5-2016

A Comprehensive Safety Analysis of Diverging Diamond Interchanges

Holly Lloyd
Utah State University

A COMPREHENSIVE SAFETY ANALYSIS OF DIVERGING DIAMOND INTERCHANGES

by

Holly Lloyd

A thesis submitted in partial fulfillment
of the requirements for the degree

of

MASTER OF SCIENCE

in

Civil Engineering

Approved:

Ziqi Song, Ph.D. Major Professor	Anthony Chen, Ph.D. Committee Member
Joe Caliendo, Ph.D. Committee Member	Mark McLellan, Ph.D. Vice President for Research and Dean of the School of Graduate Studies

UTAH STATE UNIVERSITY
Logan, Utah

2016

ABSTRACT

A Comprehensive Safety Analysis of Diverging Diamond Interchanges

by

Holly Lloyd, Master of Science

Utah State University, 2016

Major Professor: Dr. Ziqi Song
Department: Civil and Environmental Engineering

As the population grows and the travel demands increase, alternative interchange designs are becoming increasingly popular. The diverging diamond interchange is one alternative design that has been implemented in the United States. This design can accommodate higher flow and unbalanced flow as well as improve safety at the interchange. As the diverging diamond interchange is increasingly considered as a possible solution to problematic interchange locations, it is imperative to investigate the safety effects of this interchange configuration. This report describes the selection of a comparison group of urban diamond interchanges, crash data collection, calibration of functions used to estimate the predicted crash rate in the before and after periods and the Empirical Bayes before and after analysis technique used to determine the safety effectiveness of the diverging diamond interchanges in Utah. A discussion of pedestrian and cyclist safety is also included. The analysis results demonstrated statistically significant decreases in crashes at most of the locations studied. This analysis can be used by UDOT and other transportation agencies as they consider the implementation of the diverging diamond interchanges in the future.

(125 pages)

PUBLIC ABSTRACT

A Comprehensive Safety Analysis of Diverging

Diamond Interchanges

Holly Lloyd

With the implementation of new roadway configurations, there is a great need to study the influence of the roadway design on the crash rate. Utah is one of the leading states in the implementation of the Diverging Diamond Interchange.

In order to determine the effects of the new roadway configuration on the safety of the intersection, this study employs an Empirical Bayes before-after study of the crash rates at selected Diverging Diamond Interchanges in Utah. The results of the Empirical Bayes method were also used to calculate crash modification factors. The total number of crashes at each site were analyzed. In addition, total crash data for each site was analyzed at varying crash severity levels. This was done with the intention of looking at the total safety impact of the interchange design as well as the specific effects on crashes at different levels of crash severity. A theoretical discussion of pedestrian and cyclist safety is also included. The study supplied positive results and a helpful look into the safety effects of the Diverging Diamond Interchanges in Utah.

ACKNOWLEDGMENTS

I would like to express my gratitude to the UDOT Traffic & Safety Division for their support in this research project and for providing the data necessary for the analysis. I would also like to acknowledge Dr. Ziqi Song for his support throughout this project. I have to thank my family and friends for their love and endless support as I completed this project and master's program.

Most of all, I thank my amazing husband, Rob Lloyd, for listening to me discuss the details of my project over and over and over. Rob, your love, understanding and patience helped me throughout this program and I couldn't have done it without you!

Holly Lloyd

CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

As the US population continues to grow, and the numbers of travelers on roadways are persistently increasing, safety on the roadways is a priority of all government departments of transportation organizations. In this focus to increase the safety of all travelers, the Utah Department of Transportation (UDOT) has looked to some of the most innovative intersections and roadway configurations, including the diverging diamond interchange (DDI), also known as a double crossover diamond (DCD).

The aim of the DDI is to eliminate the need for the left turn phase of the signal timing at the intersection, thereby increasing traffic flow and reducing congestion. This is accomplished through the geometric difference between the traditional diamond interchange and the DDI in the crossover designs that lead traffic to cross to the opposite side of the road, allowing left turning movements onto the freeway on-ramp and left turn movements from the freeway off-ramp onto the crossroad. This design allows left turn movements that do not conflict with the opposing traffic flow. Traffic flows are controlled by a two-phase signal located at each on-ramp/off-ramp pair (Shroeder et al., 2014). The DDI configuration reduces the number of conflict points to 14 conflict points, compared to 26 conflict points in the standard diamond interchange (Siromaskul, 2010). The reduction of conflict points in the intersection and intersection approach is aimed at improving the safety of all travelers.

The DDI was first used in France more than 2 decades ago and was first implemented in the US in 2009 in Springfield, MO (FHWA, 2010). The DDI is helpful at interchanges that experience high volumes on the ramps and/or unbalanced through traffic on the arterial (FHWA, 2010). Missouri reported a drastic increase in traffic flow after the DDI was implemented in Springfield, MO (MoDOT, 2010). This success has also been seen with the implementation of the American Fork Main Street/Pioneer Crossing DDI in American Fork, Utah, which was the third DDI in the US. The Pioneer Crossing DDI has been operating since August of 2010 and has been fundamental in allowing continued traffic flows through diversions from I-15 closures, pothole repair and road repair closures lasting multiple days, as well as freeway backups due to accidents (UDOT, 2012). A current list of all operating DDIs in the United States is provided in Table 1 with the Utah DDI's highlighted in green (ATS/American, 2016). As of June 2016, Utah has eight operating DDIs and many others in the construction and planning stages. UDOT does have concerns about over-application of the innovative DDI and will be evaluating the benefits, limitations and best application opportunities for the design (UDOT, 2014).

Along with the improved traffic flow, it has also been claimed that the overall safety of drivers navigating the DDI intersections has been improved. Due to the decrease in conflict points as shown in Table *2* (Siromaskul, 2010), the severity of crashes is decreased to merging and diverging conflicts which, results in less severe crashes (Schroeder et al., 2014).

Table 1
DDIs in the United States as of June 2016.

Interchange	Location	Date Opened
I-44 @ MO 13	Springfield, MO	6/21/2009
US 60 @ National Ave	Springfield, MO	7/12/2010
I-15 @ American Fork	American Fork, UT	8/23/2010
I-270 @ Dorsett Rd	Maryland Heights, MO	Oct 17. 2010
US 129 @ Middlesettlements Rd*	Alcoa, TN	12/14/2010
KY 4 @ US 68*	Lexington, KY	8/14/2011
I-15 @ Timpanogos Hwy	Highland, UT	8/14/2011
SR 201 @ Bangerter Hwy	West Valley, UT	10/23/2011
I-435 @ Front Street*	Kansas City, MO	11/6/2011
I-15 @ 500 East	American Fork, UT	11/7/2011
US 65 @ MO 248	Branson, MO	1/22/2012
I-285 @ Ashford-Dunwoody Rd	Dunwoody, GA	6/3/2012
MD 295 @ Arundel Mills Blvd	Hanover, MD	6/11/2012
US 67 @ SR 221	Farmington, MO	9/5/2012
I-590 @ South Winton Road	Brighton, NY	9/11/2012
US 65 @ Chestnut Expressway	Springfield, MO	11/10/2012
I-580 @ Moana Lane	Reno, NV	11/21/2012
MO 150 @ Botts Road	Kansas City, MO	12/5/2012
I-85 @ Pleasant Hill Road	Duluth, GA	6/9/2013
I-44 @ Range Line Road	Joplin, MO	7/20/2013
US 60 @ MO 13	Springfield, MO	8/18/2013
US 52 @ New Olmsted County Road 12	Oronoco, MN	9/3/2013
I-70 @ Woods Chapel Road	Blue Springs, MO	9/26/2013
I-86 @ Yellowstone Ave (US 91)	Chubbuck, ID	10/7/2013
I-35 @ Homestead Lane	Gardner, KS	10/11/2013
I-70 @ Stadium Blvd	Columbia, MO	10/14/2013
I-25 @ College Drive	Cheyenne, WY	10/14/2013
SR 15 @ SR 120 Stearns County Rd.	St. Cloud, MN	10/17/2013
I-270 @ Roberts Road	Columbus, OH	10/21/2013
I-70 @ Mid-Rivers Mall Drive	St. Peter's, MO	10/28/2013
I-494 @ 34th Ave	Bloomington, MN	11/17/2013
I-15 @ St. George Blvd	St. George, UT	11/19/2013
I-64 @ US 15	Zion Crossroads, VA	2/22/2014
I-70 @ US 6 / US 50	Grand Junction, CO	2/27/2014
Dalma Mall Interchange	Abu Dhabi, UAE	5/2/2014

Table 1
Continued.

Interchange	Location	Date Opened
I-77 @ Catawba Avenue	Cornelius, NC	6/29/2014
I-29 @ Tiffany Springs Pkwy	Kansas City, MO	7/12/2014
I-15 @ UT 130, Cross Hollow Rd	Cedar City, UT	8/25/2014
Loop 375 @ Spur 601	El Paso, TX	9/2/2014
I-85 @ Poplar Tent Road	Concord, NC	9/7/2014
I-15, I-85 @ US 91, 1100S	Brigham City, UT	9/16/2014
I-69 @ IN 1 DuPont Rd	Ft Wayne, IN	9/22/2014
I-85 @ NC 73	Concord, NC	10/27/2014
MN 101 Main Street @ 141st Avenue*	Rogers, MN	10/29/2014
I-435 @ Roe Avenue	Overland Park, KS	10/30/2014
I-515 @ Horizon Drive	Henderson, NV	1/25/2015
US 65 @ Battlefield Road	Springfield, MO	2/14/2015
I-85 @ Jimmy Carter Blvd	Norcross, GA	3/29/2015
I-485 @ Mallard Creek Road	Charlotte, NC	5/29/2015
I-10 @ Old MS 67 D'Iberville Road	D'Iberville, MS	6/2/2015
I-15 @ UT 68	Bountiful, UT	6/15/2015
I-40 @ SR 66	Sevier County, TN	6/30/2015
K-10 @ Ridgeview Road	Olathe, KS	7/28/2015
I-57 @ Morgan Avenue	Marion, IL	8/12/2015
I-40 @ NC 66 Union Cross Road	Kernersville, NC	9/19/2015
I-88 @ SR 59	Naperville, IL	9/21/2015
I-95 @ US 301 Fayetteville Road	Lumberton, NC	9/29/2015
Highway 36 @ McCaslin Boulevard	Superior, CO	10/19/2015
I-75 @ University Drive	Auburn Hills, MI	11/10/2015
I-35W @ CR 96	Arden Hills, MN	11/13/2015
I-26 @ Airport Road	Asheville, NC	11/16/2015
I-35 @ University Blvd (RM 1431)	Round Rock, TX	11/19/2015
I-65 @ Worthsville Road	Greenwood, IN	11/25/2015
I-80 @ Grand Prairie Parkway	Waukee, IA	12/1/2015
I-25 @ CO 38 Fillmore Street	Colorado Springs, CO	3/25/2016
US 17/74/76 @ NC 133	Leland, NC	4/17/2016

A preliminary safety study performed by the Missouri Department of Transportation directly compared the crash rates before and after the construction of a

Table 2
Conflict Point Comparison.

Conflict Points		
Type	Standard Diamond	Diverging Diamond
Diverging	8	6
Merging	8	6
Crossing	10	2
Total	**26**	**14**

DDI in Missouri and concluded that total crashes dropped by 46% in the first year of operation (MoDOT, 2011). The simple before-after method, however, assumes that any changes to the safety performance can be attributed solely to the DDI design. In reality, confounding factors that change continuously, such as traffic flow, traffic composition, and weather conditions, can also affect the safety performance.

The US Department of Transportation Federal Highway Administration (FHWA) compiled a list of DDI advantages and disadvantages shown in Figure 1. These points can be used to analyze effectiveness of a DDI to meet the needs of locations of concern. Figure 1 also mentions the increased safety of the DDI compared to the diamond interchange. As the popularity of the DDI is increasing and more DDIs are being constructed, the need has arisen to measure the actual safety of the DDI as related to the traditional diamond interchange. The major objective of this study is to conduct a comprehensive before-after study to assess the overall safety impact of DDIs.

Advantages	Disadvantages
Non-motorized users	
• Reduces conflicts between vehicles and pedestrians for most crossing movements • Opportunity to safely accommodate pedestrians and bicyclists through interchange • Creates shorter pedestrian crossing distance for some movements • Opportunity for bicycle lanes and multi-use paths through additional right-of way (no left turn lanes needed) • Provides two-stage crossing opportunities	• Pedestrians may have to cross unsignalized, channelized right and left turns onto freeway • Pedestrians may be unaware what direction traffic is coming from • Center walkway may be unfamiliar to pedestrians
Safety	
• Reduced conflict points over other interchange forms • Reduction from 10 to 2 crossing conflicts compared to standard diamond	• May have potential for wrong-way maneuvers at crossovers • Unusual sight distance considerations at crossovers and ramp movements
Operations	
• Two-phase signals reduce lost time at interchange and increase capacity • Left-turns onto freeway may be free-flowing • Ability to coordinate through traffic or left turns from freeway • Potential for significant delay and travel time savings over standard diamond • Significantly reduced queue spillback potential, especially between ramp terminals	• Potential lower right-turn capacity from freeway where RTOR not allowed • Challenging to coordinate through traffic in both directions • Increased operational challenges at adjacent intersections due to crossover, two-phase operation at DDI • Potential driver unfamiliarity with crossover design and merges from left
Access Management	
• Provide full access control through interchange	• Does not allow exit ramp to entrance ramp movement • May require access control beyond interchange to prevent weaving maneuvers • May require relocating or removal of adjacent streets/driveways to accommodate crossover and reverse curves
Traffic Calming	
• Reduced speed through the interchange from crossover geometry and reverse curves	• Turns onto the freeway may have high speeds due to lack of signal control
Space	
• Generally fits within existing interchange right-of-way and bridge structure • Lower footprint compared to parclo interchanges	• Some additional ROW width may be needed to accommodate crossovers at tight diamonds due to reverse curves
Construction and Maintenance	
• Generally fits on or under existing bridge structure • Low-cost design compared to other interchange forms • Less queuing on the arterial may reduce pavement rutting and wear	• May require additional lighting due to unique geometry • Additional signage and pavement markings are needed beyond the levels of a conventional diamond interchange • More complex signal design requires early consideration in design

Figure 1

FHWA DDI Advantages & Disadvantages (FHWA, 2014).

