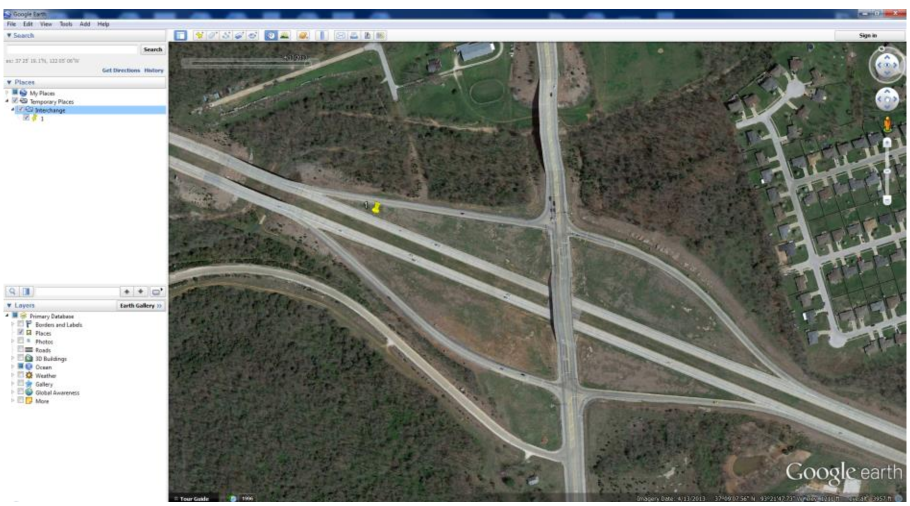


Figure 3.11. Location Interchange 1
(Image Lansat/Copernicus, Google 2016)

With the image number (column: Image #) 3110016727, search and open the crash report from the folder Crash reports as can be observed in Figure 3.12.

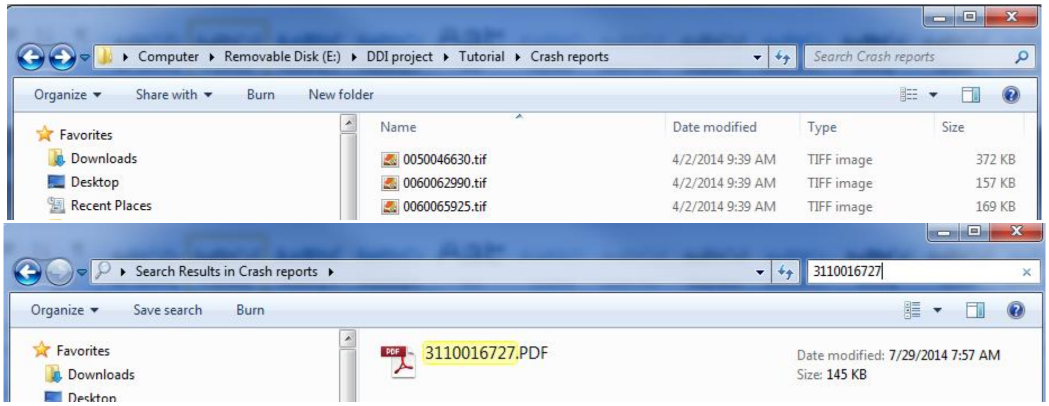
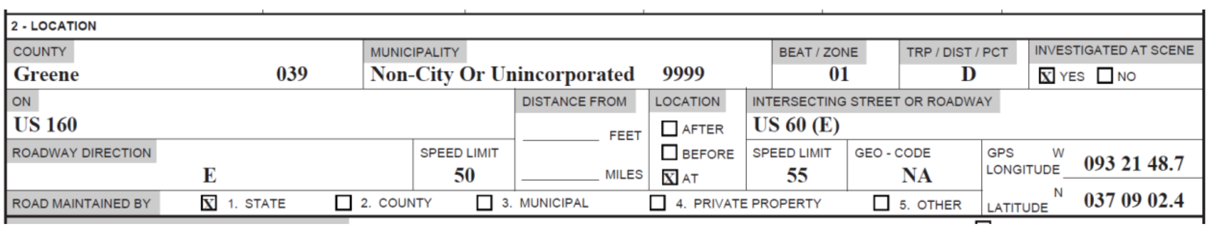


Figure 3.12. Search of Crash Report

After finding the crash report, the reviewer should start looking at section 2 of the report where the location of the crash is described with the different fields covered previously. Figure 3.13. shows the corresponding location section of the crash report of Example 1.


2 - LOCATION

COUNTY		MUNICIPALITY		BEAT / ZONE	TRP / DIST / PCT	INVESTIGATED AT SCENE
Greene	039	Non-City Or Unincorporated	9999	01	D	☒ YES ☐ NO

ON	DISTANCE FROM	LOCATION	INTERSECTING STREET OR ROADWAY		GPS
US 160	____ FEET	☐ AFTER	US 60 (E)		
ROADWAY DIRECTION: E; SPEED LIMIT: 50	____ MILES	☐ BEFORE ☒ AT	SPEED LIMIT: 55	GEO - CODE: NA	W LONGITUDE 093 21 48.7
ROAD MAINTAINED BY: ☒ 1. STATE ☐ 2. COUNTY ☐ 3. MUNICIPAL ☐ 4. PRIVATE PROPERTY ☐ 5. OTHER					N LATITUDE 037 09 02.4

Figure 3.13. Section 2 of Crash Report: Example 1

3.1.7.1.2. Step 2 The rest of the crash report information should be reviewed to verify the location and circumstances of the crash. Section 7 of the crash report contains the collision diagram. It can be observed in the diagram that a right angle collision occurred in the ramp terminal when vehicle 1 (V1) travelling southbound U.S. 160 in the through lane collided in a right angle with vehicle 2 (V2). V2 was traveling eastbound on the exit ramp from U.S. 60 taking a left turn to continue northbound on U.S. 160. Also, a witness (W) was included in the diagram. The collision diagram can be observed in Figure 3.14.

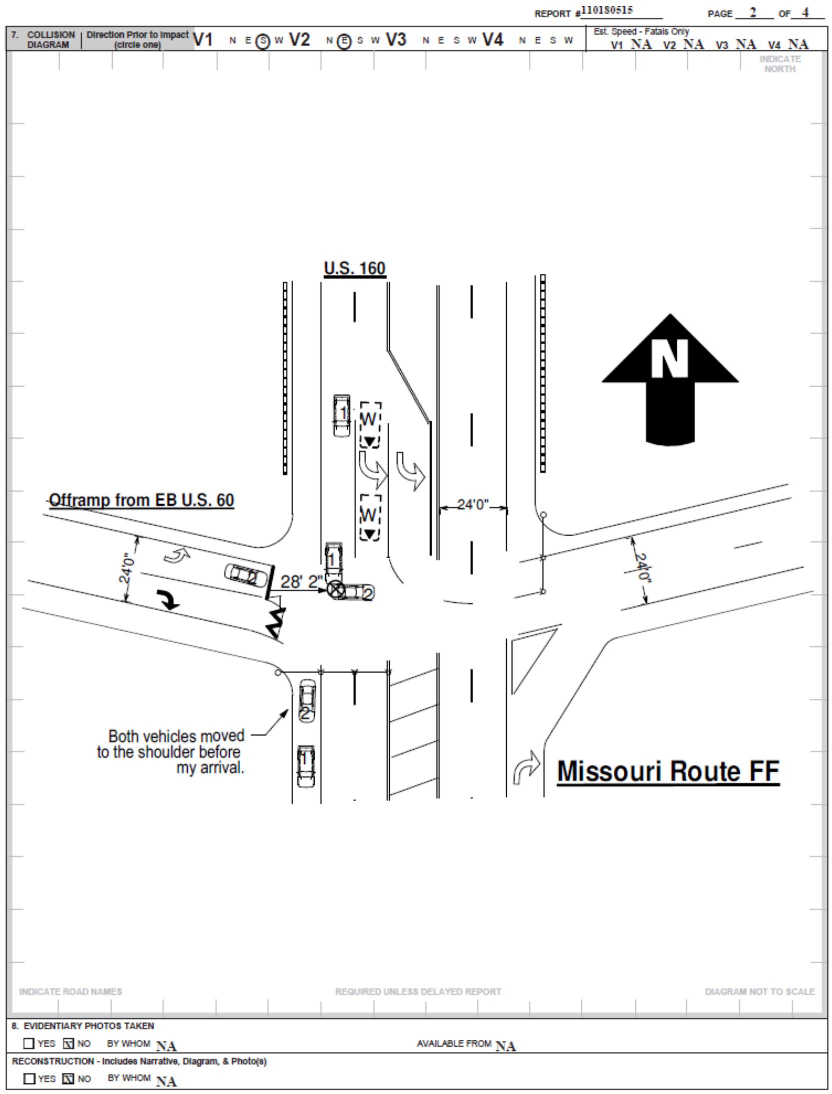


Figure 3.14. Collision Diagram Example 1

3.1.7.1.3. Step 3 It can be observed in Figure 3.15., section 28 of the crash report that the description and the narrative of the crash specifically describe the investigation conducted by the officer, the statements of the drivers and witness. Usually, this section alone could help determine the assignment of the crash. In conclusion, the crash occurred in the south (S) ramp terminal of interchange 1 due to a driver running the red signal on the crossroad and hitting a vehicle coming from the exit ramp. The assignment can be observed again in Table 3.3., in column "ramp terminal" where the assignment S was included.

28 - NARRATIVE / STATEMENTS (If additional room is necessary, attach a separate sheet.)

1. This accident apparently occurred as vehicle 1 was southbound on U.S. 160 at U.S. 60 and struck vehicle 2. Vehicle 2 was eastbound on U.S. 60, attempted a left turn to go north on U.S. 160 and was hit by vehicle 1 in the intersection.

2. Driver 1 said, "I ran the red light. I was in a hurry, late for an appointment. I saw her in the intersection. I looked up and my light was red. I slammed on my brakes." Driver 1 was inattentive by not seeing the red light until after he saw vehicle 2.

3. Driver 2 said, "I was stopped. My light turned green. I started through. I saw him coming and floored it to get out of his way. He hit me and spun me around. I got out and looked at the light over his lane. It was red."

4. Baker said, "I was stopped at a red light. The guy beside me went on through the red light and hit the car and spun it around." She clarified the light was red as they approached it. She stopped. He did not.

Figure 3.15. Narrative/Statements Example 1

3.1.7.2. Example 2 The procedures in Step 1 of Example 1 are the same for all the examples. Table 3.4. contains the information of the crash. Also, Figure 3.16. shows interchange 2.

Table 3.4. Crash Data Example 2

County	Desg	Travelway	Dir	Cont Log	Accident Class	Accident Date	Severity Rating	Image #	Intersection #	Log Unit	Intrsc	Intrchg	Grpd	Light Cond	Road Surf Cond	Weather Cond	Tway Id	Property Damage	Day of Week	Time	Interchange	Ramp terminal
GREENE	CST	NATIONAL AVE	S	2.268	REAR END	4/11/2008	MINOR INJURY	80044140	0	2.268		Y		DAYLIGHT	DRY	CLEAR	92536	NONE	FRI	1725	2	N

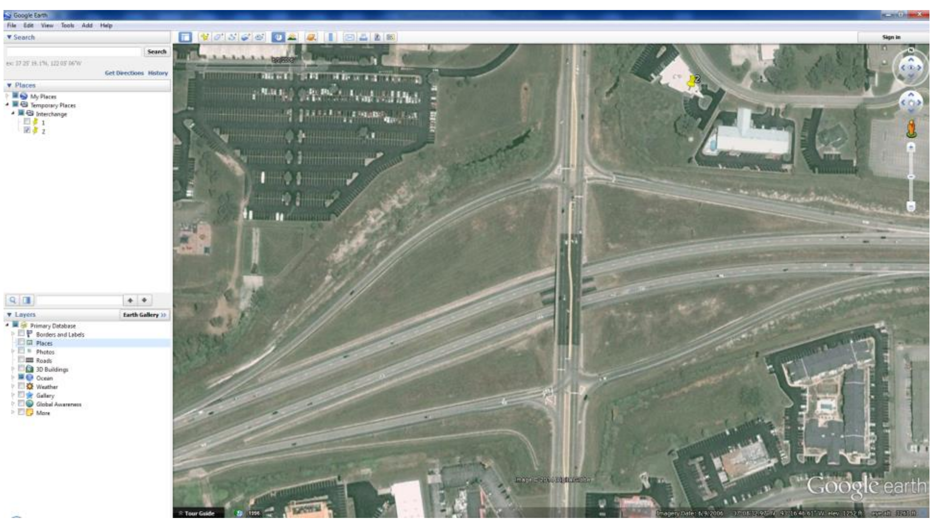


Figure 3.16. Location of Interchange 2
(Image Lansat/Copernicus, Google 2016)

3.1.7.2.1. Step 2 The crash report information should be reviewed to determine the location and circumstance of the crash. From the collision diagram in Figure 3.17., it can be observed that the crash occurred in the inside leg of the north ramp terminal of the interchange. The crash was a rear end with a stopped vehicle. According to the narrative/statements, the cause of the crash was that the driver from vehicle 1 (V1) was unable to stop on time because his foot slid off the brake pedal. The crash is not only within the 250 ft. threshold, but it is also ramp terminal related.

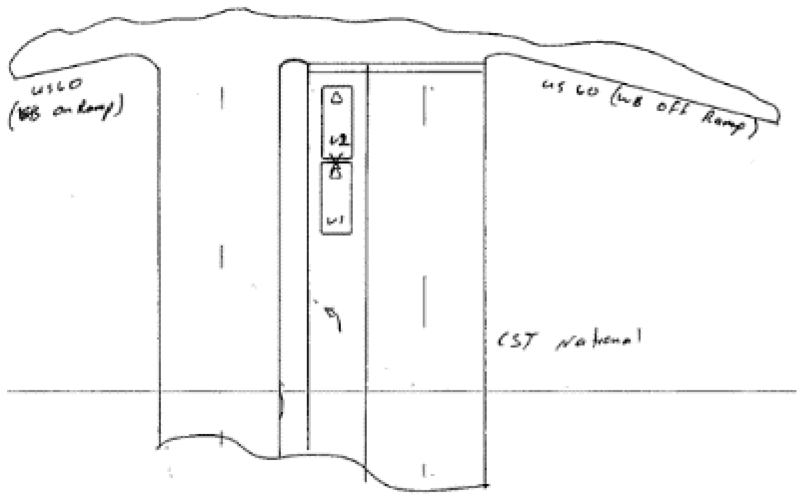


Figure 3.17. Collision Diagram Example 2

3.1.7.2.2. Step 3 Therefore, the assignment of the crash is to the north ramp terminal (N). There might be some cases in which the crash might have occurred in the middle of the segment (and beyond 250 ft.) in between the ramp terminals, and it might be difficult to decide to which ramp terminal should be assigned. However, the intersection related criteria should be used or considering the entering traffic to the ramp terminal only.

3.1.7.3. Example 3 The procedures in Steps 1 in example 1 are the same for all the examples. Table 3.5. contains the information of the crash. Also, Figure 3.18. shows interchange 2 in a different orientation than before.

Table 3.5. Crash Data Example 3

County	Desg	Travelway	Dir	Cont Log	Accident Class	Accident Date	Severity Rating	Image #	Intersection #	Log Unit	Intrsc	Intrchg	Grpd	Light Cond	Road Surf Cond	Weather Cond	Tway Id	Property Damage	Day of Week	Time	Interchange	Ramp terminal
GREENE	US	60	W	255.625	REAR END	7/10/2007	MINOR INJURY	70079057	0	12.272		Y		DAYLIGHT	DRY	CLOUDY	7783	NONE	TUE	843	2	N

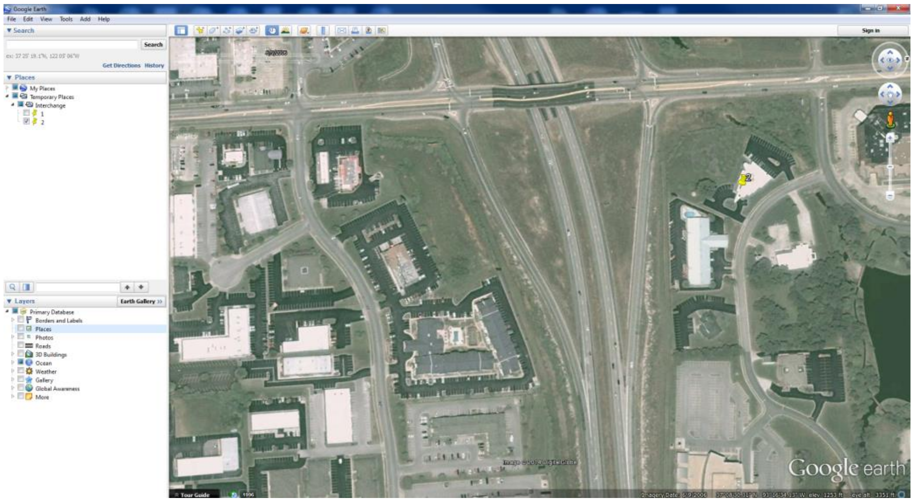


Figure 3.18. Location of Interchange 2 (other view)
(Image Lansat/Copernicus, Google 2016)

3.1.7.3.1. Step 2 The crash report information should be reviewed to determine the location and circumstance of the crash. From the collision diagram in Figure 3.19., the collision occurred on the exit lane of the freeway. There was queuing coming from the ramp terminal trough the ramp reaching the freeway. Three vehicles were involved in the crash. Vehicle A (VA) was able to avoid collision and went off the roadway towards the shoulder. The second vehicle (V2) was unable to stop in time, and hit the stopped vehicle (V1).

3.1.7.3.2. Step 3 Since the crash was caused by the queue originated at the ramp terminal, it should be assigned to the ramp terminal north (N).

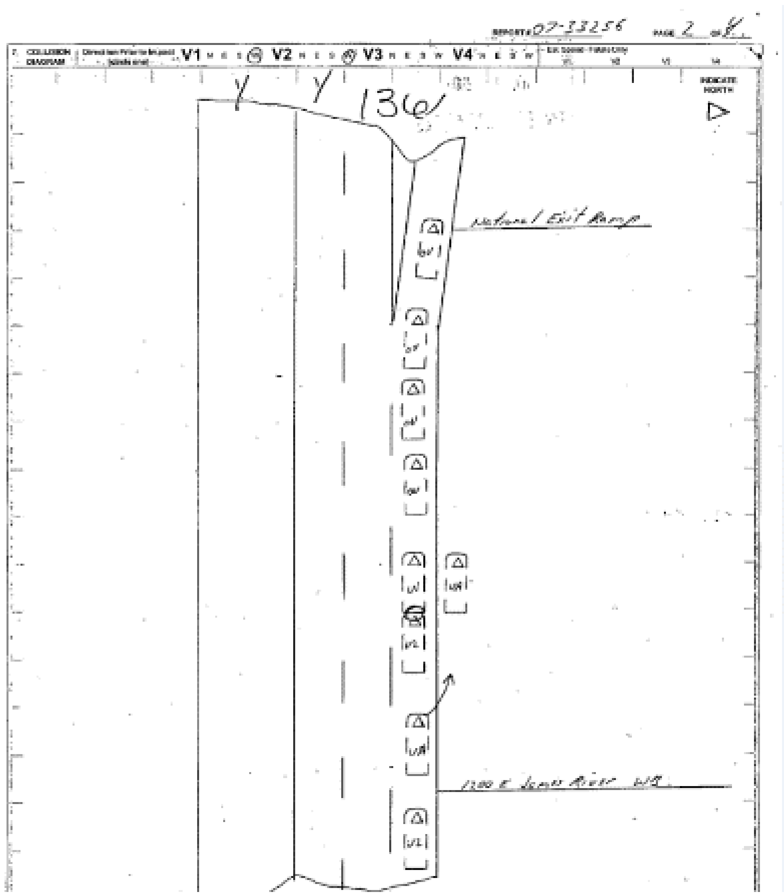


Figure 3.19. Collision Diagram Example 3

3.1.8. Training Test for the Reviewer Seventy five crash reports were carefully selected to test the reviewer learning process with respect to the assignment and coding of the crashes. The crash reports include different scenarios to observe the response and the comprehension of the material explained in this tutorial. Additionally, the crash data is also provided, so the reviewer can include the coding according to the review. Finally, the results of the test will be evaluated by a designated specialist to provide feedback to the reviewer and correct undesired inconsistences in the review of the crash reports before the reviewer starts reviewing the real data.

3.1.9. Supplemental Files The files for the test are:

a) Crash report images from the highway patrol

b) Worksheet with the summary crash data from all crashes

c) Google Earth file containing the location of interchanges

3.2. Phase 2: Speed Change Lanes, Freeway, and Ramp Segments

3.2.1. Introduction Phase 1 of the crash review tutorial focused on ramp terminal related crashes. Those crashes were assigned with the notation: N, S, E, and W. On the other hand, crashes that were not ramp terminal related were assigned the letter X. Phase 2 of the tutorial focuses on assigning the non-terminal related crashes (X) to the corresponding facility of the interchange. These facilities are the freeway segment, speed change lanes, and ramp segments. Each facility is described in detail in terms of operations, geometry, influence over drivers, and type of crashes. Followed by the description of each facility, the Phase 2 report contains a methodology to assign crashes based on a notation for each facility and the criteria to determine the assignment. The criteria developed in this phase maintains consistency in the crash review by setting uniform standards, using available information, and illustrating with an example that provides the reviewer the capability to assign crashes to the correct facility. This tutorial only establishes the guidelines for uniform crash assignment; it also important that the crash reviewer applies the guidelines consistently and accurately. Therefore, a self-diagnostic test was developed so that a reviewer can test his/her understanding before the actual crash review is performed.

3.2.2. Description of Conventional Diamond Interchange Facilities A conventional diamond interchange is a grade separated intersection of a freeway with a crossroad that is composed of a set for facilities: freeway segments, speed change lanes, ramp segments, and ramp terminals. Figure 3.20. shows the facilities of a conventional interchange. The speed change lanes are shown in magenta and the ramps in yellow. The ramp terminals will not be covered since they were previously discussed earlier in this report. There are different types of speed change lanes and ramp segments, and they will be explained in detail in the next sections.

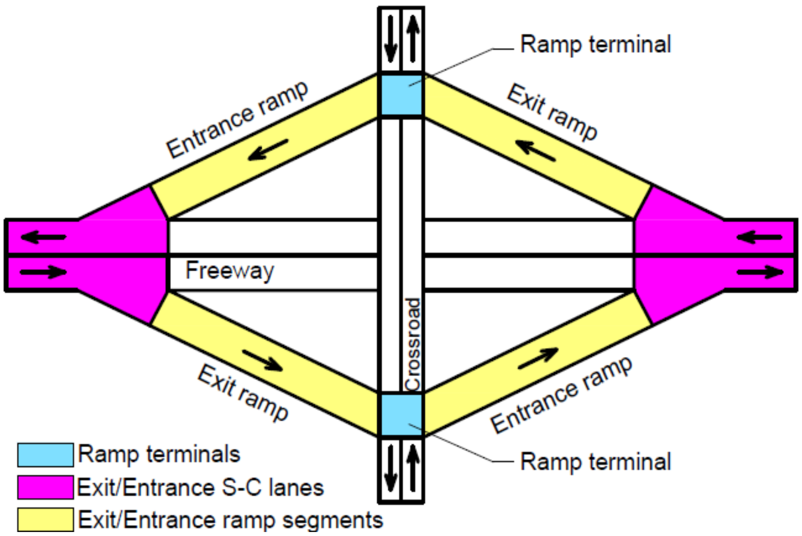


Figure 3.20. Facilities at a Conventional Diamond Interchange

3.2.2.1. Freeway Segment The freeway segment of an interchange is the section of a freeway (both directions) that it is bounded by the speed change lanes as shown in Figure 3.20. The gore point is a reference location to determine where the freeway segment begins and ends. The gore point is the location at which a ramp segment diverges or merges with a freeway segment, and it should be 2 ft. wide.

At interchanges there could be barriers on both sides of the road, bridge infrastructure, and grade differentials which increase the risk of crashes. Therefore, the crashes occurring in the freeway segment within the interchange area might be different than non-interchange continuous segments that are not influenced by speed change lane interactions, interchange geometry, and structural features.

3.2.2.2. Ramp Segments Ramp segments are unidirectional auxiliary roadways located between speed change lanes and ramp terminals (AASHTO, 2010). There are two types of ramp segments: 1) exit ramp segments and 2) entrance ramp segments. An exit ramp segment allows through traffic to leave the freeway and connect with the crossroad using the ramp terminal. An entrance ramp provides the crossroad traffic access to the freeway through the ramp terminal. The length of a segment is measured from the gore point. For the exit ramp segment, the length is from the gore point to the stop line at the ramp terminal. For the entrance ramp segment, the length is from the edge of the crossroad to the gore point with the freeway. Figure 3.21. shows the locations and lengths of ramp segments at a conventional diamond interchange.

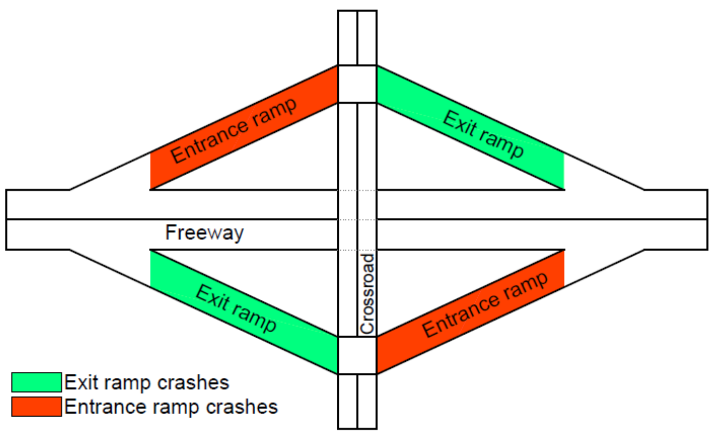
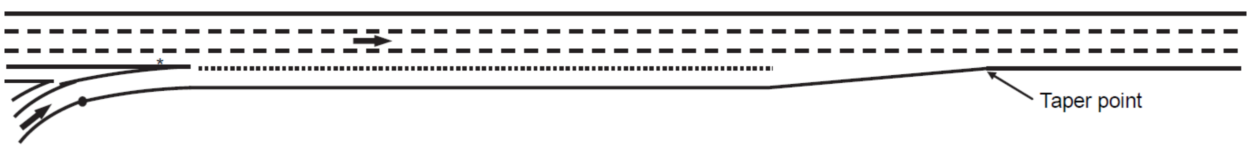


Figure 3.21. Ramp Segments Locations

3.2.2.3. Speed Change Lanes A speed change lane is a unidirectional uncontrolled terminal between the freeway and ramp segments. There are also two types of speed change lanes: 1) exit speed change lanes and 2) entrance speed change lanes. Typically, an interchange has 4 speed change lanes. An exit speed change lane gradually adds additional lane(s) to separate exiting traffic from the through freeway traffic and connects with the exit ramp segment. This gradual transition area in the speed change lane is called the taper. An entrance speed change lane merges the lanes from the ramp to the freeway by gradually dropping the ramp lane(s), allowing vehicles to merge safely with the freeway through traffic. The length of speed change lanes are measured from the gore point to the beginning or end of the taper. Figure 3.22. shows a common entrance ramp and an exit ramp with the associated speed change lanes, including the gore point and the taper.

Entrance Ramp w/ Parallel Design



Exit Ramp w/ Taper Design

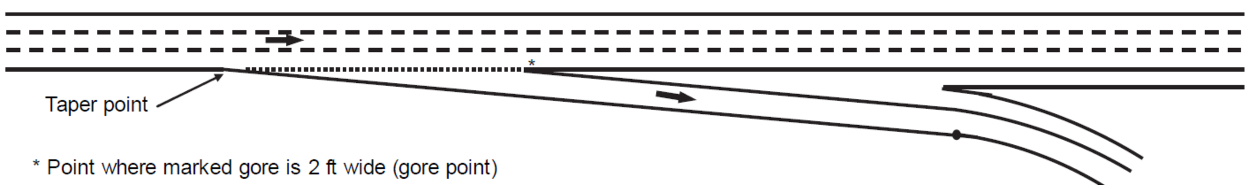


Figure 3.22. Speed Change Lanes (AASHTO, 2010)

Important note: Speed change lanes are different from add or drop lanes. An add lane is a lane that is added to the mainline and does not end with a taper. Figure 3.23. shows an example of a westbound add lane where the additional lane continues without terminating at a taper. A drop lane is a mainline lane that is terminated via an off ramp. Figure 3.23. also shows an example of an eastbound drop lane where the drop lane did not begin with a taper. Speed change lanes associated with a ramp that merges with (or diverges from) the freeway should not exceed a length of 0.30 mi (1,600 ft). If in the process of the crash review add or dropped lanes are identified, and the crash occurred in the limits of that facility, they are considered freeway segments related crashes (AASHTO, 2010). These segments are different than the freeway segment of the interchange between ramps. Therefore, if there are crashes on add or drop lanes, they should be assigned with the notation X.

Figure 3.23. Example of Add and Droop Lanes
(Image Lansat/Copernicus, Google 2016)

3.2.3. Description of Crash Reports This section briefly describes the contents of the crash reports that are used in crash review and assignment. The crash report sections used for assignment consist of the image number, collision diagram, and narrative/statements of the crash.

The ***image number*** is a unique number assigned by the Missouri DOT to identify a crash report, and it is compatible with the electronic crash report. Each crash report filename includes the image number identification (e.g. 40073302.PDF).

The second section of the crash report presents a specific description of the ***location*** of the crash. The fields in this sections help identify the road on which the crash occurred and the distance from the intersecting road. Note that the accuracy of the distances and the reference point varies according to the person who filled out the form. The location information should be used in conjunction with the collision diagram and statement/narrative.

A ***collision diagram*** shows the circumstances and the location of the crash. The legend of the collision diagram is located on the header of the page. The collision diagram might have limited or no information. If that is the case, then other resources, such as the narrative and statements of the crash, need to be used to locate the crash.

The ***narrative*** contains a written description of the crash and statements collected from witnesses and/or people involved in a crash. The details in this section are also subject to the experience and expertise of the reporting personnel.

3.2.4. Review and Assignment Procedure Each crash report should be reviewed by following the criteria and methodology developed in this tutorial. The goal is to locate the crash on the appropriate freeway interchange facility. Similarly to Phase 1, there are three important steps:

STEP 1: Crash Location Review

STEP 2: Crash Circumstances Review

STEP 3: Assignment of Crashes

The ***first step*** in reviewing a crash report is to determine the specific location of the crash using the information provided in the location field, collision diagram, and statement/narrative. If the sections are inconsistent with each other, then, as a general rule, select the location in which ***2 out of the 3*** sections are in agreement.

The ***second step*** of the review consists of the analysis of the crash circumstances. The statements provided by witnesses and parties involved in the crash should be carefully interpreted, because they are personal opinions, perceptions, and claims. Understanding the different factors involved in a particular crash scenario helps the reviewer to correctly assign the crash to the corresponding facility.

The ***third step*** is the assignment of the crash to the appropriate interchange facility. The reviewer should be careful to avoid misplacing or misclassifying crashes to the wrong interchange facility. As mentioned previously, Phase 2 of this tutorial focuses on assigning the non-terminal crashes from Phase 1. The following section describes in detail the new Phase 2 notation for crash facility assignment.

3.2.5. Crash Assignment Notation Several characters are used in the crash assignment notation to locate crashes on ramps, speed change lanes, or freeway segments. First, identify crashes as entry or exit related. Next, locate crashes in the north, south, east, or west side of the interchange. The notation for crash assignment has three components: **1) facility type**, **2) exit or entry**, and **3) location** with respect to the orientation of the interchange.

3.2.5.1. Interchange Facility Type Characters The notation for the three facilities that are considered for assignment in this phase of the tutorial are: F = freeway segment, S = speed change lanes, and R = ramp segments.

3.2.5.2. Exit or Entry Designation Characters There are three designations: D = diverging or exiting from the freeway, M = merging, entrance, or entry to the freeway, and F = freeway mainline. Since a freeway segment does not have entry or exit, D or M, characters should not be assigned, and it should be skipped to proceed with the next character describing the location.

3.2.5.3. Location Character For consistency purposes, the notation used for Phase 1 is also used in Phase 2. Therefore, the crash location will be designated based on the compass direction relative to the freeway centerline. The characters are: N = North, S = South, E = East, and W = West.

If the freeway runs in the north-south direction, the crash location should be coded as E if the crash is being assigned to the facility located on the east side of the freeway and as W if the crash is being assigned to the facility located on the west side of the freeway. If the freeway runs in the east-west direction, the crash location should be coded as N if the crash is being assigned to the facility located on the north side of the freeway and as S if the crash is being assigned to the facility located on the south side of the freeway. If the freeway runs in a diagonal direction, the reviewer should estimate visually if the freeway runs closer to the north-south direction or east-west direction to make the crash location assignment. The use of an aerial photograph from sources such as Google Earth or Google Maps is recommended to determine the location and orientation of the ramp terminals with the freeway. ***The use of an aerial photograph is critical*** to accurately performing the crash assignment.

As an example of the aforementioned notation, assume there is an exit speed change lane on the east side of the interchange. The notation for the assignment should be: SDE (S = speed change lane, D = diverging and E = east). Similarly, an interchange with a freeway segment going east/west should be: FS (F = freeway and S = south). Figure 3.24. shows examples of facility assignments with different interchange orientations.

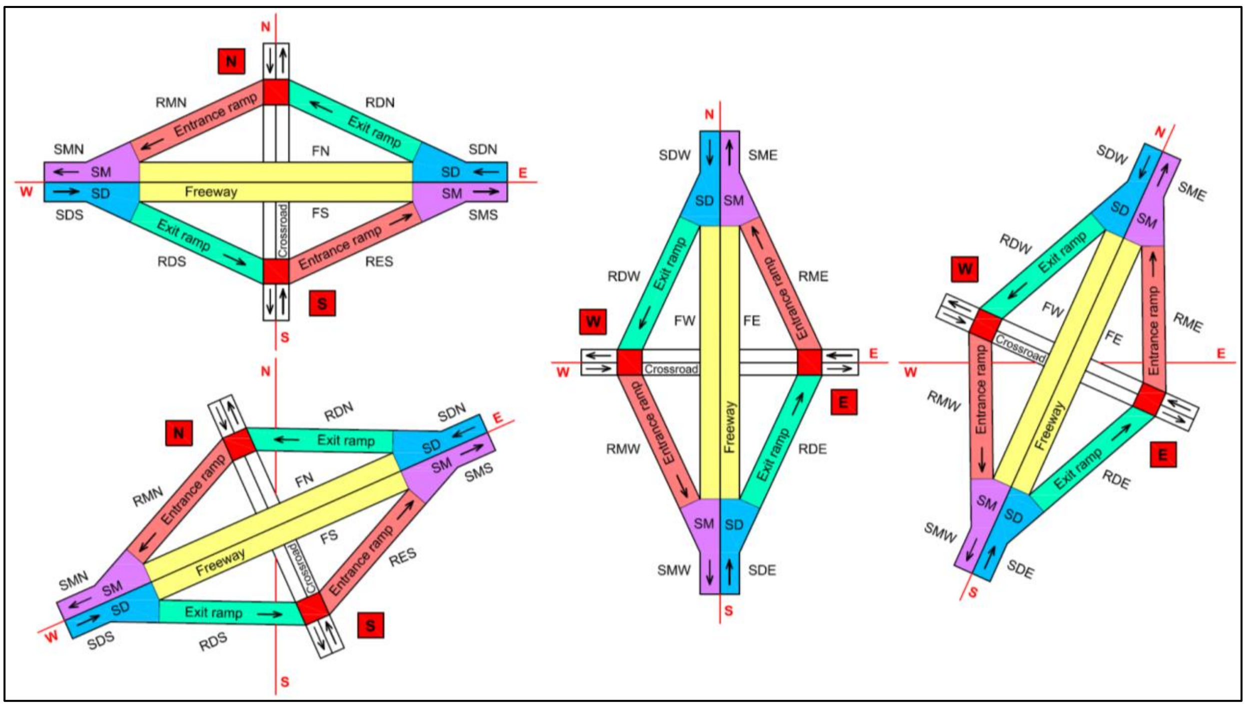


Figure 3.24. Examples of Facility Assignment Notation

Similarly to Phase 1, the assignments with the notations just described will be recorded in the crash data spreadsheet via an Interchange Facility column. Table 3.6. shows the spreadsheet with the new column and the corresponding assignment highlighted in green.

Table 3.6. Phase 2 Interchange Facilities Assignment

County	Desg	Travelway	Dir	Cont Log	Accident Class	Accident Date	Severity Rating	Image #	Log Unit	Intrchg	Light Cond	Road Surf Cond	Weather Cond	Tway Id	Property Damage	Day of Week	Time	Interchange	Period	Ramp terminal	Interchange Facility
GREENE	US	160	E	95.436	OUT OF CONTROL	11/30/2004	MINOR INJURY	1040046510	24.917		DARK	ICE	CLEAR	7806	NONE	TUE	2150	1	B	X	SDE
GREENE	US	160	E	95.619	OUT OF CONTROL	1/27/2005	MINOR INJURY	1050012588	25.1	Y	DAYLIGHT	DRY	CLOUDY	7806	NONE	THU	1110	1	B	X	FS
GREENE	US	160	E	95.619	REAR END	9/10/2005	DISABLING INJURY	1050036079	25.1	Y	DAYLIGHT	DRY	CLEAR	7806	NONE	SAT	755	1	B	X	
GREENE	US	160	E	95.599	REAR END	10/13/2005	PROPERTY DAMAGE ONLY	50116612	25.08	Y	DAYLIGHT	DRY	CLEAR	7806	NONE	THU	1819	1	B	X	

3.2.6. Crash Assignment Criteria for Interchange Facilities The crashes that occur on the different interchange facilities could have different characteristics in terms of traffic operations and roadway geometrics. The following sections will discuss the criteria for each facility type for making location assignment decisions.

3.2.6.1. Speed Change Lanes As mentioned previously, there are two types of speed change lanes: exit and entrance. Crashes at these facilities are usually due to speed differential and distracted drivers. Vehicles exiting the freeway usually reduce speed considerably and change lanes to be able to exit the freeway and continue to the exit ramp. However, following vehicles might not be able to adjust in time to the movements of the exiting vehicle which might lead to a crash. Cases 1, 2, and 4 of Figure 3.25. illustrate this type of crashes. In case 3, a distracted driver midjudges the proximity of the exit leading to a collision with the gore or runoff the road. Case 5 is a loss of control of the vehicle because of lane changing, braking, or any other factors influenced by the exit speed change lane geometry or operation. This type of crash is considered speed change related if the information of the crash report suggest that the driver lost control of the vehicle before the gore point. Case 6 shows a particular crash type in which a driver aborts exiting the freeway and returns to the through lanes causing a collision with a vehicle on the freeway.

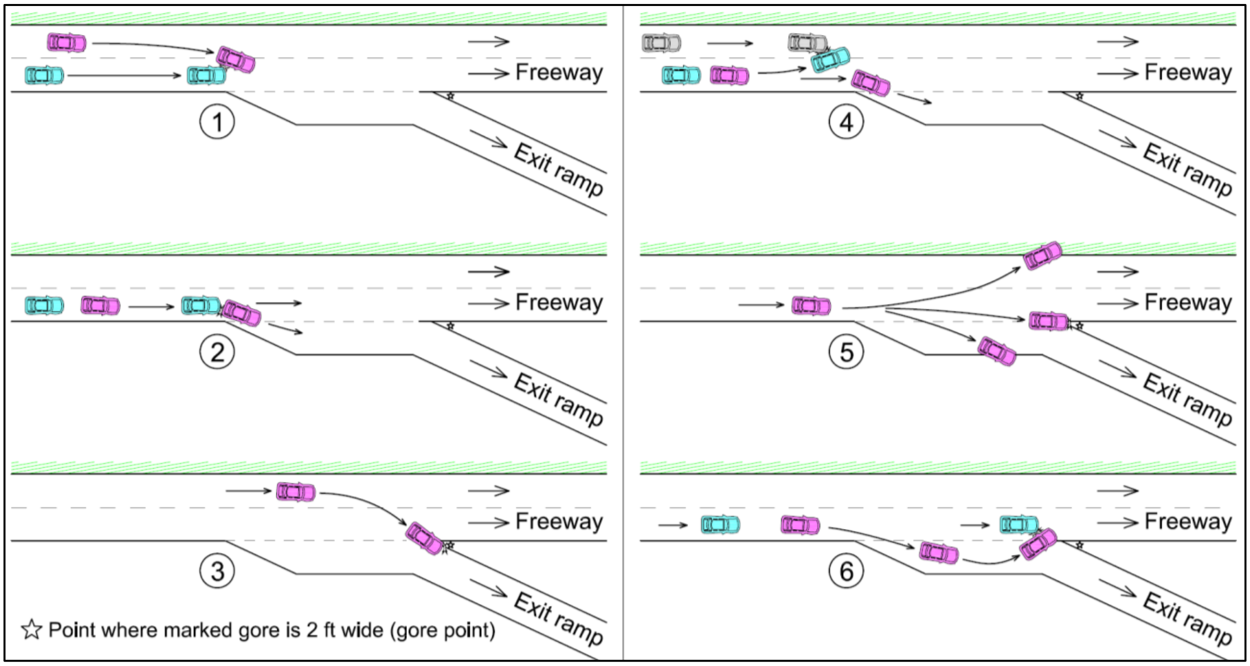


Figure 3.25. Common Crash Types at Exit Speed Change Lanes

Similar cases could occur at an entrance ramp where an entering vehicle might not be able to develop the necessary speed soon enough to keep up with mainline freeway traffic, resulting in a collision with approaching vehicles. Cases 1 and 3 of Figure 3.26. illustrate this type of crashes. Case 2 shows a crash due to a congested freeway where ramp vehicles have difficulty finding a gap to merge. Therefore, a queue is generated on the ramp, and a distracted driver rear ends the end of the queue. Case 4 shows an example of run off the road or loss of control crash. Usually these crashes are generated because of distracted drivers trying to find a gap on the freeway traffic to merge. This crash is considered speed change related if the crash report information suggests that it was after the gore point.

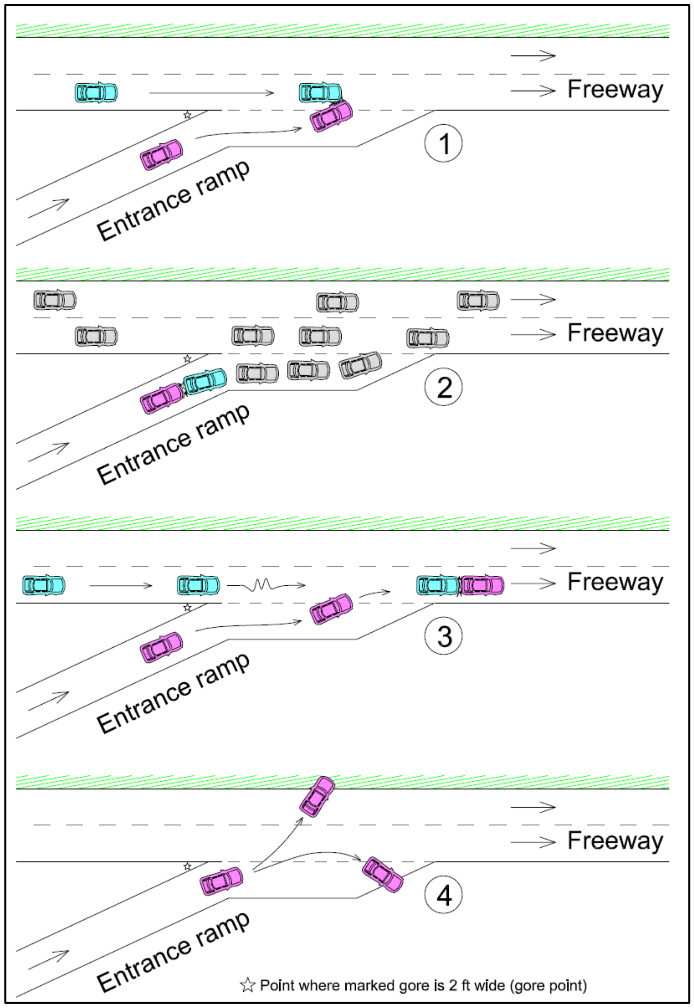


Figure 3.26. Common Crash Types at Entrance Speed Change Lanes

Geometric characteristics of speed change lanes might influence crashes as well. The taper configuration, number of lanes, width of lanes, and horizontal and vertical curves might significantly influence crashes. For instance, speed change lanes with multiple lanes add more exposure because of more vehicle interactions. Short tapers might cause vehicles to perform late merging or diverging movement decisions.

Crashes should be assigned to speed change lanes if the geometry and vehicle operations influenced the crash. As was extensively discussed in Phase 1 of this tutorial, the fact that a crash occurred within the boundaries of a facility type does not guarantee that the crash should be assigned to that facility. When ramp terminals were reviewed in Phase 1, crashes that were caused by queuing from the ramp terminals were considered ramp terminal related. Those crashes occurred either on the boundaries of exit ramp segments or even on the freeway mainline, depending on the length of the queue. Figure 3.27. shows the boundaries and other areas of interest for speed change lane related crashes. As shown in the figure, a speed change lane related crash could occur on a freeway mainline or even on an entrance ramp.

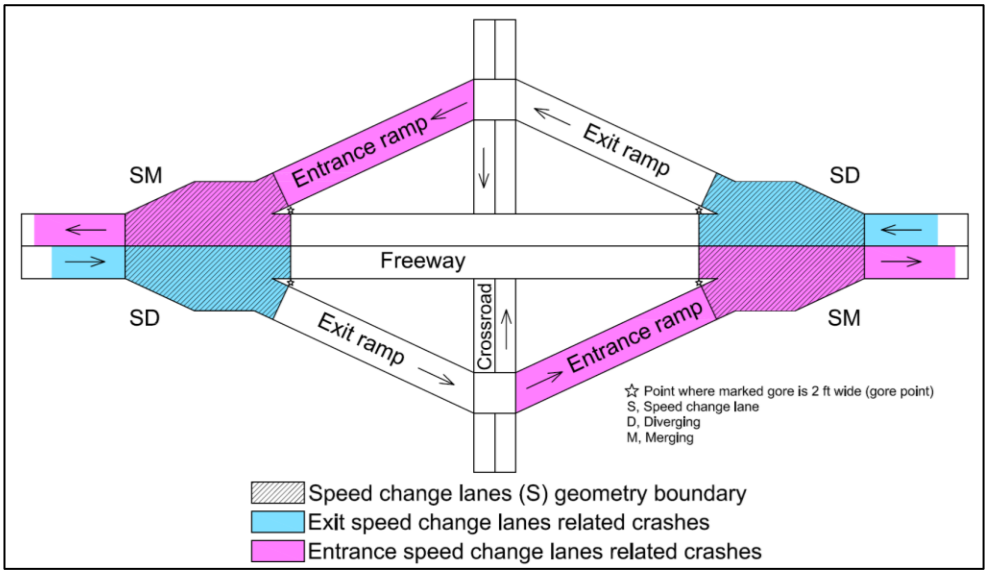


Figure 3.27. Speed Change Lanes Related Crashes and Boundaries

3.2.6.2. Ramp Segments Ramp segment crashes are similar to other segment crashes. The most common crashes at segments are vehicle departures from the roadway, collision with object/wild animal on the road, or less frequent types like sideswipes or rear end. However, at interchanges, the most influential geometric factor on crashes are horizontal and vertical curves and the related superelevation for horizontal curves. For example, the speed limit on a freeway is higher than on an exit ramp segment. Since vehicles on the freeway can circulate at higher speeds, drivers might not be able to adjust to a curved exit ramp segment while exiting on the exit ramp, and they might lose control of the vehicle running off the road after the gore point. A similar case could occur at an entrance ramp in which the driver, while looking for a gap on the freeway, loses control of the vehicle before gore point. Figure 3.28. illustrates run off the road crash types for exit and entrance ramps.

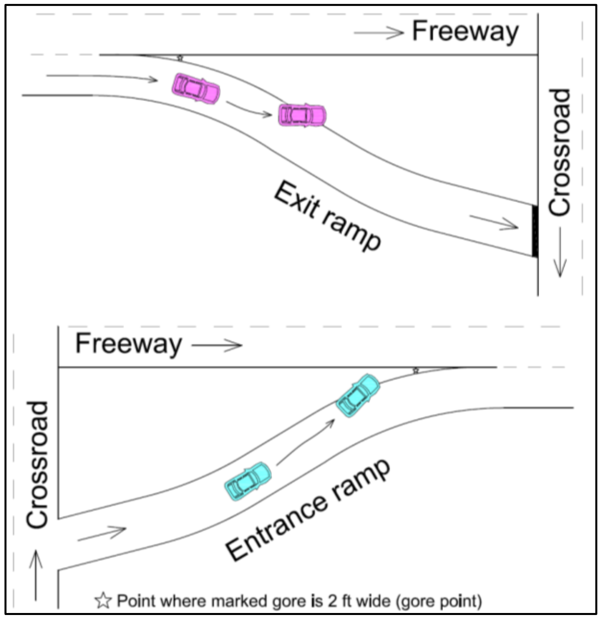


Figure 3.28. Run off the Road Crashes at Exit/Entrance Ramp Segments

Another common crash type at segments is vehicle collisions with abandoned or dropped objects, or wild animals on the road. This crash should be within the limits of the exit (after gore point) or entrance (before gore point) ramp segments. Figure 3.29. illustrates this crash type.

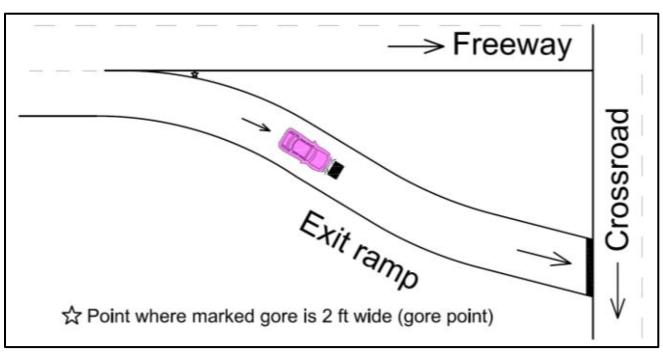


Figure 3.29. Collision with Object/Wild Animal on Ramp Segment

Important note: In Phase 1, object or animal related crashes were considered rare events and were not considered. However, for freeway and ramp segments, object or animal collisions are segment related crashes, and they should be assigned to the facility in which the collision occurred.

3.2.6.3. Freeway Segment The same crash types that apply to ramp segments apply to freeway segments. Figure 3.30. illustrates these crash cases on the interchange freeway segment. Case 1 shows a collision with an object or animal. Case 2 illustrates a sideswipe crash in the limits between the gore points. Case 3 shows loss of control of a vehicle between gore points.

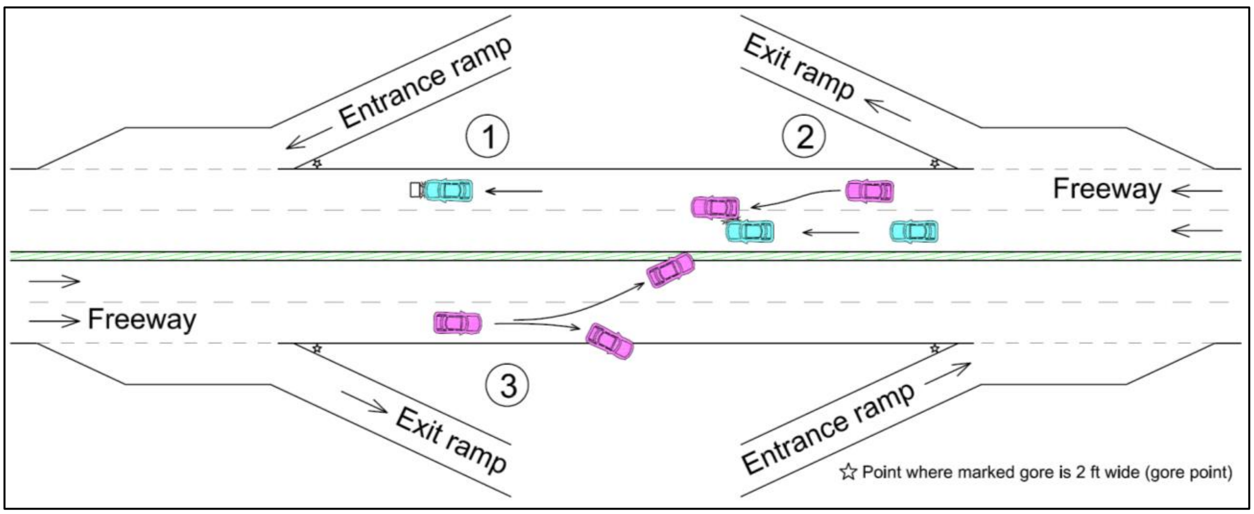


Figure 3.30. Crashes at Interchange Freeway Segment

Crashes occurring at an interchange freeway segment should not be influenced by the speed change lanes. If vehicles perform maneuvers on the freeway segment influenced by the speed change lanes, this crash should be assigned to the corresponding speed change lane and not the freeway segment. Figure 3.31. shows the boundaries of segment related crashes at an interchange.

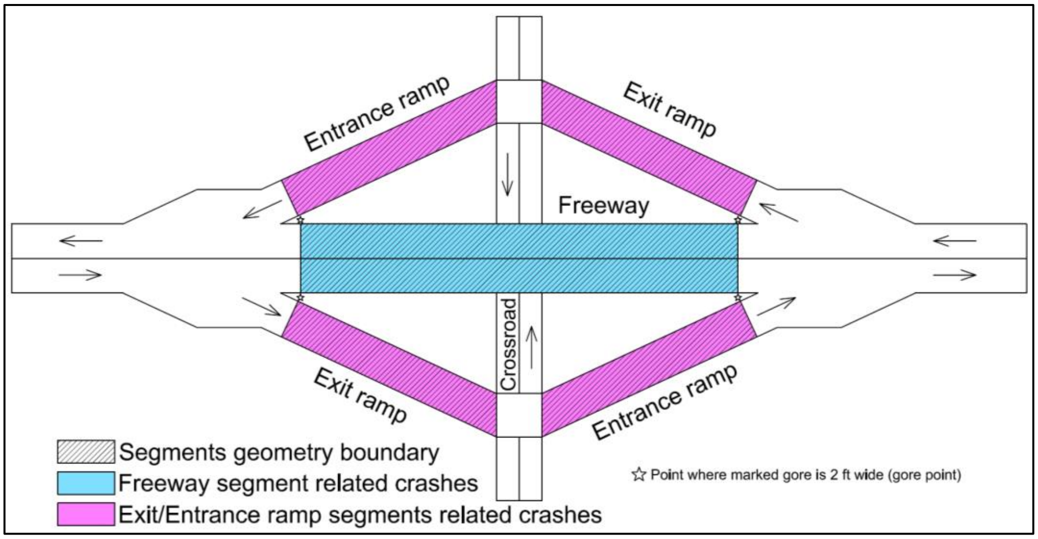


Figure 3.31. Interchange Segment Related Crashes and Boundaries

3.2.7. Examples A set of examples was developed to describe the procedure and methodology of this tutorial. The purpose of the examples is to illustrate in detail the most common scenarios, interpretation of the crash reports, and the different tools used in the procedure.

3.2.7.1. Example 1 The following is a step by step example of the use of the criteria and methodology for crash report revision and assignment. Table 3.7. contains the crash data necessary to start the review of the crash report.

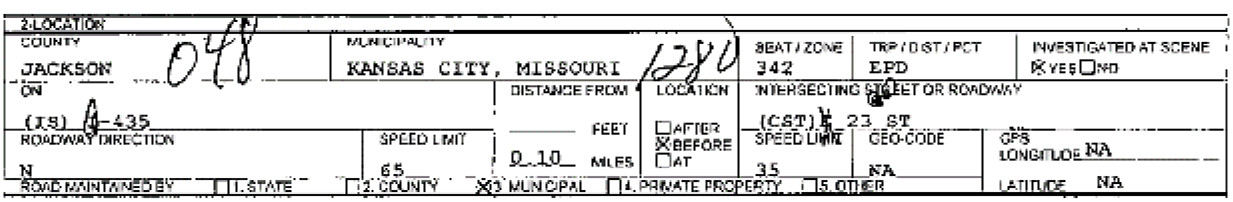
Table 3.7. Crash Data Example 1

County	Desg	Travelway	Dir	Cont Log	Accident Class	Accident Date	Severity Rating	Image #	Intersection #	Log Unit	Intrsc	Intrchg	Grpd	Light Cond	Road Surf Cond	Weather Cond	Tway Id	Property Damage	Day of Week	Time	Interchange	Period	Ramp terminal	Interchange Facility
JACKSON	IS	435	N	14.486	RIGHT TUR	10/4/2009	MINOR INJURY	90101021	0	14.486		Y		DAYLIGHT	DRY	CLEAR	6039	NONE	SUN	1444	5	B	X	SDE

3.2.7.1.1. Step 1 Locate the interchange in Google earth (use Phase2.kmz file). In Table 3.7., "Interchange" column provides the number of the interchange. In this example it is number 5. Figure 3.32. shows the aerial image of interchange 5.

Figure 3.32. Aerial Image Interchange 5
(Image Lansat/Copernicus, Google 2016)

Additionally, with the image number being 90101021, search and open the crash report from the crash report folder. After finding the crash report, the reviewer should start looking at section 2 of the report where the location of the crash is described with the different fields covered previously. Figure 3.33. shows the corresponding location section of the crash report of the example. The example interchange is located in Kansas City, Missouri, on I-435 northbound, 0.1 miles before 23rd St. crossover. Verify that the crash location is the same as the one on the aerial photograph.



2-LOCATION

COUNTY: JACKSON 048
MUNICIPALITY: KANSAS CITY, MISSOURI 1280
BEAT / ZONE: 342
TRP / DIST / PCT: EPD
INVESTIGATED AT SCENE: ☒ YES ☐ NO

ON: (IS) I-435
DISTANCE FROM: ____ FEET; 0.10 MILES
LOCATION: ☐ AFTER ☒ BEFORE ☐ AT
INTERSECTING STREET OR ROADWAY: (CST) E 23 ST

ROADWAY DIRECTION: N
SPEED LIMIT: 65
SPEED LIMIT: 35
GEO-CODE: NA
GPS LONGITUDE: NA
LATITUDE: NA

ROAD MAINTAINED BY: ☐ 1. STATE ☐ 2. COUNTY ☒ 3. MUNICIPAL ☐ 4. PRIVATE PROPERTY ☐ 5. OTHER

Figure 3.33. Section 2: Location Information of Crash Report Example 1

3.2.7.1.2. Step 2 The rest of the crash report information should be reviewed to verify the location and circumstances of the crash. Section 7 of the crash report contains the collision diagram. The diagram facilitates the identification of the circumstances and the direction of travel of vehicle(s).   For this example, Figure 3.34. shows two vehicles involved (V1 and V2). V1 was a small car and V2 was a motorcycle. The point of impact (P.O.I), exit/entrance ramp segments, and crossroad orientations are labeled. The north arrow is also provided. At this point of the crash report review, there is significant information to make the assignment to the exit speed change lane in the east side of the interchange (assignment notation: SDE). However, it is required to also review the narrative to confirm that all the information from sections 2 and 7 of the crash report are accurate. As a general rule a minimum of 2 out of 3 the sections should be consistent to proceed with the crash assignment. Otherwise, the crash should not be assigned to any facility, and should be kept with the designation X.

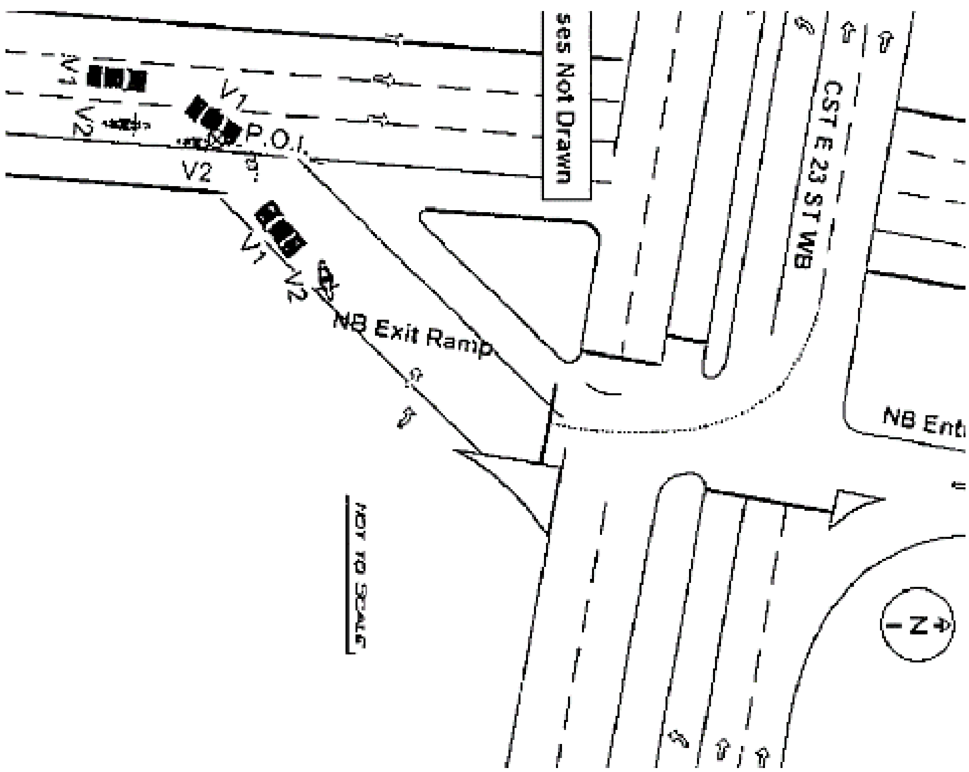


Figure 3.34. Section 7: Collision Diagram of Crash Report Example 1

3.2.7.1.3. Step 3 The narrative section of the example crash report is shown in Figure 3.35. The narrative supports the information from the location section and the collision diagram. In summary, the crash occurred when a vehicle tried to make the exit from the through lane closest to the median of the freeway. The driver claimed the she was not able to see the motorcycle coming on the right-most lane. She cut off the path of the motorcycle causing the crash. The crash should be assigned to the exit speed change lane in the east side of the interchange (assignment notation: SDE). It should be recorded in column "Interchange Facility" as shown in Table 3.7.

On 10/4/09, at approximately 1446 hours, PO Buske (Radio #343) and I (Radio #342) were dispatched to I-435 and 23rd St. Trafficway in regard to an Injury Accident.

Driver #1 reported she was travelling north on I-435 in the #2 lane of traffic when she realized she missed her exit (23rd St. Trafficway off ramp). Driver #1 reported she did not see anyone in the #3 lane of traffic and proceeded across the #3 lane of traffic onto the off-ramp to 23rd St. Trafficway. She stated she did not realize she struck Vehicle #2 until she saw Driver #2 airborne on the side of her vehicle. Driver #1 stated she then struck the guardrail face and her vehicle spun around coming to rest facing south on the north bound off-ramp. Driver #1 denied sustaining any injuries. She also denied any injuries to her passenger, her daughter.

Driver #2 reported he was travelling north on I-435 near the 23rd St. Trafficway off-ramp. Driver #2 reported Vehicle #1 was travelling north on I-435 in the #2 lane of traffic. Driver #2 reported he saw Vehicle #1 begin to change lanes travelling toward the off-ramp. ·Driver #2 stated he attempted to exit with Vehicle #1 to avoid being struck. However, he reported Vehicle #1 struck his vehicle causing him to be ejected. He stated he then skidded across the pavement, as well as, his vehicle. He stated he came to rest at the guardrail. He stated his vehicle came to rest north of him also striking the guardrail. Driver #2 sustained multiple contusions, scratches and scrapes. He also sustained a large cut to his right lower leg. He was transported to Liberty Hospital via MAST #140. MAST employees advised Driver #2 had a possible broken leg, as well.

Witness #1's statement is consistent with both Driver #1 and Driver #2. Witness #1 stated he saw the accident in his rear view mirror. He stated he observed Driver #1 exit from the #2 lane of I-435 onto the off-ramp. He stated Driver #2 was travelling north in the #3 lane of I-435 and was struck by Driver #1.

My investigation revealed the statements of Driver #1, Driver #2, and Witness #1. I observed damage consistent with their statements. Vehicle #1 sustained heavy damage to the front bumper, front fenders, and hood. Vehicle #1 also sustained damage to the passenger side rear bumper and the driver's side where it struck the guardrail. Vehicle #2 sustained moderate damage to the left side of the fuel tank, handle bars, front headlamps, and front fender. Vehicle #2 had a sidecar attached and the frame and wheel appeared to be bent.

Vehicle #1 was towed from the scene by private tow. Vehicle #2 was released at the scene to a family member to be towed at a later time by private means.

Figure 3.35. Section 28: Narrative/Statements of Crash Report Example 1

3.2.7.2. Example 2 The following is a step by step example of the use of the criteria and methodology for crash report revision and assignment considering a drop lane. Table 3.8. contains the crash data necessary to start the review of the crash report.

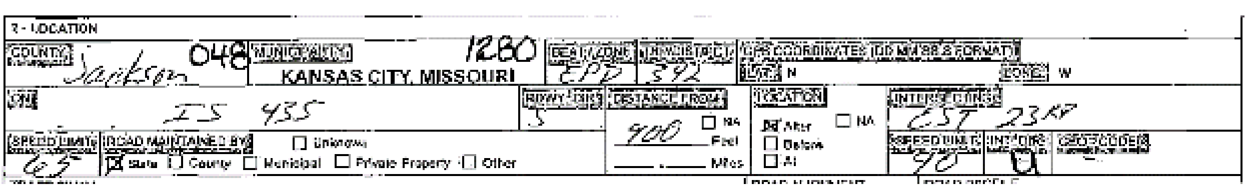
Table 3.8. Crash Data Example 2

Travelway	Dir	Cont Log	Accident Class	Accident Date	Severity Rating	Image #	Intersection #	Log Unit	Intrsc	Intrchg	Grpd	Light Cond	Road Surf Cond	Weather Cond	Tway Id	Property Damage	Day of Week	Time	Interchange	Ramp terminal	Interchange Facility
435	S	40.708	OUT OF CONTROL	3/3/2012	MINOR INJURY	120031880	0	5.102		Y		DAYLIGHT	DRY	CLEAR	6042	MODOT	SAT	830	3	X	X

3.2.7.2.1. Step 1 Locate the interchange in Google earth (use Phase2.kmz file). Table 3.9., "Interchange" column provides the number of the interchange. In this example it is number 3. Figure 3.36. shows the aerial image of interchange 3.

Figure 3.36. Aerial Image Interchange 3
(Image Lansat/Copernicus, Google 2016)

Additionally, with the image number being 120031880, search and open the crash report from the crash report folder. After finding the crash report, the reviewer should start looking at section 2 of the report where the location of the crash is described with the different fields covered previously. Figure 3.37. shows the corresponding location section of the crash report of the example. The interchange is located in Kansas City, Missouri, on I-435 southbound, 400 ft. after 23rd St crossover. Verify that the crash location is the same as the one on the aerial photograph.



2 - LOCATION
COUNTY Jackson 048
MUNICIPALITY 1280 KANSAS CITY, MISSOURI
BEAT/ZONE EPD
TH/DISTRICT 392
GPS COORDINATES (DD MM SS.S FORMAT) LAT N LONG W
ON IS 435
RDWY DIR S
DISTANCE FROM 900 Feet ☐ NA Miles
LOCATION ☒ After ☐ Before ☐ At ☐ NA
INTERSECTING CST 23RD
SPEED LIMIT 65
ROAD MAINTAINED BY ☒ State ☐ County ☐ Municipal ☐ Private Property ☐ Other ☐ Unknown
SPEED LIMIT 40
INT DIR
GEO CODE

Figure 3.37. Section 2: Location Information of Crash Report Example 2

3.2.7.2.2. Step 2 The rest of the crash report information should be reviewed to verify the location and circumstances of the crash. Section 7 of the crash report contains the collision diagram. The diagram facilitates the identification of the circumstances and the direction of travel of vehicle(s). Figure 3.38. provides the collision diagram form the crash report. V1 was traveling southbound on I-435 when a vehicle invaded its lane coming from the on ramp. V1 avoided collision, but it was not able to keep control of the vehicle since it started sling. Ultimately, the driver lost control of the vehicle and collided with the right shoulder barrier.

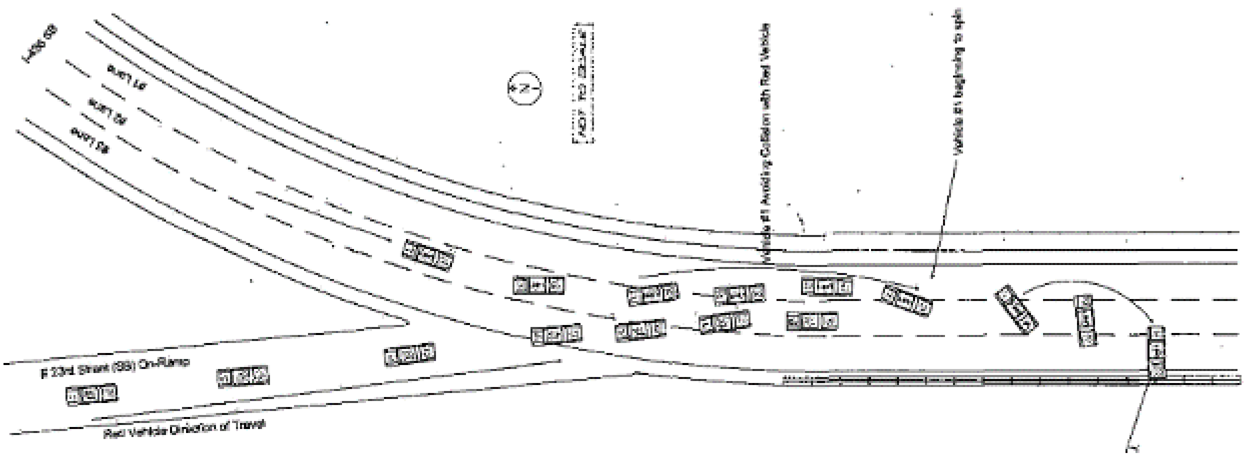


Figure 3.38. Section 7: Collision Diagram Crash Report Example 2

At this point, section 2 and section 7 of the crash report provide consistent information about the location of the crash. Before going further in to investigation, let's examine Figure 3.39. The figure shows a closer aerial image of the location where the crash occurred. It can be observed that the facility is add lane because there is not a visible taper within 1,600 ft from the gore point reference measured on the side shoulder of the road (yellow line). Therefore, this is not a speed change lane, it is an add lane freeway segment outside the footprint of the interchange and should be assigned with the letter X.