CHAPTER 2

LITERATURE REVIEW

As transportation officials increasingly implement the DDI in the United States, it is important to study the design, performance, and safety of the configuration. This chapter will provide a comprehensive review of DDI studies as well as before-after study methodology.

Diverging Diamond Interchange

This section will provide a review of various aspects of the DDI including design, performance, and safety concerns and studies.

Design Considerations

Due to the crossover of the lanes, there is no longer a need for a left turn phase in the signal timing for DDIs. The left turn movements off of the through traffic are free to turn without yielding to oncoming traffic. This lane configuration allows the left turn phase to be eliminated from the signal timing. The extra time can be allocated to the through traffic or it can be completely eliminated resulting in shorter signal cycle times. Both of these options create more efficiency of traffic flow through the interchange. If the extra green time is allocated to the through movement, the capacity is greatly increased. Studies performed by UDOT observed that the addition of green time at the end of the green phase can increase the capacity of the interchange by 30%-50% (UDOT, 2014). The additional green time is added to the end of the phase when traffic is already traveling at speed, this allows more vehicles to travel through the interchange without holding up the opposite direction any longer than with the normal signal timing. Elimination of the additional saved green time provides shorter total cycle lengths which can also improve efficiency and allows more traffic movement without long waits in either direction (UDOT, 2014).

There are many design elements that must be well-thought-out in the planning of a DDI. The FHWA (2010) recommends the following design elements for consideration:

- Relocation and turning radius of the left turn lane including radius requirements for heavy vehicles
- Reverse curvature on high speed minor streets
- Appropriate median widths for standard lanes and lanes with reverse curvature as found in the Green Book
- Adequate signage to deter wrong way driver error

Pedestrian and bicycle walkway designs must also be considered if needed. These considerations, as well as any site specific needs, can vary and must be evaluated for each individual location.

The Missouri Department of Transportation (MoDOT) conducted an extensive study comparing the tight urban diamond interchange to the DDI. The FHWA (2010) reported the following improvements after the use of the DDI:

- Number of required lanes under bridges are reduced from five to four

- Number of lanes needed on cross street extending outside the interchange is reduced
- Provides more storage capacity between the ramp terminals
- Provides increased sight distance
- Interchange geometry includes traffic-calming features through reduced speeds while increasing throughput
- Geometry theoretically results in fewer and less severe crashes

Another design measure used to increase safety of all traffic in the DDI is the use of medians. Medians are used to separate the opposing traffic flows in order to reduce the risk of conflicts at the crossover areas and to help direct drivers to the correct side of the road inside the interchange. The use of medians, adequate road markings and signage are vital to the safety and correct navigation of drivers through the interchange.

Non-Motorized Traffic

Cyclists follow the same crossover movement as vehicles. Before analyzing the movement of bicycle traffic through the DDI, two types of cyclists should be considered. The first type of cyclist is familiar and comfortable moving along with the vehicle traffic on the road. These cyclists will follow the normal roadway path in a bike lane alongside vehicle traffic. The other type of cyclist, identified as a “recreational cyclist,” will be less comfortable moving with the vehicle traffic. These cyclists could be encouraged to use the median as a safer route to pass through the interchange. Figure *2* shows these two optional paths (UDOT, 2014).

Pedestrian and bicycle walkways can be located on the outside of the interchange or through the middle of the interchange. Both walkways may put pedestrians and cyclists at risk of being involved in an accident due to lower visibility of pedestrians and drivers at the crossing areas of the interchange. Depending on the placement of the

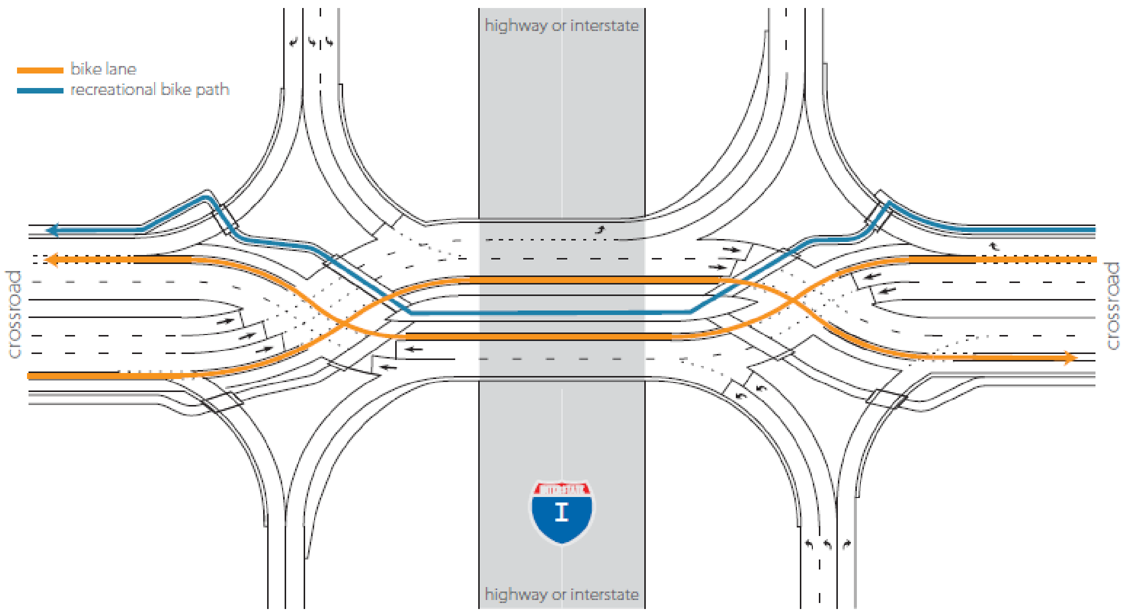

Figure 2

DDI Bicycle Paths (UDOT, 2014).

walkway, pedestrians and cyclists will cross two directions of traffic when traversing the interchange. With the walkway in the center of the interchange, pedestrians and cyclists must cross the path of right-turning vehicles coming from the freeway off-ramps as well as the through traffic at the crossover. If the walkway is located on the outside of the interchange, pedestrians and cyclists cross the path of the vehicles turning right from the freeway off-ramp as well as the path of the vehicles turning left onto the freeway on-ramp. Vehicles on the ramps could be traveling very quickly with limited visibility. Drivers may be slowing to merge with traffic; however, they are not necessarily required to stop at this merge area. Pedestrians should be extremely alert and cautious as they cross through the DDI (UDOT, 2014). Pedestrian and cyclist safety will be further discussed in Chapter 7.

Operational Performance

Using a VISSIM simulation, a MoDOT study found a decrease in average delay time per vehicle during times with higher volumes within the total DDI network configuration. MoDOT also observed decreased back-ups from traffic due to Friday night tourists and PM peak periods when compared to back-up levels of up to a mile or more before the DDI was implemented. However, morning commute back-ups were found at the Springfield, MO DDI. The implementation of a dual right and dual left off-ramp and greater signal spacing between the DDI ramps and adjacent intersections are thought to have caused the decrease in delay and back-up. Furthermore, operational improvement was even seen in the PM peak hours during a power outage. Traffic moved through the interchange as if it were a two way stop with minimal delay (Chilukuri et al., 2011).

A study performed by Gilbert Chlewicki had similar results to the MoDOT study. Using Synchro 5 for the simulation modeling to compare the DDI to the traditional diamond interchange, Chlewicki (2003) observed the following improvements:

- Total delay was decreased by two thirds
- Stop delay was decreased by three quarters
- The total number of stops was reduced by half

These simulations support the theoretical expectation that the DDI will improve capacity and flow when compared to the traditional diamond interchange.

However, the DDI is not appropriate for all intersections. When weighing the options for a particular location, the benefits and disadvantages of the DDI should be analyzed, along with other interchange configurations, to determine if the DDI is a good fit or if another option would better serve the users of the interchange. One major limitation of the DDI is the risk to pedestrians as they cross the right turn (freeway off-ramp) and left turn (freeway on-ramp) lanes. A second consideration is the risk of a “wrong-way maneuver” through the interchange. There is a learning curve for local drivers, which will help decrease the “wrong-way maneuver” risk; however, a “wrong-way maneuver” may still occur as drivers who are unfamiliar with the intersection operations drive through the DDI. A third concern is the increased capacity at the DDI location which can create problems for adjacent intersections that cannot handle the DDI capacity levels resulting in queue spillback. Another disadvantage is the elimination of access to the freeway on-ramp from the freeway off-ramp that is common in the traditional diamond interchange (Schroeder et al., 2014).

Each of these limitations must be analyzed against the benefits of the DDI, and other configurations, and the most appropriate and beneficial interchange selected for each individual location.

Safety

Safety is also a large concern when introducing a new interchange configuration such as the DDI. As reviewed in Chapter 1, the total number of conflict points decreases from 26 in the diamond interchange to 14 in the DDI. In theory, the decrease in conflict points deems the DDI safer than the traditional diamond interchange; however, statistical studies on the before and after analysis of crash frequency are necessary to truly determine if implementation of the DDI can improve the safety at a given location. As the DDI is gaining popularity, more studies are being performed on this matter; however, at this time, there are still only a few conclusive studies. Table *3* shows a compilation of the study summary and results of the recent DDI safety studies.

The VISSIM simulation study performed by the FHWA in 2010, listed first in Table 3, analyzed 74 licensed drivers in the Washington, DC area and found minimal wrong-way maneuvers. Also, when comparing the VISSIM DDI simulation to the standard diamond interchange, no change was observed in erroneous navigation and red light violations (FHWA, 2010). The Versailles, France DDI has only experienced 11 light injury crashes in the first 5 years after implementing the DDI. This is a large decrease when compared to the average 23 fatal and injury crashes at US diamond interchanges (Poorbaugh and Houston, 2006).

The majority of the studies summarized in Table 3 utilize the naïve before-after method with only the most recent MoDOT study applying the comparison and EB methods. While the naïve studies are a starting point in the safety analysis of DDIs, it is important to continue the safety research efforts. As time continues, more before and after crash data will be available, allowing for more accurate study results. Employing more advanced before-after study methods will also provide more reliable results accounting for changes in input variables from the before period to the after period as well as the regression-to-the-mean tendency. This study aims to utilize increased data in after periods and the EB analysis to provide safety analysis methodology and results. As an additional study measure, a crash modification factor (CMF) will be developed for the DDI. The FHWA mentions that a DDI CMF will be coming soon in an upcoming edition of the Highway Safety Manual (HSM) and on their CMF Clearinghouse (FHWA, 2014). The establishment of the DDI CMF will be a helpful tool in assessing the safety performance of the DDI. This study will calculate a DDI CMF from the Empirical Bayes analysis results. The CMF creation will be discussed in Chapter 5.

BEFORE-AFTER STUDY METHODOLOGY

Safety studies generally employ a before-after study method in order to determine if an improvement has in fact resulted in an increase in safety. Three before-after study methods will be discussed in this section.

Naïve Before-After Method

Before-after studies are used frequently in safety studies in the transportation field. As seen in Table *3*, a common approach to measure the effectiveness of implemented roadway improvements/changes is the naïve before-after study method. This approach makes the assumption that the observed annual average crash rate in the before period can be used as the projected expected annual average crash rate in the after period had the treatment not been implemented as shown in equation 1. The data is then analyzed by comparing the observed annual average crash rate of the after period to the expected annual average crash rate. The success of the executed improvement is determined as shown in equation 2 with the percent improvement and percent effectiveness shown in equations 3 and 4 respectively.

$$N_{obs-b} = N_{exp-a} \tag{1}$$

$$\Delta_{cr} = N_{exp-a} - N_{obs-a} \tag{2}$$

$$\%\,\Delta_{cr} = \frac{N_{obs-a}}{N_{obs-b}} \times 100 \tag{3}$$

$$\%\,Effectiveness = (1 - \%\,\Delta_{cr}) * 100 \tag{4}$$

where:

N_{obs-b} = number of observed crashes in the before period

N_{exp-a} = number of expected crashes in the after period

N_{obs-a} = number of observed crashes in the after period

Δ_{cr} = change in crash rate due to treatment

$\%\Delta_{cr}$ = percent change in crash rate due to treatment

Table 3
DDI Safety Studies.

Year of Report	Agency	Location	Before Data (Years)	After Data (Years)	Study Method	Results	Source
2010	FHWA	NA	N/A	N/A	Naïve Before-After	Positive	FHWA 2010
2010	MoDOT	Springfield, MO	5	1	Naïve Before-After	Decrease in Crashes	MoDOT 2010
2010	AASHTO	Lexington, KY	4	2	Naïve Before-After	Mixed; Some decrease, some increase within crash types	AASHTO 2010
2014	FHWA/ NYSDOT	Rochester, NY	3	0.667	Naïve Before-After	Mixed; Some decrease, some increase within crash types	FHWA 2014, NYSDOT
2015	MoDOT	Missouri	2.9-4.25	.83-4.25	Naïve, Comparison Group, Empirical Bayes	All Positive	MoDOT 2015

Hauer (1997) takes an in-depth look at the naïve before-after approach to safety studies. Five factors are identified that render this approach insufficient and problematic: 1) factors that change naturally over time, i.e., traffic patterns, annual average daily traffic (AADT), weather, driver behaviors etc., 2) other treatments and programs that have been put in place, other than the treatment being studied, that would affect the area of the studied treatment, 3) the number of reported 'property damage only' accidents that may fluctuate due to changed reportability limits or costs of repairs, 4) the probability of accidents actually being reported may vary between study periods, and 5) the uniqueness of the entities chosen for study create an unstable foundation for estimating what may naturally be expected.