Figure 3.39. South Section of Interchange 3
(Image Lansat/Copernicus, Google 2016)

3.2.8. Training Test for Reviewer Twenty five crash reports were carefully selected to test the reviewer's understanding with respect to the criteria and procedures presented in this tutorial. The crash reports include different scenarios. The crash data is provided in a worksheet, so the reviewer can include the crash assignment in the "Interchange Facility" column according to the tutorial. The results of the test will be evaluated by a designated specialist to provide feedback to the reviewer and correct inconsistences in the review before the reviewer undertakes the review of actual data.

3.2.9. Supplemental Files The three files required for the reviewer test are:

a) Crash report images from the highway patrol

b) Worksheet containing crash data from all crashes

c) Google Earth file containing the location of interchanges

4. SAFETY EVALUATION OF DIVERGING DIAMOND INTERCHANGE

4.1. Introduction

The safety evaluation of the Diverging Diamond Interchange (DDI) includes ramp terminals site specific safety effectiveness, interchange project level safety effectiveness, and collision diagram analysis before and after implementation. Also, the effect of the operation of the DDI over adjacent facilities such as speed change lanes and signalized intersection was evaluated.

4.2. Ramp Terminal Site Specific Safety Effectiveness

Developing a jurisdiction specific model instead of calibrating existing models is likely to increase the accuracy and reliability of the predictions (AASHTO, 2010; Garber et al., 2010; Srinivasan and Carter, 2011; Sacchi et al. 2012; Srinivasan et al., 2013; Lu et al., 2014). The focus of this study was to improve the accuracy of the safety effectiveness estimates of the DDI implementations in Missouri by developing D4 signalized ramp terminals jurisdiction specific Safety Performance Functions (SPF) and associated Crash Modification Factors (CMF).

The dataset to develop the models consisted of 117 signalized four leg ramp terminals with diagonal ramps (D4), randomly selected across the state. The period of analysis considered included three years from 2010 to 2012. Geometric, signal control, traffic, and crash data were collected for the period of analysis. In order to developed the ramp terminal SPFs, accurate crash data was required. With the inconsistencies observed in crash reporting and assignment, all crash reports were carefully reviewed on a case by case basis. A methodology was developed for the review process and to accurately assign ramp terminal related crashes to the corresponding facility. After the crash data was reviewed and correctly assigned, prediction models for Fatal and Injury (FI) and Property Damage Only (PDO) crashes were developed using a customized spreadsheet and the solver in Excel following current SPF development guidance (Srinivasan and Bauer, 2013;

Hauer, 2015). Measures of goodness of fit included the Log-likelihood, overdispersion, parameter estimates variability, and cumulative (CURE) plots. The obtained jurisdiction specific models and corresponding overdispersion parameter (k) were used to determine the safety effectiveness of 18 DDI ramp terminals (9 interchanges) using the Empirical Bayes method.

4.2.1. Jurisdiction Specific Crash Prediction Model The development of the SPFs and CMFs was performed through a series of steps which included: sampling, data collection, modeling, and diagnostics. The following sections explain in detail the approach and process of each step.

4.2.1.1. Ramp Terminal Sampling The sampling of ramp terminals was conducted from an inventory of all interchanges in the state of Missouri. The sites were randomly selected to verify the ramp terminals type and signal control. Interchanges that included signalized four leg ramp terminals with diagonal ramps (D4) were included in the sample set for model development. A total of 117 ramp terminals were sampled with the selection process just mentioned. Following the sampling process, the data collection and crash report revision was performed.

4.2.1.2. Data Collection The data collection consisted of ramp terminal geometry, signal control, traffic, and crashes during the period of analysis (2010-2012). The geometric variables were collected using aerial imaging and map application of Missouri DOT (MoDOT) Transportation Management Systems (TMS). The following geometric data was collected:

- Number of through or shared lanes on the crossroad in both travel directions at the ramp terminal;
- Number of lanes on exit ramp (lanes fully developed for 100 ft. or more before intersecting crossroad);

- Number of left turn lanes on the crossroad inside part of the ramp terminal nearest to the freeway;
- Number of outside crossroad approach opposing lanes to the inside crossroad left turn lanes;
- Outside exclusive right turn lane of bay in the outside crossroad;
- Right turn channelization on the outside crossroad approach and exit ramp;
- Crossroad median in ft. (including the left-turn bay if present);
- Distance adjacent ramp terminal in mi.;
- Distance to next public street intersection on the outside crossroad approach in mi.;
- Number of driveways and public street approaches within 250 ft. on the outside crossroad approach.

The signal control of interest were the inside crossroad approach left turn and exit ramp right turn. This information was directly requested to MoDOT for each ramp terminal during the period of analysis. MoDOT issued a request to all the districts to provide the information from each jurisdiction since the authors had no direct access to historical signal data which was essential for modeling. The signal control for the left turn movements were permissive, protective/permissive, or protected only. For the exit right turn, the signal control was free merge, yield, stop, or signal controlled. The traffic data was available through MoDOT ODBC database in combination with the map application. The traffic data was collected for each individual year and ramp terminal approach—crossroad, exit, and entrance ramps. The dataset included AADTs by direction, and in the case of the crossroad the traffic of each direction was added to have the overall crossroad traffic volume.

The crash data was collected for a buffer influence area beyond the interchange physical and functional footprint to capture ramp terminal queue related crashes on the crossroad and freeway. On the crossroad outside approaches, the additional buffer of the footprint was of 500 ft. from the end of the functional area of the ramp terminal—or until next intersection. On the freeway, the buffer of the footprint of the interchange was extended from the beginning of the taper of the exit speed change lane with an additional 1,000 ft. If no taper was present, in the case of freeway exit drop lanes, 1,500 ft. from the gore was considered. Since there was inconsistency in crash reporting (landing) and assignment, all crash reports were reviewed to correct the data and specifically assign crashes to the ramp terminals of interest if the crashes were ramp terminal related. The revision of the crash reports consisted of identifying the specific location and circumstances of the crash. The crash report location description, collision diagram, and statements/narrative were carefully reviewed to make a determination of the accurate location and contributing factors for the crash. If evidence in the crash report led to determine that the ramp terminal geometric, operational and subsequent driving behavior contributed to the occurrence of the crash, the crash was assigned to the corresponding ramp terminal. Otherwise, it was discarded form the crash dataset. An approximate of 10,000 crash reports were reviewed from all sites, and 3,201 were ramp terminal related crashes.

4.2.1.3. Model Approach Model parameter estimation was performed through maximum likelihood estimation. The negative binomial was used to represent the crash data. The general form of the model included the base SPF and CMFs for the different ramp terminal traits by severity level. Equation 4.1 illustrates the approach to develop the prediction model for jurisdiction specific signalized D4 ramp terminals in Missouri.

$$N_{pred} = N_{spf} \times \begin{pmatrix} CMF_{ex} \times CMF_{lt} \times CMF_{plt} \times CMF_{mw} \times CMF_{ap} \\ \times\, CMF_{sl} \times CMF_{rl,xrd} \times CMF_{ch,xrd} \times CMF_{ch,ex} \end{pmatrix} \tag{4.1}$$

Where,

N_{pred} , predicted crash frequency (crashes/year);

N_{spf} , base safety performance function crash frequency (crash/year);

CMF_{ex} , exit ramp capacity crash modification factor;

CMF_{lt} , inside crossroad approach left turn lanes crash modification factor;

CMF_{plt} , opposing crossing lanes to inside crossroad left turn lanes crash modification factor;

CMF_{mw} , inside crossroad median width crash modification factor;

CMF_{ap} , access point frequency (driveways and public street) crash modification factor;

CMF_{sl} , segment length crash modification factor (adjacent ramp terminal and intersection);

$CMF_{rl,xrd}$, crossroad right turn lane crash modification factor;

$CMF_{ch,xrd}$, channelization right turn lane on crossroad;

$CMF_{ch,ex}$, channelization right turn lane on exit ramp.

4.2.1.4. Predictor Variables Functional Form The method used for predictor variable selection, order of introduction, and functional form were Exploratory Data Analysis (EDA) and Variable Introduction Exploratory Data Analysis (VIEDA) (Hauer et al., 1997; Hauer, 2004; Hauer et al., 2004; Hauer, 2015). For instance, the functional form of a variable is sought through visualization of the ratio between the observed crashes and base model predicted crashes (model with some introduced variables) versus values of the variable that is being considered for the model. When the ratios and variable values are combined, bins are generated with specified variable ranges to aggregate the data and look for orderly distributions which a functional form could ultimately represent in the model. With this process, the deficiencies of the base model are observed and the new variable considered for the model improves the model in ranges where the model is not able to predict accurately. Therefore, the new variable introduces additional information with a functional form to improve the quality of the overall model. The data distribution and resulting functional forms of the model variables are illustrated in the following figures. Figure 4.1. illustrates the distribution of the data over the crossroad and ramp AADTs.

Figures 4.2. to 4.4. show the results of the VIEDA analysis to determine the functional form of the variables AADT and crossroad number of lanes which are the main components of the SPF.

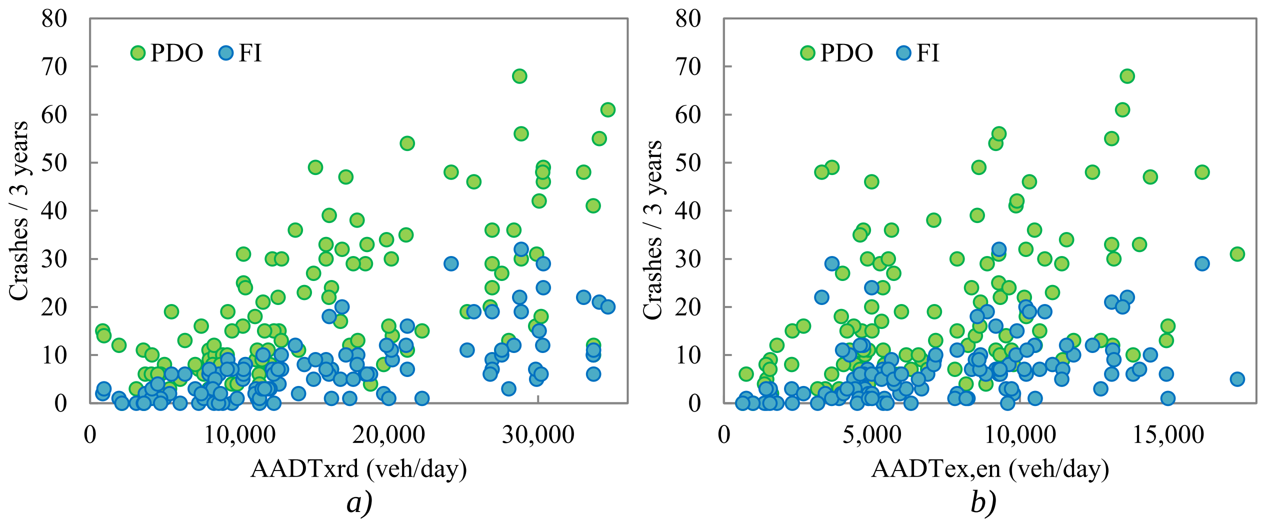


Figure 4.1. a) Crash Data versus Crossroad AADT, *b)* Crash Data versus Ramps AADT

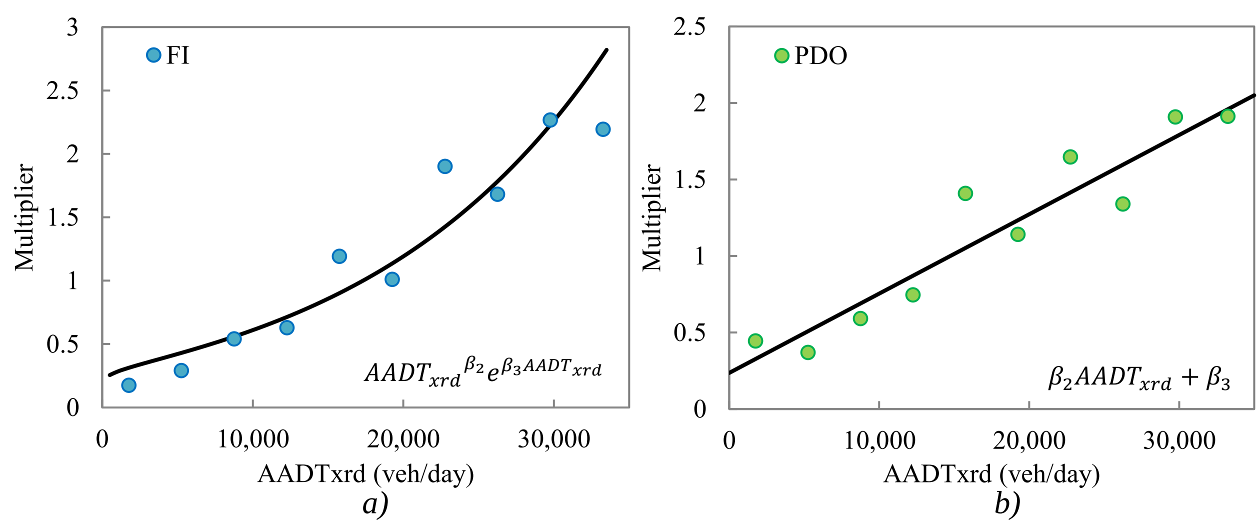


Figure 4.2. Crossroad AADT VIEDA Analysis Results for *a)* FI and *b)* PDO

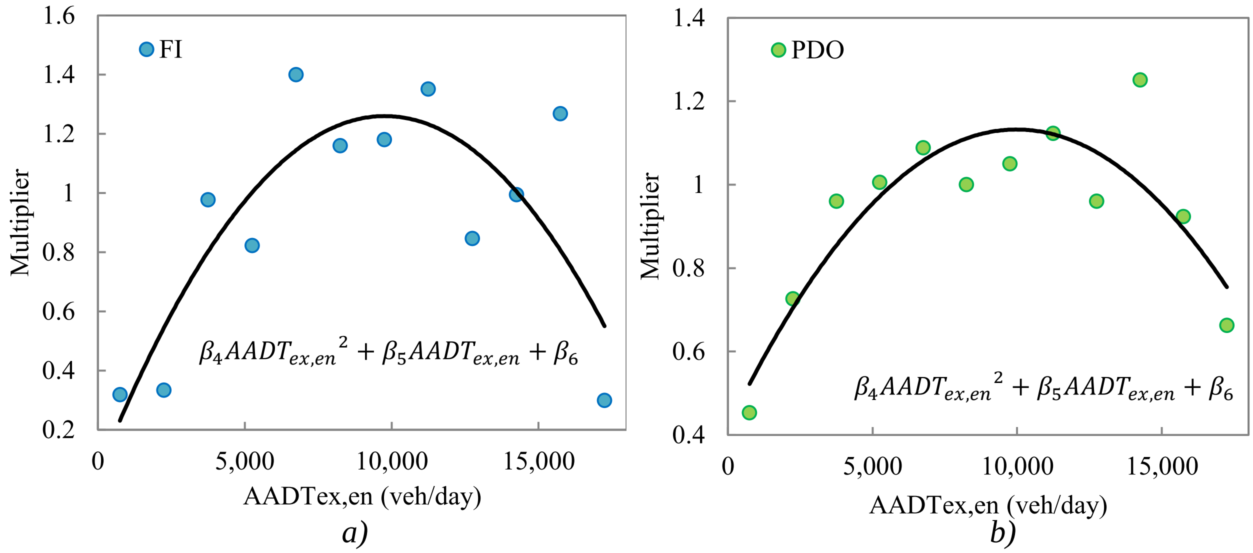


Figure 4.3. Ramps AADT VIEDA Analysis Results for *a)* FI and *b)* PDO

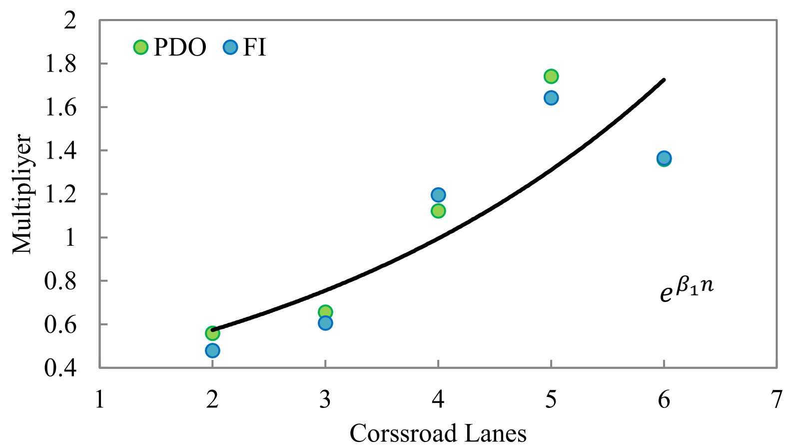


Figure 4.4. Number of Through or Shared Lanes on Crossroad

The same analysis was performed for the rest of the variables (CMFs) in the model to determine the corresponding functional form as each variable was introduced in the model. The rest of the variables did not show complex functional forms since the range of values are reduced (e.g. crossroad lanes in Figure 4.4., and binary variables in many cases) and the variables were introduced after most significant measures of exposure were represented in the model. The functional forms for the rest of the variables were also quadratic, linear, or exponential. Variables were added one by one in the model according to the relevance. The order was changed multiple times as well as the functional forms to find the best combination and optimal fit. Similarly, available modeling of freeway facilities (Bonneson et al., 2012) include impressive results with conventional model equation structure and functional forms for ramp terminals. These functional forms were also explored with Missouri data while developing the predictive model, but the results showed inaccurate representation supporting the argument that safety prediction models should be developed according to site specific characteristics and data distribution instead of using models from other states thorough statistically unsupported calibrations.

4.2.1.5. Safety Performance Functions The prediction model includes the safety performance function (SPF) to estimate the average crash frequency (crashes/year) for signalized D4 ramp terminals by severity (FI and PDO) with base conditions. The base conditions for the SPFs are:

• Inside crossroad left turn lane	Not present
• Inside crossroad protected left phase turn lane	Not present
• Outside crossroad right turn lane	Not present
• Channelization right turn on outside crossroad	Not present
• Channelization right turn on exit ramp	Not present
• Inside crossroad median	Not present
• Access points (driveway or intersection) within 250 ft.	Not present
• Distance to adjacent ramp terminal	Not present within 0.2 mi.
• Distance to adjacent intersection	Not present within 0.8 mi.

Each SPF has an associated overdispersion parameter k which provides a measure of the statistical reliability of the SPF. The smaller k is (close to 0), the more statistically reliable the SPF is. The overdispersion parameter is used in the Empirical Bayes method. The SPFs developed in this study are presented in Equations 4.2 and 4.3. The SPFs coefficients and inverse overdispersion parameter ($1/k$) are provided in Table 4.1.

For Fatal and Injury (FI) crashes:

$$N_{spf,fi} = e^{\beta_0+\beta_1\times n} \times \left[\left(\frac{AADT_{xrd}}{1{,}000}\right)^{\beta_2} e^{\beta_3\left(\frac{AADT_{xrd}}{1{,}000}\right)}\right] \times \left[\beta_4\left(\frac{AADT_{ex,en}}{1{,}000}\right)^2 + \beta_5\left(\frac{AADT_{ex,en}}{1{,}000}\right) + \beta_6\right] \quad (4.2)$$

For Property Damage Only (PDO) crashes:

$$N_{spf,pdo} = e^{\beta_0+\beta_1\times n} \times \left[\beta_2\left(\frac{AADT_{xrd}}{1{,}000}\right) + \beta_3\right] \times \left[\beta_4\left(\frac{AADT_{ex,en}}{1{,}000}\right)^2 + \beta_5\left(\frac{AADT_{ex,en}}{1{,}000}\right) + \beta_6\right] \quad (4.3)$$

Where,

$N_{spf,fi}$, predicted crash frequency D4 ramp terminal (base conditions and severity FI);

$N_{spf,pdo}$, predicted crash frequency D4 ramp terminal (base conditions and severity PDO);

n , number of through or shared lanes on the crossroad (both directions at the ramp terminal) applicable in the range of 2 to 6 lanes;

$AADT_{xrd}$, AADT volume for crossroad applicable range of 1,000 to 35,000 vehicles per day;

$AADT_{ex,en}$, added AADTs of exit and entrance ramps (veh/day); applicable in the range of 200 to 10,000 vehicle per day by ramp.

Table 4.1. SPF Coefficients

Severity	Model Coefficients							
	β_0	β_1	β_2	β_3	β_4	β_5	β_6	$1/k$
FI	0.2938	0.1638	0.2537	0.0382	-0.0009	0.0173	-0.0008	7.8741
PDO	0.1610	0.1849	0.1194	0.8352	-0.0071	0.1508	0.3958	6.5095

4.2.1.6. Crash Modification Factors This section includes the different CMFs applicable to the base SPF to adjust the prediction to specific site conditions characteristics. Different functional forms and variable combinations were explored from commonly used CMFs, and in the majority of the cases the functions did not represent the data accurate. Therefore, CMFs were developed by severity (FI and PDO) with the optimal functional forms and variable combination to Missouri data.

4.2.1.6.1. Exit Ramp Capacity The CMF is used to describe the influence of exit ramp number of lanes and capacity to capture the influence of queuing. Long queues may generate unsafe conditions (Bonneson et al., 2012). The CMFs by severity are represented using the following equations.

$$CMF_{ex,fi} = \beta_7 {n_{ex}}^{\beta_8} \quad (4.4)$$

$$CMF_{ex,pdo} = (1 - P_{ex}) + P_{ex} \times e^{\left(\beta_7 \frac{AADT_{ex}}{1,000 \times n_{ex,eff}}\right)} \quad (4.5)$$

With,

$$P_{ex} = \frac{AADT_{ex}}{AADT_{ex} + AADT_{en} + AADT_{xrd}} \quad (4.6)$$

$$n_{ex,eff} = \begin{cases} 0.5 \times (n_{ex} - 1) + 1 & : merge\ or\ free\ flow\ right\ turn \\ 0.5 \times n_{ex} & : signal,\ stop,\ or\ yield\ control\ right\ turn \end{cases} \quad (4.7)$$

Where,

$CMF_{ex,fi}$, exit ramp crash modification factor for severity FI (β_7 = 1.1282, β_8 = 0.3095);

$CMF_{ex,pdo}$, exit ramp crash modification factor for severity PDO (β_7 = 0.1182)

P_{ex} , proportion of total AADT on exit ramp;

$AADT_{ex}$, exit ramp AADT (veh/day);

$AADT_{en}$, entrance ramp AADT (veh/day);

$AADT_{xrd}$, crossroad AADT (veh/day);

$n_{ex,eff}$, effective number of lanes on the exit ramp (AASHTO, 2010);

n_{ex} , number of lanes on the exit ramp.

The CMF for PDOs has a functional form and approach proposed by Bonneson et al. (2012). Other functional forms and relevant variable combinations were tried for this CMF, but no other variation was found to have the accuracy to represent the variable for property damage only crashes in the data. This CMF is the only functional form used from common safety modeling practice.

4.2.1.6.2. Crossroad Left Turn Lanes The CMF describes the influence on crash frequency due to the presence of left turn lanes on the inside crossroad approach to the ramp terminal. The base condition is no left turn present. The CMFs by severity are represented using the following equation.

$$CMF_{lt,i} = \left[\beta_j {n_{lt}}^{\beta_k}\right]^{I_{lt}} \qquad (4.8)$$

Where,

$CMF_{lt,i}$, left turn lanes crash modification factor for severity *i*:

i = *fi*, $\beta_{j,9}$ = 1.9655, $\beta_{k,10}$ = -0.5932,

i = *pdo*, $\beta_{j,8}$ = 1.3022, $\beta_{k,9}$ = 0.2681;

n_{lt} , number of left turn lanes inside crossroad approach to ramp terminal.

I_{lt} , left turn indicator (= 1.0 if left turn present, 0.0 otherwise).

4.2.1.6.3. Protected Left Turn Operation The CMF is used to represent the influence in crash frequency of protected only left turn lanes in the inside approach of the crossroad. The base condition is permissive or protected-permissive left turn operation. The signal control effect is captured with the number of through lanes the left turn movement faces as opposing traffic. The CMFs by severity are represented using the following equation.

$$CMF_{plt,i} = \left[\beta_j n_{olt}{}^2 + \beta_k n_{olt}\right]^{I_{po}} \quad (4.9)$$

Where,

$CMF_{plt,i}$, protected only left turn lanes operation crash modification factor for severity *i*:

i = *fi*, $\beta_{j,11}$ = 0.0930, $\beta_{k,12}$ = -0.2849,

i =*pdo*, $\beta_{j,10}$= 0.0299, $\beta_{k,11}$ = -0.1196;

n_{olt} , number of through lanes on opposing traffic to the left turn movement (1 to 4);

I_{po} , left turn protected only (PO) operation indicator (= 1.0 if PO present, 0.0 otherwise).

4.2.1.6.4. Median Width Hardwood et al. (1995) conducted a study of operational and safety effects of median widths at intersections. Using data from California the relationship between median width and crash frequency at intersections were studied. These results were adapted (Bonneson et al., 2012) to build a CMF for median width at ramp terminals. This adaptation was explored with Missouri data and there was no clear representation and the contribution to the predictive model was not optimal. Therefore, a different CMF specific for the ramp terminal data with a simple power function was developed. The base condition is no median present.

$$CMF_{mw,i} = \left[\beta_j W_m{}^{\beta_k}\right]^{I_m} \quad (4.10)$$

Where,

$CMF_{mw,i}$, median width crash modification factor for severity *i*:

i = *fi*, $\beta_{j,13}$ = 1.1559, $\beta_{k,14}$ = -0.1013,

i = *pdo*, $\beta_{j,12}$ = 1.5783, $\beta_{k,13}$ = -0.1690;

W_m , median width (ft.);

I_m , median presence indicator (= 1.0 if median present, 0.0 otherwise).

4.2.1.6.5. Access Point Frequency This CMF is used to represent the influence of access points within 250 ft. to the center of the ramp terminal. The base condition is no driveways or public street approaches present in the outside crossroad approach.

$$CMF_{ap,i} = \left[\beta_j N_{dw,ps} + \beta_k\right]^{I_{ap}} \qquad (4.11)$$

Where,

$CMF_{ap,i}$, access point crash modification factor for severity *i*:

i = *fi*, $\beta_{j,15}$ = 0.5700, $\beta_{k,16}$ = 0.4107,

i = *pdo*, $\beta_{j,14}$ = 0.4844, $\beta_{k,15}$ = 0.6732;

$N_{dw,ps}$, number of driveway or public street approaches within 250 ft.;

I_{ap} , access points indicator (= 1.0 if driveway/public street approach present, 0.0 otherwise).

4.2.1.6.6. Segment Length The distance to the adjacent ramp terminal and intersection was represented with the segment length CMF. There is significant influence in the frequency of crashes in relation to the proximity of other facilities because of combined signal control operations, speed, exclusive lanes, or queues.

$$CMF_{sl,fi} = [\beta_{17} L_{rm\times st} \times 10.0 + \beta_{18}]^{I_{sl}} \qquad (4.12)$$

$$CMF_{sl,pdo} = \left[\beta_{16} {L_{rm+st}}^{\beta_{17}}\right]^{I_{sl}} \qquad (4.13)$$

Where,

$CMF_{sl,fi}$, segment length crash modification factor for severity FI

(β_{17} = 0.5744, β_{18} = 1.2403);

$CMF_{sl,pdo}$, segment length crash modification factor for severity PDO

(β_{16} = 0.7160, β_{17} = - 0.0126);

$L_{rm\times st}$, distance to adjacent ramp terminal (mi.) times distance to intersection (mi.);

L_{rm+st} , distance to adjacent ramp terminal (mi.) plus distance to intersection (mi.);

I_{sl} , segment length indicator (=1.0 if $l_{rm} \leq 0.2$ or $l_{st} \leq 0.8$ $mi.$, 0.0 otherwise).

4.2.1.6.7. Crossroad Right Turn Lane The CMF describes the relationship between crash frequency and the presence of an exclusive right turn lane to the entrance ramp from the outside crossroad road approach through the ramp terminal. The base condition is no right turn lane present.

$$CMF_{xrl,xrd,i} = e^{\beta_j I_{rl,xrd}} \qquad (4.14)$$

Where,

$CMF_{rl,xrd,fi}$, crossroad right turn lane crash modification factor for severity *i*:

i = *fi*, $\beta_{j,19}$ = 0.1581,

i = *pdo*, $\beta_{j,18}$ = 0.0972;

I_{rl} , crossroad right turn lane indicator (= 1.0 if present, 0.0 otherwise).

4.2.1.6.8. Channelization The CMF represents the influence in crash frequency due to the presence of channelization for the right turn lanes on the outside crossroad approach and exit ramp. The base condition is no channelization present.

$$CMF_{ch,z,i} = e^{\beta_j I_{ch,z}} \qquad (4.15)$$

Where,

$CMF_{ch,z,i}$, channelization for right turn at approach z and severity i,

z = *crossroad*: $i = fi$, $\beta_{j,20}$ = -0.2370 and $i = pdo$, $\beta_{j,19}$ = -0.2392,

z = *exit ramp*: $i = fi$, $\beta_{j,21}$ = 0.2160 and $i = pdo$, $\beta_{j,20}$ = 0.0501;

$I_{ch,z}$, channelization for right turn lane indicator for z (= 1.0 for right turn lane present, 0.0 otherwise, z = *crossroad* or *exit ramp*).

4.2.1.7. Model Diagnostics Common statistical modeling focuses on parameter estimates rather than the prediction itself and bias is overlooked. When a single number is used to measure the goodness of fit for the overall model, the measure is too general and does not provide useful information for safety prediction models (Hauer, 2015). Therefore, goodness of fit for the prediction model of this study was evaluated throughout the evolution following measures for all traits such as: variation of parameter coefficients, Log-likelihood, Cumulative Residuals (CURE) plots, and Overdispersion. These measures are considered appropriate for safety prediction modeling and current approach. (Srinivasan et al., 2013; Hauer, 2015).

4.2.1.7.1. Model Evolution Tables 4.2. and 4.3. show the model development progress as the variables were introduced in the model. The table includes the model coefficient found through maximum likelihood, Log-likelihood estimate, and inverse overdispersion. These measures of goodness of fit show the sequence of model improvement.

Table 4.2. Sequence of Models and Parameters for Predictive Model Severity FI

No.	0	1	2	3	4	5	6	7	8	9	10	11	12
Log -lik.	**973.762**	**984.470**	**1014.177**	**1025.688**	**1028.049**	**1032.892**	**1034.881**	**1035.210**	**1035.398**	**1036.111**	**1036.520**	**1037.444**	**1038.348**
$1/k$	1.2260	1.5544	3.6271	4.5713	5.0089	6.0449	6.7724	6.8601	6.8858	7.1911	7.2250	7.5716	7.8741
β_0	0.8239	-0.5740	-1.6761	0.7180	3.3802	2.6236	2.7125	2.7975	2.7232	2.6686	2.4177	2.2231	0.2938
β_1		0.3573	0.1671	0.1602	0.1084	0.1481	0.1459	0.1401	0.1465	0.1451	0.1454	0.1597	0.1638
β_2			0.4180	0.2343	0.2102	0.2957	0.2912	0.2693	0.2626	0.2452	0.2714	0.2794	0.2537
β_3			0.0371	0.0424	0.0428	0.0332	0.0355	0.0373	0.0376	0.0407	0.0372	0.0376	0.0382
β_4				-0.0016	-0.0001	-0.0001	-0.0001	-0.0001	-0.0001	-0.0001	-0.0001	-0.0001	-0.0009
β_5				0.0336	0.0025	0.0025	0.0025	0.0025	0.0025	0.0025	0.0025	0.0024	0.0173
β_6				0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.0008
β_7					0.9509	0.9542	0.9593	0.9619	1.0095	1.0344	1.1476	1.2953	1.1282
β_8					0.3839	0.3512	0.2802	0.2874	0.2945	0.3050	0.3037	0.2805	0.3095
β_9						1.8632	1.9036	1.9890	2.0091	1.9559	1.9559	2.1282	1.9655
β_{10}						-0.7055	-0.5915	-0.5574	-0.5653	-0.5148	-0.5944	-0.6107	-0.5932
β_{11}							0.0925	0.0768	0.0707	0.0823	0.0895	0.0967	0.0930
β_{12}							-0.3002	-0.2444	-0.2267	-0.2587	-0.2730	-0.2868	-0.2849
β_{13}								1.0793	1.1292	1.0651	1.1281	1.0441	1.1559
β_{14}								-0.0698	-0.0885	-0.0793	-0.1009	-0.0664	-0.1013
β_{15}									0.3788	0.4279	0.5275	0.3566	0.5700
β_{16}									0.5651	0.5703	0.4521	0.6489	0.4107
β_{17}										0.5171	0.5507	0.6197	0.5744
β_{18}										0.9882	1.0764	1.1598	1.2403
β_{19}											0.1187	0.1571	0.1581
β_{20}												-0.2104	-0.2370
β_{21}													0.2160

$$N_{pred,fi} = e^{\beta_0+\beta_1\times n} \times \left[\left(\frac{AADT_{xrd}}{1{,}000}\right)^{\beta_2} e^{\beta_3\left(\frac{AADT_{xrd}}{1{,}000}\right)}\right] \times \left[\beta_4\left(\frac{AADT_{ex,en}}{1{,}000}\right)^2 + \beta_5\left(\frac{AADT_{ex,en}}{1{,}000}\right) + \beta_6\right] \times \beta_7 {n_{ex}}^{\beta_8} \times \left[\beta_9 {n_{lt}}^{\beta_{10}}\right]^{I_{lt}} \times \left[\beta_{11} {n_{olt}}^2 + \beta_{12} n_{olt}\right]^{I_{po}}$$
$$\times \left[\beta_{13} {W_m}^{\beta_{14}}\right]^{I_m} \times \left[\beta_{15} N_{dw,ps} + \beta_{16}\right]^{I_{ap}} \times \left[\beta_{17} L_{rm\times st} \times 10.0 + \beta_{18}\right]^{I_{sl}} \times e^{\beta_{19} I_{rl,xrd}} \times e^{\beta_{20} I_{ch,xrd}} \times e^{\beta_{21} I_{ch,ex}}$$

Note: All model sequences were optimized. Therefore, models 1 to 12 can be used in practice according to the availability of data (e.g. model 3 can be used when crossroad number of lanes and AADTs are available with coefficients β_0 to β_6). The most accurate prediction model is 12 with all coefficients.

Table 4.3. Sequence of Models and Parameters for Predictive Model Severity PDO

No.	0	1	2	3	4	5	6	7	8	9	10
Log -lik.	**5223.779**	**5238.639**	**5262.464**	**5273.091**	**5274.318**	**5274.784**	**5276.117**	**5278.996**	**5280.389**	**5280.390**	**5282.843**
$1/k$	1.9667	2.5802	4.2380	5.1116	5.2530	5.3104	5.5107	5.9125	6.0852	6.0851	6.5095
β_0	1.8866	0.6934	0.3406	0.3196	0.2786	0.2569	0.2518	0.0574	0.0578	0.0585	0.1610
β_1		0.3077	0.1710	0.1577	0.1570	0.1607	0.1748	0.1705	0.1772	0.1771	0.1849
β_2			0.0408	0.0400	0.1797	0.1756	0.1870	0.1564	0.1037	0.1052	0.1194
β_3			0.0432	0.0363	0.0305	0.0304	0.0308	0.0314	0.0355	0.0354	0.8352
β_4				-0.0096	-0.0065	-0.0056	-0.0053	-0.0065	-0.0071	-0.0077	-0.0071
β_5				0.2113	0.1386	0.1191	0.1099	0.1429	0.1531	0.1661	0.1508
β_6				0.3002	0.3092	0.2668	0.2665	0.3395	0.3483	0.3809	0.3958
β_7					0.1071	0.1071	0.1179	0.1073	0.0964	0.0970	0.1182
β_8						1.1912	1.2218	1.2263	1.2074	1.2087	1.3022
β_9						0.0789	0.2037	0.3133	0.3460	0.3454	0.2681
β_{10}							0.0497	0.0157	0.0176	0.0171	0.0299
β_{11}							-0.1929	-0.0807	-0.0861	-0.0849	-0.1196
β_{12}								1.8212	1.6539	1.6556	1.5783
β_{13}								-0.2409	-0.2082	-0.2083	-0.1690
β_{14}									0.7064	0.7037	0.4844
β_{15}									0.4348	0.4347	0.6732
β_{16}										0.9074	0.7160
β_{17}										-0.0053	-0.0126
β_{18}											0.0972
β_{19}											-0.2392
β_{20}											0.0501

$$N_{pred,pdo} = e^{\beta_0+\beta_1\times n} \times \left[\beta_2\left(\frac{AADT_{xrd}}{1{,}000}\right)+\beta_3\right] \times \left[\beta_4\left(\frac{AADT_{ex,en}}{1{,}000}\right)^2+\beta_5\left(\frac{AADT_{ex,en}}{1{,}000}\right)+\beta_6\right] \times (1-P_{ex}) + P_{ex} \times e^{\left(\beta_7\frac{AADT_{ex}}{1{,}000\times n_{ex,eff}}\right)} \times \left[\beta_8 {n_{lt}}^{\beta_9}\right]^{I_{lt}}$$

$$\times \left[\beta_{10}{n_{olt}}^2+\beta_{11}n_{olt}\right]^{I_{po}} \times \left[\beta_{12}{W_m}^{\beta_{13}}\right]^{I_m} \times \left[\beta_{14}N_{dw,ps}+\beta_{15}\right]^{I_{ap}} \times \left[\beta_{16}{L_{rm+st}}^{\beta_{17}}\right]^{I_{sl}} \times e^{\beta_{18}I_{rl,xrd}} \times e^{\beta_{19}I_{ch,xrd}} \times e^{\beta_{20}I_{ch,ex}}$$

Note: All model sequences were optimized. Therefore, models 1 to 10 can be used in practice according to the availability of data (e.g. model 3 can be used when crossroad number of lanes and AADTs are available with coefficients β_0 to β_6). The most accurate prediction model is 10 with all coefficients.

4.2.1.7.2. Log-likelihood The Log-likelihood is an indicative measure of the probability to observe the crash counts from the data in the prediction of the model. When the measure is increased with the addition of a variable or changes in functional form, the more significant the improvement to the accuracy of the model (Hauer, 2015). For instance, in Table 4.3., the Log-likelihood for models 7 and 6 is 5278.996 and 5276.117 respectively. The increase is of 2.879 which numerically is an improvement of approximately 18 times ($e^{2.879} = 17.796$). According to the contribution of each variable with its optimized functional form the variables were sorted in Figure 4.5. for FI and Figure 4.6. for PDO prediction model.

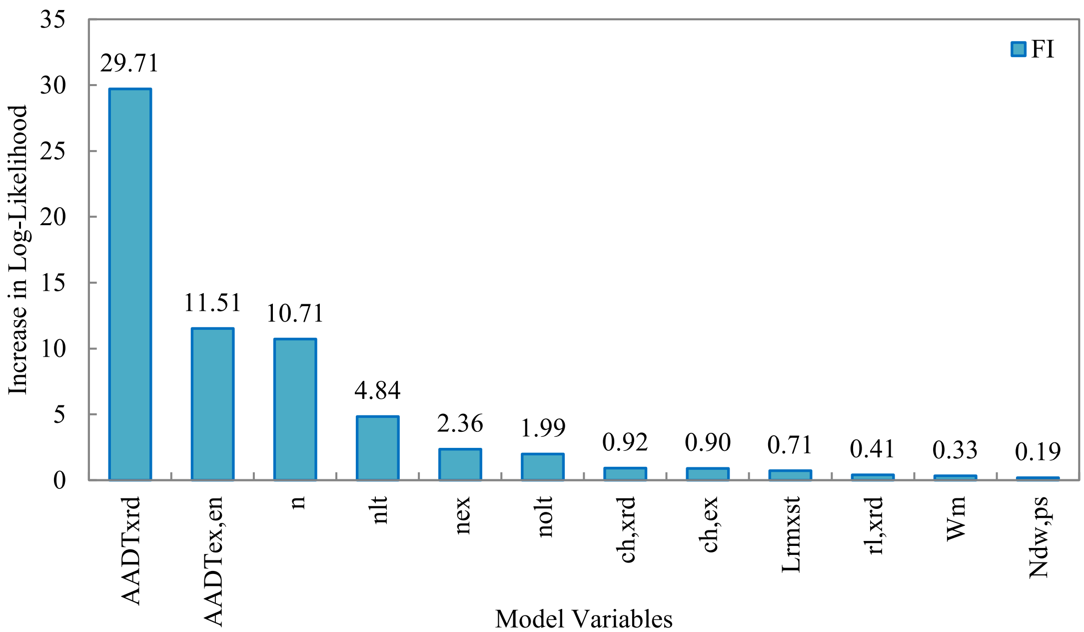


Figure 4.5. Increase in Log-likelihood by Variable for FI Prediction Model

The variables that contributed the most to the prediction models were the AADTs and number of lanes on the crossroad. The range of contributions and order was significantly different by severity. Whereas in the FI model, the median width was one of the least contributing variables; in the PDO model, it was the fourth most contributing variable to the model. In addition to the

Log-likelihood estimate, the overdispersion was also evaluated as variables were added to the model.

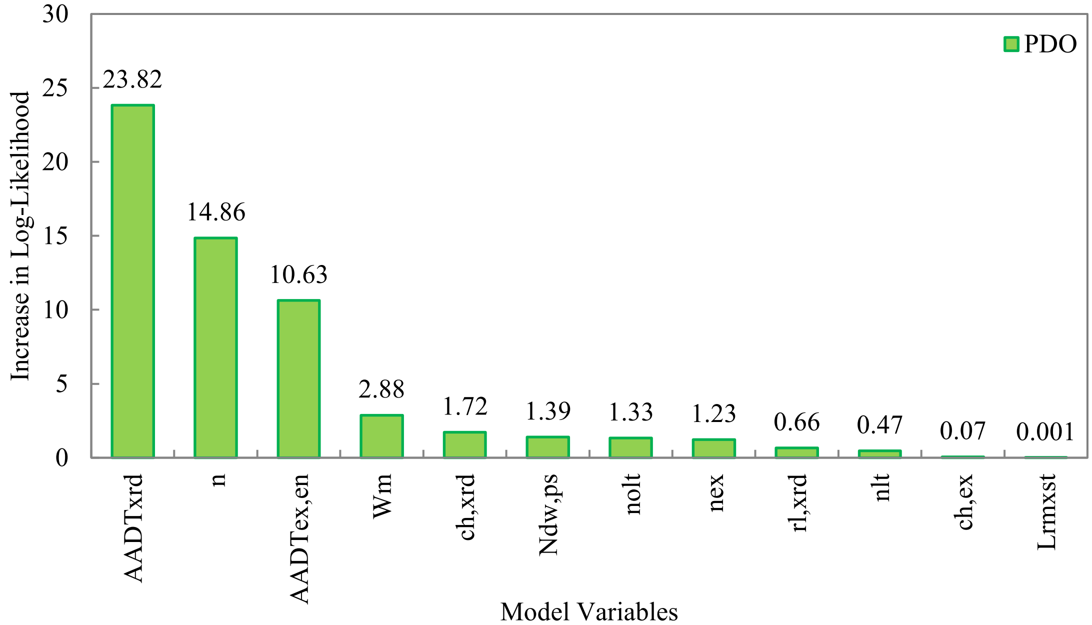


Figure 4.6. Increase in Log-likelihood by Variable for PDO Prediction Model

4.2.1.7.3. Overdispersion The overdispersion (k) is also estimated along the parameters estimates of the model. It is represented in the estimated model $E(u)$ by the variance term $V(u) = [E(u)]^2 / b$. The larger the value of b, the smaller $V(u)$ and consequently less overdispersion (k=1/ b) in comparison to the Poisson distribution. Whatever increases b is good for practice because the prediction model is more accurate and influential for the application in methods such us the Empirical Bayes (Hauer, 2015). Figures 4.7. and 4.8. illustrate the overdispersion improvement sorted by variable contribution. The contribution of the number of lanes for left turns, opposing traffic, and exit ramp significantly contributes to the model after the AADTs.

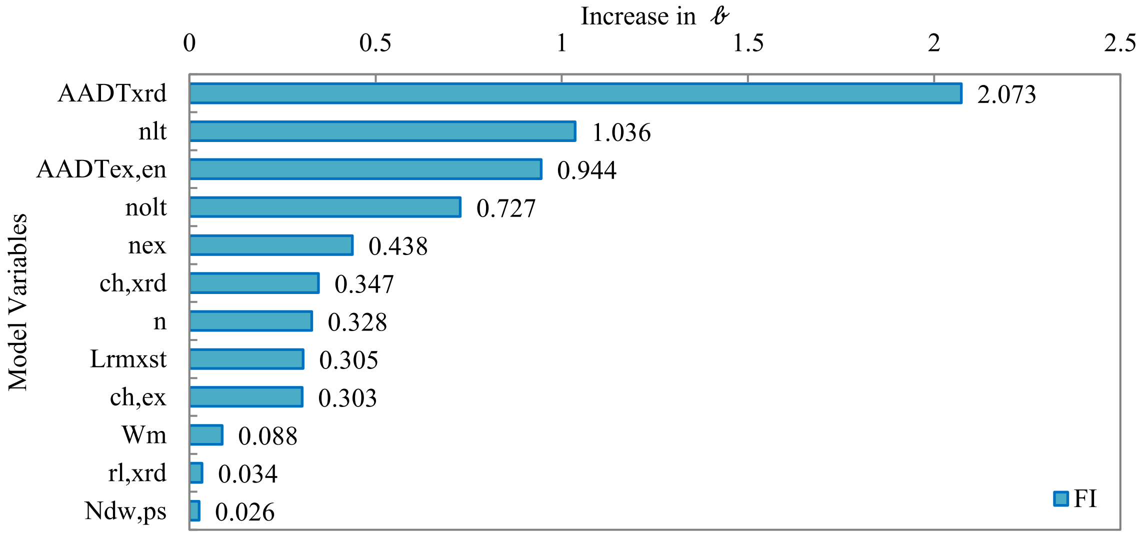


Figure 4.7. Increase in ℓ by Variable for FI Prediction Model

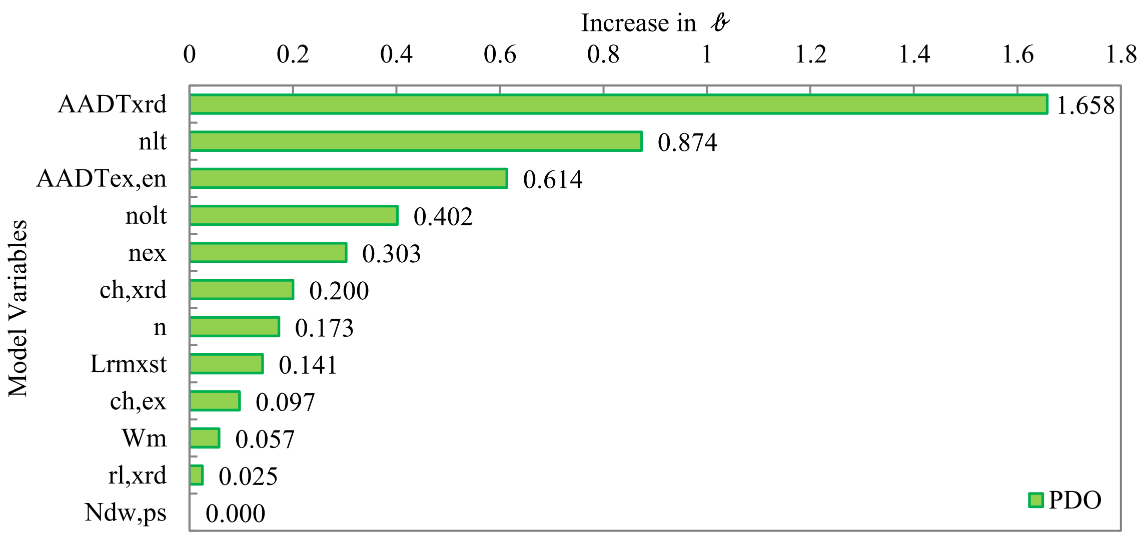


Figure 4.8. Increase in ℓ by Variable for PDO Prediction Model

4.2.1.7.4. Cumulative Residuals (CURE) plots Residuals are the difference between the number of recorded and predicted crashes—considered the basic element to judge goodness-of-fit (Hauer, 2004; Hauer, 2015). Cumulative residuals display the nature of fit of the model in the range of values of a predictor variable. The walk (cumulative residuals) indicates the performance of the model in a range of values. It shows ranges in which the prediction overpredicts, underpredicts, or jumps to extreme predictions (outliers). The CURE plots in Figures 4.9. and 4.10. are the final walks for the final models.

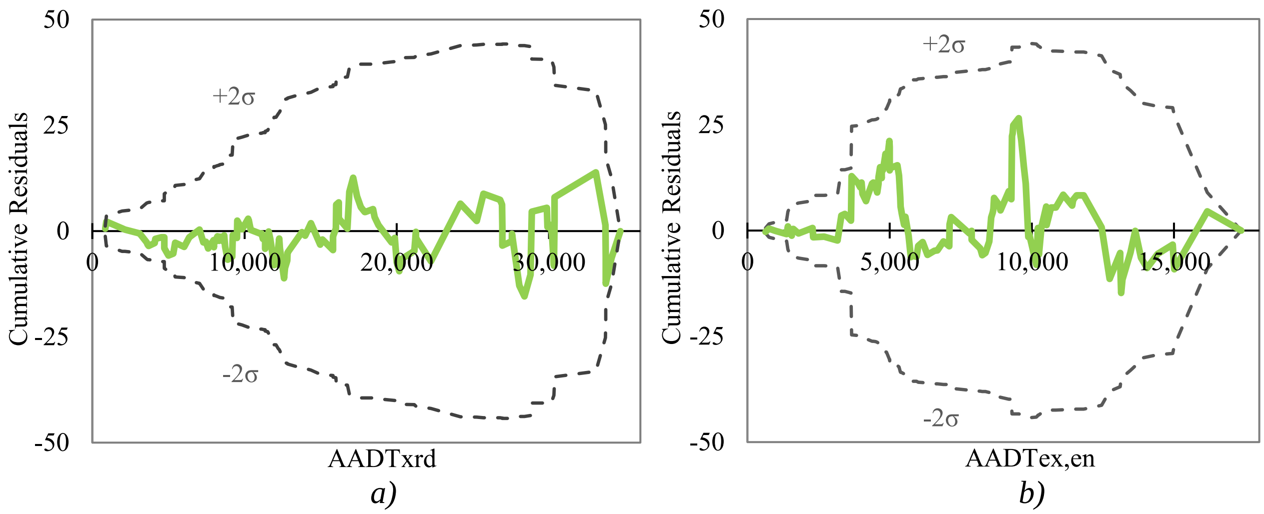


Figure 4.9. FI Model CURE Plot for *a)* Crossroad AADT and *b)* Ramps AADT

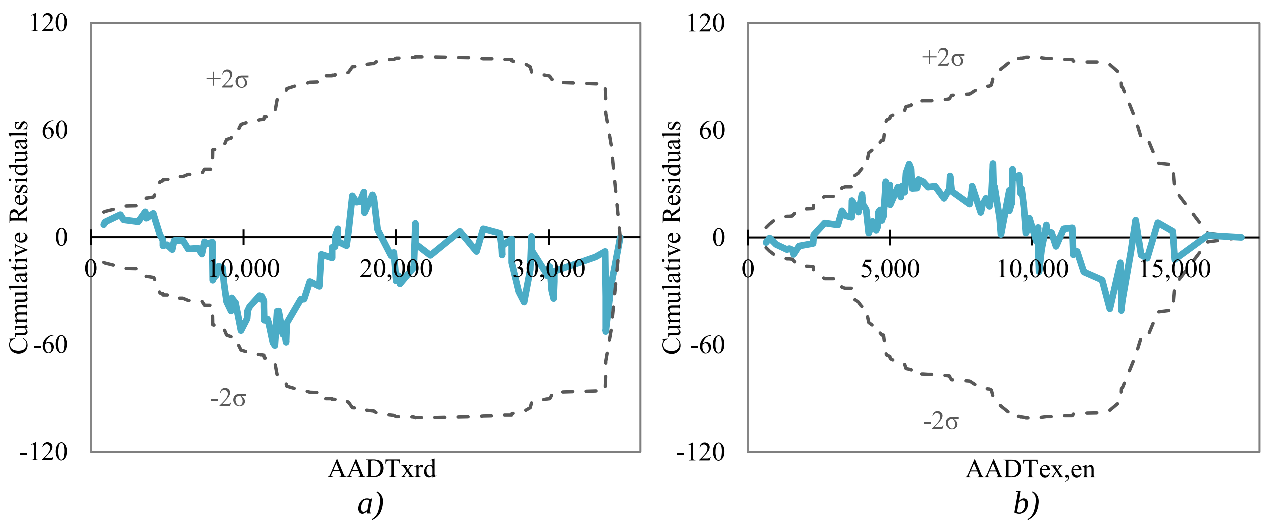


Figure 4.10. PDO Model CURE Plot for *a)* Crossroad AADT and *b)* Ramps AADT

In the process of developing the models, the CURE plots were constantly monitored to observe the changes in the walk as the variables were added or changes in functional form were made. The main interpretations of a CURE plot are (Hauer, 2015):

- Good fit is when the walk oscillates around 0
- Bad fit is when the CURE plot is entirely above or below 0
- Vertical jumps indicate possible outliers
- The walk should remain between the confidence limits

The final CURE plots meet all the criteria for optimal modeling. The walks remain between ±2σ intervals, oscillate around 0, and no other model variable or functional form was found to improve them further more.

The objective to develop Missouri specific ramp terminal prediction models was to accurately determine the safety effectiveness of Diverging Diamond Interchanges (DDI). The models developed allow the use of the Empirical Bayes method through the use of the overdispersion parameter (*k*). The Empirical Bayes is the most accurate method and is commonly used in practice (AASHTO, 2010).

4.2.2. Site Specific Safety Effectiveness The dataset included for the safety evaluation included 9 DDIs with signalized D4 ramp terminals. A total of 18 ramp terminals—2 ramp terminals by DDI. Table 4.4. provides geometric and operational characteristics of the DDIs. The interchanges were implemented between years 2009 and 2013. The observational before and after study required data for each period. The before period ranged from 51 to 36 months (4.25 to 3 years), and the after period ranged from 51 to 12 (4.25 to 1 year).

Table 4.4. DDI Sites Characteristics for Site Specific Safety Evaluation

DDI Site Location	Opening Date	Period (Month)		Crossroad			Conf. Type[4]	Ped. Acc.[5]	Adj. Ramp Ter. (ft.)	Adj. Inter. (ft.)
		Before	After	Speed (mph)[1]	AADT[2]	Lanes[3]				
RT-13 and I-44, Springfield	6/21/2009	51	51	40	25,215	4	O	M	530	320/ 685
I-270 and Dorsett Rd., Maryland Heights	10/17/2010	35	35	35	31,405	6	U	R	480	265/ 635
J. River Exp. and National Av., Springfield	7/12/2010	38	38	40	31,809	6	O	M	630	530/ 580
US-65 and MO-248, Branson	11/20/2011	44	22	35	31,640	3	O	M	740	580/ 1,795
I-435 and Front St., Kansas City	11/6/2011	44	22	40	23,926	4	U	M	420	530/ 1,955
Chestnut Exp. and Route 65, Springfield	11/10/2012	36	24	40	23,351	4	U	R	370	160/ 475
US-60 and Kansas Expy., Springfield	8/18/2013	36	12	45	26,002	5	O	M	600	470/ 1,000
IS-70 and Woods Chapel Rd., Blue Springs	9/26/2013	36	12	40	16,704	5	O	R	550	580/ 400
IS-70 and Stadium Blvd., Columbia	10/14/2013	36	12	40	37,951	3	O	M	420	380/ 550

Notes: [1]Posted speed limit; [2]AADT of 2014 for reference purpose only; [3]Lanes between ramp terminals; [4]Crossover overpass (O) or underpass (U); [5]Pedestrian accommodation through median (M) or roadside (R).

The construction period was omitted and the dates in which the crash data was collected was matched in both periods to account for seasonality. The posted speed limit on the crossroad was similar across sites with 35-45 mph. The AADT ranged from 11,000 to 32,000 vehicles per day. The total number of lanes between ramp terminals varied from 3 to 6. Some differences existed in the type of crossover where some sites were over or underpasses. Pedestrian accommodation was allocated trough the median or roadside. The distances to adjacent ramp terminal and intersections were similar across sites.

4.2.2.1. Data Collection The data collection was performed following the same procedure and sources used for the samples used in the development of the safety prediction model. Also, all the crash reports before and after the implementation the DDI were reviewed and assigned based on the criteria developed for ramp terminal related crashes established in this study. Approximately, 3,000 more crashes reports were reviewed from which 1,288 were found to be ramp terminal related.

4.2.2.2. Results The safety effectiveness of the DDI ramp terminals was performed using the jurisdiction specific safety prediction models in combination with the Empirical Bayes method. Table 4.5. shows the results of the evaluation with the observed, expected crashes in the after period, and corresponding safety effectiveness (standard error in parenthesis) in percentage.

Table 4.5. DDI Ramp Terminal Safety Effectiveness Results

DDI Site Locations	Ramp Terminal	FI[1]			PDO[2]			TOT[3]		
		Observed	Expected	Safety Effec.% (St. Error%)	Observed	Expected	Safety Effec.% (St. Error%)	Observed	Expected	Safety Effec.% (St. Error%)
RT-13/I-44,	1	**10**	**27**	63.5(12.9)	**52**	**54**	4.5(18.3)[4]	**62**	**82**	24.2(12.5)
Springfield	2	**9**	**40**	77.7(8.1)	**32**	**79**	59.4(8.6)	**41**	**119**	65.6(6.3)
I-270/ Dorsett Rd.,	3	**5**	**6**	15.7(40.2)[4]	**32**	**45**	28.5(15.2)[4]	**37**	**51**	27.0(12.6)
Maryland Heights	4	**9**	**6**	-48.1(54.5)[4]	**47**	**80**	41.6(9.9)	**56**	**86**	35.3(9.9)
J.R. Exp./National Av.,	5	**6**	**16**	63.0(16.1)	**44**	**30**	-48.2(32.3)[4]	**50**	**46**	-9.0(20.6)[4]
Springfield	6	**12**	**23**	47.0(17.0)	**43**	**51**	16.3(16.3)[4]	**55**	**74**	25.7(12.3)
US-65/MO-248,	7	**2**	**5**	63.0(26.2)	**1**	**8**	87.8(11.9)	**3**	**14**	78.0(12.9)
Branson	8	**1**	**5**	80.4(19.1)	**9**	**18**	50.7(17.9)	**10**	**23**	57.2(14.2)
I-435/Front St.,	9	**1**	**4**	75.0(24.3)	**5**	**21**	76.4(10.8)	**6**	**25**	76.2(10.0)
Kansas City	10	**0**	**3**	100.0(0.3)	**4**	**15**	74.0(13.3)	**4**	**19**	78.8(10.8)
Chestnut Exp./RT-65,	11	**2**	**7**	69.9(21.2)	**6**	**16**	63.4(16.1)	**8**	**23**	65.3(13.2)
Springfield	12	**8**	**8**	1.5(39.5)[4]	**17**	**14**	-23.1(37.1)[4]	**25**	**22**	-14.0(28.0)[4]
US-60/Kansas Exp.,	13	**1**	**1**	21.3(76.2)[4]	**10**	**4**	-131.8(84.3)[4]	**11**	**6**	-97.0(67.5)[4]
Springfield	14	**4**	**4**	-8.1(56.1)[4]	**9**	**12**	26.4(26.2)[4]	**13**	**16**	18.4(24.4)[4]
IS-70/W. Chapel Rd.,	15	**3**	**2**	-46.4(87.1)[4]	**10**	**9**	-6.6(37.1)[4]	**13**	**11**	-13.7(33.5)[4]
Blue Springs	16	**1**	**1**	22.9(74.7)[4]	**3**	**5**	39.0(36.0)[4]	**4**	**6**	35.6(32.5)[4]
IS-70/Stadium Blvd.,	17	**0**	**3**	100.0(0.4)	**1**	**4**	77.3(22.1)	**1**	**7**	85.6(14.2)
Columbia	18	**1**	**4**	75.1(24.3)	**0**	**5**	100.0(0.2)	**1**	**9**	89.4(10.5)
ALL SITES	All	**75**	**166**	55.0(5.9)	**325**	**473**	31.4(4.7)	**400**	**639**	37.5(3.7)

Note: [1]Fatal and Injury crashes, [2]Property Damage Only Crashes, [3]Total crashes, [4]not significant at the 95% confidence level. Negative values represent increase in crashes.

The overall safety effectiveness results showed a reduction of 55.0% (5.9%) in Fatal and Injury crashes (FI), 31.4%(4.7%) of Property Damage Only crashes (PDO), and 37.5%(3.7%) in Total crashes (TOT) significant at the 95 percent significance level. At the disaggregate level, ramp terminal 12 to 16 showed not significant results in all severities. At some of these sites, an increase in crashes was recorded. These variations and extreme values could be attributed to different factors. Crash reporting at the corresponding jurisdiction may not be as consistent as other districts. These facilities had 1 to 2 years after period duration and reduced number of crashes. On the other hand, the DDI may not be the appropriate treatment at the specific sites where an increase of crashes was recorded. Safety evaluations at individual sites provide good reference of trends at the disaggregated level, but they were not predominant in the completion of the analysis. Methods such us the Empirical Bayes adjust the influence of each individual site in the overall result according to the variability of the corresponding estimate. Therefore, the influence of extreme results does not bias the aggregated estimate, and it is expected to have locations in which there are some inconsistencies from the general trend.

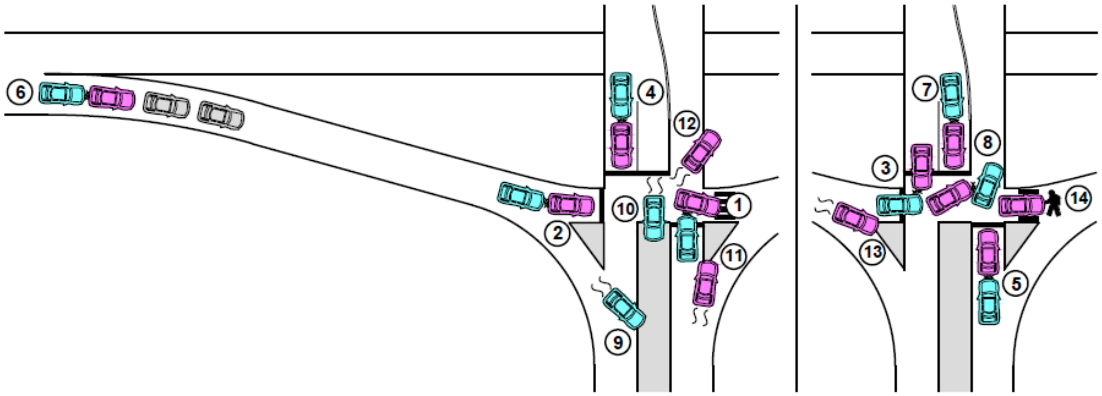
4.2.3. Collision Diagram Analysis The crash report images for fatal and injury crashes occurring at ramp terminals were reviewed to identify any differences in the types of crashes occurring at a conventional diamond versus a DDI. Collision diagrams were created to visualize the frequency of types of crashes.

All crashes within the footprint of the interchange were landed at the specific reported location of the crash for both periods separately. Although crashes occurring at all interchange facilities were reviewed, only the crashes occurring at the ramp terminals or related to the ramp terminals were analyzed using the collision diagrams. This focus on ramp terminals was due to the fact that the primary difference between the conventional diamond and a DDI is the configuration of ramp terminals and the interaction between traffic movements at the terminals.

The collision diagrams generated for the before and after period are shown in Figure 4.11. In generating Figure 4.11., crashes occurring over the same period before and after DDI were included for each site. Sites 1 to 12 (6 interchanges) were used for the collision diagram analysis (see Table 4.4.). Sites 1 to 6 had the same duration of before and after periods. The duration of before period for sites 7 to 12 was reduced to match the shorter after period. This adjustment in the duration allowed for a fair comparison of the before crashes (traditional diamond) with the after crashes (DDI), since they occurred over the same duration. For the collision diagram shown in Figure 4.11., the crashes were classified into 14 different types for the before and after periods. The top two crash types in the before period at the conventional diamond ramp terminals were, 1) collision of left turn movements from inside the crossroad and the oncoming through movement, and 2) rear end collisions on the exit ramp at the intersection.

In the after period for the DDI design, the top two crash types were, 1) rear end collisions between right turning movements on the exit ramp at the intersection, and 2) rear end collisions on the outside crossroad approach leg to the ramp terminal (see Figure 4.11.). It was also observed that some other types of crashes distributed across the different legs of the DDI ramp terminal increased, but all these crashes were of lower severity. For instance, sideswipes at the different merging and diverging locations, and the loss of control in the bays while making turning movements, increased with the DDI; however, none of these types of crashes resulted in any severe injuries. Thus, the DDI design traded a severe crash type, right angle left turn crash, with less severe rear end, sideswipe, and loss of control crash types. Wrong way crashes inside the crossroad between ramp terminals accounted for 4.8% of the crashes occurring at the DDI.

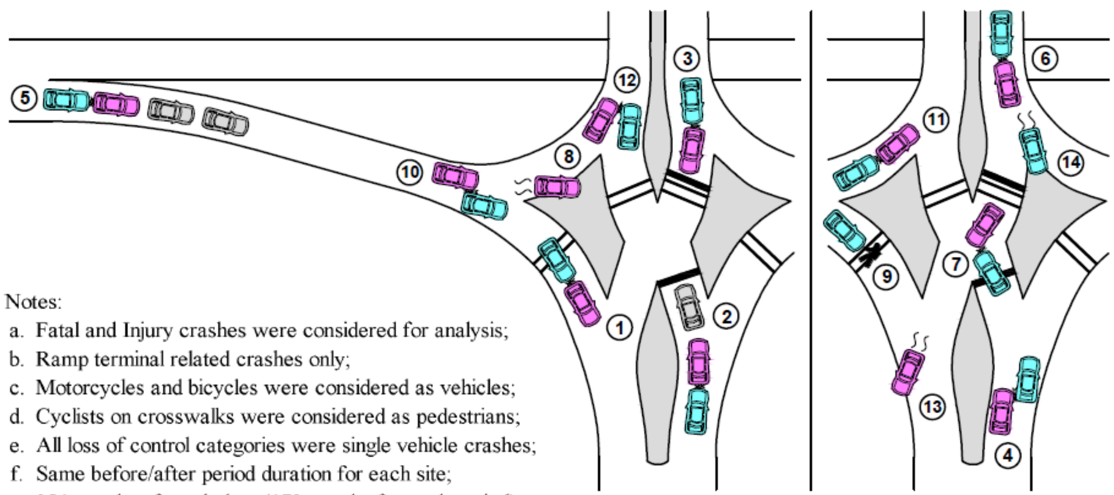
BEFORE PERIOD
Conventional Diamond Interchange (D4 Ramp Terminals)



BEFORE

	Crash Type	#	%
1	Crossing left turn inside crossroad leg	57	34.3
2	Rear end at exit ramp	51	30.7
3	Crossing left turn at exit ramp	14	8.4
4	Rear end inside crossroad through lane	14	8.4
5	Rear end outside crossroad	8	4.8
6	Rear end due to queuing from ramp terminal	8	4.8
7	Rear end inside crossroad left turn lane	5	3.0
8	Sideswipe exit ramp left turn	2	1.2
9	Run off exit ramp right turn	2	1.2
10	Loss of control left turn lane inside crossroad	1	0.6
11	Loss of control outside crossroad bay	1	0.6
12	Run off exit ramp left turn	1	0.6
13	Loss of control exit ramp bay	1	0.6
14	Pedestrian on crosswalk	1	0.6

AFTER PERIOD
Diverging Diamond Interchange (DDI)



AFTER

	Crash type	#	%
1	Rear end at exit ramp right turn	23	37.1
2	Rear end outside crossroad	6	9.7
3	Rear end inside crossroad	5	8.1
4	Sideswipe on crossroad	4	6.5
5	Rear end due to queuing from ramp terminal	4	6.5
6	Wrong direction inside crossroad	3	4.8
7	Crossing at ramp terminal	3	4.8
8	Loss of control left turn at exit ramp bay	3	4.8
9	Pedestrian on crosswalk	2	3.2
10	Sideswipe on exit ramp	2	3.2
11	Rear end exit ramp left turn	2	3.2
12	Sideswipe exit ramp left turn and inside crossroad	2	3.2
13	Run off ramp terminal	2	3.2
14	Loss of control left turn inside crossroad bay	1	1.6

Notes:
a. Fatal and Injury crashes were considered for analysis;
b. Ramp terminal related crashes only;
c. Motorcycles and bicycles were considered as vehicles;
d. Cyclists on crosswalks were considered as pedestrians;
e. All loss of control categories were single vehicle crashes;
f. Same before/after period duration for each site;
g. 356 months of crash data (178 months for each period).