Because of the possible uniqueness of the sites selected, a bias can occur caused by the regression-to-the-mean tendency of data. This bias can be attributed, in part, to the fact that in many instances the locations chosen for improvement are chosen due to high reports of crashes and incidents (AASHTO, 2010). These high levels are believed to have the tendency to naturally regress back to the actual long term mean as time progresses, as seen in Figure *3* (FHWA, 2010). These extreme values can cause high estimations of expected values in the after period resulting in exaggerated improvement results including high increases and decreases in safety. The risk of regression-to-the-mean bias can be decreased as the number of years of data included in the study increases (AASHTO, 2010).

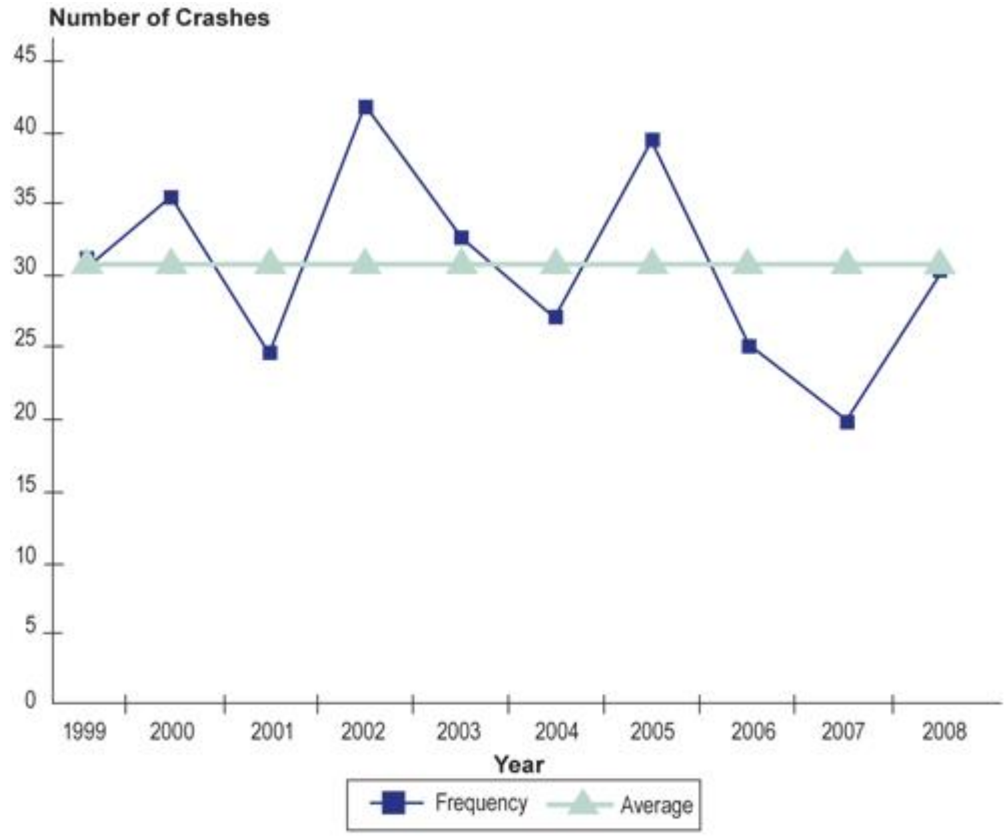

Figure 3
Regression-to-the-Mean Illustration (FHWA, 2010) Highway Safety Improvement Program Manual (Section 2.3).

Comparison Group Before-After Method

An alternative method for before-after studies is the comparison group method. This can be seen as a better option to the naïve before-after method since it does not assume that expected annual average crash rates in the after period will be the same as the observed annual average crash rates in the before period. This method uses a comparison group which is a group of sites that are similar to the site being treated. This group is used to calculate the expected annual average crash rate for the after period if the treatment were not implemented. This number is then compared with the actual observed crash rate to measure the increased or decreased safety of the study site.

Hauer (1997) indicates the two main assumptions that are involved in this method. The first is that the factors which affect the safety will change in exactly the same way for the study site and the comparison group sites from the before period to the after period. The second assumption is that as these various factors change from the before to the after period, their influence on the safety of the study site and comparison group sites is the same. However, these factors are hard to identify and understand. It is also difficult to isolate the factors' individual effect on the safety of the sites. The comparison group method helps to account for the changes in the factors without deep understanding and calculations regarding each factor's effects. The general form of the comparison group formulation is shown in equation 5.

$$N_{exp-a-t} = \frac{N_{obs-a-u}}{N_{obs-b-u}} \times N_{obs-b-t} \tag{5}$$

where:

$N_{exp-a-t}$ = number of expected crashes in the after period at the treated site

$N_{obs-a-u}$ = total number of observed crashes in the after period at the untreated comparison group sites

$N_{obs-b-u}$ = total number of observed crashes in the before period at the untreated comparison group sites

$N_{obs-b-t}$ = number of observed crashes in the before period at the treated site

This method can be a good alternative to the naïve approach; however, there is still room for improvement in order to most accurately predict the expected crashes for the after period. Hauler (1997) notes that as professionals are capable of greater calculations and understandings of the factors that affect safety, the comparison group method should decrease in use.

Empirical Bayes Before-After Method

The Empirical Bayes (EB) method and calculations are introduced and discussed in depth by Hauer (1997). Hauer's (1997) discussion introduces one data characteristic that factors into the safety of an entity include the traits of the individual drivers, i.e., age and gender, and the traits of the entity, i.e., rural, urban, number of lanes and more. Another available data characteristic is the "history of accident occurrence" for the entity. The data characteristics are used to estimate the safety of the entity. The first data type is used to calculate the "mean" to which the data is regressing toward. The second data type helps determine how much the expected number of accidents differs from the group mean. A reference population with similar characteristics provides necessary knowledge about the entity being studied. The data from the reference group is used in the EB calculations for the before period. The use of the reference group and the EB calculations counteract the regression-to-the-mean bias and create a more stable data foundation to be used in the formulations.

The EB method will also account for the factors that are likely to change over time, including traffic patterns, AADT, weather and driver behaviors, as mentioned before.

This is accounted for when the predicted number of accidents is calculated from the reference group data. Two methods are available for this calculation. One method that has been used frequently in before-after studies is a regression approach as suggested by Hauer. The data collected from the reference group sites can be analyzed and a regression fit to the data that will be used to calculate the predicted number of crashes for the before period. Many probability distributions are available for transportation data and have been used in regression analysis for before-after safety studies. A Gamma distribution can be used; or, if the accident count follows the Poisson distribution and the population expected number of accidents is Gamma distributed, then the negative binomial regression can be used in the EB calculations (Hauer, 1997; Ahmed et al., 2014). The Poisson distribution assumes the mean and variance are the same. This is not usually the case in the real world data collected for safety studies. Often, the variance is larger than the mean, showing the data is overdispersed. The negative binomial regression accounts for this overdispersion and has been used frequently in recent studies (Zhou et al., 2013; Schultz et al., 2010; Wu et al., 2015; Wang et al., 2011).

The other method used in calculating the predicted number of crashes in the before period is the use of a Safety Performance Function (SPF) provided in various sources including the Highway Safety Manual (HSM), FHWA Interchange Safety Analysis Tool (ISAT), and other empirical studies. The HSM is published by the American Association of State Highway and Transportation Officials (AASHTO) as a resource for transportation professionals in order to facilitate informed decision making. It contains the most current and innovative methods on safety performance and aims to increase the inclusion of safety parameters in roadway designs. The ISAT is a spreadsheet based tool used to assist transportation professional analyze the safety effects of proposed geometric designs and traffic measures (FHWA, 2007).

The HSM provides multiple SPFs for various road and intersection configurations including rural two-lane and two-way roads, intersections on rural two-lane and two-way, undivided and divided rural multilane highways, intersections on rural multilane highways, urban and suburban arterials roadway segments, intersections on urban and suburban arterials, freeway segments, speed-change lanes, ramp segments, collector-distributor roadways and ramp terminals (AASHTO, 2010).

Similar to the HSM, the ISAT provides SPFs for freeway mainline roadways, freeway interchange ramps, interchange crossroad segments and ramp terminals and intersections. Other empirical studies generally aim to develop and utilize SPFs for specific roadway types as well.

SPFs are generally based on the negative binomial distribution, which is better suited to modeling the high natural variability of crash data than traditional modeling techniques based on the normal distribution (AASHTO, 2010). One commonly selected independent variable for the SPF is the AADT or ADT with the dependent variable being crashes per mile per year (Zhou et al., 2013). These SPFs are calculated according to base conditions which are specified in their respective source material. The SPFs need to be calibrated for areas similar to the treatment sites in characteristics and location. Calibration is accomplished by applying crash modification factors (CMF) and calibration factors to the SPFs.

Data from a group of selected reference sites will be used for the calibration of the appropriate SPF. The reference group used, discussed in Chapter 3, is a much broader group of sites than a comparison group. The reference sites will vary more in variables such as the AADT, geometric characteristics and crash rates. This variation helps to correct the regression-to-the-mean bias (Ahmed et al., 2014). An evaluation study can be performed with fewer sites (recommended 10-20) or shorter time periods (recommended 3-5 years), or both, with the understanding that statistically significant results are less likely. A minimum of 30-50 selected reference sites is recommended. Crash frequencies at each site need not be considered. A buffer period of several months is usually allowed for traffic to adjust to the presence of the treatment (AASHTO, 2010).

The EB method is going to return a much more reliable and accurate measure of the change in safety due to the implementation of a roadway treatment. Calibration of the SPFs requires time and a fair amount of data for each study. Due to the data requirements, the EB method is limited to sites where all observed crash data, AADT and geometric data is available in the before period for all comparison group and study site locations. Chapter 5 will discuss the calculations necessary for this method.

CHAPTER 3

DATA COLLECTION

Two forms of data, i.e., AADT and crash counts, were used in this study, which were obtained from UDOT. The details regarding the selection of study sites as well as the collection process for crash counts and AADT will be discussed in this chapter.

STUDY SITE SELECTION

Currently, Utah has eight operational DDIs spanning from St. George to Brigham City; five of which have been selected for this study. The selected DDI study sites are shown in Table *4*. Selection of the DDI study sites is based on available data before and after the construction of the new DDIs. The use of three to five years of before and after data is recommended which limits the use of more recent DDIs in Utah due to the lack of after data. Before and after pictures of the selected study sites are shown in Appendix B.

Table 4
Selected DDI Study Sites.

Exit #	Interchange Location	City	Year Implemented	Before Years	After Years
278	I-15 & Main Street	American Fork	August 2010	3	4
284	I-15 & Timpanogos Hwy	Highland	August 2011	4	3
13	SR-201 & Bangerter Hwy	West Valley	October 2011	4	3
276	I-15 & 500 East	American Fork	November 2011	4	3
8	I-15 & St. George Blvd	St. George	November 2013	6	1

Comparison Group Site Selection

The EB before and after method involves the use of SPFs in the beginning calculations. Chapter 4 will discuss the calibration of safety performance functions using a group of urban diamond interchanges along I-15, SR-201, I-80 and I-215. All urban diamond interchanges along I-15 were selected with additional diamond interchange sites pulled from SR-201, I-80 and I-215 totaling 26 sites which are listed in Table *5*. These sites will be used in calibrating the SPFs employed in the EB analysis.

When comparing Table *4* and Table *5*, it can be seen that some of the interchanges that have been converted to DDIs are included in the list of sites used as the comparison group for the SPF analysis. It should be noted that only the data from before the DDI conversion was included in the sample data. The inclusion of the before data for any DDI locations for the SPF calibration does not affect the EB analysis or the integrity of the data set and analysis of this study.

Crash Count Data Collection

The crash count data was provided by the UDOT Traffic & Safety Division. Using the provided data, the appropriate route numbers and latitude and longitude coordinate ranges were selected for the interchanges in order to extract only the crashes that happened at each study site. The HSM defines an intersection related crash as occurring on any intersection approach within 250 ft from the center of the intersection

Table 5
Selected Diamond Interchanges for SPF Calibration.

Exit #	Road Name	Route #	Intersecting Highway	County
6	Bluff Street	SR-18	I-15	Washington
8	St. George Blvd	SR-34	I-15	Washington
13	Washington Parkway	FR-3153	I-15	Washington
62	Main Street - Cedar City	SR-130	I-15	Iron
273	1600 North	SR-241	I-15	Utah
275	Pleasant Grove Blvd	FR-2978	I-15	Utah
276	500 East	SR-180	I-15	Utah
278	Main Street	SR-145	I-15	Utah
282	1200 West	SR-85	I-15	Utah
284	Timpanogos Highway	SR-92	I-15	Utah
288	14600 South	SR-140	I-15	Salt Lake
305C	1300 South	FA-2290	I-15	Salt Lake
315	2600 South	SR-93	I-15	Davis
316	500 South	SR-68	I-15	Davis
319	Parrish Lane	SR-105	I-15	Davis
328	200 North	SR-273	I-15	Davis
331	Hill Field Road	SR-232	I-15	Davis
332	Antelope Drive	SR-108	I-15	Davis
334	700 South	SR-193	I-15	Davis
335	650 North	SR-103	I-15	Davis
341	31st Street	SR-79	I-15	Weber
343	21st Street	SR-104	I-15	Weber
344	12th Street	SR-39	I-15	Weber
349	2700 North	SR-134	I-15	Weber
113	5600 West	SR-172	I-80	Salt Lake
124	State Street	US-89	I-80	Salt Lake
125	700 East	SR-71	I-80	Salt Lake
11	5600 West	SR-172	SR-201	Salt Lake
23	700 North	FR-2354	I-215	Salt Lake

(AASHTO, 2010). This definition was applied in this project as shown in Figure *4*. Traditionally, the crossroad section more than 250 ft beyond the ramp terminal/intersection would not be included in the terminal; however, as this study is concerned with all areas affected by the implementation of the DDI, the crossroad section is included. Therefore, each terminal extends to the center of the crossroad section. Any crashes occurring within 250 ft of the ramp terminal and the crossroad section are assigned to the ramp terminal.

The route number and coordinate range sort was adequate to select the crashes occurring on the crossroad and at the ramp terminals at each interchange; however, the I-15 data was further sorted according to the "Roadway Type." For all crashes in Utah, UDOT has indicated which type of roadway the accident occurred on. All crashes within the desired

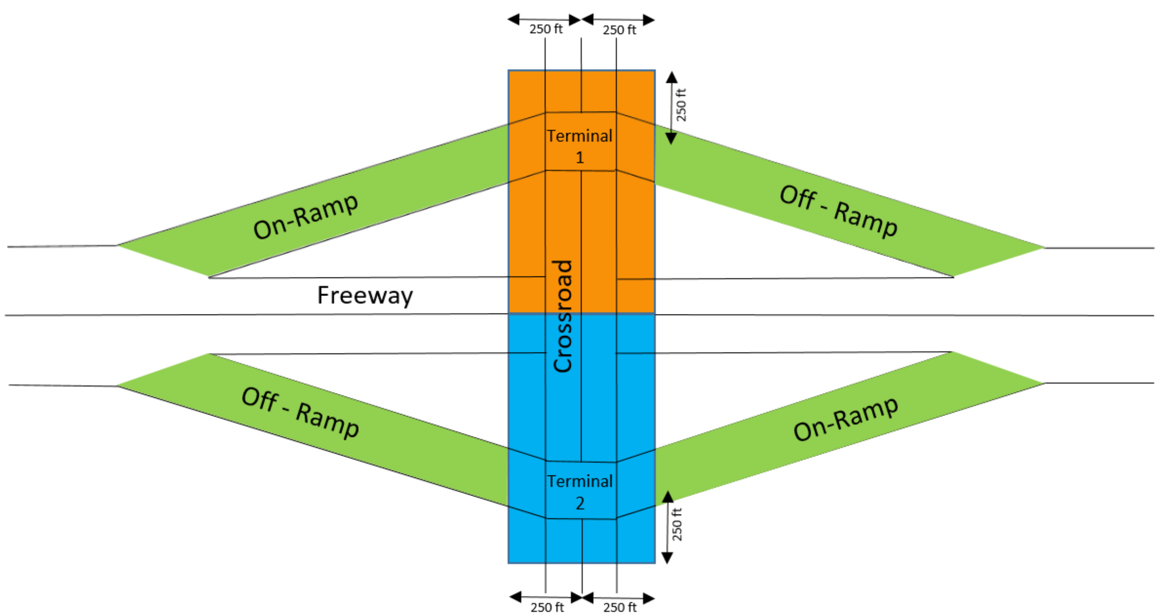

Figure 4
Crash Site Assignment Diagram.

route and coordinate range with an “R” roadway type designation, indicating a ramp segment, were selected for the study data set. These selections were mapped in ArcMap to verify the crashes were within the desired area.

The AADT for each crossroad was obtained from the UPlan UDOT Map Center accessed through the UDOT Data Portal. The AADT for the ramps at each interchange was acquired from UDOT. The data set was then converted into the appropriate format for the SPSS regression including the exit number, year, crossroad segment/ramp length obtained from ArcMap, AADT, and crash count. Once the formatting was completed, the data was ready for regression analysis in SPSS as discussed in Chapter 3. This data collection process is shown in the flow chart in Figure *5*.

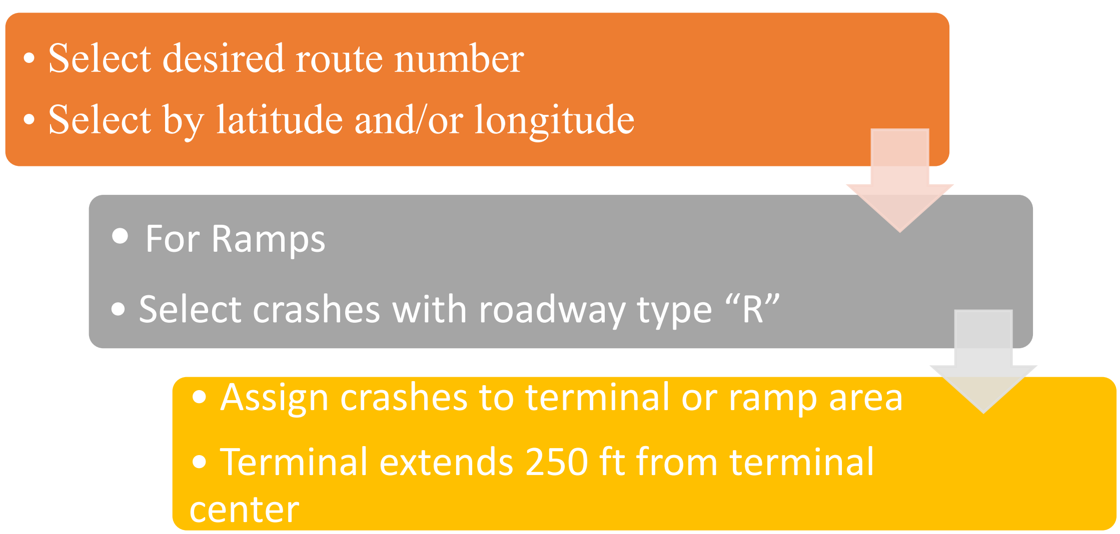

Figure 5
Comparison Group Data Scrubbing Process.

CHAPTER 4

SAFETY PERFORMANCE FUNCTION

In the transportation industry, it has become commonplace for the negative binomial regression to be used to model the crash data and formulate SPFs. The Poisson distribution, which is used frequently for modeling count data such as crash data, assumes the data's variance is equal to its mean. Crash data often experience a variance that is larger than the mean of the dataset causing the Poisson distribution to be inoperative. In the case where the variance exceeds the mean, also known as being overdispersed, the negative binomial distribution is used due to its ability to accommodate the larger variance. Crash data has been found to most frequently fall into the overdispersed-Poisson distribution lending itself to the negative binomial distribution. SPFs for the study site and the comparison group sites are used in the EB calculations, which will be discussed in Chapter 5.

Diamond Interchange Safety Performance Function Options

Multiple SPFs have been developed for specific roadway configurations. Three diamond interchange specific SPFs will be discussed in this section.

Highway Safety Manual

The HSM provides base SPFs that have been derived using a negative binomial regression based on data collected for various site types. Each function is to be used as a base equation with specified base parameters, including AADT and road segment length as well as other parameters (AASHTO, 2010). The appropriate function should be selected based on site type and should be adjusted to account for the differences between the base parameters and the actual characteristics of the study site. This adjustment is accomplished by applying crash modification factors (CMF) and a calibration factor to accommodate specific local settings.

As an example, equation 6 shows the SPF provided for a one-way stop controlled 4 leg diamond intersection. The SPF coefficients a,b,c, etc. are provided in the HSM and are specific to different factors such as crash type, crash severity and rural or urban area. The appropriate SPF and coefficients will need to be selected to match the factors of each site being studied. The CMF equations are given in the HSM for multiple site types. The CMFs are calculated similar to the SPFs and applied to the SPFs as in equation 7. The calibration factor calculation is shown in equation 8. The resulting value of equation 7 is the number of predicted crashes for the before period. It is important to note that the use of CMFs that are correlated or not fully independent from the others can cause an overestimation in their effect on the SPF through the combined modification (UDOT, 2011).

$$N_{spf,int,1\,way\,stop} = exp(a + b \times ln[c \times AADT_{xrd}] + d \times ln[c \times AADT_{ex} + c \times AADT_{en}]) \quad (6)$$

where

$a, b, c, \& \, d =$ coefficients provided in HSM

$AADT_{xrd} =$ AADT volume for the crossroad

$AADT_{ex}$ = AADT volume for the off-ramp intersection

$AADT_{en}$ = AADT volume for the on-ramp at the intersection

$$N_{pred-b} = N_{spf-b} \times \left(CMF_{1x} \times CMF_{2x} \times \ldots \times CMF_{yx}\right) \times C_x \qquad (7)$$

$$C_x = \frac{\sum N_{obs-b}}{\sum N_{spf-b}} \qquad (8)$$

where

N_{pred-b} = predicted number of crashes in the before period

N_{spf-b} = estimated number of crashes in the before period

CMF_{yx} = crash modification factor for design features y and specific site type x

C_x = calibration factor for each specific site type x

N_{obs-b} = number of observed crashes in the before period

Federal highway Administration

The Federal Highway Administration (FHWA) has developed an analysis tool to help professionals assess the safety effects of different roadway characteristics. The Interchange Safety Analysis Tool (ISAT) runs in Microsoft Excel and includes many applications including an SPF calculation function. As with the HSM, the ISAT provides predetermined SPFs which are also based on the negative binomial regression of data from selected base sites in California, Minnesota, Ohio and Washington (FHWA, 2007). Site-specific coefficients are given for the ISAT SPFs as they are in the HSM. Calibration is required for the ISAT SPFs to adjust the equation to be applicable to the specific site being studied.

When calculating the calibrated SPFs, the ISAT mentions two methods for selecting the years to be included in the analysis. The first method is to look only at the most recent year in which all the crash data is available. This would cause the SPFs to directly model after only the year of data used. The second method is to use up to ten years of the most recent data for the study sites for the calibration. This will model the trend of the crash data over the selected years chosen for calibration rather than only one year of data. Attributable to the random nature of crash data, one year of data may provide a skewed or abnormal representation of the crash trends at the location. Using more data will result in a more accurate estimation of the predicted number of crashes at the chosen location. The second method is recommended by the ISAT. Data for sites under construction during the selected analysis year should not be included as the construction activities could impact the crash rates and reflect an inaccurate safety impact of the treatment. Once the analysis period is determined, the number of crashes for the sites in the analysis period should be predicted using the appropriate SPFs. The calibration factor is determined using equation 9 and applied to the SPF as shown in equation 10.

$$C = \frac{N_{obs-b}}{N_{spf-b}} \tag{9}$$

$$N_{pred-b} = N_{spf-b} \times C \tag{10}$$

where

N_{obs-b} = number of observed crashes in the before period

N_{spf-b} = estimated number of crashes in the before period

N_{pred-b} = predicted number of crashes in the before period

SPF for Signalized Diamond Interchanges - Wang et al 2010

An additional study conducted by Wang et al. (2010) set out to develop an SPF for signalized diamond interchanges at ramp terminals, which resulted in the following SPF given in equations 11-14.

$$N_{spf-ramp\ terminal} = a \times VE^{b} \times exp(c \times Y_{dif} + d \times AR_{dif} + eRT_{dummy} + f \times S_{terminal}) \quad (11)$$

$$VE = AADT_{ramp1} \times AADT_{crd1} + AADT_{ramp2} \times AADT_{crd2} \quad (12)$$

$$Y_{dif} = Y_{obs} - Y_{ITE} = Y_{obs} - \left(T_{pr} + \frac{V_a}{2d_r + 2gG_r}\right) \quad (13)$$

$$AR_{dif} = AR_{obs} - AR_{ITE} = AR_{obs} - \left(\frac{S+L}{V_a}\right) \quad (14)$$

where

a, b, c, d & e are the parameters that will be estimated by the model

RT_{dummy} = dummy variable identifying the existence of an exclusive right turn phase on the off-ramp where1=right turn phase on either of the two off-ramps, 0=no right turn phase

L_{cr} = length of the crossroad segment between the two ramp terminals

$AADT_{ramp1}$ = AADT ramp volume of the first ramp at the project site

$AADT_{cr1}$ = AADT crossroad volume of the crossroad segment outside of the first ramp terminal

$AADT_{ramp2}$ = AADT ramp volume of the second ramp at the project site

$AADT_{cr2}$ = AADT crossroad volume of the crossroad segment outside of the second ramp terminal

Y_{dif} = difference between the yellow phase time of the intersection and the ITE recommended yellow phase time

Y_{obs} = observed yellow phase time at the intersection

T_{pr} = driver perception-reaction time; generally 1 second

V_a = vehicle's speed; posted speed limit is used

d_r = deceleration rate; generally 10 ft/s^2

g = gravitational acceleration; 32.2 ft/s^2

G_r = grade of the intersection approach, ft/ft

AR_{dif} = difference between the all-red phase time of the intersection and the ITE recommended all-red phase time

AR_{obs} = observed all-red phase time at the intersection

S = path length of the left turn curve, ft

L = vehicle length, 20 ft is used here

While this SPF is valid, it will not be used in this study for the following reasons. The study performed by Wang et al. (2010) considered the entire ramp terminal as a whole entity with one SPF for the study site. The HSM and ISAT SPFs look at each section separately, i.e., ramps and crossroad segments, with an SPF for each section type. The section SPF predictions are summed to provide the final predicted number of crashes at the ramp terminal. Also, this SPF includes the signal timing data which differs from the most common SPFs used in safety studies. It can be argued that the signal timing, specifically the length of yellow and all-red phases, could have an effect on driver behaviors and crash frequency; however, this study is not focusing on the effects of signal timing on crash rates. Collection of accurate signal timing at all sites for the before and after periods would be difficult to acquire.