Figure 4.11. Before and After Collision Diagrams for Fatal and Injury Crashes

4.2.4. Discussion Safety evaluations have become an important resource to estimate the effectiveness of treatments. Project planning can be evaluated based on the expected safety contribution of a proven treatment. Therefore, the importance of accurate estimates should not be overlooked since it may mislead to wrong engineering practice. Current practice allows the use of prediction models developed from different parts of the United States and transfer the application of these models to other states. Unfortunately, the accuracy of the prediction through calibrations is not statistically supported. Calibrations are performed based on a factor found from a ratio between observed and predicted observations. Researchers agree that the calibration process needs to be improved and should not depend on a single factor. It should rather be calibrated based on a composite of factors or even a function of different variables (Persaud, et al. 2002; Srinivasan et al., 2013; Hauer, 2015). Despite the improvements that can be made to the calibrations, there is no better prediction model than the one developed with jurisdiction specific data when the resources and data are available. In this research study, throughout the process of developing the prediction model, some differences were found from the models developed in other states.

First, there was a lack of representation of the measures of exposure with general functional forms. Variables such as the AADT provide the most significant information to the model and finding an optimal functional form can increase the accuracy of the model significantly. It was not necessary to explore complicated functions, simple composite (Hoerl's function), linear, or polynomial functions were sufficient.

Second, the flexibility of using a spreadsheet with the solver in Excel provides all necessary tools to develop quality prediction models. It may be considered old fashioned and outdated, but the principles of modeling do not change. Conventional software provides optimization tools in which the modeler has no access to interpretation and assumptions made (likelihood function, local maxima, global maxima, etc.). There is no need of gross statistical estimates to provide the significance of variables; the focus should be in the purpose of the model and not the optimization of parameters. The solver in Excel allows the modeler to develop a close relationship with the data; it allows seeing changes as the model evolves in each step.

Third, goodness of fit measures in conventional statistical practice are good estimates of overall statistical significance of a model, but it is not the most appropriate for safety prediction models. The use of CURE plots (Hauer, 2015) is a major contribution because the model can be improved to meet the purpose of unbiased prediction throughout the range of values of predictor variables. The final CURE plots are the accumulation of work throughout the evolution of the model in which each variable as it is added, provides additional information. This new information is tailored to meet the needs of the model in ranges in which the prediction lacked accuracy, and the CURE plot was an appropriate indicator of the contribution of each variable with its optimized functional form.

Developing safety prediction models is not an easy task (Srinivasan et al., 2013). It requires a significant amount of time, resources, data availability, and statistical knowledge. Additional effort was required for this study since there were crash data issues that needed to be corrected. Close to 13,000 crash reports were reviewed for the model development and safety evaluation. However, the purpose of this study was met—to estimate accurate safety effectiveness of the DDI implementation at the ramp terminal site specific. The DDI replacing a convectional diamond signalized ramp terminal (D4) reduced Fatal and Injury (FI) crashes by 55.0%, decreased Property Damage Only (PDO) crashes by 31.4%, and total (TOT) crashes by 37.5%. The results showed a reduction in crashes in all severity levels.

Finally, the collision diagram analysis revealed that right angle crashes were predominant in the before period at the ramp terminals of a conventional diamond. Specifically, 34.3% of ramp terminal-related fatal and injury crashes occurred due to collisions between the crossing left turn from inside the crossroad and the oncoming through traffic. Due to the crossover design, the DDI completely eliminated this crash type from occurring. One of the potential concerns of a DDI is the possibility of wrong-way crashes. This study found that only 4.8% of all fatal and injury crashes occurring at the ramp terminal of a DDI were wrong-way crashes. The review of remaining crash types found that the DDI exchanged high severity crash types, such as those occurring at a conventional diamond, for lower severity crash types.

4.3. Project Level Safety Evaluation

The before-after safety analysis of DDI designs implemented in Missouri was conducted using data from six DDI sites. Six additional sites were used as comparison sites for comparison group analysis. Table 4.6. contains the following characteristics of the six DDI locations: traffic volume, date open to traffic, the duration of before and after periods, and geometric characteristics.

The duration of before and after periods was determined by taking into account seasonality and construction effects. Initially, five years of crash data were processed for the before period, and the after period duration varied depending on the opening date of the DDI. The after period ranged from 1 year to 4 years for the six sites. In order to avoid the effect of construction activity on crashes, four months of data before the opening date were not used for each site. It was assumed that the work activity at the interchange during 1 year period prior to opening of DDI directly impacted traffic and crashes, and thus crash data was not used from this period. Seasonality was also accounted for by matching the months included in the before period with that of the after period. All six DDI designs replaced conventional diamond interchanges. Pedestrian crossings were implemented in the median or roadside as listed in Table 4.6.

The data necessary for conducting the before-after analysis were obtained from several sources. Aerial photographs were used to measure distances and determine geometric characteristics. The Automated Road Analyzer (ARAN) viewer from the MoDOT Transportation Management System (TMS) database allowed for facilities to be viewed for different years and at specific log miles, which enabled the estimation of short distances such as lane widths and median widths. Computer Aided Design tools were used to measure horizontal curve distances and radii of ramps and freeway facilities. Traffic data was obtained from the MoDOT TMS database for different locations and years within the study period.

Table 4.6. DDI Site Characteristics for Project Level Safety Evaluation

Site Location		RT-13 an d I-44 Springfield, MO	I-270 and Dorsett Rd Maryland Heights, MO	James River Exp. and National Av. Springfield, MO	US 65 and MO248 Branson, MO	I-435 and Front Street Kansas City, MO	Chestnut Exp. and Route 65 Springfield, MO
Opening Date		6/21/2009	10/17/2010	7/12/2010	11/20/2011	11/6/2011	11/10/2012
Periods (Months)	Before	51	35	38	44	44	40
	After	51	35	38	22	22	10
Crossroad	Speed (mph)[1]	40	35	40	35	40	40
	AADT[2]	27082	29275	26891	19842	16087	24513
	Lanes[3]	4	6	6	3	4	4
Freeway	Speed (mph)[1]	60	60	60	65	65	60
	AADT[2]	47734	151923	68179	32604	75276	62207
	Lanes	4	8	4	4	6	6
Configuration Type		Overpass	Underpass	Overpass	Overpass	Underpass	Underpass
Pedestrian Accommodation		Median	Roadside	Median	Median	Median	Roadside
Ramp Terminal Spacing (ft.)		530	480	630	740	420	370
Dist. to Adjacent Street (ft.)		320/685	265/635	530/580	580/1795	530/1955	160/475

Notes: [1] Posted speed limit; [2] AADT of 2013 for reference purpose only; [3] Lanes between ramp terminals.

Crash data was collected for the entire interchange footprint for the study periods reported in Table 4.6. The footprint included the influence areas of all interchange components. For the freeway, crashes were included from the beginning of speed change lanes to end of speed change lanes in both directions of travel. For the crossroad, the influence area included 250 ft. (76 m.) from the ramp terminals, and crashes were collected for the ramp terminals and the crossroad segment in between the terminals.

4.3.1. Crash Severity Analysis The severity of crashes was studied during the before and after periods. The crash data was classified into four severity types: minor injury, disabling injury, fatal, and property damage only (PDO). The crash data was aggregated across all six sites by severity type, and the annual crash frequency was calculated and shown in Figure 4.12. ('All Facilities'). The percentage reductions in crash frequency for all facilities were 57.7% for FI, 26.4% for PDO, and 34.7% for TOT after DDI implementation. There were no fatal crashes at any of the six sites before the installation of DDI. There was one pedestrian fatality that occurred during the after period at one site, but the details of that crash were unknown since it was a hit and run that occurred late at night. Since the fatal crash occurred within the footprint of the DDI, it was still included in the safety evaluation in this study. Figure 4.12. also presents the aggregate crash frequency of all injury crashes denoted by FI (fatal and injury), Property Damage Only (PDO) crashes, and the total number of crashes denoted by TOT.

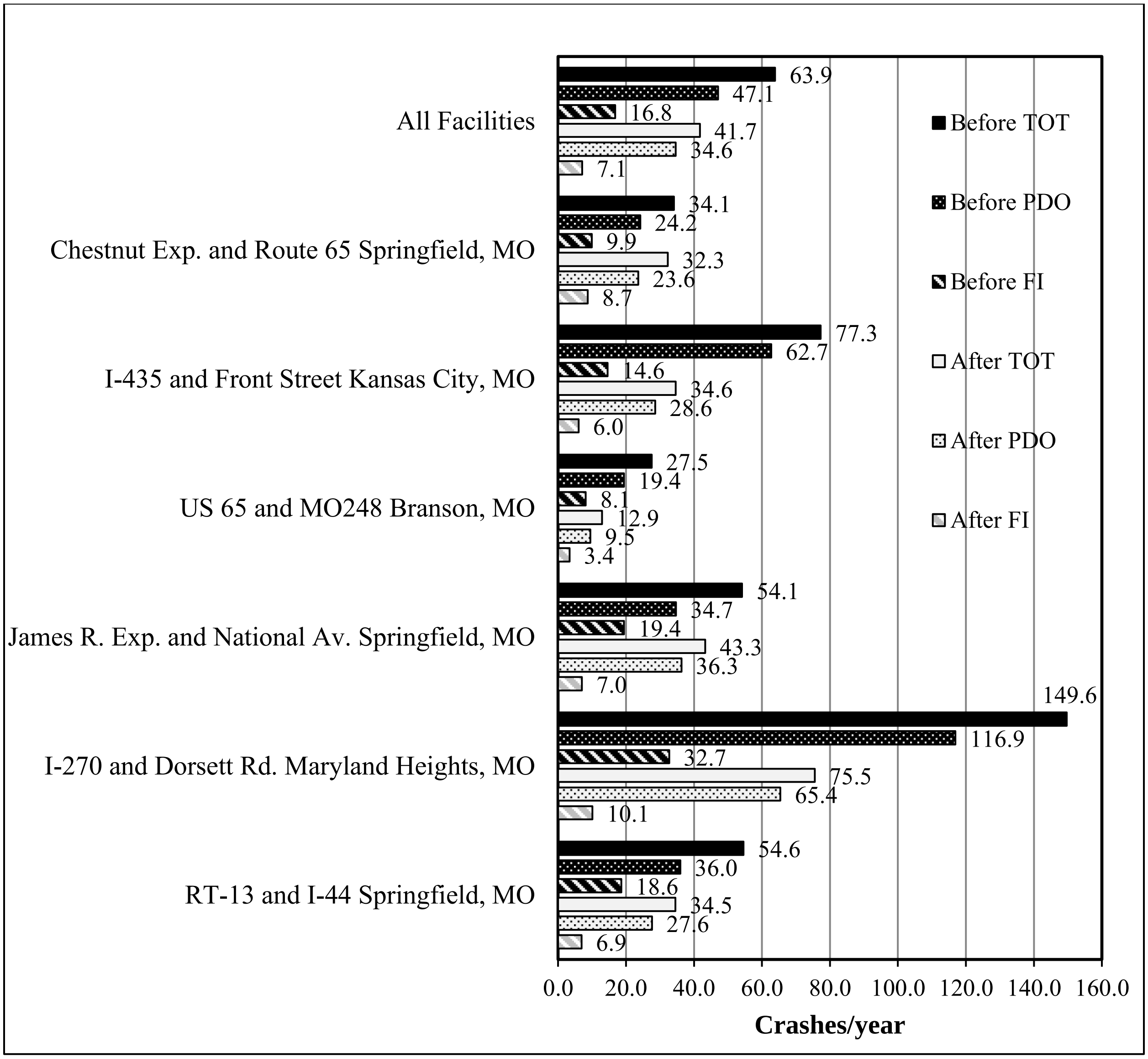


Figure 4.12. Crashes per Year by Severity during the Before and After Period

4.3.2. Project Level Safety Effectiveness Safety effectiveness evaluations use quantitative estimates of how a treatment, project, or a group of projects affected crash frequencies or severities. The effectiveness estimate is useful for future decision-making and policy development (AASHTO, 2010). The observational before and after evaluation methods used in this study compared the anticipated safety of a site without the treatment in the after period to the actual safety of the entity with the treatment in the after period (Hauer, 1997). Three different methods were selected to evaluate the safety effectiveness of the DDI: Naive, Empirical Bayes (EB), and Comparison Group (CG). These methods were selected due to their different approach and use in previous safety research (AASHTO, 2010; Hummer et al., 2010). An interchange was considered as the entire facility or project, by aggregating the various facilities within its footprint. This approach is commonly known as the "Project Level" analysis in the HSM (AASHTO, 2010). The facilities within the interchange footprint include ramp terminals, ramp segments, speed-change lanes, and freeway segment.

4.3.2.1. Results The safety evaluation consisted of Naïve, Empirical Bayes (EB), and Comparison Group (CG).

4.3.2.1.1. Naïve Method The odds ratio and safety effectiveness were computed for three types of crashes – fatal and injury only crashes (FI), property damage only crashes (PDO), and total crashes (TOT). The safety effectiveness results showed a 41.7% (2.9%) reduction in total crash frequency after DDI implementation. The value in the parenthesis denotes the standard error of the estimated safety effectiveness. The FI crash frequency experienced the greatest reduction of 63.2% (4.1%), while the PDO crash frequency decreased by 33.9% (3.7%). All reductions were statistically significant at the 95% confidence level.

As previously discussed, the Naïve method can only estimate the cumulative effect of all changes that have occurred at the treatment sites during the study period. It is however, not possible to ascertain the individual effects of the safety treatment using the Naïve method. Variability of traffic, road user behavior, weather, and many other factors could change over time (Hauer, 1997). Nevertheless, the Naïve method still serves as a good starting point for the safety analysis due to its statistical accuracy, and it has been frequently used in safety evaluations as it provides a precise upper bound (1997).

4.2.3.1.2. Empirical Bayes Method The project-level EB method was applied to conduct the safety evaluation. The safety effectiveness values were calculated for the three correlations previously discussed: independent, fully correlated, and partially correlated. The results for the three crash types are shown in Table 4.7. In Table 4.7., the observed crashes, the EB expected crashes, and the safety effectiveness values for each site are reported in different rows. The standard error values are also reported in parenthesis next to each safety effectiveness value. The right-most column provides the results for the entire treatment group (combination of all six sites).

Since the actual correlation among the interchange facilities is not known, the safety effectiveness values obtained assuming partial correlation can be used for determining the crash modification factors for the DDI (Bonneson et al., 2012). The safety effectiveness values for partial correlation are highlighted in red bold text in Table 4.7. For the entire treatment group ('All Sites' column in Table 4.7.), the percentage reduction in crashes was the greatest for FI crashes, at 62.6% compared to the 35.1% for PDO and 40.8% for TOT crashes. These findings are consistent with the results of the crash severity analysis and the Naïve method. The left turn angle crashes that were predominant in the traditional diamond design (before period) were completely eliminated in the DDI design (after period), which accounts for the reduction in severe crashes.

The EB results for individual sites (see Table 4.7.) showed that the DDI was effective at decreasing the FI crashes at all six sites, although the reduction at the sixth site was not statistically significant at the 95% confidence level. The PDO crashes also decreased at all six sites with the reductions statistically significant at all but sites 3 and 6. The TOT crashes also decreased at all six sites and the reductions were statistically significant at all but site 6. The lack of statistical significance of the EB results for site 6 was due to two reasons. First, the duration of the after period for site 6 was the smallest among all six sites at 10 months. Second, the observed crash frequencies per year before DDI (10 FI, 24 PDO, 34 TOT) and after DDI (9 FI, 24 PDO, 32 TOT) were not considerably different.

Table 4.7. Project-level EB Results

Severity	Correlation	Parameter	RT-13 and I-44 Springfield, MO (Site 1)	I-270 / Dorsett Rd Maryland Heights, MO (Site 2)	James River Exp. National Av. Springfield, MO (Site 3)	US 65 / MO248 Branson, MO (Site 4)	I-435 / Front St. Kansas City, MO (Site 5)	Chestnut Exp. Route 65 Springfield, MO (Site 6)	**All Sites**
FI		Observed Crashes	29	29	22	6	11	7	104
	I[1]	EB Expected Crashes[4]	74	82	61	15	27	9	269
		SE (St.E.)[5]	61.0(8.1)	64.8(7.2)	63.9(8.5)	60.8(16.3)	59.6(12.4)	20.3(30.4)[6]	61.4(4.2)
	C[2]	EB Expected Crashes	83	88	64	16	26	9	286
		SE (St.E.)	65.1(7.5)	67.0(6.8)	65.4(8.4)	63.4(15.4)	57.5(13.3)	18.2(31.4)[6]	63.7(4.1)
	P[3]	EB Expected Crashes	79	85	62	16	27	9	277
		SE (St.E.)	**63.2(7.8)**	**65.9(7.0)**	**64.7(8.4)**	**62.1(15.8)**	**58.6(12.8)**	**19.3(30.9)[6]**	**62.6(4.1)**
PDO		Observed Crashes	116	188	114	17	52	19	506
	I	EB Expected Crashes	164	302	119	37	98	18	739
		SE (St.E.)	29.3(9.0)	37.8(5.6)	4.4(12.5)[6]	53.9(11.7)	47.2(7.7)	-3.0(24.1)[6]	31.6(3.8)
	C	EB Expected Crashes	198	326	126	41	106	20	818
		SE (St.E.)	41.5(7.7)	42.4(5.2)	9.7(12.3)[6]	58.4(10.7)	51.1(7.1)	3.0(22.8)[6]	38.2(3.5)
	P	EB Expected Crashes	181	314	123	39	102	19	779
		SE (St.E.)	**36.0(8.3)**	**40.2(5.4)**	**7.1(12.4)[8]**	**56.3(11.2)**	**49.2(7.4)**	**0.1(23.5)[6]**	**35.1(3.7)**
TOT		Observed Crashes	145	217	136	23	63	26	610
	I	EB Expected Crashes	233	383	163	52	126	27	984
		SE (St.E.)	37.9(6.6)	43.3(4.6)	16.6(9.1)[6]	55.8(9.6)	49.9(6.6)	4.7(19.1)[6]	38.1(3.0)
	C	EB Expected Crashes	274	412	172	57	132	28	1076
		SE (St.E.)	47.2(5.8)	47.4(4.3)	20.8(9.0)	59.7(8.9)	52.3(6.3)	7.8(18.6)[6]	43.4(2.8)
	P	EB Expected Crashes	254	398	167	55	129	28	1030
		SE (St.E.)	**42.9(6.2)**	**45.4(4.5)**	**18.8(9.0)**	**57.8(9.2)**	**51.1(6.4)**	**6.2(18.8)[6]**	**40.8(2.9)**

Notes: [1] I denotes independent correlation, [2] C denotes full correlated, [3] P denotes partial correlation, [4] The expected crash values are rounded (up) to facilitate comparison with observed crash values, [5] SE denotes Safety Effectiveness (%). ST.E denotes Standard Error (%). Negative SE values represent an increase in crashes; [6] Not significant at the 95% confidence level.

4.2.3.1.3. Comparison Group Method For computing the sample odds ratio, a time frame of five years was chosen (2004 to 2009) before any DDI in the treatment group was implemented. The mean, standard error, and the 95% confidence interval of the sample odds ratio were computed. The mean value for FI, PDO, and TOT crashes were 0.97 (0.31 standard error), 1.01 (0.20), and 1.00 (0.22), respectively, all close to 1.0. All 95% confidence intervals also included 1.0. Based on the sample odds ratio results and confidence intervals, the comparison group was deemed to be suitable for comparison with the treatment group following the FHWA guidelines for developing crash modification factors (Gross et al., 2010).

The safety effectiveness was then calculated using the comparison group method previously discussed. The CG method produced safety effectiveness values (and standard errors) of 59.3% (4.8%) reduction in FI crashes, 44.8% (3.3%) reduction in PDO crashes, and 47.9% (2.7%) reduction in TOT crashes, all significant at the 95% confidence level.

The safety effectiveness results obtained from the Naïve, EB, and CG methods are compared in Table 4.8. The safety effectiveness values for each category (FI, PDO, TOT) are shown in different rows for the three methods. Again, the standard error values are reported in parenthesis next to each safety effectiveness value. The overall safety effectiveness values for the entire treatment group are also shown in the right-most column.

The Naïve results for individual sites shown in Table 4.8. revealed that the DDI was effective at decreasing FI crashes at all six sites, PDO crashes at five out of six sites (one site witnessed an increase that was not statistically significant), and total crashes at all six sites. The variation in the safety effectiveness values for FI crashes across the sites was not high. However, PDO and TOT crashes showed higher variation across the six sites. The EB results for individual sites were previously discussed. The CG results for individual sites, shown in Table 4.8., indicated statistically significant reductions in FI crashes for sites 1, 2, and 3 only. Site 6 actually showed an increase in FI crashes, although it was not statistically significant. For the CG method, statistically significant reduction in PDO and TOT crashes were observed for the first five sites. Again, site 6 showed increases in PDO and TOT crashes that were statistically significant. In addition to the short duration of the after period and the lack of considerable variation in the observed crash frequency before and after DDI for site 6, one additional reason may have contributed to the CG results for site 6. Comparison site 6 witnessed higher crash reductions for FI and TOT crashes. For comparison site 6, the observed crash frequencies per year in the before period were: 12 FI, 31 PDO, 42 TOT and in the after period were: 2 FI, 34 PDO, 36 TOT crashes. All three before-after evaluation methods for all sites combined showed that the DDI was effective at improving safety, especially for reducing FI crashes. The results for individual sites also demonstrated that FI, PDO, and TOT crashes decreased at most sites after DDI implementation.

Table 4.8. Safety Effectiveness Results by Site for the Three Methods

Severity	Method	RT-13 and I-44 Springfield, MO (Site 1)	I-270 and Dorsett Rd. Maryland Heights, MO (Site 2)	James River Exp. and National Av. Springfield, MO (Site 3)	US 65 and MO248 Branson, MO (Site 4)	I-435 and Front Street Kansas City, MO (Site 5)	Chestnut Exp. And Route 65 Springfield, MO (Site 6)	**All Sites**
FI	Naïve	63.3 (7.9)	69.5 (6.4)	64.5 (8.7)	60.0 (17.3)	59.3 (13.2)	15.1 (34.4) [1]	**63.2 (4.1)**
	EB	63.2 (7.8)	65.9 (7.0)	64.7 (8.4)	62.1 (15.8)	58.6 (12.8)	19.3 (30.9) [1]	**62.6 (4.1)**
	CG	69.8 (6.8)	70.8 (6.4)	69.0 (7.9)	36.1 (29.8)[1]	20.3 (27.3)[1]	-204.5 (143.2) [1]	**59.3 (4.8)**
PDO	Naïve	23.7 (9.4)	44.2 (5.1)	-3.6 (13.8)[1]	51.5 (13.0)	54.6 (6.9)	3.7 (24.3) [1]	**33.9 (3.7)**
	EB	36.0 (8.3)	40.2 (5.4)	7.1 (12.4)[1]	56.3 (11.2)	49.2 (7.4)	0.1 (23.5) [1]	**35.1 (3.7)**
	CG	57.7 (5.5)	57.3 (4.1)	32.3 (9.4)	39.3 (17.0)	26.9 (11.7)	-191.7 (82.4)	**44.8 (3.3)**
TOT	Naïve	37.0 (6.7)	49.7 (4.2)	20.5 (9.1)	53.6 (10.6)	55.4 (6.2)	6.2 (20.3) [1]	**41.7 (2.9)**
	EB	42.9 (6.2)	45.4 (4.5)	18.8 (9.0)	57.8 (9.2)	51.1 (6.4)	6.2 (18.8) [1]	**40.8 (2.9)**
	CG	60.2 (4.4)	59.4 (3.6)	43.9 (6.7)	38.8 (14.7)	25.1 (10.9)	-191.6 (70.5)	**47.9 (2.7)**

Notes: Standard error values are shown in the parenthesis next to the safety effectiveness; Negative values represent the percentage increase in crashes; [1] Not significant at the 95% confidence level.

4.4. Safety Effect of Diverging Diamond Interchanges on Adjacent Roadway Facilities

Two types of facilities adjacent to DDI were examined. First, speed change lanes adjacent to the entrance and exit ramps of DDI were studied. In Figure 4.13., the highlighted areas on the freeway show the different types of speed change lane facilities. Second, major signalized intersections adjacent to the DDI ramp terminal were studied. Minor intersections with outer roads, driveways, and other service roads were not included in the analysis. Figure 4.14. illustrates a DDI location with the adjacent major four-leg signalized intersection. This study utilized data from 11 DDIs during an accumulated time period of 76.5 years—43 years before and 33.5 years after the DDI implementation. Thirty-two speed change lane facilities and twelve major signalized intersections were investigated. A total of 4,073 crash reports were reviewed manually to accurately assign crashes to the appropriate facilities. These facilities included: freeway, ramp terminals, crossroad segment, ramp segment, speed change lanes, and adjacent intersections. The Empirical Bayes method was used to quantify the safety effects of DDI on speed change lanes and adjacent intersections.

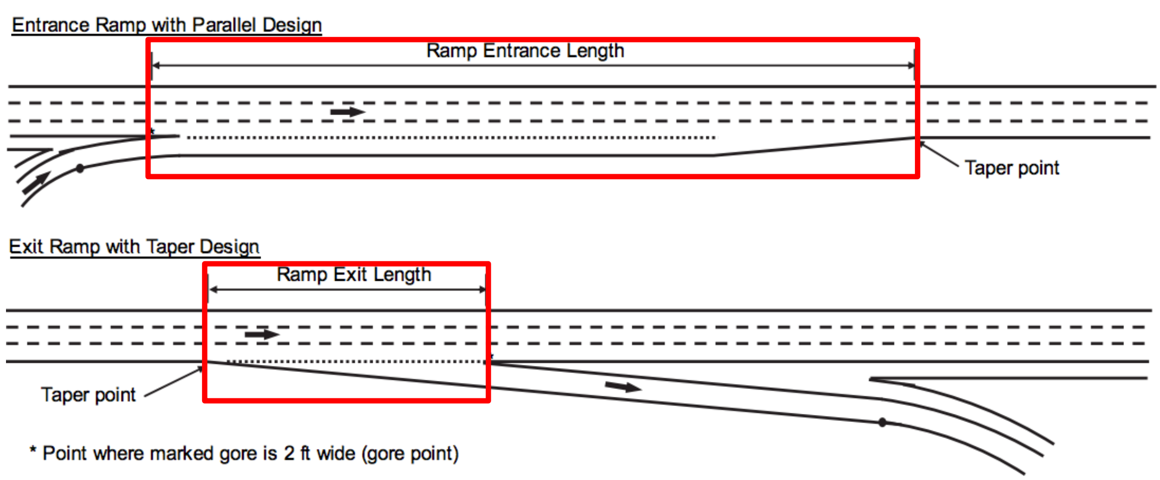


Figure 4.13. Example Entrance and Exit Speed Change Lanes (AASHTO, 2010)

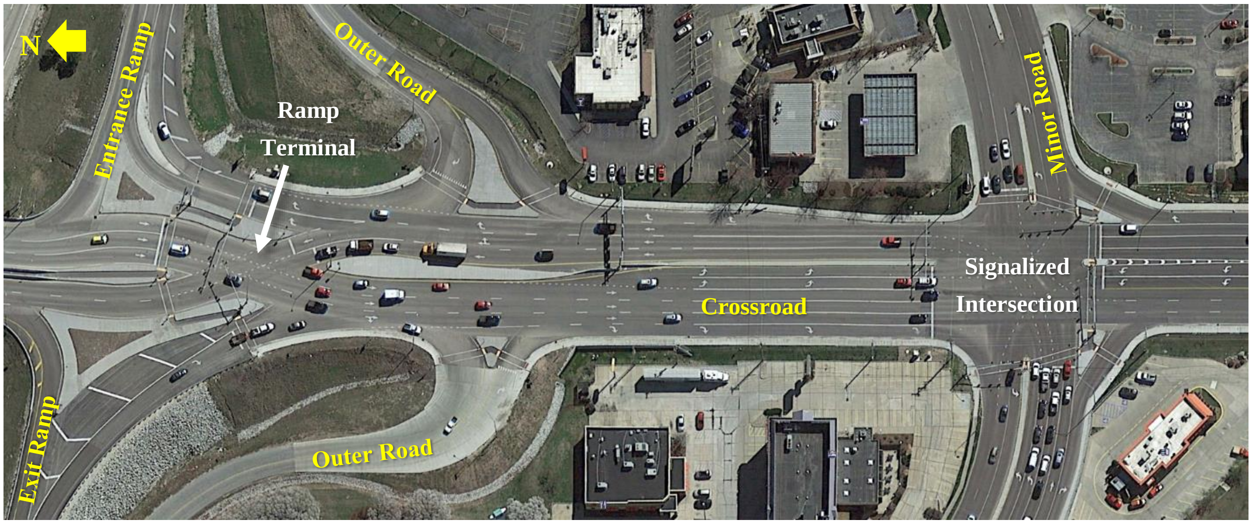


Figure 4.14. Adjacent Signalized Intersection at Stadium Blvd./ IS-70, Columbia, MO

4.4.1. Site Selection A total of 11 interchange facilities where the DDI was implemented were considered. Table 4.9. presents the list of facilities used in the analysis. A few speed change lanes did not meet the site selection criteria previously explained, as additional lanes were added or the taper was extended during DDI implementation to facilitate merging or diverging movements. The Annual Average Daily Traffic (AADT) values were collected for all years, before and after periods, for all sites. Table 4.9. shows 2015 AADTs only for reference purposes (to illustrate how the AADTs varied across the sites).

In Table 4.9., the number of lanes over the freeways were 4 and 6 (total through lanes both directions). The crossroad lanes varied from 3, 4, 5, and 6 lanes (total number of lanes in both directions between interchange ramp terminals). There were some DDIs with an unbalanced number of crossroad lanes such as Site 3: MO-248 and US-65 with 2 lanes in one direction and 1 lane in the opposite direction. The same was observed at Site 6: Kansas Expy. and US-60 where one direction had 3 lanes and the opposite direction had only 2 lanes. The number of lanes recorded at the ramps were the number of exit or entry lanes at the freeway.

Table 4.9. Sites Description for Adjacent Facilities Analysis

Site Location	Code[1]	AADT (vpd)[2]	Lanes[3]	Speed (mph)[4]	Distance	Other
MO-13 and I-44, Springfield, MO	1	25,620/51,604	4/4	40/60	-	
I-44 East and Exit 77 to MO-13	SDS	51,604/3,809	2/1	60/40	440	**Open**
I-44 West and Exit 77 to MO-13	SDN	51,604/8,010	2/1	60/40	740	6/21/2009
MO-13 South to I-44 West	SMN	3,860/51,604	1/2	NA/60	630	**Before/After**
MO-13 North to I-44 East	SMS	7,412/51,604	1/2	NA/60	745	4/6 years
Norton Rd. and MO-13	IN	3,177/25,620	4/4	35/40	710	
National Av. and US-60, Springfield,	2	32,320/56,870	6/4	40/60	-	**Open**
US-60 West and Exit to National Ave.	SDN	56,870/6,725	2/1	60/40	810	7/12/2010
National Av. North to US-60 East	SMS	5,229/56,870	1/2	NA/60	755	**Before/After**
Republic Rd. and National Ave.	IS	10,018/32,320	4/4	35/40	1,100	4/5 years
Primrose St. and National Ave.	IN	13,273/32,320	4/4	40/40	1,480	
MO-248 and US-65, Branson, MO	3	31,640/43,017	3/4	35/65	-	11/20/2011
US-65 North and Exit to MO-248	SDE	43,017/2,541	2/1	65/40	810	4/4 years
MO-248 Southeast to US-65 South	SMW	3,129/43,017	1/2	NA/65	770	
Front St. and I-435, Kansas City, MO	4	25,343/79,945	4/6	40/65	-	
I-435 North and Exit 57 to Front St.	SDE	79,945/8,722	3/1	65/40	570	**Open**
I-435 South and Exit 57 to Front St.	SDW	79,945/5,951	3/1	65/40	870	11/6/2011
Front St. West to I-435 North	SME	5,698/79,945	1/3	NA/65	910	**Before/After**
Corrington Ave. and Front St.	IE	2,238/25,343	3/4	NA/40	510	4/4 years
Universal Av. And Front St.	IW	1,064/25,343	2/4	35/40	1,840	
Chestnut Expy. and US-65,	5	23,430/65,170	4/6	40/60	-	**Open**
US-65 and Exit to Chestnut Expy.	SDW	65,170/4,939	3/1	60/40	900	11/10/2012
Chestnut Expy. West to US-65 North	SME	4,350/65,170	1/3	NA/60	730	**Before/After**
Chestnut Expy. East to US-65 South	SMW	7,451/65,170	2/3	NA/60	1,580	4/3 years
Belcrest Ave. and Chestnut Ave.	IW	5,810/23,430	2/4	30/40	2,220	
Kansas Expy. and US-60, Springfield,	6	26,125/35,394	5/4	45/60	-	**Open**
US-60 East and Exit to Kansas Expy.	SDS	35,394/2,261	2/1	60/40	780	8/18/2013
Kansas Expy. South to US-60 West	SMN	3,238/35,394	1/2	NA/60	830	**Before/After**
Republic Rd. and Kansas Expy.	IS	15,077/26,125	4/4	25/45	1,160	4/2 Years
Chesterfield Blvd. and Kansas Expy.	IN	3,773/26,125	4/4	30/45	1,000	
Columbia St. and US-67, Farmington,	7	11,780/12,621	4/4	40/60	-	
US-67 North and Exit to Columbia St.	SDE	12,621/1,339	2/1	60/NA	440	**Open**
US-67 South and Exit to Columbia St.	SDW	12,621/4,168	2/1	60/NA	420	9/5/2012
Columbia St. West to US-67 North	SME	4,070/12,621	1/2	NA/60	650	**Before/After**
Columbia St. East to US-67 South	SMW	1,471/12,621	1/2	NA/60	700	4/3 years
Westmount Dr. and Columbia St.	IE	2,924/11,780	2/2	25/40	850	
Woods Ch. Rd and IS-70, Blue	8	99,131/16,994	5/6	40/65	-	**Open**
IS-70 East and Exit 18 to Woods Ch. Rd.	SDS	16,994/8,592	3/1	65/40	860	
IS-70 West and Exit 18 to Woods Ch.	SDN	16,994/2,147	3/1	65/40	900	9/26/2013
Woods Chapel Rd. North to IS-70 East	SMS	2,079/16,994	1/3	NA/65	860	
Woods Chapel Rd. North to IS-70 West	SMN	8,491/16,994	1/3	NA/65	850	**Before/After**
Stadium Blvd. and IS-70, Columbia,	9	38,834/47,186	6/4	40/60	-	**Open**
US-70 East and Exit 124 to Stadium	SDS	47,186/3,971	2/1	60/NA	250	10/14/2013
Stadium Blvd. to US-70 West	SMN	3,855/47,186	1/2	NA/60	670	**Before/After**
Bernadette Dr. and Stadium Blvd.	IS	12,633/38,834	6/6	30/40	850	4/2 years
Business Loop 70 W and Stadium Blvd.	IN	4,430/38,834	2/4	35/40	560	
T. Springs Pkwy and I-29, Kansas	10	3,936/59,829	5/6	40/70	-	7/12/2014
I-29 South and Exit 10 to Tiffany Springs	SDW	59,829/7,160	3/1	70/40	800	3/1.5 years
Tiffany Springs Pkwy. to I-20 North	SME	2,027/59,829	1/3	NA/70	700	
Battlefield Rd. and US-65, Springfield,	11	22,812/66,619	6/6	40/60	-	
US-65 North and Exit to Battlefield Rd.	SDE	66,619/4,491	3/1	60/40	480	**Open**
US-65 South and Exit to Battlefield Rd.	SDW	66,619/5,437	3/1	60/40	420	2/14/2015
Battlefield Rd. to US-65 North	SME	6,321/66,619	1/3	NA/60	800	**Before/After**
Battlefield Rd. to US-65 South	SMW	5,661/66,619	1/3	NA/60	790	4/1 years
Moulder Ave. and Battlefield Rd.	IW	1,916/22,812	2/4	35/40	820	

Notes: [1]Intersection facility notation; [2]AADT in vehicles per day (vpd) from 2015 for reference purposes only; [3]Total number of lanes; [4]Posted speed limit in miles per hour (mph); [5]Distance gore to taper for speed change lanes and from the center of the DDI ramp terminal to the center of the adjacent intersection.