The HSM and ISAT SPFs will be calibrated for use in the EB before-after method. The use of these two SPFs will substantiate the returned EB results.

SPF Calibration Analysis

As discussed previously in this chapter, the HSM and ISAT provide base SPFs as well as the predetermined parameters specific to different roadway configurations and various characteristics specific to a study site. It is prescribed in the HSM and ISAT that the appropriate coefficients be selected to match the characteristics of the site being studied. For this study, the parameters of the base SPFs from the HSM and the ISAT will be determined using a regression analysis which will lead to a more accurate estimation of expected crashes.

Using crash data sets from UDOT, as discussed in Chapter 3, the base SPFs for diamond interchanges found in the HSM and ISAT will be calibrated. Interchange SPFs are divided into ramps and crossroad terminals which will each be calibrated separately. This will provide an accurate, Utah-specific SPF fit to the crash patterns of urban diamond interchanges along Utah's freeways. The HSM and ISAT SPFs are shown in equations 15-16 and 17-18 respectively (AASHTO, 2010; FHWA, 2007).

$$N_{HSM\ ramp} = L \times exp\left(a + b \times ln\left[c \times AADT_{ramp}\right] + d \times \left[c \times AADT_{ramp}\right]\right) \tag{15}$$

$$N_{HSM\ terminal} = \exp[\mathrm{a} + \mathrm{b} \times \ln(\mathrm{c} \times AADT_{crossroad}) + d \times \ln(c \times AADT_{exit} + c \times AADT_{entrance})] \tag{16}$$

where

L = length of ramp

$AADT_{ramp}$ = AADT for the selected ramp

$AADT_{crossroad}$ = AADT for the crossroad

$AADT_{exit}$ = AADT for the freeway exit ramp entering the terminal

$AADT_{entrance}$ = AADT for the freeway entrance ramp leaving the terminal

$a, b, c,$ & d = parameters to be determined in regression analysis

$$N_{ISAT\ ramp} = e^{a} \times AADT_{ramp}{}^{b} \times RL^{e} \tag{17}$$

$$N_{ISAT\ terminal} = e^{a} \times AADT_{crossroad}{}^{b} \times AADT_{exit}{}^{c} \tag{18}$$

where

$AADT_{ramp}$ = AADT for the selected ramp

RL = ramp length

$AADT_{crossroad}$ = AADT for the crossroad

$AADT_{exit}$ = AADT for the freeway exit ramp entering the terminal

$a, b,$ & e = parameters to be determined in regression analysis

The data sample consists of crash data for the 2006-2014 period. The number of crashes were totaled for each year at each location. Each data point in the sample consists of the AADT and the length of each road segment as independent variables and the number of crashes as the dependent variable for one year at one location.

SPSS, a statistical analysis program, will be used to calculate the regressions for calibration. The regression function will fit a trend line to the provided data and determine the parameters of each defined independent variable. The standard form of a linear regression equation follows the format in equation 19.

$$Y_i = \beta_0 + \beta_1 x_{i1} + \beta_2 x_{i2} + \cdots + \beta_{ik} x_{ik} + \varepsilon_i \tag{19}$$

Where Y_i is the dependent variable, β_k is the parameter associated with each respective independent variable, x_{ik} is the independent variable, and ε_i is the error term. Due to the exponential components in the SPFs, the equations must be linearized into the form of equation 4.14 before the regression can be implemented. The linearization is performed by applying the natural log to the entire equation. The regression can then be run to estimate the unknown parameters in the SPFs. SPSS generates the output information including descriptive statistics, regression parameter results and significance measures, goodness of fit, and various other statistical analysis values. A brief summary of the regression output is provided in Table *6*. The full results can be found in Appendix A. With these output measures, the accuracy and validity of the regression can be checked. The goodness of fit measures should be reviewed to ensure a good fit and accurate estimations. The deviance divided by degrees of freedom (deviance/df) is a good indicator of the goodness of fit. If this value is close to one, either below or above the value of one, then the fit can be declared good. A goodness of fit measure too far above or below a value of one indicates the inability for the regression to accurately estimate parameters based on the given data. The regression software will provide parameter estimates with or without an acceptable goodness of fit measure. It is the user's responsibility to check this measure and deem the regression estimates valid or not. The statistical significance of the estimated parameters should be checked as well. For these parameters to be considered valid at a 95% confidence level, the parameter significance should be less than or equal to .05. If the significance values are below this threshold, the parameters are significant and can be used in the SPFs.

The estimated parameters provided by the SPSS regression will then be used to solve for the parameters indicated in the SPFs. With the parameters now known, the SPFs have been calibrated to diamond interchanges in urban freeway zones in Utah. These calibrated SPFs are shown in equations 20-31. As a crosscheck, the data was also analyzed using SAS, a statistical analysis program, with very similar results with negligible differences in parameter estimations, supporting the SPSS regression results.

$$N_{HSM\ ramp,total} = L \times exp\left(-11.477 + 1.466 \times ln\left[1 \times AADT_{ramp}\right] - (5.442 \times 10^{-5}) \times \left[1 \times AADT_{ramp}\right]\right) \quad (20)$$

$$N_{HSM\ ramp,pdo} = L \times exp\left(-13.311 + 1.66 \times ln\left[1 \times AADT_{ramp}\right] - (8.161 \times 10^{-5} \times \left[1 \times AADT_{ramp}\right]\right) \quad (21)$$

Table 6
Regression Analysis Results Summary.

			Goodness of Fit		Parameter Estimate Significance				
		Site Description	Deviance	Pearson Chi-Square	a	b	c	d	e
HSM	Ramp	Total Crashes	1.056	1.437	0.000	0.000	1.000	0.173	--
		Property Damage Only Crashes	1.546	2.281	0.000	0.000	1.000	0.018	--
		Injury/Fatality Crashes	0.697	1.329	0.001	0.002	1.000	0.209	--
	Terminal	Total Crashes	1.061	1.04	0.000	0.000	1.000	0.000	--
		Property Damage Only Crashes	1.089	1.022	0.000	0.000	1.000	0.001	--
		Injury/Fatality Crashes	1.137	1.033	0.000	0.000	1.000	0.006	--
ISAT	Ramp	Total Crashes	0.932	1.165	0.000	0.000	--	--	0.634
		Property Damage Only Crashes	0.947	1.239	0.000	0.000	--	--	0.864
		Injury/Fatality Crashes	0.593	1.043	0.000	0.000	--	--	0.541
	Terminal	Total Crashes	1.061	1.013	0.000	0.000	0.000	--	--
		Property Damage Only Crashes	1.09	0.997	0.000	0.000	0.000	--	--
		Injury/Fatality Crashes	1.131	1.025	0.000	0.000	0.000	--	--

$$N_{HSM\ ramp,injury/fatality} = L \times exp\big(-15.896 + 1.832 \times ln\big[1 \times AADT_{ramp}\big] - (8.155 \times 10^{-5}) \times \big[1 \times AADT_{ramp}\big]\big) \quad (22)$$

$$N_{HSM\ terminal,total} = \exp[-6.062 + .391 \times \ln(1 \times AADT_{crossroad}) + .451 \times \ln(1 \times AADT_{exit} + 1 \times AADT_{entrance})] \quad (23)$$

$$N_{HSM\ terminal,pdo} = \exp[-5.387 + .325 \times \ln(1 \times AADT_{crossroad}) + .411 \times \ln(1 \times AADT_{exit} + 1 \times AADT_{entrance})] \quad (24)$$

$$N_{HSM\ terminal,injury/fatality} = \exp[-9.866 + .692 \times \ln(1 \times AADT_{crossroad}) + .409 \times \ln(1 \times AADT_{exit} + 1 \times AADT_{entrance})] \quad (25)$$

$$N_{ISAT\ ramp,total} = e^{-8.875} \times {AADT_{ramp}}^{.979} \times RL^{-.117} \quad (26)$$

$$N_{ISAT\ ramp,pdo} = e^{-8.703} \times {AADT_{ramp}}^{.936} \times RL^{-.042} \quad (27)$$

$$N_{ISAT\ ramp,injury/fatality} = e^{-11.058} \times {AADT_{ramp}}^{1.061} \times RL^{-.208} \quad (28)$$

$$N_{ISAT\ terminal,total} = e^{-4.604} \times {AADT_{crossroad}}^{.414} \times {AADT_{exit}}^{.299} \quad (29)$$

$$N_{ISAT\ terminal,pdo} = e^{-3.833} \times {AADT_{crossroad}}^{.351} \times {AADT_{exit}}^{.243} \quad (30)$$

$$N_{ISAT\ terminal,injury/fatality} = e^{a} \times {AADT_{crossroad}}^{b} \times {AADT_{exit}}^{c} \quad (31)$$

With the goodness of fit and parameter significance checked and the individual unknowns solved for, these equations are now ready to be implemented in the EB calculations.

CHAPTER 5

BEFORE-AFTER SAFETY ANALYSIS METHODOLOGY

The Empirical Bayes before-after method involves a series of calculations which will determine the predicted and expected crash counts for the before and after periods of the study if the treatment was not implemented. These values are then compared to the observed crash counts to determine how the treatment affected the crash frequency at the study site. A decrease in crashes would indicate that the treatment was successful in increasing the safety of that site. Adversely, an increase in crash counts will show a negative effect on the safety of the site.

EMPIRICAL BAYES ANALYSIS

When performing the Empirical Bayes (EB) analysis for a study site, it is necessary to determine whether the study site will be viewed at a project level, including the entire on-ramp/off-ramp terminal as one entity, or at a site-specific level with differentiable site types that will be summed together. This will depend on the data available for the site being studied (AASHTO, 2010). If a single rural or urban highway segment is being studied that has no exits, entrances, or intersections, the level of analysis performed will not affect the calculations; because, there is only one site type in the whole project. In this study, a site-specific analysis will be performed on diamond interchanges at ramp terminals. This site can be broken down into the following site types:

- On-ramps, typically one in each direction
- Off-ramps, typically one in each direction
- Ramp terminal intersections, one at each entrance/exit pair
- Crossroad segments

It is important to make this distinction before the process begins as it effects the selection of SPFs and data required. At the site-specific level, crash data, AADT, and other included factors will need to be detailed enough to assign each reported accident to the appropriate site type within the project. If this detailed data is not available, the analysis will need to be performed at the project level.

The lengths of the before and after periods will also need to be predetermined. The before and after periods need not be the same length. The before period must be the same for each study site, and the after periods need to be the same length for each study site as well. Periods should not include times when construction was being performed at the selected study sites.

The EB analysis that will be used in this study comes from the HSM recommended method (AASHTO, 2010) and employs a number of calculations in multiple steps to determine the effectiveness of the implemented treatment being studied. The general flowchart for these steps is shown in Figure *6* followed by a description of each step.

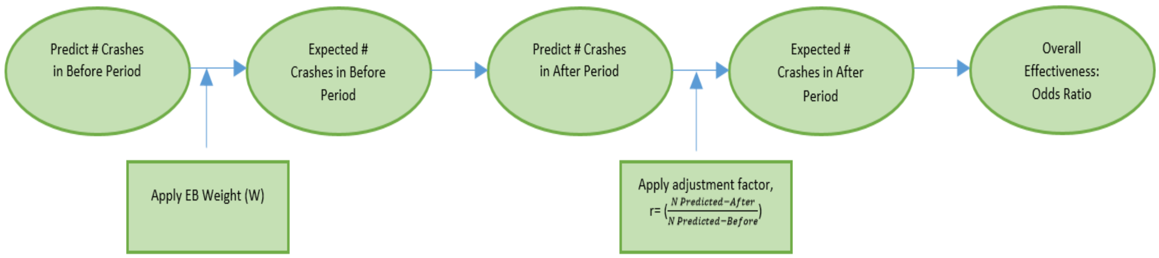

Figure 6

Empirical Bayes Method Flow Chart.

Step 1 – Predicted Number of Crashes for the Before & After Periods

As mentioned earlier in this chapter, the SPF is used as the base point in the EB method. Once the site types are determined, the SPFs can be selected. The SPF is applied to the data collected for the before and after periods and the predicted number of crashes for each site is returned. SPFs provided by the HSM, ISAT or any other source will only be base models or models based on factors that may vary from one state or location to another. The differences between the SPF bases and the study sites can cause major discrepancies. In order to account for these differences, the SPFs need to be adjusted and calibrated. There are many different ways to calibrate an SPF as mentioned earlier in this section. It is important to calibrate the selected SPF the correct way as suggested by the source of the SPF. The general calibration approaches for the HSM and ISAT SPFs are mentioned in the respective sections in Chapter 4. If a site-specific SPF is modeled using data from the actual study sites and local comparison groups, the SPF does not need to be calibrated. The SPFs used in this study were calibrated using Utah specific comparison group data.

Calculations can be performed for each separate year at each site. The predicted values are summed over the before and after periods in order to get the total number of predicted crashes for each period respectively.

Step 2 – Overdispersion Parameter

When using the HSM SPF, the overdispersion parameter is provided specific to each SPF. The ISAT does not provide this parameter. This study will use the regression data to calculate the data specific overdispersion parameter. It is common in the field of statistics to use the Pearson Chi-Square/degrees of freedom as the overdispersion parameter; therefore, this value will be used in this study.