All intersecting ramp lanes at the speed change lanes had only one lane. For intersections, the total number of through lanes were reported without counting any exclusive right or left turn lanes. The distances reported in Table 4.9. are the gore to taper distance for speed change lanes and the distance from the center of the DDI ramp terminal to the center of the adjacent intersection. The major signalized intersections were located between 500 to 2,220 feet away from the center of the DDI ramp terminal.

4.4.2. Safety Effect on Adjacent Facilities The safety evaluation consisted of estimating the change in safety over adjacent facilities due to the DDI implementation using the Empirical Bayes analysis. Two prediction methods were utilized within the Empirical Bayes analysis: Sample Moments method for speed change lanes and the HSM prediction models for signalized intersections (AASHTO, 2010; Hauer, 1997).

4.4.2.1. Results The results of the Empirical Bayes analysis for speed change lanes are presented in Table 4.10. The observed crashes, expected crashes, and safety effectiveness values are presented for each site for property damage only (PDO) and total (TOT) crashes. The number of fatal and injury crashes on speed change lanes were too few and were not modeled in this study.

4.4.2.1.1. Speed Change Lanes As previously discussed, the speed change lanes experienced few crashes. Table 4.10. shows that after the DDI implementation, exit speed change lanes experienced 56 PDO and 73 TOT crashes over a combined period of 49.5 years across all sites i.e. about 1.5 total crashes per year per site. The safety effectiveness values varied across the exit speed change lane sites, with 11 sites witnessing an increase and 5 sites witnessing a decrease in TOT crashes; the majority of the changes were not statistically significant. The only statistically significant change was experienced at IS-70 E/Exit 18 to Woods Ch. Rd. (Site 8, SMS), which experienced a 53.3% reduction in total crashes after DDI implementation.

Table 4.10. Safety Effectiveness Results for Speed Change Lanes

	Site Location	DDI	Code	PDO[1]			TOT[2]		
				Obs.[3]	Exp.[4]	SE%(St. E. %)[5]	Obs.	Exp.	SE%(St. E. %)
Exit Speed Change Lanes	I-44 E/Exit 77 to MO-13	1	SDS	3	1	-132.0%*(135.3%)*	6	4	-48.0%*(77.5%)*
	I-44 W/Exit 77 to MO-13	1	SDN	10	10	-4.2%*(43.9%)*	13	11	-17.5%*(45.4%)*
	US-60 W/Exit to National Ave.	2	SDN	2	2	3.3%*(64.8%)*	3	2	-24.8%*(76.4%)*
	US-65 N/Exit to MO-248	3	SDE	2	3	38.2%*(42.9%)*	3	3	13.9%*(52.6%)*
	I-435 N/Exit 57 to Front St.	4	SDE	10	10	-4.6%*(41.3%)*	16	10	-52.5%*(52.5%)*
	I-435 S/Exit 57 to Front St.	4	SDW	8	11	28.3%*(29.7%)*	8	13	37.7%*(25.4%)*
	US-65/Exit to Chestnut Expy.	5	SDW	3	1	-142.0%*(147.5%)*	4	3	-25.2%*(69.7%)*
	US-60 E/Exit to Kansas Expy.	6	SDS	1	1	-21.0%*(99.5%)*	1	1	-4.0%*(87.8%)*
	US-67 N/Exit to Columbia St.	7	SDE	3	2	-63.7%*(100.3%)*	3	2	-48.0%*(90.6%)*
	US-67 S/Exit to Columbia St.	7	SDW	0	1	100.0%*(0.0%)*	1	1	-16.6%*(89.9%)*
	IS-70 E/Exit 18 to Woods Ch. Rd.	8	SDS	3	6	46.2%*(32.0%)*	3	6	53.3%*(27.7%)*[6]
	IS-70 W/Exit 18 to Woods Ch. Rd.	8	SDN	6	3	-87.6%*(90.0%)*	7	4	-71.6%*(76.5%)*
	US-70 E/Exit 124 to Stadium Blvd.	9	SDS	1	1	-21.0%*(99.5%)*	1	1	26.1%*(65.2%)*
	I-29 S/Exit 10 to Tiffany S. Pkwy.	10	SDW	3	2	-85.5%*(113.3%)*	3	2	-40.8%*(85.6%)*
	US-65 N/Exit to Battlefield Rd.	11	SDE	1	0	-142.0%*(199.0%)*	1	1	-48.0%*(130.4%)*
	US-65 S/Exit to Battlefield Rd.	11	SDW	0	1	100.0%*(0.0%)*	0	1	100.0%*(0.0%)*
	All Exit Speed Change Lanes		**16**	**56**	**54**	**-2.8%*(18.6%)***	**73**	**67**	**-7.9%*(17.2%)***
Entrance Speed Change Lanes	MO-13 S to I-44 W	1	SMN	6	7	13.8%*(42.1%)*	10	8	-32.6%*(57.7%)*
	MO-13 N to I-44 E	1	SMS	9	9	0.0%*(42.6%)*	10	13	24.6%*(30.0%)*
	S National Av. N to US-60 E	2	SMS	9	7	-35.4%*(58.7%)*	13	9	-42.3%*(54.9%)*
	MO-248 S to US-65 S	3	SMW	4	2	-108.4%*(119.6%)*	4	2	-101.1%*(115.7%)*
	Front St. W to I-435 N	4	SME	5	4	-26.3%*(65.7%)*	9	7	-37.5%*(58.7%)*
	Chestnut Expy. W to US-65 N	5	SME	1	2	48.8%*(45.7%)*	1	2	51.6%*(43.0%)*
	Chestnut Expy. E to US-65 S	5	SMW	1	2	48.8%*(45.7%)*	3	2	-45.5%*(89.0%)*
	Kansas Expy. S to US-60 W	6	SMN	0	1	100.0%*(0.0%)*	0	1	100.0%*(0.0%)*
	Columbia St. W to US-67 N	7	SME	0	1	100.0%*(0.0%)*	0	1	100.0%*(0.0%)*
	Columbia St. E to US-67 S	7	SMW	0	1	100.0%*(0.0%)*	0	1	100.0%*(0.0%)*
	Woods Ch. Rd. N to IS-70 E	8	SMS	2	4	54.2%*(32.4%)*[6]	3	5	37.5%*(37.3%)*
	Woods Ch. Rd. N to IS-70 W	8	SMN	8	5	-70.3%*(70.6%)*	10	7	-34.2%*(49.6%)*
	Stadium Blvd. to US-70 W	9	SMN	3	1	-130.8%*(141.1%)*	3	3	-19.3%*(72.2%)*
	Tiffany S. Pkwy. to I-20 N	10	SME	0	1	100.0%*(0.0%)*	0	2	100.0%*(0.0%)*
	Battlefield Rd. to US-65 N	11	SME	1	1	-1.1%*(93.4%)*	1	1	6.4%*(86.4%)*
	Battlefield Rd. to US-65 S	11	SMW	0	1	100.0%*(0.0%)*	1	1	6.4%*(86.4%)*
	All Entrance Speed Change Lanes		**16**	**49**	**49**	**0.7%*(18.6%)***	**68**	**64**	**-4.2%*(17.1%)***

Notes: [1]Property Damage Only crashes; [2]Total crashes; [3]Observed crashes; [4]Expected number of crashes estimated with the Empirical Bayes method; [5]Safety effectiveness % *(Standard Error %)*; [6]Significant at the 90% confidence level; Negative values represent increase in crashes.

The Empirical Bayes results of all exit speed change lanes were not statistically significant for PDO crashes. Thus, there was no evidence showing the DDI design had any effect, positive or negative, on the crashes occurring at the exit speed change lanes.

The observed crash frequencies at entrance speed change lanes were even smaller than those observed at exit speed change lanes. While nine sites witnessed a decrease in total crashes and seven sites witnessed an increase, none of these changes were statistically significant. The overall safety effectiveness values for all sites combined were also not statistically significant for PDO or TOT crashes. Thus, the DDI did not have any effect on the crash frequency of entrance speed change lanes.

4.4.2.1.2. Adjacent Intersections The results of the Empirical Bayes analysis for intersections are presented in Table 4.11. The results are shown for fatal and injury (FI), property damage only (PDO) and total (TOT) crashes. For FI crashes, nine of the twelve sites witnessed a decrease in crashes after DDI (positive safety effectiveness values in Table 3); however, only two of those reductions were statistically significant only at the 90% confidence level. Of the three sites that experienced an increase in FI crashes, none were statistically significant. Similarly, seven of the twelve sites witnessed a decrease in PDO crashes after DDI; four of which were statistically significant. Two sites experienced a statistically significant increase in PDO crashes after DDI. For TOT crashes, eight of the twelve sites witnessed a decrease after DDI; half of them being statistically significant. Two sites experienced a statistically significant increase in TOT crashes after DDI. When all sites were analyzed together as a whole, the FI crashes decreased by 6.5% (not statistically significant), while PDO and TOT crashes increased by 19.5% and 12.0%, respectively (both statistically significant only at the 90% confidence level).

Table 4.11. Safety Effectiveness Results for Adjacent Signalized Intersections

Site Location	DDI	Code	FI[1]			PDO[2]			TOT[3]		
			Obs.[4]	Exp.[5]	SE% *(St. E. %)*[6]	Obs.	Exp.	SE% *(St. E. %)*	Obs.	Exp.	SE% *(St. E. %)*
W Norton Rd. and MO-13	1	IN	33	18	-82.5% *(51.5%)*	106	66	-61.8% *(26.3%)*[7]	139	84	-66.2% *(24.0%)*[7]
E Republic Rd. and S National Ave.	2	IS	17	23	26.0% *(22.6%)*	63	57	-10.7% *(19.8%)*	80	80	-0.2% *(15.7%)*
E Primrose St. and S National Ave.	2	IN	28	20	-41.1% *(41.0%)*	91	33	-175.1% *(58.0%)*[7]	119	53	-124.8% *(38.6%)*[7]
N Corrington Ave. and Front St.	4	IE	3	4	22.9% *(47.0%)*	15	17	12.9% *(28.3%)*	18	21	14.8% *(25.1%)*
Universal Av. and Front St.	4	IW	4	2	-69.2% *(96.9%)*	7	3	-152.9% *(129.7%)*	11	5	-114.3% *(85.0%)*
N Belcrest Ave. and E Chestnut Ave.	5	IW	6	10	40.7% *(26.9%)*	11	12	5.4% *(34.9%)*	17	22	21.8% *(22.9%)*
W Republic Rd. and S Kansas Expy.	6	IS	1	4	71.9% *(26.4%)*[7]	21	19	-11.5% *(27.6%)*	28	29	4.5% *(20.4%)*
W Chesterfield Blvd. and S Kansas Expy.	6	IN	6	18	66.0% *(14.7%)*[7]	6	11	45.0% *(23.9%)*[7]	9	16	44.7% *(19.9%)*[7]
Westmount Dr. and W Columbia St.	7	IE	4	4	11.1% *(48.2%)*	12	19	37.5% *(20.6%)*[7]	13	23	42.9% *(17.9%)*[7]
Bernadette Dr. and N Stadium Blvd.	9	IS	3	3	13.4% *(51.6%)*	9	32	71.8% *(10.0%)*[7]	15	49	69.7% *(8.4%)*[7]
B. Loop 70 W and N Stadium Blvd.	9	IN	7	10	33.1% *(26.9%)*	5	8	34.2% *(32.2%)*	9	12	25.6% *(28.3%)*
S Moulder Ave. and E Battlefield Rd.	11	IW	3	5	44.1% *(33.2%)*	1	14	92.8% *(7.2%)*[7]	4	17	76.9% *(11.8%)*[7]
All Sites		**12**	**115**	**122**	**6.5% *(11.5%)***	**347**	**289**	**-19.5% *(9.1%)*[7]**	**462**	**412**	**-12.0% *(7.3%)*[7]**

Notes: [1]Fatal and Injury crashes, [2]Property Damage Only (PDO) crashes; [3]Total (TOT) crashes; [4]Observed crashes; [5]Expected number of crashes estimated with the Empirical Bayes method; [6]Safety effectiveness % *(Standard Error %)*; [7]Significant at the 90% confidence level; Negative values represent increase in crashes.

Of all intersection sites, the intersection located at Primrose St. and National Ave. (Row 3 in Table 4.12.) experienced the highest increase in crashes. Possible contributing factors were explored to examine this increase. The AADT on National Ave. increased from 28,477 (vpd) in 2009 to 32,320 (vpd) in 2015. Also, an upstream unsignalized intersection at Bradford Pkwy. and National Ave. experienced a major change after DDI as illustrated in Figure 4.15. in bright red—the left turn movements from National Ave. to Bradford Pkwy were prohibited by a newly built non-traversable median from the DDI ramp terminal to Primrose St. Also, an underpass was implemented connecting establishments along both sides of National Ave. If the Primrose St.

intersection was not included in the safety evaluation, due to these unusual changes in access and traffic volumes, the overall safety effectiveness values indicate a 0.4% reduction in PDO (with a standard error of 8.4%) and a 4.6% reduction in TOT crashes (with a standard error of 6.8%) after DDI implementation; although neither values were statistically significant at the 90% confidence level.

a) Before DDI (06/09/2006 dated image)

b) After DDI (03/14/2015 dated image)
Figure 4.15. Aerial Images at Primrose St. and National Ave., Springfield, MO

4.5. Conclusions

In this chapter, the results of safety evaluation of Diverging Diamond Interchanges in Missouri were presented. Missouri was ideal for such a study because it was the first state to implement DDIs in the US, thus significant after treatment data was available. This study used crash data from six sites in Missouri to conduct a comprehensive before-after evaluation of the DDI. The safety evaluation consisted of three types of observational before-after evaluation methods: Naïve, Empirical Bayes (EB), and Comparison Group (CG). Collision diagram analysis was also conducted to determine the differences in crash types at a DDI and a conventional diamond.

All three before-after safety evaluation methods produced consistent results. The DDI design replacing a conventional diamond decreased crash frequency for all severities. The most significant crash reduction was observed for fatal and injury crashes – 63.2% (Naïve), 62.6% (EB) and, 59.3% (CG). Property damage only crashes reduced by 33.9% (Naïve), 35.1% (EB), and 44.8% (CG). The total crash frequency also decreased by 41.7% (Naïve), 47.9% (EB), and 52.9% (CG). The safety effectiveness results for the six sites also demonstrated that FI, PDO, and TOT crashes decreased at most sites after DDI implementation. This study documented the safety benefits of DDI, which complements the existing knowledge on the operational benefits of DDI. In future research, data from DDIs in different states may be jointly analyzed to develop a nation-level crash modification factor for the DDI.

Little is known about how the Diverging Diamond Interchange affects safety of facilities adjacent to the interchange. This dissertation examined the safety of two adjacent facilities: the speed change lanes and major signalized intersections. Analysis of 32 speed change lane facilities from 11 DDI sites in Missouri found no evidence of any changes in crash frequency at these facilities post DDI implementation.

Signalized intersections adjacent to the DDI ramp terminals that did not experience any geometric changes due to DDI implementation were also studied. The individual site results were mixed, with several sites not witnessing statistically significant changes post DDI implementation. When all sites were combined, statistically significant increases in PDO and total crashes resulted post DDI implementation but only at the 90% confidence level. The FI crashes decreased, though not statistically significant. One intersection site experienced unusually higher number of crashes as compared to other sites. Further investigation of this site revealed some unique access management changes that occurred between the DDI ramp terminal and the signalized intersection (although the intersection itself remained unchanged). The safety analysis when repeated without this intersection site did not reveal any statistically significant changes in FI, PDO, or total crashes due to DDI implementation. The conclusion from this study is that there is no strong evidence that DDIs impacted the safety of adjacent roadway facilities, either positively or negatively. In future research, additional sites from Missouri and other states should shed further light on DDI's effect on adjacent signalized intersections.

5. SAFETY EVALUATION OF J-TURN INTERSECTION

The safety evaluation of the J-turn intersections consisted of the safety effectiveness analysis at the project level, collision diagram analysis, and the evaluation of a safe U-turn spacing.

5.1. Project Level Safety Evaluation

5.1.1. Safety Effectiveness A safety evaluation was performed by analyzing the crashes occurring before and after the implementation of the J-turn design. The safety evaluation was performed using two methods. The first method compared the crash frequency for different severity levels and types for the before and after period. Nine intersections in Missouri with J-turns were considered, but only five had adequate data after implementation to be included in the safety evaluation. Table 5.1. shows the characteristics of each J-turn used in the study. All sites had high speed limits of 65 or 70 mph (105 or 113 kph).

Table 5.1. Characteristics of the J-turn Sites Included in the Safety Evaluation

J-Turn Location	Average AADT After (Before)		Type	Speed Limit	Before period	After period
	Major	Minor	3 or 4 leg	Mph	Years	Years
US 63 and Deer Park Rd. Columbia, Boone County, MO	26470 (25807)	987 (1059)	4-leg	70	2.50	1.25
US 54 and Honey Creek Rd. Jefferson City, Cole, MO	18922 (18848)	505 (508)	4-leg	65	2.25	2.25
US 54 and Route E Henley, Cole, MO	15591 (15541)	1340 (1389)	4-leg	65	2.25	2.25
MO 13 and NE 364 Rd. Osceola, St. Clair, MO	10630 (10630)	447 (447)	4-leg	65	3.00	3.00
Route M and Old Lemay Ferry Rd. Imperial, Jefferson, MO	10326 (10326)	434 (434)	3-leg	65	3.50	3.50

The annual crash frequency was computed for each site by dividing the total number of crashes by the duration. Crash frequencies were computed for four severity levels: 1) Property Damage Only (PDO), 2) Minor Injury (MI), 3) Disabling Injury (DI), and 4) Fatality (F). The effects of the J-turn on specific crash types were also analyzed. Specifically, the following intersection-related crash types were analyzed: 1) Right Angle, 2) Right Turn, 3) Right Turn Right Angle, 4) Left Turn, 5) Left Turn Right Angle, 6) Rear End, 7) Side Swipe, and 8) Passing. The aforementioned eight crash types were the most relevant to J-turn and TWSC intersections. The angle crash types were included since they are common occurrences at at-grade intersections and are typically of high severity. The J-turn design introduces new weaving maneuvers between the minor road and the U-turn. Thus, rear end, sideswipe, and passing types of crashes were also analyzed.

The second method, Empirical Bayes (EB), was more statistically rigorous and has been used in previous studies to evaluate the safety effectiveness of alternative intersection designs (Inman and Hass, 2012; Hummer et al., 2010; Persaud et al., 2001). The EB method is also recommended by the Highway Safety Manual (HSM) for conducting safety evaluations. The EB method was used to compute the safety effectiveness of the J-turn design replacing a two-way stop controlled intersection. The method uses Safety Performance Functions (SPF) to predict crashes with specified base conditions for a facility type. Crash Modification Factors (CMF) are used to adjust the base SPF predictions to the site geometric, signal, and traffic conditions. The analysis was conducted at the project level, meaning that the entire footprint of the treatment was covered. The project-level EB used in this research is different from the site-specific analysis performed by Hummer et al. (2010). In the site-specific analysis conducted by Hummer et al. (2010), only intersection-related crashes occurring at the main intersection were included in the safety evaluation. However, a project-level analysis considered the entire footprint including the main intersection, the two U-turns, and the segments between them. Since multiple facilities (intersections and segments) are included in the analysis, a correlation among the facilities was incorporated. According to Hauer et al (Hauer et al., 2012), there are two bounds of correlation: perfectly correlated and independent facilities. The weight adjustment factors for the two bounds of correlation were computed. For partial correlation conditions, averaging the expected crash estimate of the perfect correlation and independent conditions is recommended (AASHTO, 2010).

The analysis period was adjusted by removing the actual construction period for each J-turn and by matching the seasons (months) exactly in the before and after period. The durations of before and after period are reported in Table 5.1. The predictions for each site were performed for fatal and injury (FI) and total (TOT) crashes, as those were the only two currently available SPFs in the HSM. To accurately predict crashes using the HSM functions, the functions must be calibrated for Missouri conditions. Calibration factors for FI and TOT crashes were developed for rural multilane intersections and rural multilane divided segments. The sampling criteria recommended by the HSM were followed to randomly generate samples of intersections and segments for calibration. Additional information on calibration of different facilities for Missouri can be found in Sun et al. (2013). The calibration factors for rural multilane four-leg intersection with minor road stop control were (sample size of 66 intersections): 0.64 for FI, 0.73 for TOT crashes, rural multilane three-leg intersection with minor road stop control were (sample size of 71 intersections): 0.85 for FI, 1.08 for TOT crashes, and rural multilane divided segments were (sample size of 37 segments): 0.59 for FI, 0.98 for TOT crashes.

5.1.1.1. Results Two methods were used to compare before and after crash frequency and severity: a graphical comparison by severity and crash type and EB analysis. Figure 5.1. presents the graphical comparison by severity and by crash type for all five J-turn sites combined.

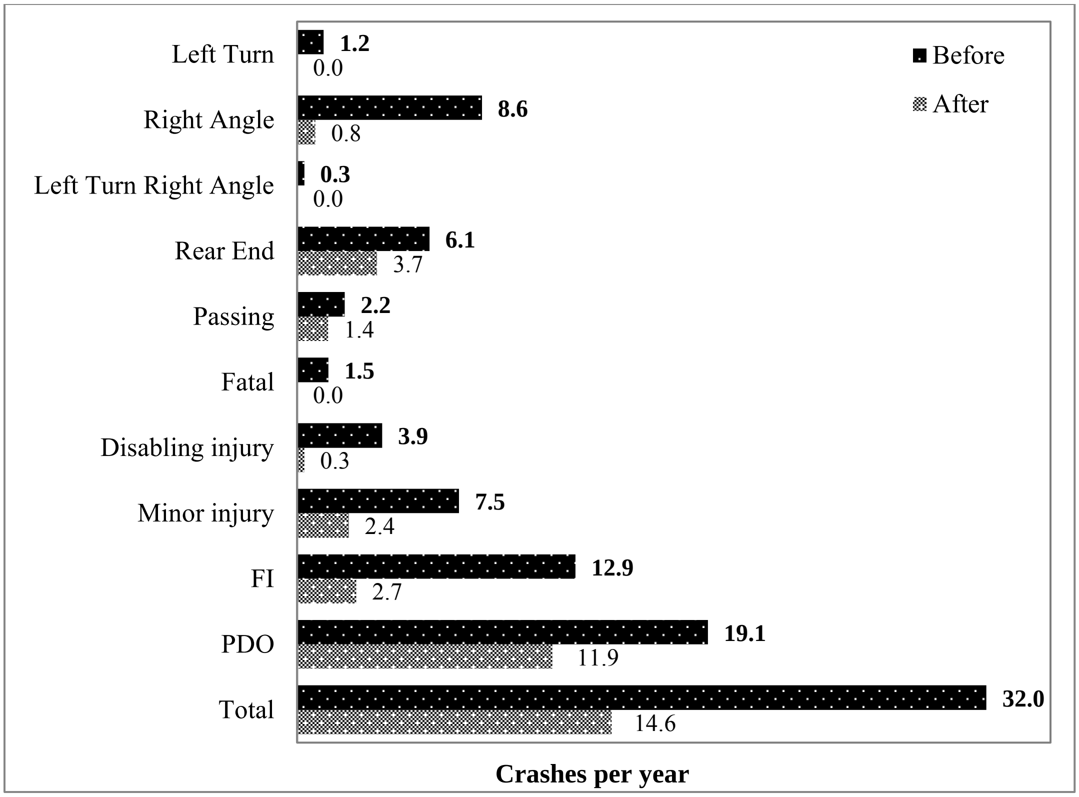


Figure 5.1. Annual Crash Frequency Before and After J-turn (sum of all sites)

Figure 5.1. illustrates that the total number of crashes per year combined across all sites and all severities decreased from 32.0 to 14.6 (54.4% reduction) after the J-turn treatment. There were no fatal crashes at any of the sites in the after period. Disabling injury crashes per year decreased from 3.9 to 0.3 (91.6% reduction). The elimination of fatal crashes and a significant reduction in disabling injury crashes are substantial safety improvements offered by the J-turn treatment. Minor injury crashes per year also decreased from 7.5 to 2.4 (67.9% reduction). Property damage only crashes per year decreased from 19.1 to 11.9 (37.8% reduction). The J-turn is designed to decrease angle crashes. Figure 5.1. shows this goal was accomplished: right angle crashes per year decreased from 8.6 to 0.8. One of the most severe crash types, left turn right angle crashes, was totally eliminated by the J-turn. Rear-end and passing crashes also decreased post J-turn implementation.

The project-level EB method compared the predicted crash frequency without the J-turn to the actual crash frequency with the J-turn. Calibration factors and correlations discussed previously were used in the predictions. The safety effectiveness values for the three correlation conditions were found to be: 1) independent – 60.4% reduction in FI crashes, 28% reduction in TOT crashes, 2) fully correlated – 66.7% reduction in FI crashes, 34.2% reduction in TOT crashes, and 3) partially correlated – 63.8% reduction in FI crashes, 31.2% reduction in TOT crashes. All reductions were significant at the 95% confidence level. These safety effectiveness values are comparable to, but higher than, the site-specific effectiveness values reported by Hummer et al. (2010)—a reduction of 51% in FI crashes and a 27.2% reduction in TOT crashes.

The safety effectiveness results for individual sites are presented in Table 5.2. The safety effectiveness values for FI and TOT crashes are shown in the far right columns. For each site, the expected crash frequency obtained from the EB method (assuming partial correlation) for the after period, the observed crash frequency for the after period, and the safety effectiveness value are reported. The standard error values are also reported in parenthesis next to each safety effectiveness value. The overall safety effectiveness for all sites combined is shown in the last row of Table 5.2.

Table 5.2. EB Results for Individual Sites

J-turn site	Location	Crash measure	FI	TOT
Site 1	US 63 and Deer Park Rd. Columbia, Boone County, MO	EB Expected Crashes	6.6	23.4
		Observed Crashes	0	8
		SE % (Std. Error %)	100.0 (0.0)	65.8 (12.5)
Site 2	US 54 and Honey Creek Rd. Jefferson City, Cole County, MO	EB Expected Crashes	3.5	6.8
		Observed Crashes	1	5
		SE % (Std. Error %)	71.7 (26.2)	26.9 (36.7)[1]
Site 3	US 54 and Route E Henley, Cole County, MO	EB Expected Crashes	3.1	6.7
		Observed Crashes	1	3
		SE % (Std. Error %)	68.2 (29.2)	55.2 (26.8)
Site 4	MO 13 and NE 364 Rd. Osceola, St. Clair County, MO	EB Expected Crashes	4.4	10.2
		Observed Crashes	3	15
		SE % (Std. Error %)	32.3 (40.6)[1]	-47.7 (50.9)[1]
Site 5	Route M and Old Lemay Ferry Rd. Imperial, Jefferson County, MO	EB Expected Crashes	4.0	12.0
		Observed Crashes	3	10
		SE % (Std. Error %)	24.1 (45.6)[1]	16.6 (32.1)[1]
All sites		**EB Expected Crashes**	**21.7**	**59.0**
		Observed Crashes	**8**	**41**
		SE % (Std. Error %)	**63.8 (4.6)**	**31.2 (2.5)**

Notes: EB Expected Crashes (assuming partial correlation) and Observed Crashes are for the after period (see Table 2 for duration of after period for each site). SE is the safety effectiveness expressed as a percentage. Negative SE values of represent the percentage increase in crashes. Standard error values are shown in the parenthesis next to the safety effectiveness. [1] Not significant at the 95% confidence level.

The EB results for individual sites showed that the J-turn was effective at decreasing the FI crashes at all five sites. The reductions in FI crashes were statistically significant, at the 95% confidence level, only for the first three sites, reductions in FI crashes witnessed at sites 4 and 5 were not statistically significant. The TOT crashes decreased at four out of the five sites, although only two of those sites (sites 1 and 3) witnessed a statistically significant decrease. One site, site 4, witnessed an increase in TOT crashes but the increase was not statistically significant. The results of site 4 were further investigated. The observed numbers of crashes during the 3-year before period were: 13 TOT with 5 PDO and 8 FI, while the observed numbers of crashes during the 3-year after period were: 15 TOT with 12 PDO and 3 FI. Based on the observed crash frequency at site 4 it appears that the J-turn traded higher severity FI crashes with lower severity PDO crashes.

5.1.2. Collision Diagram The information extracted from crash reports contributes to identify particular patterns in crashes according to geometric and operational features of facilities.

The collision analysis of J-turn facilities consisted of collecting crash data after the treatment and categorizing crashes by type. The following steps were taken: sampling, crash data collection, and crash type analysis.

5.1.2.1. Sampling The master list of J-turns in Missouri consisted of 18 facilities. All of the facilities were evaluated, and a selection criterion was developed which consisted of: crash data availability, regular geometric configuration, comparable features to the rest of the facilities, no influence of other facilities, and no changes during the period of analysis. Table 5.3. contains the selected J-turns.

Table 5.3. J-turn Facilities Selected

J-turn	City	Location	Open	Distance (ft.)	
				U-turn 1	U-turn 2
1	Imperial	RT M and Old Lemay Ferry Connector	Sep-07	800	1,900
2	Byrnes Mill	MO 30 and Upper Byrnes Mills Rd	Dec-12	1,500	1,700
3	Jefferson City	US 54 and Honey Creek Rd	Nov-11	1,900	1,900
4	Jefferson City	US 54 and Route E	Oct-11	1,700	N/A
5	Columbia	US 63 and Route AB	Nov-12	2,300	3,000
6	Columbia	US 63 and Bonne Femme Church Rd	Nov-12	900	1,400
7	Osceola	MO 13 and Old MO 13/364 E	Jul-09	1,100	980
8	Ridgedale	US 65 and Rochester Rd	Dec-12	730	990
9	Sheridan	US 65 and MO 215/ RT O	Nov-09	630	630
10	Jackson	US 65 and MO 38	Nov-09	630	630
11	Jackson	US 65 and Ash St/ Red Top Rd	Nov-09	630	630
12	Sheridan	US 65 and RT AA	Nov-09	650	1,300

The designated area and AADTs of the minor/major road were also considered for analysis as detailed in Table 5.4. The following sections describe geometric details of each facility analyzed.

Table 5.4. Designation Area and AADTs

J-turn	Location	Area	AADT Major Road	AADT Minor Road
1	RT M and Old Lemay Ferry Connector	Urban	9,320	358
2	MO 30 and Upper Byrnes Mills Rd	Urban	23,091	2,226
3	US 54 and Honey Creek Rd	Rural	18,213	435
4	US 54 and Route E	Rural	15,097	1,017
5	US 63 and Route AB	Rural	26,956	1,020
6	US 63 and Bonne Femme Church Rd	Urban	26,388	1,504
7	MO 13 and Old MO 13/364 E	Rural	11,109	467
8	US 65 and Rochester Rd	Rural	11,584	486
9	US 65 and MO 215/ RT O	Rural	7,573	982
10	US 65 and MO 38	Rural	6,975	822
11	US 65 and Ash St/ Red Top Rd	Rural	6,631	524
12	US 65 and RT AA	Rural	9,407	932

Each facility has specific geometric and operational features. The specific considerations to evaluate the J-turns included the distance to the U-turn from the minor road, facilities with open median to provide access to the minor road from the main road (left turning movements). The presence of acceleration and deceleration for the U-turns and minor road. Also, the inclusion of additional areas to facilitate turning movements at the U-turns for larger vehicles.

5.1.2.2. Crash Data Collection Crash data was collected considering an area of influence of the J-turn. It consisted of 1,000 ft. from the U-turn in each direction for the major road, and 250 ft. for the minor road. Crashes were queried using the accident browser application of MoDOT TMS. The periods of analysis for each facility were from the date the facilities open to traffic with the new geometric until the end of 2014. A total of 183 crashes were found considering the footprint established.

5.1.2.3. Collision Diagram Analysis The analysis was divided in four phases: 1) crash landing within j-turn footprint, 2) selection of J-turn related crashes, 3) classification of J-turn related crashes, and 4) additional crash statistics.

5.1.2.3.1. Crash Landing All 183 crash reports were reviewed manually and located in a CAD drawing according to type of crash and severity. In this stage, crashes that were outside the limits of the footprint were not included since there are errors with crash location in the data. Figure 5.2. shows the landing of crashes for the J-turn at RT M and Old Lemay Ferry Connector.

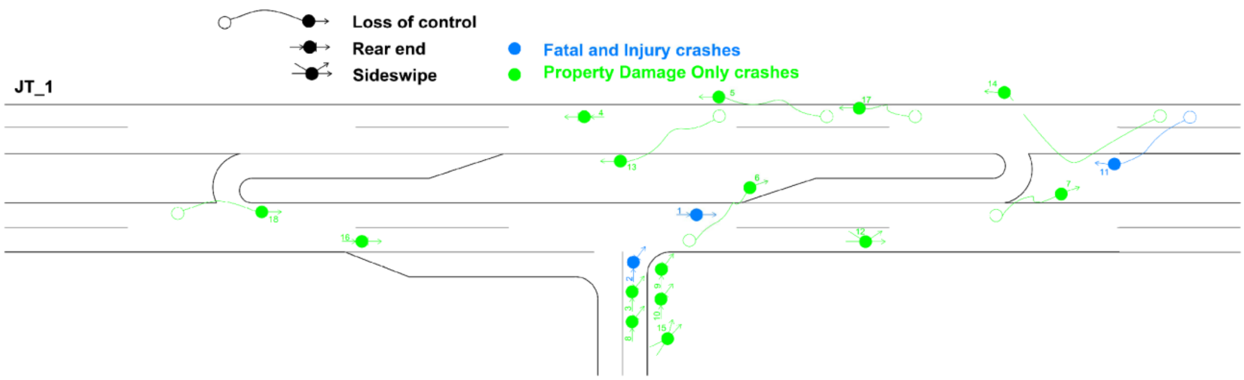


Figure 5.2. Crash Landing at RT M and Old Lemay Ferry Connector

5.1.2.3.2. J-turn Related Crashes As observed in Figure 5.2., there were many landed crashes involving loss of control throughout the footprint of the J-turn. The landed crashes were reviewed once again and the circumstances and details of the crash report were interpreted. Crashes that were generated because of the influence of geometry and operations of the J-turn were selected. Since many crashes were strictly related to severe weather conditions or impaired drivers these crashes would have biased the results. Figure 5.3. shows the J-turn related crashes.

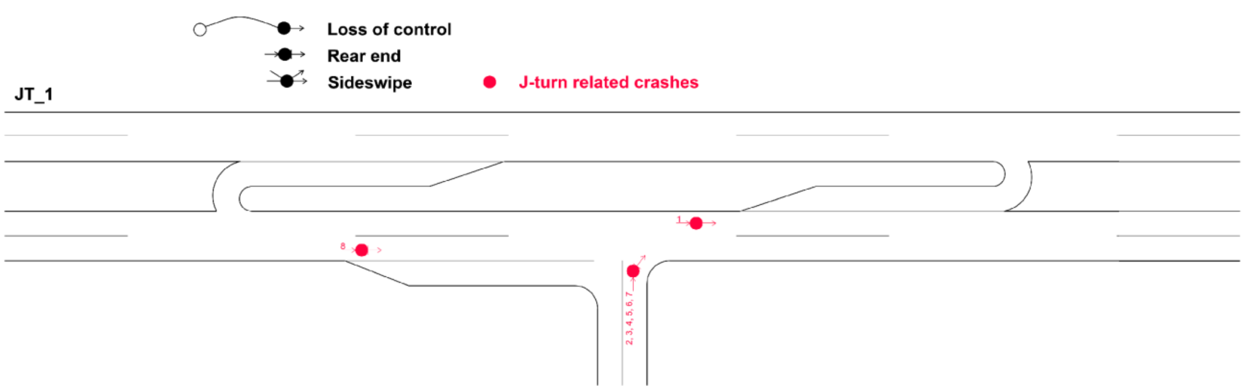
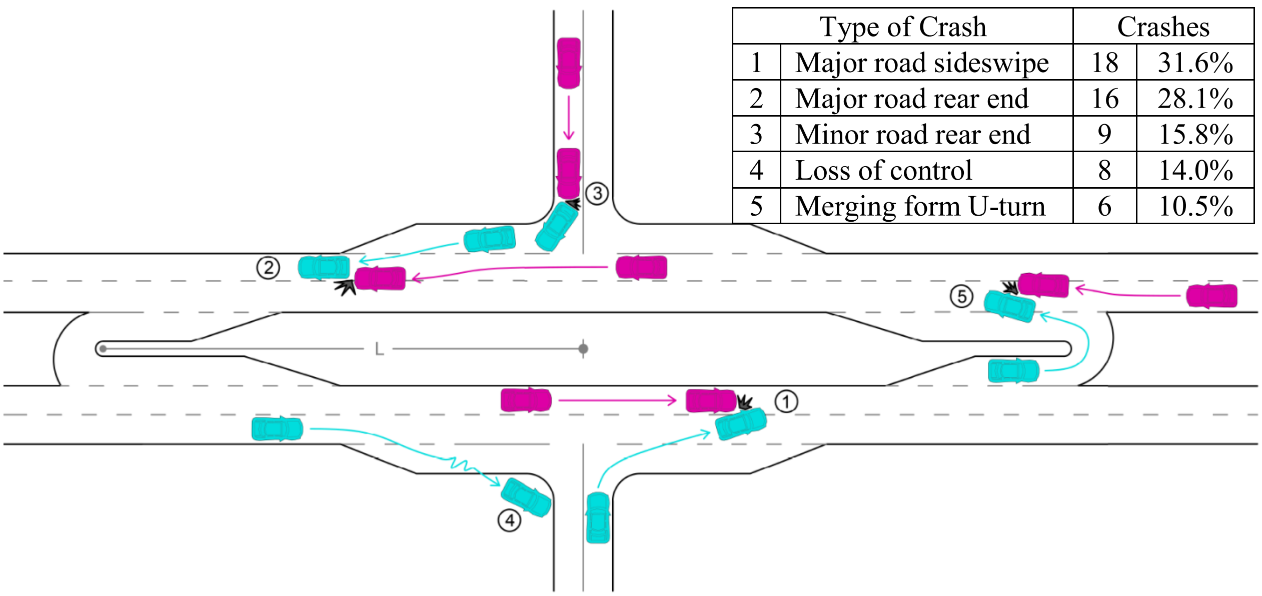


Figure 5.3. J-turn Related Crashes at RT M and Old Lemay Ferry Connector

5.1.2.3.2. Additional Crash Statistics Crash Reports provided relevant information to perform additional crash statistics. The type of vehicle, age and gender of drivers involved in the crash were studied.

5.2.2.4. Results and Conclusions The collision diagram analysis provided significant information to identify crashes according to the location and geometry. The most recurrent crashes were sideswipe with 31.6% and rear end with 28.1% on the main road involving J-turn related crashes. Most cases involved vehicles merging with traffic and changing lanes to enter the U-turn. Speed differential and inattention were common in most of the cases. Vehicles traveling on the main road were not able to react on time when traffic from the minor road were traveling on the same lane or cutting across lanes. Figure 5.4. illustrates the results of the collision diagram analysis.



	Type of Crash	Crashes	
1	Major road sideswipe	18	31.6%
2	Major road rear end	16	28.1%
3	Minor road rear end	9	15.8%
4	Loss of control	8	14.0%
5	Merging form U-turn	6	10.5%

Figure 5.4. Results of Collision Diagram Analysis

Conventional crashes at minor road approaches were rear ends with 15.8%. Drivers were unable to stop in time when they were trying to look for upcoming traffic on the main road and the vehicle ahead stopped or slowed down. There were significant cases of loss of control due to inattention, improper lane use, or high speeds. Most of the cases were at deceleration lanes and accounted for 14.0% of the crashes. Also, vehicles maneuvering through the U-turn and merging with traffic accounted for 10.5% of the crashes.

Sideswipes and rear ends in the main road were reviewed according to the distance between the minor road and the U-turn. The analysis of this type of crashes was evaluated with crash rates since crashes were compared among facilities taking into account time and exposure. The distance limits considered were less than 1,000 ft.; between 1,000 and 1,500 ft.; and larger than 1,500 ft. The following equation was used to calculate the crash rate as a function of exposure. Figure 5.5. illustrates the results.

$$Crashes\ per\ Million\ Vehicle\ Miles\ Traveled\ (MVMT) = \frac{A \times 1{,}000{,}000}{L \times AADT \times 365} \quad (5.1)$$

Where,

A , average crashes per year;

L , segment length (miles);

$AADT$, total entering vehicles per year.

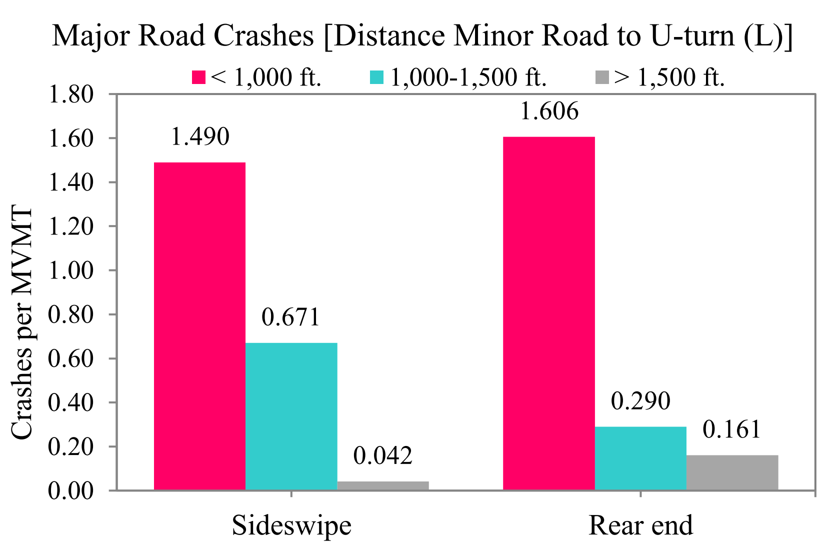


Figure 5.5. Major Crashes Sideswipe and Rear end Crashes

The analysis showed that sideswipe crashes decrease as the distance between U-turn and the minor road increases. Rear ends were less frequent between 1,000 and 1,500 ft. However, they were more frequent at location in which there were either short or prolong distances to the U-turn. This trend could be supported by the acceleration and deceleration lanes when there are short and prolonged distances. The shorter the distance, slower lane changes are from minor road merging vehicles. The longer the distance, merging vehicles may not fully develop their speed conflicting with major road through vehicles going at a faster speed.

Additional data was collected from the crash reports to evaluate other characteristics involving crashes at the J-turn. The analysis of light conditions showed that crashes at the J-turn happened during daylight time (77.2%). Despite the presence of illumination, 14% of total crashes happened compared to 5.3% with no lights or 3.5% with lights off. From the analysis of vehicle types, there were numerous cases in which slow moving vehicles were involved. For instance, semi-trucks with 4.7%; single unit trucks and pickups with trailer unit with 3.8% each type. There were 3.8 % of the crash cases with motorcycles generated by vehicles cutting across them and speeding that ended in loss of control. There was one specific case in which there was a buggy pulled by a horse traveling on the shoulder and when it merged to the through traffic lanes the buggy was rear ended. The vehicle on the main road was unable to make any evasion maneuver when the buggy merged to the lane. The crash was property damage only. It was during day time, but it was snowing. The driver of the buggy stated that the vehicle was coming at the distance and slowly when decided to merge.

The research findings of the J-turn study provides an insight for geometric design. It provides a range of safe distances for U-turn spacing and the benefits of including acceleration lanes to provide adequate merging and lane changing maneuvers at the J-turn.

6. SAFETY EVALUATION OF RED LIGHT CAMERAS

6.1. Introduction

The first Red Light Camera (RLC) installed in Missouri was in the city of Arnold in 2006, and many municipalities followed suit. In building a data sample, a master list of locations with RLC across the state was developed. From the list, facilities were randomly selected and validated to obtain a consistent sample. The sampling criteria consisted of four leg intersections, urban locations, no influence from other facilities, and crash data availability. A total of 24 intersections were selected for this study. The HSM recommends a sample of 20-40 sites for safety evaluations (AASHTO, 2010). Additionally, 35 comparable intersections with no RLC treatment were also selected to estimate crashes by type (right angle and rear end). The periods of analysis consisted of two years before and two years after RLC implementation. The crash data was collected from the Missouri State Highway Patrol (MSHP) database.

6.2. Data Collection

The data collection included intersection geometry, signal control operation, traffic volume, surrounding features, and crash data. The data was collected using tools such aerial photographs and MSHP crash records database. The geometry required for the analysis was the number of left/right turn lanes and the length of pedestrian's crossings. The traffic volume was the annual average daily traffic (AADT). It was important to identify educational facilities, bus stops, and alcohol sale establishments in the area (within 1,000 feet of the center of the intersection), since they significantly influence crashes. Table 6.1. summarizes the data collected for the treated facilities.

Table 6.1. Site Data Characteristics

No.[1]	Enforcement date	AADT[2]		Turning lanes		Signal		Pedestrians		Establishments[3]		
		Major road	Minor road	Approach with left turn lanes	Approach with right turn lanes	Approach Pro/Perm Left	Approach Protected Left	Ped. volume[4]	Max crossing lanes	Bus stops	School[5]	Alcohol
1	2/2/2008	17,337	15,782	4	1	3	1	3	7	4	N	3
2	3/5/2009	30,296	11,451	4	4	0	4	3	8	5	N	3
3	11/14/2007	38,357	5,473	2	0	2	0	5	5	0	N	2
4	11/28/2007	13,254	2,966	2	2	2	0	4	6	4	Y	1
5	6/17/2008	24,793	6,667	4	4	3	1	3	6	6	N	2
6	9/4/2009	29,944	14,364	4	3	0	4	3	7	1	N	2
7	3/9/2007	60,793	10,944	4	3	0	4	3	7	1	N	4
8	6/5/2008	39,121	12,205	4	3	4	0	4	7	8	Y	1
9	4/10/2008	19,134	4,746	3	0	2	0	4	5	3	Y	0
10	4/10/2008	29,171	9,529	3	3	2	1	3	5	3	Y	2
11	11/1/2011	29,559	6,946	4	4	4	0	5	5	0	N	2
12	10/27/2005	17,240	19,315	4	3	0	3	4	7	2	N	3
13	9/26/2008	24,019	18,749	4	4	0	4	4	8	2	N	3
14	3/16/2008	23,087	15,782	4	0	2	2	3	5	8	N	2
15	6/3/2009	27,068	15,608	4	4	0	4	4	7	7	N	2
16	2/5/2010	40,198	28,596	4	4	0	4	3	7	2	Y	1
17	9/4/2009	31,641	11,275	4	4	0	4	4	7	1	N	1
18	4/30/2009	21,586	19,487	4	3	0	4	4	7	6	Y	2
19	3/22/2010	30,162	9,057	4	3	0	2	3	7	4	N	3
20	2/11/2009	41,331	19,259	4	4	0	4	4	8	3	Y	3
21	3/5/2009	29,524	7,322	4	2	2	0	5	5	3	N	1
22	3/9/2007	24,707	32,975	4	4	0	4	4	7	7	N	2
23	10/5/2010	23,221	18,860	4	4	0	4	4	9	6	N	1
24	2/24/2008	15,782	16,909	4	3	4	0	4	9	8	N	3

Notes: [1] The locations are not identified with further description for liability purposes (listed from 1 to 24); [2] Average AADT during period of study; [3] Educational, transportation, or sale establishments within 1,000 ft. from the center of intersection; [4] Pedestrian volume 1 (3,200 ped/day), 2 (1,500 ped/day), 3 (700 ped/day), 4 (240 ped/day), and 5 (50 /day) *(13)*; [5] Presence of educational establishment Y = yes or N = no.

Table 6.1. contains the date in which automated enforcement was implemented. The traffic volumes for the major road ranged from 13,000 to 60,000 (vehicles/day) and the minor road between 2,000 and 33,000 (vehicles/day). The characteristics of turning lanes on approaching legs are listed, including the left and right turn lanes. Two signal control types for left turns exist—permissive/protective and protected only. Pedestrians had not been considered in previous research, and this study considered this as pedestrian volume in relation to the maximum number

of lanes crossed at the intersection (AASHTO, 2010). The maximum number of crossing lanes was between 5 and 9. Several bus stops were common around the intersections, and in some cases, there were up to 8 stops. Site 11 was the only intersection without any bus stops. Alcohol sale establishments were common in the surrounding areas of the intersections. Site 9 was the only location without an alcohol sale establishment in the area. The crash data was collected using the functional area of the intersections as illustrated in Figure 6.1. for the before and after periods.

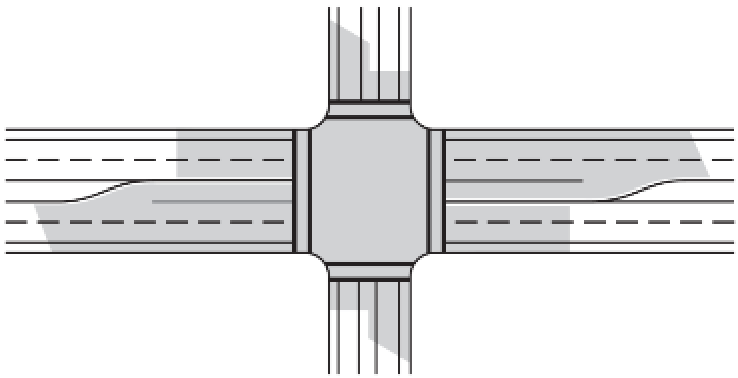
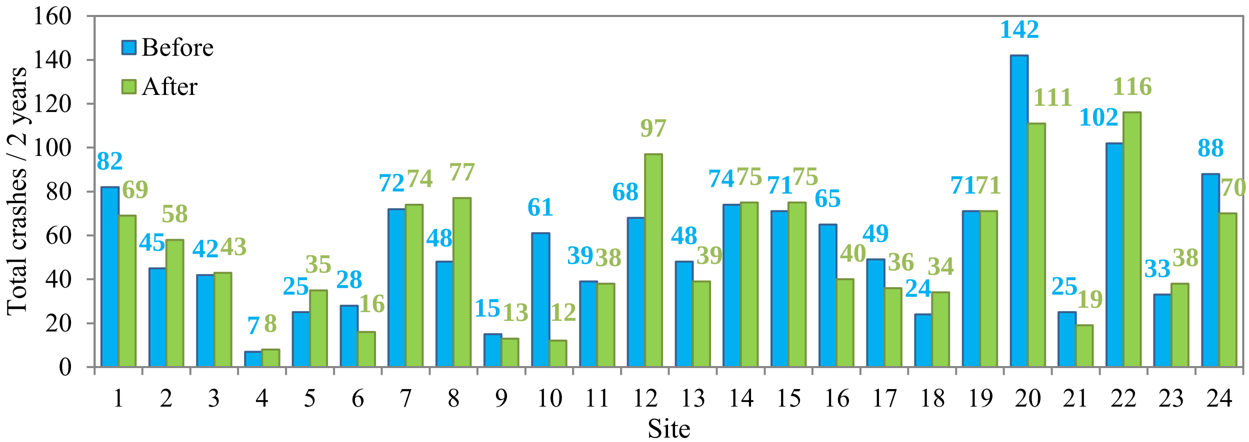
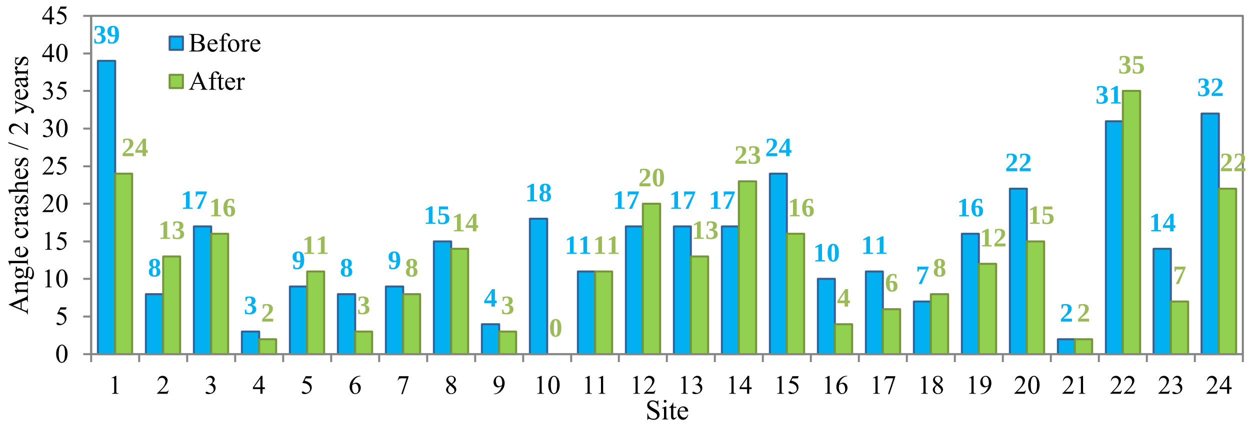


Figure 6.1. Functional Area of an Intersection (AASHTO, 2010)

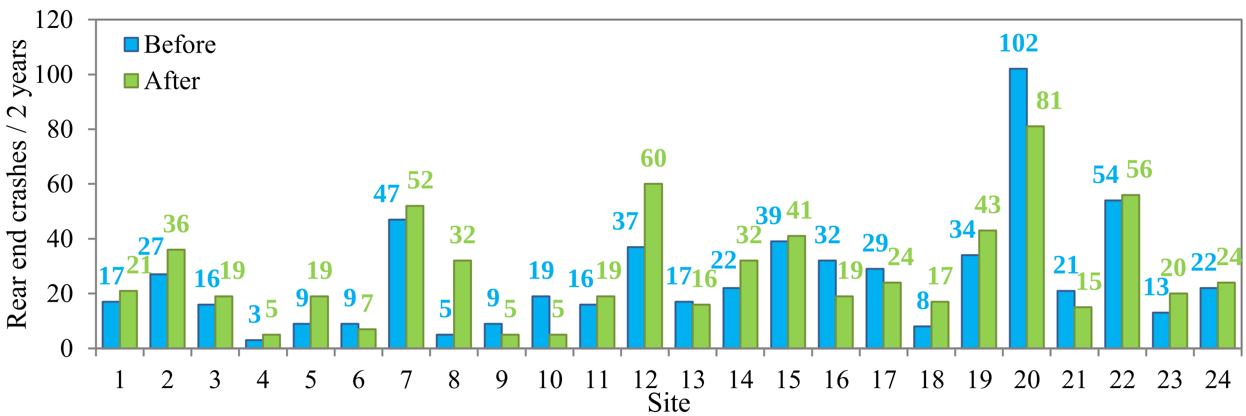
The crash data for the before and after periods is shown in Figure 6.2. Total, angle, and rear end crashes are shown individually. Notice that Figure 6.2. is only descriptive statistics before more rigorous statistical methods were applied in this study. Figures 6.2.a and 6.2.b show that 11 and 16 sites experienced reduction in observed crashes for total and angle crashes respectively. On the other hand, there was an increase of rear end crashes at 15 sites as shown in Figure 6.2.c. The changes in crashes between before and after periods were not large in magnitude.



a)



b)



c)

Figure 6.2. Before and After *a)* Total, *b)* Angle, and *c)* Rear End Crashes

6.3. Site Specific Safety Effectiveness Results

As previously discussed, the main objective of the Empirical Bayes method is to determine an unbiased expected crash frequency in the after period had the treatment not been implemented (Hauer, 1997). The predicted crashes are obtained using the prediction methodology of the HSM (AASHTO, 2010), involving Safety Performance Functions (SPF), Crash Modification Factors (CMF), and Calibration factors (C) by facility and severity type. All these functions and factors account for local site characteristics, refining the prediction of crashes. The base model SPF has an additional parameter called overdispersion (k) which forms the basis for the application of the Empirical Bayes method.

It was important to determine the distribution of crashes at non-treated facilities to accurately estimate the effect of the treatment by crash type. Therefore, an additional 35 non-treated comparison sites were used to determine crash distribution by type (e.g. angle, read end). Angle crash distribution refers to right angle crashes or collisions between vehicles in converging directions (front end and lateral crashes). Also, the distribution of type of crashes was further identified by the severity categories of total (TOT), fatal and injury (FI), and property damage only (PDO). The results are summarized in Table 6.2.

Table 6.2. Crash Distribution Results

Crash Type	TOT	FI	PDO
Rear end	62.60%	56.00%	64.80%
Angle	33.40%	40.10%	30.90%
Other	4.00%	3.90%	4.30%

Table 6.3. shows the combined results of the safety effectiveness across all sites. The implementation of RLC in Missouri resulted in a reduction in FI crashes by 7.4%, increase in PDO crashes by 3.8%, and increase in TOT crashes of 1.6%. Additionally, right angle crashes were reduced across all severities, including 14.5% for FI. Rear end crashes increased by 16.5% overall

but were decreased by 10.9% for FI crashes. These results are in agreement with previous studies (Høye, 2013).

Table 6.3. Aggregated RLC Safety Effectiveness Results

Type	Severity		
	TOT	FI[1]	PDO[2]
All crashes	-1.6% (3.9%)[3]	7.4% (7.3%)	-3.8% (4.5%)
Angle crashes	**11.6% (1.7%)**	**14.5% (11.4%)**	**11.2% (7.9%)**
Rear end crashes	**-16.5% (6.3%)**	**10.9% (8.5%)**	**-23.1% (7.5%)**

Notes: [1] Fatal and Injury; [2] Property Damage Only; [3] Safety effect. % (St. error %); Negative values represent increase in crashes. **Black Bold** values indicates 95% confidence and **Gray Bold** indicates 80% confidence.

6.4. Crash Cost Benefit

The economic benefit of RLC was calculated using aggregated crash costs by crash types and severity levels. It involved placing a monetary value on crashes, including material and life losses. An adaptation of the Empirical Bayes Method was used for the economic estimates (Council et al., 2005, 2005a). The change in crash costs over all treated facilities in a jurisdiction for specific crash types was estimated. Based on the method used for the safety effectiveness described previously, the Empirical Bayes method measures the difference between net crash costs expected without treatment and observed with treatment in the after period.

6.4.1. Comprehensive Crash Costs The analysis performed focused on crashes at urban intersections with speed limits equal to or less than 45 mph. Although there are crash costs for every individual KABCO severity scale (K=fatal; A, B, C=injury; O=no injury), fatal and injury crash costs were aggregated for this analysis as per common practice. This was done to limit the potential bias from fatal crashes which have large values but very few samples. Table 6.4. shows the crash cost used for the analysis. An adaptation of the Empirical Bayes Method was used for the economic estimates (Council et al., 2005, 2005a).

Table 6.4. Comprehensive Crash Costs (Council et al., 2005b)

Severity	Cost		
	Angle Crash	Rear end crash	All crashes
FI[1]	\$64,468 (\$11,919)[3]	\$44,687 (\$9,276)	\$91,917 (\$12,881)
PDO[2]	\$8,673 (\$1,285)	\$11,463 (\$3,338)	\$7,068 (\$547)

Notes: [1] Fatal and Injury; [2] Property Damage Only; [3] Crash cost \$ (St. error \$) in 2001 dollars.

6.4.2. Results The Empirical Bayes estimates of crash cost benefit results are presented in Table 6.5. RLC in Missouri showed a positive net economic benefit of \$35,269 per site per year in 2001 dollars (approximately \$47,000 in 2015 dollars). It translated into an overall 5.0% economic crash benefit. The results are similar to the estimates from previous research (Council et al., 2005, 2005a).

Table 6.5. Economic Effects

Empirical Bayes Estimates	Crash Type		
	Right Angle	Rear End	All
Crash cost without RLC	\$8,220,077	\$11,080,705	\$33,960,325
Crash cost after RLC	\$7,128,809	\$11,644,164	\$32,267,427
Dollar crash cost benefit, all treated facilities	\$1,091,268	-\$563,459	\$1,692,898
Dollar crash cost benefit by treated facility per year	**\$22,735 (\$3,374)[1]**	**-\$11,739 (\$3,779)**	**\$35,269 (\$6,433)**
% Crash cost benefit	**12.3% (1.8%)[2]**	**-5.1% (1.7%)**	**5.0% (0.9%)**

Notes: [1] \$Crash cost (\$St. error) in 2001 dollar costs; [2] Crash cost benefit% (St. error%), all significant at the 95% confidence; Negative values indicate increase in costs.

6.5. Legislation

In 2010, the Missouri Supreme Court voided a \$100 fine levied against a driver for allegedly running a Springfield red light. This case challenged the legality of RLC programs administered by municipalities in Missouri. The issue originated when the city of Springfield created an administrative system to prosecute violators with a municipal ordinance Code 106-161. Many municipalities classify a red light violation captured by the camera as a non-moving violation and assign no points to the violation. The violations are assigned to the vehicle and the owner rather than the driver. Missouri state law assesses two points against the driver's license that commits a moving violation and suggests that red light violations are moving violations (Copeland, 2015; Vock, 2015; Schlinkmann, 2015).

The Springfield RLC program conflicts with state statutory requirements involving violations of municipal ordinances to be heard only before divisions of the circuit court. The exception as provided in Section 479.011 states: "any city not within a county or any home rule city with more than four hundred thousand inhabitants and located in more than one county may establish ... an administrative system for adjudicating parking and other civil, non-moving municipal code violations." Therefore, only Kansas City and St. Louis are allowed to create such a system (Wolf, 2012).

After the opinion of the case in the city of Springfield was announced, the city terminated its automated enforcement program which included thirteen RLCs. Many other cases were raised using the same legal theories from the case in Springfield, and many programs in the state were shelved as a result. Twenty-seven lawsuits against Missouri cities and RLC providers were recently settled (Currier, 2015; Horsley, 2015). The circuit judge certified up to $18 million in class action lawsuits that challenged Missouri's red light camera laws. Approximately $2 million is expected to be paid out to people who paid the fines. There has also been movement in the legislature to eliminate RLC programs statewide (Currier, 2015; Horsley, 2015). In summary, the major legal challenge in Missouri involved the due process issue of the adjudication body and not any substantive issues related to safety effectiveness. The deference given by courts to municipalities for safeguarding public safety means that challenges based on safety effectiveness would most likely be unsuccessful.

6.6. Discussion and Conclusions

Automated enforcement programs have significantly contributed to the development of more efficient and safer transportations systems. RLC is an advanced technological tool to identify violators and process information; however, these tools should be developed while considering local conditions.

Effective automated enforcement could be accomplished with the application of transportation safety research. The candidate intersections should be evaluated considering geometric and operational features. Crash rates are measures of the past. The HSM methodology and the application of rigorous statistical methods such as Empirical Bayes provide accurate estimates accounting for regression to the mean bias. Right angle and rear end crashes are two primary crash types of interest. The distribution of these types of crashes could be analyzed to identify facilities with abnormally high crash frequencies. It is also important to consider measures of exposure including speed limits and traffic volumes by movements (left/right turns and through movements).

Revenue generation as a motivation is often mentioned whenever RLC are discussed. Programs are usually run by private providers and municipalities. Despite the common accusation, revenue is not significant compared to the cost of a life and societal harm. On average, 130 lives are lost at intersections in Missouri every year (MCRS, 2011). Although the American Automobile Association (AAA) in a recent study (AAA, 2011) concluded that a fatal motor vehicle crash is more than $6 million, nothing can replace the suffering of a family and society with the loss of a loved one.

RLC should not be ruled out simply because they represent enforcement and fines. In a recent survey (AAA, 2015) of 361 Missouri drivers, 78% perceived that running a red light was very or somewhat a serious threat. However, only 40% of the drivers supported laws and regulations of red light running cameras in urban areas. RLC have been proven to have positive benefits in this and many other studies conducted by safety experts independent from the RLC industry and politics. Unfortunately, ten years later the discussion goes on and RLC programs are being terminated because of ancillary issues.

The integration of automated enforcement should be closely related to local safety, legislation, and economy. RLC technology has developed as its own industry, a provider of a service rather than a provider of safety. State legislation could adopt guidelines promoted by federal agencies and uniform law committees to develop statutes that would balance procedural safeguards with safety. The involvement of different stakeholders could contribute to the effective selection of sites and implementation of RLCs.

7. AIRFIELD RUNWAY INCURSION SAFETY ANALYSIS

7.1. Introduction

The Safety Management System (SMS) is a formalized and proactive approach to system safety which directly supports the Federal Aviation Administration's (FAA) mission to provide the safest and most efficient aerospace system. The SMS is an integrated set of processes, consisting of four components: Safety Policy, Safety Assurance, Safety Promotion, and Safety Risk Management (SRM). The SRM describes the system, identifies the hazards, and analyzes/assesses/controls risk (FAA, 2014). The SRM requires specific quantitative measures to determine risk likelihood. For instance, the expected number of runway incursions in a system is a crucial measure for runway safety analysis. This dissertation proposed the enhancement of the SRM with quantitative runway incursion frequency models as a function of airport geometry, operations, construction activity, and weather variables. The most important components of the SRM are summarized in order to illustrate the application of these models. Also, a review of current aviation safety modeling is presented.

7.2. Development of Airfield Incursion Functions

7.2.1. Sampling Airports that shared similar operational, funding, and administrative characteristics were sampled. The National Plan of Integrated Airport Systems (NPIAS) lists around 3,300 airports eligible to receive federal grants under the Airport Improvement Program (AIP) (FAA, 2014b). From this list, airports were selected which fell under the following categories:

- Commercial Service: Publicly owned airports with at least 2,500 passenger boardings per year
- Primary: Airports that have more than 10,000 passenger boardings per year

- Hub: Central airport with concentrated operations, classified by the percentage of annual passenger boarding: large (1% or more), medium (0.25% to 1%), and small (0.05% to 0.25%)

The sampled airports are listed numerically (No.) in Table 7.1. according to the facility identification code (ID) and corresponding hub classification (Hub). A total of 137 airports were selected, from which 29 were large hubs, 33 medium hubs, and 75 small hubs.

Table 7.1. Sampled Airports

No.	**ID[1]**	**Hub[2]**	**No.**	**ID[1]**	**Hub[2]**	**No.**	**ID[1]**	**Hub[2]**	**No.**	**ID[1]**	**Hub[2]**	**No.**	**ID[1]**	**Hub[2]**
1	ORD	L	**29**	TPA	L	**57**	SMF	M	**85**	RIC	S	**113**	FSD	S
2	ATL	L	**30**	SNA	M	**58**	OMA	M	**86**	TYS	S	**114**	GEG	S
3	DFW	L	**31**	ANC	M	**59**	BDL	M	**87**	TUL	S	**115**	STT	S
4	LAX	L	**32**	MEM	M	**60**	JAX	M	**88**	BHM	S	**116**	EUG	S
5	DEN	L	**33**	PDX	M	**61**	ONT	M	**89**	BTR	S	**117**	BLI	S
6	CLT	L	**34**	HOU	M	**62**	RSW	M	**90**	LIT	S	**118**	SYR	S
7	LAS	L	**35**	OAK	M	**63**	LGB	S	**91**	ROC	S	**119**	AMA	S
8	IAH	L	**36**	STL	M	**64**	IWA	S	**92**	ITO	S	**120**	HSV	S
9	SFO	L	**37**	RDU	M	**65**	SFB	S	**93**	SAV	S	**121**	JAN	S
10	JFK	L	**38**	DAL	M	**66**	MYR	S	**94**	LBB	S	**122**	ECP	S
11	PHX	L	**39**	AUS	M	**67**	HPN	S	**95**	BIL	S	**123**	PSP	S
12	PHL	L	**40**	BNA	M	**68**	SDF	S	**96**	ACY	S	**124**	DAY	S
13	MSP	L	**41**	SAT	M	**69**	TUS	S	**97**	BZN	S	**125**	EYW	S
14	MIA	L	**42**	SJU	M	**70**	ICT	S	**98**	MSN	S	**126**	MHT	S
15	EWR	L	**43**	IND	M	**71**	COS	S	**99**	GSO	S	**127**	CAE	S
16	DTW	L	**44**	SJC	M	**72**	KOA	S	**100**	MAF	S	**128**	MDT	S
17	LGA	L	**45**	PBI	M	**73**	BOI	S	**101**	FAR	S	**129**	CID	S
18	BOS	L	**46**	PIT	M	**74**	LIH	S	**102**	GRR	S	**130**	GPT	S
19	SEA	L	**47**	CVG	M	**75**	FAI	S	**103**	CAK	S	**131**	PWM	S
20	SLC	L	**48**	CLE	M	**76**	PIE	S	**104**	PVD	S	**132**	ILM	S
21	IAD	L	**49**	ABQ	M	**77**	OKC	S	**105**	ORF	S	**133**	GSP	S
22	HNL	L	**50**	OGG	M	**78**	FAT	S	**106**	RNO	S	**134**	HRL	S
23	MCO	L	**51**	MCI	M	**79**	ISP	S	**107**	BTV	S	**135**	XNA	S
24	DCA	L	**52**	MSY	M	**80**	PNS	S	**108**	ALB	S	**136**	FNT	S
25	FLL	L	**53**	CMH	M	**81**	CHS	S	**109**	GUM	S	**137**	MLI	S
26	MDW	L	**54**	BUR	M	**82**	SBA	S	**110**	GSN	S			
27	BWI	L	**55**	BUF	M	**83**	SRQ	S	**111**	DSM	S			
28	SAN	L	**56**	MKE	M	**84**	ELP	S	**112**	LEX	S			

Notes: [1]ID: Airport identification code; [2]Hub: L, large; M, medium; S, Small.