Step 3 – Empirical Bayes Weight Factor

The EB weight factor is used to apply different weights to the predicted and observed number of crashes. The assigned weight depends on the predicted number of crashes in the before period and the overdispersion parameter from the negative binomial regression model. This calculation is shown in equation 32. This number will range between 0 and 1. A weight close to 1 indicates the predicted number of crashes for the before period is close to the actual mean number of crashes of the comparison group. A weight close to 0 indicates the expected number of crashes will be close to the observed number of crashes in the before period (Hauer, 1997).

$$w_b = \frac{1}{1+k\sum N_{pred-b}} \quad (32)$$

where

w_b = weight used in the Empirical Bayes method

k = dispersion parameter

N_{pred-b} = predicted number of crashes in the before period

Step 4 – Expected Number of Crashes for the Before Period

The expected number of crashes for the before period is calculated using a combination of the predicted number of crashes in the before period and the observed number of crashes in the before period as shown in equation 33.

$$N_{exp-b} = w_b \times N_{pred-b} + (1 - w_b) \times N_{obs-b} \tag{33}$$

where

N_{exp-b} = expected number of crashes in the before period

w_b = weight used in the Empirical Bayes method

N_{pred-b} = predicted number of crashes in the before period

N_{obs-b} = number of observed crashes in the before period

Step 5 – Adjustment Factor

A ratio is used to adjust for the variance between the predicted number of crashes in the before and after periods shown in equation 34. This will account for the differences in period duration and AADT between the periods (AASHTO, 2010).

$$r = \frac{\sum N_{pred-a}}{\sum N_{pred-b}} \tag{34}$$

where

$r =$ adjustment factor

$\sum N_{pred-b}$ = sum of predicted number of crashes for all years in the before period

$\sum N_{pred-a}$ = sum of predicted number of crashes for all years in the after period

Step 6 – Expected Number of Crashes in the After Period

The expected number of crashes for the after period can be calculated by applying the adjustment factor to the expected number of crashes that was calculated for the before period as shown in equation 35. The adjustment factor will either increase or decrease the expected number of crashes from the before period based on the ratio between the predicted number of crashes for the before and after periods.

$$N_{exp-a} = N_{exp-b} \times r \tag{35}$$

where

N_{exp-a} = expected number of crashes in the after period

N_{exp-b} = expected number of crashes in the before period

$r =$ adjustment factor

Step 7 – Estimated Effectiveness of Treatment for Each Site

The calculated expected number of crashes in the after period if the treatment were not implemented is compared to the observed number of crashes with the implemented treatment. This will show the change in crash counts from what would have been observed without the treatment and give the effect of the treatment on the safety conditions of the roadway. This is done by calculating the odds ratio shown in equation 36 for each site individually. This value shows the effectiveness of each site individually.

$$OR_i = \frac{N_{obs-a,i}}{N_{exp-a},i} \tag{36}$$

where

OR_i = increase or decrease in crashes due to the treatment at site i

$N_{obs-a,i}$ = number of observed crashes in the after period at site i

$N_{exp-a,i}$ = expected number of crashes in the after period at site i

Step 8 – Safety Effectiveness

Using equation 37, the effectiveness of the total location can be measured.

$$OR = \frac{\sum N_{obs-a}}{\sum N_{exp-a}} \tag{37}$$

where

OR = odds ratio

$\sum N_{obs-a}$ = sum of number of observed crashes in the after period for all sites

$\sum N_{exp-a}$ = sum of number of expected crashes in the after period for all sites

Step 9 – Adjusted Odds Ratio: Unbiased Safety Effectiveness

The HSM points out that the value found in equation 37 could be bias and needs to be adjusted resulting in an unbiased effectiveness value for the treated site. Equations 38 and 39 show this calculation.

$$OR_{adj} = \frac{OR}{1 + \frac{Var\left[\sum N_{exp-a}\right]}{\left(\sum N_{exp-a}\right)^2}} \tag{38}$$

$$Var\left[\sum N_{exp-a}\right] = \sum\left[(r)^2 \times N_{exp-b} \times (1 - w_b)\right] \tag{39}$$

where

OR_{adj} = adjusted increase or decrease in crashes due to the treatment for the sum of all sites

OR = odds ratio, value obtained from equation 37

$\sum N_{exp-a}$ = sum of number of expected crashes in the after period for all sites

r = adjustment factor

$\sum N_{exp-b}$ = sum of number of expected crashes in the before period for all sites

w_b = weight used in the Empirical Bayes method

Step 10 – Safety Effectiveness as a Percent

The calculation in equation 40 returns the percent improvement in number of crashes for each study location.

$$Safety\ Effectiveness = 100 \times \left(1 - OR_{adj}\right) \tag{40}$$

where

OR_{adj} = adjusted odds ratio, from equation 38

The variance and standard error of the odds ratio from equation 38 can be calculated. The resulting odds ratio standard error can be used to calculate the standard error of the safety effectiveness. Finally, the safety effectiveness is divided by the standard error of the safety effectiveness with the absolute value of this quotient providing the statistical significance of the safety effectiveness value.

Crash Modification Factor Construction

Once the EB analysis has been completed, creating a crash modification factor is relatively straightforward. The FHWA explains the methodology in creating the CMF for various before-after approaches including the comparison group and EB analysis, as well as other study circumstances. The results from the above EB analysis will be used in conjunction with the FHWA guide in order to develop the DDI specific CMF. Equation 41 exhibits the required calculation for creating the CMF (FHWA, 2010). Equations 42 through 44 show the CMF variance, standard error, and confidence interval calculations respectively.

$$CMF = \left(\frac{N_{observed,A}}{N_{expected,A}}\right) / \left(1 + \left(\frac{Var(N_{expected,A})}{N_{expected,A}{}^{2}}\right)\right) \tag{41}$$

$$CMF\ Variance = \left(CMF^2 * \left[\left(\frac{1}{N_{observed,A}}\right) + \left(\frac{Var(N_{expected,A})}{N_{expected,A}{}^{2}}\right)\right]\right) / \left(1 + \frac{Var(N_{expected,A})}{N_{expected,A}{}^{2}}\right) \tag{42}$$

$$CMF\ Standard\ Error = \sqrt{CMF\ Variance} \tag{43}$$

$$CMF\ 95\%\ Confidence\ Interval = CMF \pm 1.96 * CMF\ Standard\ Error \tag{44}$$

CHAPTER 6

RESULTS AND DISCUSSION

An EB before-after analysis was applied to the collected data for the selected DDIs in Utah as specified in the Study Site Selection section. The analysis results are shown in Table *7*. The effectiveness shows the percent of change that resulted after the implementation of the DDI structure. Following the guidelines and values provided in the HSM, the significance of each safety effectiveness value was calculated to determine if the result is statistically significant. A value less than 1.7 indicates insignificance of the effectiveness indicating the effectiveness of the treatment at that site is inconclusive. A significance value of 1.7 or greater indicates significance at a 90% confidence level; significance of 2 or greater indicates significance at a 95% confidence level which are bolded in Table *7*.

The data was analyzed on three different levels including total crashes, property damage only (PDO) crashes, and injury and fatality crashes. Within each level, the HSM and ISAT SPFs were applied to each individual terminal and ramp at each study site. The data was also summed across all study locations for each road type at the three levels with results showing in the "all sites combined" column in Table *7*. The terminal results returned positive safety effectiveness values with a large number of the results being significant. Overall, the ramp results were not as positive with most being insignificant. Some ramps did see positive significant improvements and some positive insignificant improvements. If no crashes were observed in the after period, the analysis returned a

100% safety effectiveness value. This did not occur at any of the terminals; though, quite a few ramps did return this result. It is important to note that all negative results reported in Table *7* are statistically insignificant. These negative results could indicate areas of concern which could benefit from further studies; however, the insignificant negative result is not condemning to the study location. The results are mostly consistent between the HSM and ISAT analyses; however, some locations do differ more than the others.

When comparing the road type results at each study location, as well as looking at the combined results of terminals and ramps respectively, the results show greater reduction in crashes for injury/fatality across all study locations with the exception of exit 284. This large decrease in the number of injury/fatality crashes is a very promising effect of the DDI implementation. As UDOT aims for "zero fatalities," the DDI can be seen as a positive aid in this effort.

A project level analysis was also conducted on the data. In the event that crash data is not specific enough to be assigned to each individual road segment at the location, the HSM advises the use of the project level EB analysis rather than the site specific analysis presented above (AASHTO, 2010). This approach looks at the entire interchange or study site as one entity instead of breaking up each road type segment to be analyzed individually. The HSM emphasizes the inability to determine if the roadway segments are statistically independent of each other or completely correlated when analyzing the interchange as a whole; therefore, an average of these two extremes is used in calculating the expected number of crashes in the before period and is used in the EB equations as listed in Chapter 5. The results of the project level analysis are presented in

Table *8* showing positive results at most of the study locations. Due to the nature of the project level calculations, it is not possible to calculate the significance of the results. Exits 284 and 13 had a mix of negative and positive results. As reported in the site specific analysis, the largest percent safety effectiveness results were seen in the injury/fatality crashes in both the HSM and ISAT analysis. Both the site specific and project level analyses provide positive results in the improvement of safety levels at locations with DDI implementation.

As noted in Chapter 3, exit 13 was constructed recently enough that only one year of after data was available. The negative results at this location could be attributed to this lack of available data. It would be interesting to analyze this location again in a few years with more data to obtain more significant results.

In depth research into why some locations would see better or worse results from DDI implementation including causes of increased crashes and insignificant results could also be studied. For example, in this study the EB analysis concluded that Exit 284 had a negative safety improvement. This location happened to be the only location with the DDI as an underpass under I-15. Is the location of the DDI the cause for the negative improvement? Or are there other factors contributing to the negative result? Are there incorrect or ineffective geometric designs at the DDI? Is there a rapid increase in AADT due to increased businesses in the area? Are construction projects in surrounding areas affecting traffic through the DDI? There are many events that could affect the crash frequency and before-after study results. Further research into these questions could lead to a deeper understanding of the safety effects of this interchange design.

Crash modification factors were also calculated as discussed in Chapter 5. The site specific and project level crash modification factors are reported in Table 9 and Table 10 respectively.

As a whole, the implementation of the DDIs in Utah has resulted in a positive improvement in crash occurrence at these locations. Each interchange has varying results with some showing great improvement in crash frequency and others with insignificant safety effectiveness results. These insignificant results are not to be seen as negative results of the DDI implementation but are merely inconclusive on the effectiveness of the DDI at the given location.

Table 7

Site Specific Empirical Bayes Before-After Results - % Safety Effectiveness.

			HSM						ISAT					
	Road Type	Direction	Exit 8	Exit 276	Exit 278	Exit 284	Exit 13	All Sites Combined	Exit 8	Exit 276	Exit 278	Exit 284	Exit 13	All Sites Combined
Total Crashes	Terminal	E/S	**64.55**	**73.23**	**59.93**	**45.02**	25.66	**49.65**	**63.21**	**70.96**	**56.15**	**44.45**	22.81	**46.52**
	Terminal	W/N	-1.86	**86.27**	**87.21**	-30.62	1.06		-4.33	**84.33**	**86.37**	-36.71	-1.64	
	Ramp	EB/SB Off	100.00	0.56	100.00	33.63	3.31	**33.96**	100.00	5.95	100.00	29.88	-17.01	**25.60**
	Ramp	EB/SB On	100.00	100.00	43.61	**69.52**	**62.03**		100.00	100.00	8.11	**63.66**	28.16	
	Ramp	WB/NB Off	-73.39	**69.94**	-6.11	-109.15	**59.14**		-99.01	**69.49**	-53.41	-68.35	**64.10**	
	Ramp	WB/NB On	100.00	**51.37**	100.00	-79.69	13.31		100.00	**63.14**	100.00	-121.98	-15.70	
PDO Crashes	Terminal	E/S	**58.41**	**72.94**	37.50	**52.22**	2.94	**35.96**	**56.69**	**70.48**	30.25	**51.66**	-1.23	**32.04**
	Terminal	W/N	-21.70	**84.78**	**88.71**	-79.10	-11.98		-24.98	**82.39**	**88.14**	-88.61	-15.20	
	Ramp	EB/SB Off	100.00	-25.41	100.00	55.78	34.17	23.87	100.00	-23.36	100.00	51.82	11.70	10.39
	Ramp	EB/SB On	100.00	100.00	53.55	**58.90**	**54.88**		100.00	100.00	16.42	50.04	18.47	
	Ramp	WB/NB Off	100.00	57.75	-22.63	-196.53	52.59		100.00	53.14	-87.53	-112.22	**56.96**	
	Ramp	WB/NB On	100.00	13.29	100.00	-32.06	8.61		100.00	35.65	100.00	-67.57	-32.32	
Injury/ Fatality Crashes	Terminal	E/S	**78.06**	**75.03**	**82.57**	26.43	**53.96**	**67.90**	**77.59**	**74.64**	**83.15**	25.47	**52.15**	**66.81**
	Terminal	W/N	32.32	**87.11**	**85.69**	42.92	40.67		32.31	**86.72**	**84.38**	42.77	41.03	
	Ramp	EB/SB Off	100.00	60.31	100.00	12.87	-44.89	**50.45**	100.00	54.36	100.00	-3.15	-86.34	**41.63**
	Ramp	EB/SB On	100.00	100.00	59.62	100.00	100.00		100.00	100.00	16.17	100.00	100.00	
	Ramp	WB/NB Off	-221.38	100.00	20.85	7.55	**64.73**		-275.79	100.00	-20.40	29.66	**65.69**	
	Ramp	WB/NB On	100.00	**76.88**	100.00	-115.34	100.00		100.00	**81.19**	100.00	-247.70	100.00	

*Bold denotes results significant at 95% confidence interval

Table 8
Project Level Empirical Bayes Before-After Results - % Safety Effectiveness.