7.2.3. Data Collection The data contained runway incursion records and predictor data, including operational, geometric, construction, and weather data. In addition to publicly available data, FAA's archive of airport diagrams was used to track changes in airport configuration, construction, and technology. Table 7.2. provides descriptive statistics of runway incursions data by severity and type at the 137 sampled airports during the period of analysis (2009-2013).

Table 7.2. Descriptive Statistics Runway Incursion Data

Severity[1]	**A**	**B**	**C**	**D**
Count	15	16	1,577	1,616
Average	0.12	0.11	11.51	11.80
Variance	0.22	0.13	214.02	106.06
Type	**OI**	**PD**	**VPD**	**Total**
Count	861	1,854	509	3,224
Average	6.28	13.53	3.72	23.53
Variance	66.01	210.57	16.12	526.62

Note: [1]Runway incursion severities A to D; [2]Runway incursion type OI = Operational Error, PD = Pilot Deviation, and VPD = Vehicle Pedestrian Deviation.

7.2.2.1. Geometric Variables The geometric design of an airport affects operating conditions and the general state of the system. The geometric variables included in the models were the length of runways, type of runway (single, parallel, crossing, or mixed), type of taxiways (entry/exit and high speed exit), and hotspots. These data were collected for each individual year of study by reviewing aerial imaging and airport diagram archives. For each airport, the FAA provided two diagrams, before and after changes were made. The changes were highlighted to indicate the affected area in the airfield. A total of 1,702 airport diagrams were reviewed.

7.2.2.2. Airport Operations The primary measure of exposure used in modeling was the number of airport operations. The Air Traffic Activity System (ATADS) contains the official National Airspace System (NAS) air traffic operations data for public release. According to the FAA, airport operations are defined as the number of Instrument Flight Rules (IFR) and Visual Flight Rules (VFR) itinerant operations (arrivals and departures), but overflights are not included. The number of airport operations was divided by the categories of air carrier, air taxi, general aviation, military, and total operations (FAA, 2009a).

7.2.2.3. Construction Projects During construction, an airport may potentially experience an increased risk of runway incursions. According to 14 C.F.R. §77.9, notification must be made of any construction or alterations located on a public airport regardless of height or location (GOP, 2004). Notifications are divided into permanent and temporary. Construction data consisted of the notifications per year at each airport. The FAA keeps an archive of notifications in the Obstruction Evaluation/Airport Airspace Analysis (OE/AAA) portal (FAA, 2015a).

7.2.2.4. Airport Weather Conditions The National Climatic Data Center (NCDC) was used to collect annual weather measurements from stations at the airports in the sample set (NOAA, 2015). The data collected consisted of total precipitation, total snowfall, and number of days a year with more than 0.1 inch precipitation.

7.2.3. Model Development Runway incursions are rare random events represented as discrete non-negative integers. The Poisson distribution has often been used to model the probability that an event occurs given a certain incident rate, λ. Several studies determined that Poisson distribution can be used to approximate count incident generation when the incident rate is small (Nicholson, 1985; Quine, 1987; Nicholson and Wong, 1993). However, the observed total runway incursions for this study showed that sample mean was 3.8 and the variance 18.7 (variance greater than the mean). When the count data is overdispersed (greater variability than would be expected), the Poisson distribution is no longer appropriate since it does not allow for the variance to be modeled apart from the mean. Instead, the Negative Binomial distribution is used. Two fundamental assumptions are made in modeling with the Negative Binomial distribution: 1) runway incursions are Poisson distributed and 2) the populations of means are Gamma distributed (Hauer, 2015). But for longitudinal clustered count data, such as the number of runway incursions from a set of airports, the observations from the same airports may not be mutually independent.

This is because some of the measured airport traits, such as the number of operations or weather, fluctuate over time. Since the Negative Multinomial regression explicitly allows for dependent observations, the distribution was chosen to represent runway incursion in the frequency model *(35-36)*. The year to year variation in the model was derived for each year between 2009 and 2013 for the 137 airport dataset. The likelihood function used for the Negative Multinomial modeling is shown in Equation 1. The reader is referred to Hauer (2004, 2015) for details on the derivation of the likelihood function for the Negative Multinomial. In Equation 7.1, *i* denotes entity and *j* denotes time period. The mean incident count for entity *i* in time period *j* is u_{ij}. The traits of *i* and *j* define population of entities that are assumed to be Gamma distributed with mean $E\{u_{ij}\}$ and variance $E\{u_{ij}\}^2/\mathscr{b}$. The term $\mathscr{b}$ is the dispersion parameter ($1/\mathscr{b}$ = overdispersion), reciprocal of the variance $V\{u_{ij}\}$ for a given $E\{u_{ij}\}$. The likelihood function that maximizes the estimates are those that maximize the sum of $ln\left[\mathcal{L}_i^*\left(\beta_0,\ \beta_1,\ \ldots,\ \mathscr{b}\right)\right]$ resulting in the Negative Multinomial likelihood function of the form:

$$ln\left[\mathcal{L}_i^*(\varphi,\ \beta_1,\ \ldots,\ \mathscr{b})\right] =$$

$$\mathscr{b}ln(\mathscr{b}) + \left[\sum_{j=1}^{m_i} k_{ij} ln\left(\hat{E}\{u_{ij}\}\right)\right] + ln\Gamma\left(\sum_{j=1}^{m_i} k_{ij} + \mathscr{b}\right) - ln\Gamma(\mathscr{b}) - \left(\sum_{j=1}^{m_i} k_{ij} + \mathscr{b}\right) ln\left[\left(\sum_{j=1}^{m_i} \hat{E}\{u_{ij}\}\right) + \mathscr{b}\right] \quad (7.1)$$

7.2.3.1. Predictor Variable Introduction Through exploratory analysis, predictor variables were evaluated to determine if the variables were safety-related, i.e., variables that captured significant information to estimate runway incursion frequency. Each predictor variable was then represented by an optimal functional form. The variable that captures the measure of exposure in the system, total annual operations (*TO*), was introduced first. Figure 7.1a shows the Exploratory Data Analysis (EDA) for variable *TO*.

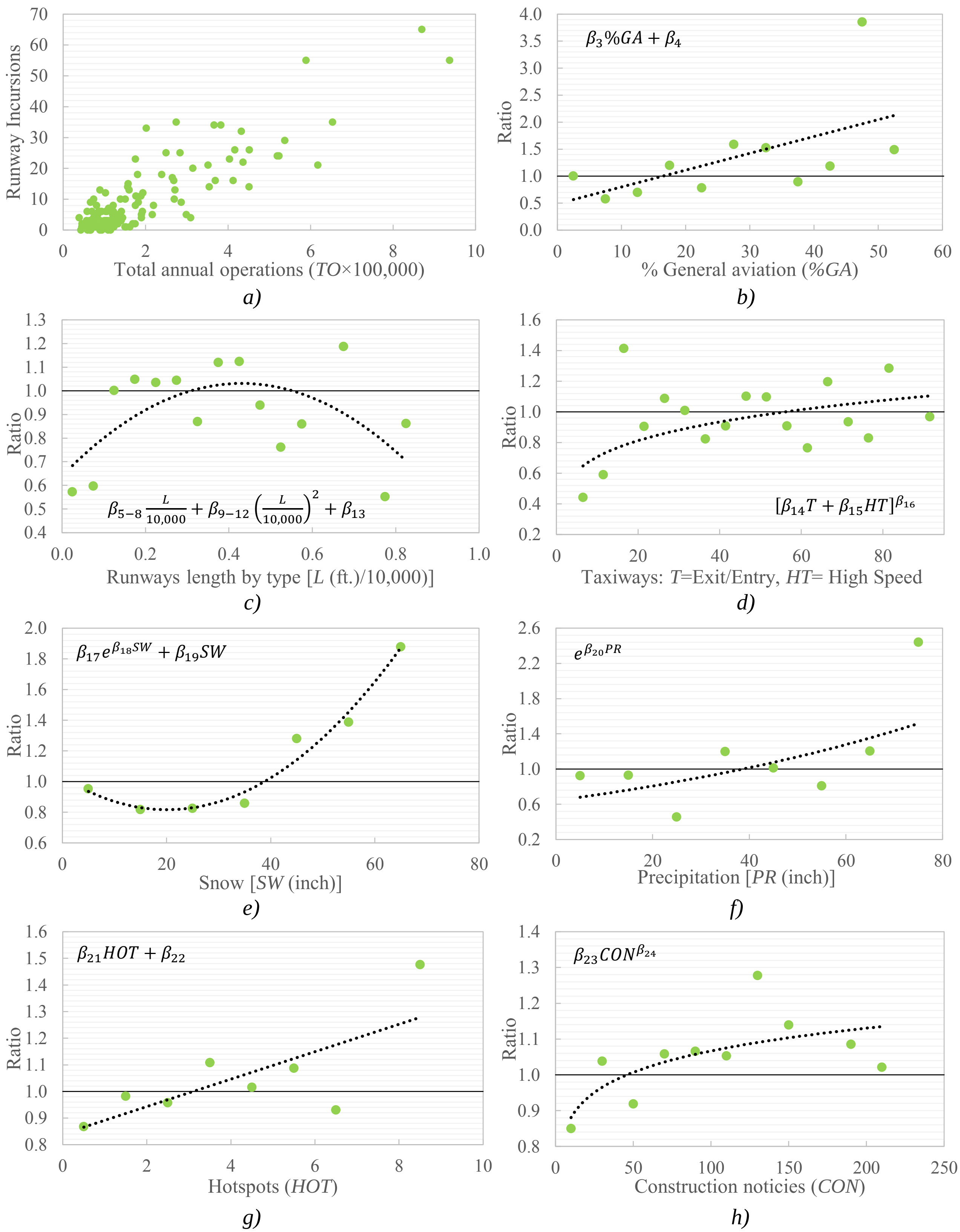


Figure 7.1. a) EDA and *(b-h)* VIEDA Analysis for Model Severities A, B, and C

The data follows an increasing trend with clustered points in the lower range of runway incursions. Different functional forms using goodness-of-fit were explored, and the Hoerl function (power and exponential composite) resulted in the best fit. The method used to select and introduce additional variables was Variable Introduction Exploratory Analysis (VIEDA) (Hauer, 2015). Step one of VIEDA involves the examination of the ratio of the observed incidents over predicted incidents with respect to the variable under consideration. If there is an orderly relationship between the ratio and the variable, then the variable is a candidate for introduction into the model. Step two involves the determination of the functional form to represent the effect of the new variable. In this step, the goodness-of-fit (e.g., CURE plot) of a functional form that includes the additional variable is examined. Figures 7.1b to 7.1h illustrate the different VIEDA analysis performed for each variable in the model for the severities of A, B, and C. Even though Figures 7.1b to 7.1h all show an orderly relationship between the ratio and a specific variable, the relationships all have different functional forms.

7.2.3.2. Runway Incursion Frequency Model Structure The frequency models developed were classified by severity as follows: 1) total runway incursions (TOT), 2) runway incursions severities A, B, C, and D. In the modeling process, severities levels A, B, and C were aggregated because severities A and B, being the most severe, have very few observations. However, severity distribution factors were obtained to distinguish each severity (A, B, and C). Equation 7.2 represents the general model structure in which the average number of runway incursions is a function of the year-specific scale parameter (φ) and multiplicative variables accounting for measures of exposure (annual operations and runways) and hazards (taxiways, weather, hotspots, and construction).

$$N = \hat{E}\{u\} = \vartheta_s \times \delta_e \times \varphi_y \times [OP \times R \times TX \times W \times H \times C] \qquad (7.2)$$

Where,

7.2.3.2.1. $\hat{E}\{u\}$*, Expected Runway Incursions* i.e., the predicted average number of runway incursions per year.

7.2.3.2.2. ϑ_s*, Runway Incursion Severity* A runway incursion is “[a]ny occurrence at an aerodrome involving the incorrect presence of an aircraft, vehicle, or person on the protected area of a surface designated for the landing and take-off of aircraft” (FAA, 2013). In 2007 (2008 fiscal year), the FAA adopted the following ICAO definition for a runway incursion as well as severity categories (s = A to E) (FAA, 2013):

- Category A - A serious incident in which a collision was narrowly avoided.
- Category B - An incident in which separation decreases and there is a significant potential for collision, which may result in a time critical corrective/evasive response to avoid a collision.
- Category C - An incident characterized by ample time and/or distance to avoid a collision.
- Category D - An incident that meets the definition of a runway incursion, such as incorrect presence of a single vehicle/person/aircraft on the protected area of a surface that is designated for the landing and take-off of aircraft, but with no immediate safety consequences.
- Category E - An incident in which insufficient or conflicting evidence of the event precludes assigning another category. This category was not modeled since there were no records with this classification.

7.2.3.2.3. δ_e*, Surface Events Classification* Surface events may be classified as runway incursions if the aircraft is still over the runway, runway protected area, or taxiway until reaching a safe maneuvering altitude (FAA, 2013). The classification of surface events (e = OI, PD, and VPD) is as follows:

- Operational Incident (OI) - A surface event attributed to airport traffic control towers (ATCT) action and inaction (FAA, 2013).
- Pilot Deviation (PD) - A surface event caused by pilot or other person operating an aircraft under its own power (FAA, 2013). The actions of a pilot that resulted in a violation of the Federal Aviation Regulation or a North American Aerospace Defense tolerance (FAA, 2010).
- Vehicle or Pedestrian Deviation (VPD) - Any unauthorized vehicle or pedestrian entry or movement in the airport movement area, including surface events involving aircrafts operated by non-pilots such as mechanics (FAA, 2010).

7.2.3.2.4. φ_y, Scale Parameter The models are more accurate for the years of study in which there are specific scale parameters (y = 2009 to 2013). To generalize the application to other years, a scale parameter for any other year, before or after the time of study, was also developed. This scale parameter was obtained through maximum likelihood by specifying a single scale parameter for all years and keeping the rest of the final model coefficients fixed. Table 7.3. shows the scale coefficient for any year is similar to the other years.

7.2.3.2.5. OP, Airport Operations The predictor variable *OP* is represented in the model as a function of airport annual total operations (*TO*) and percentage of general operations (*%GA*) in the following equation:

$$OP = \left\{\left(\frac{TO}{100{,}000}\right)^{\beta_1} exp\left[\beta_2\left(\frac{TO}{100{,}000}\right)\right]\right\} \times [\beta_3\%GA + \beta_4] \qquad (7.3)$$

The scaling of 100,000 operations is used because hub airports have a large number of annual operations.

7.2.3.2.6. R, Runways The runway predictor variable is comprised of the type of runway configuration and the length of runways. Runway type, j, includes single (j=1), parallel (j=2), crossing (j=3), and mixed (j=4).

Mixed runway configuration refers to a combination of one or more types such as single and parallel, parallel and crossing, or single and crossing. If a specific runway type is not present at an airport then $L_j = 0$ for that type. Total runway length (L_j) per type is represented in feet divided by 10,000. The scaling of 10,000 feet is used since hub airports have long runway lengths. The equation for R represents the summation over all runways types and is:

$$R = \left\{ \sum_{j=1}^{4} \left[\beta_{j+4} \left(\frac{L_j}{10{,}000} \right)^2 + \beta_{j+8} \left(\frac{L_j}{10{,}000} \right) \right] \right\} + \beta_{13} \qquad (7.4)$$

7.2.3.2.7. TX, Taxiways Two types of taxiways were incorporated into the model: conventional exit/entry (T) and high-speed exit (HT) taxiways. Conventional exit/entry taxiways are usually at a right angle and have small radii for slow aircraft maneuvering. High-speed exit taxiways are usually angled with tapered edges and used as runway exits after landing. The TX function, is presented in Equation 7.5.

$$TX = [\beta_{14} T + \beta_{15} HT]^{\beta_{16}} \qquad (7.5)$$

7.2.3.2.8. W, Weather The weather variables in the model are snowfall (SW) and precipitation (PR) in inches. The predictor variable SW was specified as a composite of a power and a linear function. The variable PR was represented by an exponential function. The function forms for SW and PR were chosen based on model performance. The equation for the variable weather is shown in Equation 7.6.

$$W = \left[\beta_{17} e^{\beta_{18} SW} + \beta_{19} SW \right] \times e^{\beta_{20} PR} \qquad (7.6)$$

7.2.3.2.9. H, Hotspots A hotspot is a surface location on an airport movement area with a history of potential risk of collision or runway incursion. This location is highlighted and brought to attention to pilots, drivers, and controllers whenever necessary. This variable was introduced in the model following the linear function:

$$H = \beta_{21} HOT + \beta_{22} \qquad (7.7)$$

7.2.3.2.10. C, Construction The variable construction is the number of construction notices per year at an airport and is represented as a power function in Equation 7.8.

$$C = \beta_{23} CON^{\beta_{24}} \qquad (7.8)$$

7.2.3.3. Runway Incursion Frequency Model Coefficients Tables 7.3. contains the estimated model coefficients. These coefficients were estimated by optimizing the Negative Multinomial Maximum Likelihood Function in Equation 7.1.

The reader is cautioned against over-interpreting individual coefficients in Table 7.3. as the goal of the modeling process was to produce good overall estimates of incursion frequency. In deriving the final model, goodness-of-fit measures were constantly re-evaluated as variables and variable functional forms were changed.

Table 7.3. Runway Incursion Frequency Model Coefficients

Severity	s	A	B	C	D	TOT
	ϑ_s	0.00871	0.00933	0.98072	1.00000	1.00000
Surface Events δ_e	***OI***	0.45274			0.08230	0.26706
	PD	0.43595			0.71349	0.57506
	VPD	0.11132			0.20421	0.15788
	All	1.00000			1.00000	1.00000
Dispersion Term	$\boldsymbol{\ell}$	6.95547			4.18394	5.11075
Scale Parameter	φ_{other}	0.90304			1.40682	1.06638
	φ_{2009}	0.76252			1.62284	1.07474
	φ_{2010}	0.77734			1.29989	0.95817
	φ_{2011}	0.80513			1.27129	0.95619
	φ_{2012}	1.13105			1.30754	1.14011
	φ_{2013}	1.04060			1.54800	1.20221
Predictor Variables Coefficients Estimates	β_1	1.13061			0.44682	0.66141
	β_2	0.05409			0.03671	0.08002
	β_3	0.01440			0.06392	0.06281
	β_4	0.50999			0.83694	1.36719
	β_5	-0.32682			-0.13363	-0.17175
	β_6	-0.15203			-0.06977	-0.06579
	β_7	-0.07307			-0.06027	-0.06779
	β_8	0.65818			-0.11942	0.03549
	β_9	0.03876			0.00346	0.01543
	β_{10}	0.02311			0.00472	0.00405
	β_{11}	-0.02007			-0.00834	-0.00885
	β_{12}	-0.16207			0.01046	-0.02385
	β_{13}	1.88207			0.89602	1.06127
	β_{14}	0.04571			0.02759	0.05055
	β_{15}	0.14693			0.01461	0.04367
	β_{16}	0.37529			0.83421	0.65461
	β_{17}	0.82277			0.80932	0.97253
	β_{18}	0.01520			0.00825	0.01123
	β_{19}	-0.01834			-0.00716	-0.01478
	β_{20}	-0.00001			-0.00102	-0.00150
	β_{21}	0.04402			0.07976	0.05456
	β_{22}	0.73730			0.74039	0.65916
	β_{23}	0.71205			0.96923	0.83182
	β_{24}	0.01549			0.08184	0.05079

7.2.4. Measures of Goodness-of-Fit Conventional statistical measures (e.g., adjusted R^2) are not the most useful here because they do not focus on count data distribution and the assumptions made in count data modeling (i.e., maximum likelihood, overdispersion). Instead, the focus should be on the performance of each predictor variable, along the ranges of its observation values, in contributing to the overall model. This focus is consistent with the ultimate goal of this research which is to help improve the SRM by providing quantitative likelihood estimates based on frequency. When evaluating conventional statistical measures of the overall (global) model fit, the individual contribution of each predictor variable is not properly quantified since it is influenced in conjunction with other variables that are part of the model (Hauer, 2015). Instead, the process used in developing the runway incursion frequency models focused on achieving good performance over the entire range of values for each predictor variable. Therefore, three measures of goodness-of-fit were used: log-likelihood, overdispersion, and CURE plots. These measures are commonly used in statistical safety modeling in other transportation modes that also deal with count data (Hauer, 2015; Hauer, 2004). Since these three measures are different, all measures are examined because a model can perform well in one measure but not in another.

7.2.4.1. Log-likelihood The model parameters that maximized the Negative Multinomial likelihood function (Equation 1) are those that maximize the sum of $ln\left[\mathcal{L}_i^*\left(\beta_0,\ \beta_1,\ \ldots,\ \mathscr{b}\right)\right]$ resulting in the in the Log-likelihood. An increase in Log-likelihood is desired when predictor variables with specified functional forms are introduced in the model. Figure 7.2 illustrates the contribution of each predictor variable as it was introduced to the models.

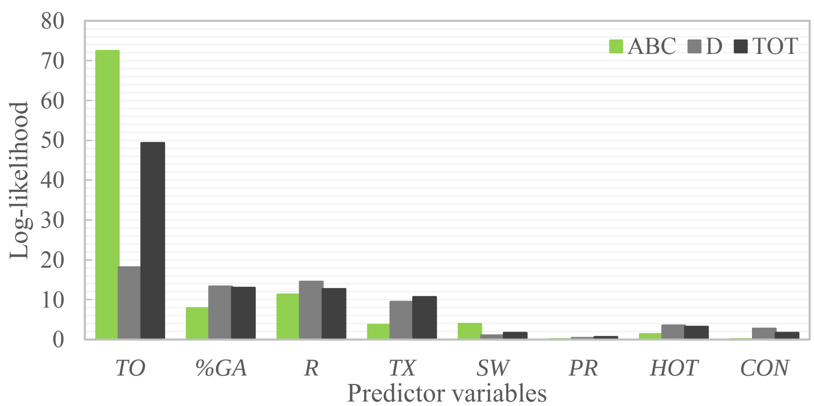


Figure 7.2. Log-likelihood Improvement by Predictor Variable

7.2.4.2. Overdispersion The overdispersion parameter indicates the variability of the model in comparison to a Poisson distribution with the same mean. The reliability of the resulting models is likely to be higher with a smaller value of the overdispersion coefficient ($k = 1/\mathscr{b}$). The decrease in overdispersion as each of the predictor variable was added is shown in Figure 7.3.

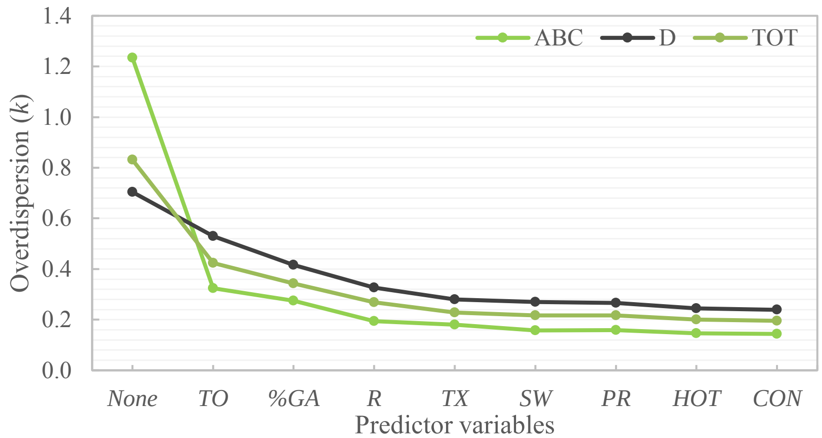


Figure 7.3. Overdispersion Improvement by Predictor Variable

7.2.4.3. CUmulative REsidual Plot In contrast to a single goodness-of-fit measure that reflects model performance over the entire range of values of a variable, CURE plots track model performance throughout the range. A satisfactory CURE plot is one that follows a symmetric random walk about the horizontal axis. In contrast, large vertical changes represent large residuals, and long increasing or decreasing runs represent regions of consistent under or over-estimation (Hauer, 2015). Throughout the process of adding more variables, trying different functional forms, or changing the order in which the variables were introduced, CURE plots were continuously evaluated for each resulting model. Figure 7.4 illustrates the CURE plot for the variable *TO* for the fully loaded model with severity A, B, and C.

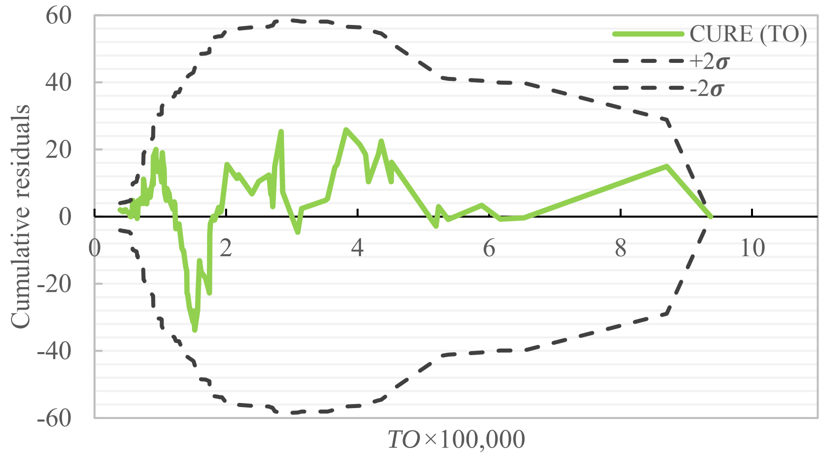


Figure 7.4. CURE Plot for Predictor Variable TO

7.2.5. Validation The frequency models were validated with additional data collected for the year 2014 at the same 137 airports (data not included for model development). The validation consisted of comparing model prediction with observed runway incursions to obtain the Mean Bias Error (MBE), Mean Absolute Error (MAE) and the Root Mean Square Error (RMSE) (Washington et al., 2005; Chai and Draxler, 2014). The models showed a MBE of -0.21 (ABC), 0.57 (D), and 0.35 (TOT). The calculated MAE for the models was 1.66 (ABC), 1.86 (D), and 2.85 (TOT). The RMSE results were 2.63 (ABC), 2.93 (D), and 4.88 (TOT). The results show that MBE is small for all severities. Although RMSE is larger than MAE the difference between the estimates is not large enough to indicate the presence of significant errors for the models with different severity levels. Therefore, the estimates show that the models performed well with the validation data and provide accurate runway incursion frequency estimates.

7.2.6. Applications The number of applications for the runway incursion models are numerous. Most importantly, they can be used in different phases of the SRM. For example, they can be used to estimate the likelihood of credible effects in Phase 4 of the SRM. The frequency models developed in this research dissertation estimate the number of times runway incursions will occur over a period of time in relation to exposure and hazards. The weighted expected runway incursion frequency according to Hauer (1997) is estimated using both the frequency model estimates and the observed data. Using predicted runway incursion frequency and the overdispersion coefficient of the model, a weighted value is calculated. With a higher accuracy model (smaller overdispersion), a larger weight is placed. The expected runway frequency can now be estimated by combining the prediction from the model, actual observations, and weight value.

7.2.7.1. Application Examples Assume Any City International Airport ("CII") is an airport classified as small hub with a single runway and 100,000 operations per year. CII has a strategic location and favorable weather conditions. The current annual operations are expected to double in the next ten years because one major air carrier is establishing its center of operations at CII. In order to accommodate the projected increase in operations, a new runway parallel to the existing runway is proposed. As part of the proposal, a SRM evaluation is developed. With the increase in operations and the additional parallel runway, runway safety is evaluated, including the potential for runway incursions. The models developed in this research dissertation provide the expected number of runway incursions taking into account the projected conditions of the new runway (e.g. parallel runways, runway length, taxiways). Weather conditions are estimated based on historical low, average, or worst conditions. Airport administrators use the model estimates in Phase 4 of the SMR to derive the runway incursion likelihood. CII administrators used this quantitative estimate to assess the level of risk of runway incursions with the SRM risk matrix. Subsequently, in Phase 5 of the SRM, CII administrators developed strategies to mitigate the risk of runway incursions. The use of accurate quantitative measures of runway incursion likelihood helps to accurately assess and mitigate risks based on the projected increase in operations and implementation of a parallel runway at CII.

Continuing with the CII example, another sample application is the evaluation of the effects of safety treatments. Assume that the new runway was successfully implemented at CII and a post evaluation of the effects of some treatments is required. Thus, after a few years after the implementation of the new parallel runway and safety mitigation strategies, CII administrators want to evaluate the effect on safety due to the treatments. Using this dissertation's methodology, the unbiased expected number of runway incursions can be estimated for the after period (after treatment implementation) as if the treatments were not implemented. Thus, the observed and expected number of runway incursions in the after period are used to estimate the effect on safety due to the treatments. The resulting unbiased safety effectiveness quantifies whether runway incursions increased or decreased due to the implementation of the new runway and mitigation strategies. The safety effectiveness estimates are statistically evaluated to determine the degree of significance. This application supports existing recommendations in Phase 5 of the SRM to track, monitor, and evaluate the effectiveness of treatments implemented at airports (FAA, 2007).

7.3. Conclusions

The Safety Management System (SMS) is FAA's approach for managing aviation safety. A major component of SMS is Safety Risk Management (SRM) which includes the analysis, assessment, and control of safety risks. The current SRM process lacks quantitative models of safety performance; thus, the SRM fails to take advantage of the wealth of quantitative data available such as operations, airfield geometrics, weather, and construction activity. Quantitative modeling of airfield safety has many benefits such as consistency across analysts, the ability to compare between alternatives, controlling for safety-related factors, and accounting for regression-to-the-mean bias. Quantitative measures are especially beneficial for Phase 4 (risk assessment) and Phase 5 (risk treatment) of the SRM.

The frequency models developed in this research dissertation estimate the number of runway incursions over a period of time in relation to exposure and hazards; thus, the likelihood of credible effects in Phase 4 of the SRM can be estimated. The proposed models can also be used in Phase 5 of the SRM to evaluate the safety of mitigation strategies. When airports are selected for treatments, these sites carry a selection bias since airports were not selected randomly, so regression-to-the-mean effect is introduced (Hauer, 1997). To adjust for this effect, the Empirical Bayes method (Hauer, 1997) can be used in observational before and after safety studies to evaluate the safety performance of treatments (effectiveness).

One significant contribution of this dissertation is the discussion of the quantitative safety data sources and the categorization of such data. The predictor variables in the model were annual airport operations (TO), percentage of general aviation operations ($\%GA$), runway length by type (R) (single, parallel, crossing, or mixed), number of taxiways intersections (exit/entry and high speed exit), snowfall (SW), precipitation (PR), number of hotspots (HOT), and number of construction notices (CON). The collection and processing of data for every year between 2009 and 2014 for 137 U.S. hub airports was a significant undertaking since some types of data required the manual review of airfield diagrams. A log of geometrics, operational, or administrative changes would be hopeful to avoid verifying several databases and sources.

The frequency models were developed using the Negative Multinomial distribution. Separate models were developed for severities A-D and total (TOT), and the surface event type (OI = operational incident, PD = pilot deviation, and VPD = vehicle pedestrian deviation). In contrast to the use of a single goodness-of-fit measure, log-likelihood, overdispersion, and cumulative residual plots were used to ensure that the models performed adequately for all levels of safety traits (i.e., all ranges of values for every predictor variable).

This dissertation contributes to the development of future safety models by providing a comprehensive example of runway incursion modeling. Other aspects of airport safety pertaining to air-traffic control, piloting, or even wildlife, can be modeled in a similar fashion by applying other sources of data. In the future, SRM can include a collection of such safety models to produce quantitative assessments of risk.

8. DISSERTATION KEY FINDINGS AND CONCLUSIONS

The field of transportation safety has witnessed a significant development over the past decades providing modeling practices that are more accessible for both practitioners and researchers in the field. Transportation safety provides elements of prediction and evaluation for decision making in transportation facilities. With the use of these methodologies, not only did this dissertation evaluate alternative geometric and enforcement designs, but also transferred these methodologies to airfield safety applications.

The most significant contributions of this dissertation are the first comprehensive safety evaluation of the Diverged Diamond Interchange (DDI), the most rigorous safety evaluation of J-turn intersections in rural areas, and the only rigorous safety evaluation study of red light running cameras in Missouri. An unprecedented effort was required to physically review crash reports to accomplish accurate safety modeling and evaluation for all treatments studied in this dissertation. In the field of aviation, the first runway incursion frequency model was developed for hub airports in the U.S.

The results in this dissertation showed that the DDI replacing a conventional diamond interchange significantly reduced crashes for all severity levels. The DDI designs transferred severe right angle crashes into less severe crashes such as rear end and sideswipe. J-turn intersections replacing two-way stop-controlled intersections in rural areas was also found to reduce fatal and injury crashes. Right angle crashes at the intersections were eliminated and the most common crashes were rear-end and sideswipes between vehicles transitioning from the minor road approach to the main road and lane changing to the U-turn. Red light cameras at signalized intersections were found to have mixed results. Although a slight increase in rear end crashes was recorded, severe right angle crashes were decreased. Thus, the crash cost benefit results showed that red light cameras have an overall positive effect.

In the field of aviation safety, quantifying runway incursions has not been available in terms of incursion frequency. This dissertation developed runway incursion models that provide quantitative measures which can be incorporated in current guidance for the evaluation of runway safety at hub airport in the US. These models are just the beginning of a potential large scale development in the field for widespread application among airport operators and stakeholders.

Despite the consensus on the suitability of Negative Binomial and Empirical Bayes (EB) methods for safety modeling, these methods have some limitations. In modeling, there are some facility types where crashes are not very frequent, such as the speed change lanes evaluated in this dissertation. The prediction of these naturally low crash facilities results in recording a vast number of sites with no crashes during the period of analysis. Thus, plenty of zeros will be found in the observations, leading to unobservable trends. In such instances, a longer analysis time period and larger sample size may be required to sufficiently analyze those facilities.

The EB method evaluates the safety effectiveness of a group of sites before and after a treatment is implemented. There are situations with mixed results (i.e. red light cameras) in which some sites experience a positive effect and others a negative effect. The concern is whether the magnitudes of the safety effectiveness of each site is properly represented since the positive effect (crash reduction) is capped at 100% (i.e., crashes can only drop to zero) while the negative effect (increase in crashes) may not have a floor. The overall safety effectiveness of the group of sites is calculated based on the sum of all observed crashes across sites in the after period versus the expected crashes in the after period as if no treatment was implemented. Thus, some sites may have disproportionally more crashes than others, so the magnitude of crash occurrence may not be properly represented. Matching sites with similar features is not an easy task especially when the implementation of treatments is evaluated and only a select number of sites are available.

For future research, the author plans to continue evaluating alternative designs and practical approaches for crash/incident prediction in roadway and aviation transportation. Some of the areas of interest are enhancing data collection approaches and evaluation of bike and pedestrian safety through naturalistic studies, crash reporting practices, and runway incursion prediction models for small airports.

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VITA

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