	HSM			ISAT		
Exit	**Injury/ Fatality** % Safety Effectiveness	**PDO** % Safety Effectiveness	**Total** % Safety Effectiveness	**Injury/ Fatality** % Safety Effectiveness	**PDO** % Safety Effectiveness	**Total %** Safety Effectiveness
8	46.22	26.76	34.52	44.89	23.00	30.12
276	79.36	65.09	70.85	79.76	63.72	69.77
278	70.05	56.71	62.26	68.15	52.22	57.80
284	23.66	-11.24	1.59	23.38	-15.52	-3.23
13	43.95	-12.49	6.61	40.68	-21.57	-2.10
Total	**56.57**	**23.27**	**35.84**	**55.11**	**18.02**	**30.75**

Table 9
Site Specific Crash Modification Factors.

	Road Type	HSM	ISAT
Total Crashes	Terminal	0.50	0.53
	Ramp	0.66	0.74
PDO Crashes	Terminal	0.64	0.68
	Ramp	0.76	0.90
Injury/Fatality Crashes	Terminal	0.32	0.33
	Ramp	0.50	0.58

Table 10
Project Level Crash Modification Factors.

	HSM	ISAT
Total Crashes	0.64	0.69
PDO Crashes	0.76	0.82
Injury/Fatality Crashes	0.43	0.44

CHAPTER 7

EVALUATING PEDESTRIAN & CYCLIST SAFETY IN DDIS

The DDI is an effective tool to increase capacity at unbalanced interchanges as well as decreasing crossing points resulting in increased safety for vehicles traveling through the interchange. While vehicles will compose the majority of the users of an interstate interchange, pedestrian and cyclist users also need to be considered in the design and implementation of a DDI.

Pedestrians naturally follow the walkway provided at the interchange; however, cyclists, based on their level of comfort with traveling with vehicles, can either follow the provided pedestrian walkway or choose to travel in the vehicle lanes. In this discussion, it will be assumed that the cyclists will follow the provided walkway with pedestrians (UDOT, 2014).

Pedestrian and cyclist walkways can be placed in one of two different locations within the DDI. The walkways can either cross the turn lanes and run along the outside of the interchange or cross the turn lanes and then the through lanes with the walkway running through the middle of the interchange. The center and outside walkway options are shown in Figure *7* (UDOT, 2014).

In either the center or outside walkway configurations, if the right turning lanes are unsignalized, precautions should be taken to increase the safety of the pedestrians at these crossing points. FHWA recommends a lower vehicle speed, increased sight distance with respect to the crosswalk and a pedestrian signal or other lighted warning system implementation could be warranted (FHWA, 2014). Pros and cons of the center and outside pedestrian and bicycle walkways, provided by FHWA in the DDI Information Guide, are shown in Table *11* and Table *12* respectively (FHWA, 2014).

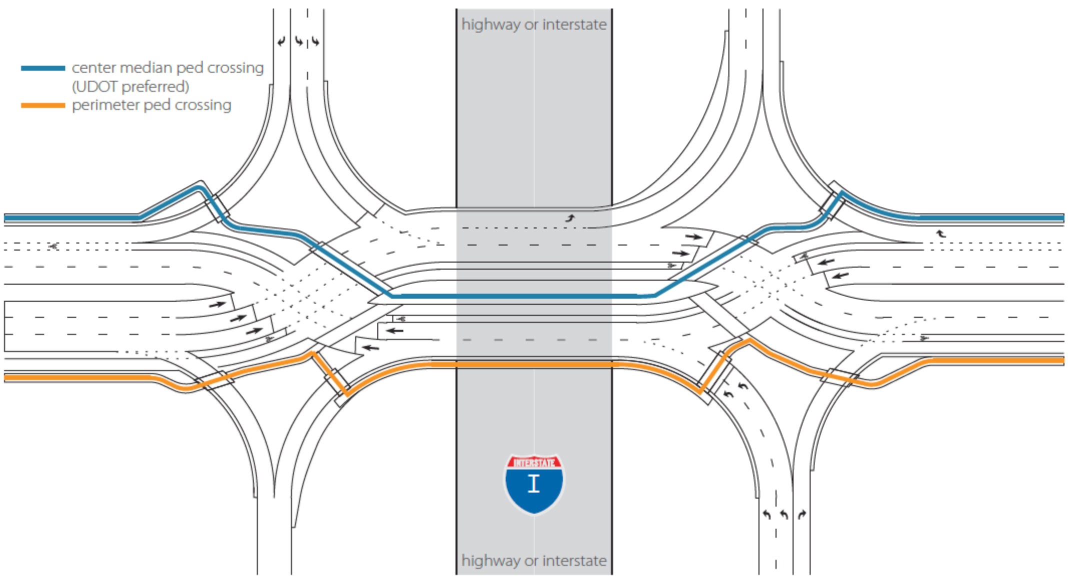

Figure 7
Center and Outside Pedestrian and Bicycle Walkways - UDOT DDI Guideline (UDOT, 2014).

The outside walkway configuration does not allow for pedestrians and cyclists to cross the crossroad at the DDI interchange. Pedestrians and cyclists would need to cross at the intersections before or after the DDI. The center walkway allows the pedestrian or cyclist to begin and end on either side of the crossroad (FHWA, 2014).

The DDI signal phases allow for longer green times which can accommodate more pedestrians and cyclists and provide longer time to cross the street at each crossing point (Chlewicki, 2003).

One large risk to pedestrians and cyclists traveling through the DDI is the unsignalized movement across the turn lanes on either end (FHWA, 2014). Pedestrians

Table 11

Center Pedestrian & Bicycle Walkway Pros & Cons.

	Advantages	Challenges
Street Crossings	Crossing of the arterial street provided at DDI for full pedestrian access	Crossing of free-flow right-turn movements to/from freeway
	Crossing one direction of traffic at a time	Pedestrians may not know to look to the right when crossing to center
	Short crossing distances	Wait at center island dictated by length of signal phase for through traffic
	No exposure to free-flowing left turns to freeway	Location of pedestrian signals can conflict with vehicular signals at crossovers
	Protected signalized crossing to walkway	
	Pedestrian clearance time generally provided in crossover signal phasing	
	Pedestrian delay to center minimized by short cycles at two-phase signals	
Walkway Facility	Side walls provide a positive barrier between vehicular movements and pedestrians	Center walkway placement counter to typical hierarchy of street design
	Walls low enough to avoid "tunnel" effect that could impact pedestrian comfort	Potential discomfort from moving vehicles on both sides of walkway
	Recessed lighting can provide good illumination of walkway	Sign and signal control clutter

and cyclists cross only one direction of traffic in a single phase resulting in shorter crossing distances allowing shorter phases (FHWA, 2014).

Chilukuri et al. (2011) administered online surveys to motorists regarding the DDI in Missouri at I-44 & Route 13 to determine the public perception of the DDI. Results showed that about 79% of those surveyed replied that the pedestrian and bicycle center walkway was easy to navigate or similar to other existing interchange configurations. Of those surveyed, 53% replied that the center walkway seemed safer than the outside walkway with another 28% replying that the outside walkways were safer. In addition to the motorist surveys, two professionals with experience in planning design and operation of pedestrian and bicycle facilities were interviewed by Chilukuri et al. (2011) about the DDI. Some of the main points of the interview include:

- Walkway path is easy to understand after first use
- Mixing pedestrians and cyclists on the same walkway could be an issue with higher volumes; however, it is acceptable for current traffic volume
- Crossing is safe at the signalized crossing points, right turn lanes are not always signalized which could create safety concerns
- Channeling of the center walkway has an increased safety level

Table 13 shows the before and after existence of pedestrian and bicycle walkways at the DDI locations selected for this study. Figure 8 through Figure 12 show images of center and outside walkways at Utah DDIs.

Table 12
Outside Pedestrian & Bicycle Walkway Pros & Cons.

	Advantages	Challenges
Street Crossings	Crossing one direction of traffic at a time	Crossing of free-flow right-turn movements to/from freeway
	Ramp crossing distances are often shorter than through traffic crossing distance due to fewer travel lanes	Conflict with free-flow left turns to freeway, where fast vehicle speeds are likely (acceleration to freeway)
		Crossing of the arterial street sometimes not provided at DDI
		Potential sight obstruction of pedestrian crossing left turns from behind barrier wall
		Pedestrians may not know which direction to look in, when crossing turn lanes
		Unnatural to look behind to check for vehicles before crossing when traveling out of the DDI (depends on angle of approach and direction of travel)
		Signalized crossings require more complicated timing
Walkway Facility	Extensions of existing pedestrian network (natural placement on outside of travel lanes)	Need for widened structure on outside for overpass
	Pedestrian typically has view of path ahead (depends on sight lines and obstructions)	Potential for additional right-of-way for underpass or construction of retaining wall under bridge
	Walkway does not conflict with center bridge piers (at underpass)	Need for additional lighting for underpass
	Opportunity to use right-of-way outside of bridge piers (at underpass)	

Table 13
Before & After Walkway Existence at DDI Study Sites.

Exit	Walkway Present Before DDI	Walkway Present After DDI
8	No	Yes (center)
276	Yes (North side)	Yes (outside - North & South)
278	Yes (North side)	Yes (outside - North & South)
284	No	Yes (outside - South side only)
13	No	No

Figure 8
St. George (Exit 8) DDI Center Walkway Aerial (ESRI ArcMap Imagery Basemap).

Edara et al. (2003) performed a simulation using VISSIM to analyze the performance of the DDI in regards to pedestrians. The simulation also studied other performance aspects of the DDI and the double crossover intersection (DXI). The pedestrian simulation results showed an average of 1.6 required stops for the pedestrian

Figure 9
St. George (Exit 8) DDI Center Walkway Crossing Point (Google Maps).

with an average delay of 35.5 sec/ped. The simulation indicated an average walk time of 39 seconds with an average pedestrian level of service C. The DDI was able to accommodate pedestrians into the existing signal phasing with minimal delay.

With the introduction of new DDIs, pedestrians and cyclists may elect a different route from origin to destination in order to avoid the new interchange. If pedestrians and cyclists change their travel patterns, crashes may occur on roads and intersections surrounding the location of the new roadway resulting in lower accident rates at the treated site and increased accident rates at adjacent and surrounding roads. This phenomenon is referred to as crash or accident migration (Maher, 1990). The safety

Figure 10
St. George (Exit 8) DDI Center Walkway (Google Maps).

Figure 11
American Fork Main Street (Exit 278) DDI Outside Walkway Aerial (Google Maps).

Figure 12
American Fork Main Street (Exit 278) DDI Outside Walkway (Google Maps).

effects of pedestrians and cyclists cannot be analyzed in this report due to lack of adequate data. It would be beneficial for future studies to be conducted to determine the impact of the DDI on pedestrians and cyclists. Data for crashes involving vehicles with pedestrians or cyclists are readily available; however, crashes involving pedestrians and cyclists without a motorized vehicle are not available. Another major limiting factor is the lack of pedestrian and cyclist volumes. For future studies, intentional volume and non-motorized crash data collection would be necessary for any statistically sound analysis.

CHAPTER 8

CONCLUSION

This study analyzed crash data at five locations along the I-15 corridor and SR-201 which had been converted from traditional diamond interchanges to DDIs. The EB before-after method, using the HSM and ISAT SPFs, was applied to the selected locations in order to provide a statistical analysis of the increase or decrease of crashes at the location since the DDI conversion. The crash data was analyzed at three levels including all crashes, property damage only crashes and fatality and injury crashes. The percent safety effectiveness results returned positive safety impacts at most study locations. Other locations resulted in insignificant negative percent safety effectiveness, which could be cause for concern but do not condemn the performance of the DDI at the given location. Injury and fatality crashes observed the greatest decrease in crashes after the DDI implementation.

As discussed in Chapter 7, another major safety concern in the DDI involves non-motorized traffic. It would be beneficial if the EB method could be applied to pedestrian and cyclist involved crashes. This would require a long term study that would include the collection of detailed pedestrian and cyclist data specifically AADT, crashes involving vehicles as well as crashes not involving motorized vehicles.

Other future studies are also recommended to continue the analysis of the safety effects of the DDI. Additional after data at DDIs across the United States will provide more comprehensive safety improvement performance measures.

REFERENCES

Ahmed, M., Abdel-Aty, M., Park, J., 2014. Evaluation of the Safety Effectiveness of the Conversion of Two-lane Roadways to Four-Lane Divided Roadways: Bayesian vs. Empirical Bayes. Proc., 94th Annual Meeting of the Transportation Research Board, Washington, DC.

American Association of State and Highway Transportation Officials (AASHTO)., 2010. Highway Safety Manual. Washington, DC.

ATS/American., The Official Website of the DDI – A Diamond Interchange with a Twist. www.divergingdiamond.com (accessed June 11, 2016).

Auger, J., Cloutier, M., and Morency, P., 2014. A Built Environment for an Ageing Society: A subpopulation analysis of pedestrian crashes at signalized intersections in Montreal, Canada. Proc., 94th Annual Meeting of the Transportation Research Board, Washington, DC.

Chewlicki, G., 2003. New Interchange and Intersection Designs: The Synchronized Split-Phasing Intersection and the Diverging Diamond Interchange. Proc. 2nd Urban Street Symposium, Anaheim, CA.

Chilukuri, V., Siromaskul, S., Trueblood, M., and Ryan, T., 2011. Diverging Diamond Interchange Performance Evaluation (I-44 & Route 13). OR11-012, Missouri Department of Transportation, Jefferson City, MO.

De Pauw, E., Daniels, S., Brijs, T., Hermans, E., and Wets, G., 2012. Redesigning Black Spots in Traffic: An Effective Evaluation. Transportation Research Board, Washington, DC.

DDI Guideline: A UDOT Guide to Diverging Diamond Interchanges, 2014. Utah Department of Transportation, Salt Lake City, UT.

Edara, P., Bared, J., Jagannathan, R., Diverging Diamond Interchange and Double Crossover Intersection – Vehicle and Pedestrian Performance. researchgate.net (accessed March 21, 2016)

Hauer, E., 1997. *Observational Before–After Studies in Road Safety*. Pergamon Press, Oxford, UK.

Highway Safety Improvement Program Manual, 2010. FHWA-SA-09-029, Federal Highway Administration, Washington, DC.

Interchange Safety Analysis Tool (ISAT): User Manual, 2007. FHWA-HRT-07-045, Federal Highway Administration, McLean, VA.

Interstate 435 and Front Street Interchange—Practical Design Solution, Presented by the Missouri Department of Transportation.

MoDOT, 2010. Missouri's Experience with a Diverging Diamond Interchange: Lessons Learned. OR 10-021, Missouri Department of Transportation, Jefferson City, MO.

New York State Department of Transportation (NYSDOT). Traffic Data.

Persaud, B., Lyon, C., 2007. Empirical Bayes before–after safety studies: lessons learned from two decades of experience and future directions. Accident Analysis & Prevention, 39, 546–555.

Poorbaugh, J. and Houston, B. 2006. *Diverging Diamond Interchange*, Institute of Transportation Engineers, Washington, DC.

Safety Performance Functions for Ramp Terminals at Diamond Interchanges, CDOT-2011-9, Colorado Department of Transportation, Denver, CO.

Saito, M., Brimley, B., Schultz, G., 2011. Transportation Safety Data and Analysis Volume 2: Calibration of the Highway Safety Manual and Development of New Safety Performance Functions. UT-10.12b, Utah Department of Transportation, Salt Lake City, UT.

Schroeder, B., Cunningham, C., Ray, B., Daleiden, A., Jenior, P., Knudsen, J., 2014. Diverging Diamond Interchange Informational Guide. FHWA-SA-14-067, Federal Highway Administration, Washington, DC.

Schultz, G., Thurgood, D., Olsen, A., Reese, C.S., 2010. Transportation Safety Data and Analysis Volume 1: Analyzing the Effectiveness of Safety Measures using Bayesian Methods. UT-10.12a, Utah Department of Transportation, Salt Lake City, UT.

Siromaskul, S., 2010. Diverging Diamond Interchange Design 101: Things to Know Before You Start. *Proc., ITE Annual Meeting and Exhibit*, ITE, Vancouver, Canada.

UDOT, 2012. Diverging Diamond Interchange (DDI) Observations and Experience, UT-12.05, Utah Department of Transportation, Salt Lake City, UT.

Wang, H., Chitturi, M., Noyce, D., 2011. Developing Safety Performance Functions for Diamond Interchange Ramp Terminals. Proc., 90th Annual Meeting of the Transportation Research Board, Washington, DC.

Wang, H., Qin, X., and Noyce, D., 2010. Development of a Safety Performance Function for Signalized Diamond Interchange Ramp Terminals. ICCTP 2010: pp. 1034-1046.

Wu, H., Han, Z., Murphy, M., Zhang, Z., 2015. Empirical Bayes Before-After Study on Safety Effect of Narrow Pavement Widening Projects in Texas. Proc., 94th Annual Meeting of the Transportation Research Board, Washington, DC.

Zhou, H., Zhao, J., Hsu, P., and Huang, J. 2013. Safety Effects of Median Treatments Using Longitudinal Channelizers: Empirical Bayesian Before-and-After Study. J. Transp. Eng. 2013.139:1149-1155.

APPENDICES

APPENDIX A: REGRESSION RESULTS

HSM Ramp

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (1)
Link Function	Log lnL
Offset Variable	

Case Processing Summary

	N	Percent
Included	784	94.1%
Excluded	49	5.9%
Total	833	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	784	.0	29.0	1.098	2.0202
Covariate	lnAADT	784	6.525029658	10.14854914	8.889134805	.5736535637
Offset	aadt lnL	784	682.0	25554.0	8339.147	4103.4205
		784	-1.83258146	-.478035801	-1.13902205	.2289975964

Goodness of Fit[a]

	Value	df	Value/df
Deviance	824.508	781	1.056
Scaled Deviance	824.508	781	1.437
Pearson Chi-Square	1122.392	781	
Scaled Pearson Chi-Square	1122.392 -	781	
Log Likelihood[b]	1101.768		
Akaike's Information	2209.536		
Criterion (AIC)	2209.567		
Finite Sample Corrected AIC (AICC)	2223.529		
Bayesian Information Criterion (BIC)	2226.529		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnAADT, aadt, offset = lnL
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
109.052	2	.000

Dependent Variable: crash
Model: (Intercept), lnAADT, aadt, offset = lnL
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III		
	Wald Chi-Square	df	Sig.
(Intercept)	18.239	1	.000
lnAADT	19.058	1	.000
aadt	1.860	1	.173

Dependent Variable: crash
Model: (Intercept), lnAADT, aadt, offset = lnL

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-11.477	2.6874	-16.744	-6.210	18.239	1	.000
lnAADT	1.466	.3357	.808	2.124	19.058	1	.000
aadt	-5.442E-5	3.9900E-5	.000	2.378E-5	1.860	1	.173
(Scale)	1[a]						
(Negative binomial)	1[a]						

Dependent Variable: crash

Model: (Intercept), lnAADT, aadt, offset = lnL

a. Fixed at the displayed value.

HSM Terminal

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (MLE)
Link Function	Log

Case Processing Summary

	N	Percent
Included	391	95.4%
Excluded	19	4.6%
Total	410	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	391	.0	46.0	9.215	6.8892
Covariate	lnaadtcr	391	6.851184927	10.76363112	9.901702093	.6716411313
	lnaadtoffon	391	8.713088868	10.54599912	9.663815558	.3509873226

Goodness of Fit[a]

	Value	df	Value/df
Deviance	410.762	387	1.061
Scaled Deviance	410.762	387	1.040
Pearson Chi-Square	402.357	387	
Scaled Pearson Chi-Square	402.357	387	
Log Likelihood[b]	-1182.773		
Akaike's Information	2373.546		
Criterion (AIC)	2373.650		
Finite Sample Corrected AIC (AICC)	2389.421		
Bayesian Information Criterion (BIC)	2393.421		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoffon
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
92.801	2	.000

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoffon
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III		
	Wald Chi-Square	df	Sig.
(Intercept)	34.863	1	.000
lnaadtcr	55.390	1	.000
lnaadtoffon	16.249	1	.000

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoffon

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-6.062	1.0267	-8.075	-4.050	34.863	1	.000
lnaadtcr	.391	.0525	.288	.494	55.390	1	.000
lnaadtoffon	.451	.1118	.232	.670	16.249	1	.000
(Scale)	1[a]						
(Negative binomial)	.292	.0293	.240	.356			

Dependent Variable: crash

Model: (Intercept), lnaadtcr, lnaadtoffon

a. Fixed at the displayed value.

ISAT Ramp

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (MLE)
Link Function	Log

Case Processing Summary

	N	Percent
Included	784	94.1%
Excluded	49	5.9%
Total	833	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	784	.0	29.0	1.098	2.0202
Covariate	lnAADT	784	6.525029658	10.14854914	8.889134805	.5736535637
	lnL	784	-1.83258146	-.478035801	-1.13902205	.2289975964

Goodness of Fit[a]

	Value	df	Value/df
Deviance	726.590	780	.932
Scaled Deviance	726.590	780	1.165
Pearson Chi-Square	908.671	780	
Scaled Pearson Chi-Square	908.671	780	
Log Likelihood[b]	-1088.705		
Akaike's Information	2185.410		
Criterion (AIC)	2185.461		
Finite Sample Corrected AIC (AICC)	2204.067		
Bayesian Information Criterion (BIC)	2208.067		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnAADT, lnL
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
78.338	2	.000

Dependent Variable: crash
Model: (Intercept), lnAADT, lnL
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III Wald Chi-Square	df	Sig.
(Intercept)	71.304	1	.000
lnAADT	72.734	1	.000
lnL	.227	1	.634

Dependent Variable: crash
Model: (Intercept), lnAADT, lnL

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-8.875	1.0510	-10.935	-6.815	71.304	1	.000
lnAADT	.979	.1148	.754	1.204	72.734	1	.000
lnL	-.117	.2455	-.598	.364	.227	1	.634
(Scale)	1[a]						
(Negative binomial)	1.272	.1362	1.031	1.569			

Dependent Variable: crash

Model: (Intercept), lnAADT, lnL

a. Fixed at the displayed value.

ISAT Terminal

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (MLE)
Link Function	Log

Case Processing Summary

	N	Percent
Included	391	95.4%
Excluded	19	4.6%
Total	410	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	391	.0	46.0	9.215	6.8892
Covariate	lnaadtcr	391	6.851184927	10.76363112	9.901702093	.6716411313
	lnaadt off	391	6.525029658	10.14854914	8.896382325	.5897026589

Goodness of Fit[a]

	Value	df	Value/df
Deviance	410.517	387	1.061
Scaled Deviance	410.517	387	1.013
Pearson Chi-Square	391.976	387	
Scaled Pearson Chi-Square	391.976	387	
Log Likelihood[b]	-1179.456		
Akaike's Information	2366.912		
Criterion (AIC)	2367.015		
Finite Sample Corrected AIC (AICC)	2382.786		
Bayesian Information Criterion (BIC)	2386.786		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
99.436	2	.000

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III		
	Wald Chi-Square	df	Sig.
(Intercept)	48.045	1	.000
lnaadtcr	68.221	1	.000
lnaadtoff	23.288	1	.000

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-4.604	.6643	-5.906	-3.302	48.045	1	.000
lnaadtcr	.414	.0502	.316	.513	68.221	1	.000
lnaadtoff	.299	.0620	.178	.421	23.288	1	.000
(Scale)	1a	.0289	.234	.348			
(Negative binomial)	.285						

Dependent Variable: crash

Model: (Intercept), lnaadtcr, lnaadtoff

a. Fixed at the displayed value.

HSM Ramp Property Damage Only

Warnings

All convergence criteria are satisfied, but the Hessian matrix is singular. The GENLIN procedure continues despite the above warning(s). Subsequent results shown are based on the last iteration. Validity of the model fit is uncertain.

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (MLE)
Link Function	Log lnL
Offset Variable	

Case Processing Summary

	N	Percent
Included	784	95.4%
Excluded	38	4.6%
Total	822	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	784	.0	21.0	.807	1.5170
Covariate	lnAADT	784	6.525029658	10.14854914	8.889134805	.5736535637
Offset	aadt lnL	784	682.0	25554.0	8339.147	4103.4205
		784	-1.83258146	-.478035801	-1.13902205	.2289975964

Goodness of Fit[a]

	Value	df	Value/df
Deviance	1205.871	780	1.546
Scaled Deviance	1205.871	780	2.281
Pearson Chi-Square	1779.270	780	
Scaled Pearson Chi-Square	1779.270 -	780	
Log Likelihood[b]	1024.522		
Akaike's Information	2057.043		
Criterion (AIC)	2057.095		
Finite Sample Corrected AIC (AICC)	2075.701		
Bayesian Information Criterion (BIC)	2079.701		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnAADT, aadt, offset = lnL
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
.	.	.

Dependent Variable: crash
Model: (Intercept), lnAADT, aadt, offset = lnL
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III Wald Chi-Square	df	Sig.
(Intercept)	26.734	1	.000
lnAADT	27.252	1	.000
aadt	5.583	1	.018

Dependent Variable: crash
Model: (Intercept), lnAADT, aadt, offset = lnL

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-13.311	2.5744	-18.357	-8.265	26.734	1	.000
lnAADT	1.660	.3180	1.037	2.284	27.252	1	.000
aadt	-8.161E-5	3.4539E-5	.000	-1.391E-5	5.583	1	.018
(Scale)	1[a]						
(Negative binomial)	.106[b]	.	.	.			

Dependent Variable: crash

Model: (Intercept), lnAADT, aadt, offset = lnL

a. Fixed at the displayed value.

b. Hessian matrix singularity is caused by the scale or negative binomial parameter.

HSM Terminal Property Damage Only

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (MLE)
Link Function	Log

Case Processing Summary

	N	Percent
Included	391	95.4%
Excluded	19	4.6%
Total	410	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	391	.0	33.0	6.343	4.7647
Covariate	lnaadtcr	391	6.851184927	10.76363112	9.901702093	.6716411313
	lnaadtoffon	391	8.713088868	10.54599912	9.663815558	.3509873226

Goodness of Fit[a]

	Value	df	Value/df
Deviance	421.530	387	1.089
Scaled Deviance	421.530	387	1.022
Pearson Chi-Square	395.522	387	
Scaled Pearson Chi-Square	395.522	387	
Log Likelihood[b]	-1066.563		
Akaike's Information	2141.125		
Criterion (AIC)	2141.229		
Finite Sample Corrected AIC (AICC)	2157.000		
Bayesian Information Criterion (BIC)	2161.000		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoffon
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
61.901	2	.000

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoffon
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III		
	Wald Chi-Square	df	Sig.
(Intercept)	24.481	1	.000
lnaadtcr	32.526	1	.000
lnaadtoffon	12.002	1	.001

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoffon

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-5.387	1.0887	-7.521	-3.253	24.481	1	.000
lnaadtcr	.325	.0570	.213	.437	32.526	1	.000
lnaadtoffon	.411	.1187	.179	.644	12.002	1	.001
(Scale)	1[a]						
(Negative binomial)	.304	.0346	.243	.380			

Dependent Variable: crash

Model: (Intercept), lnaadtcr, lnaadtoffon

a. Fixed at the displayed value.

ISAT Ramp Property Damage Only

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (1)
Link Function	Log

Case Processing Summary

	N	Percent
Included	784	95.4%
Excluded	38	4.6%
Total	822	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	784	.0	21.0	.807	1.5170
Covariate	lnAADT	784	6.525029658	10.14854914	8.889134805	.5736535637
	lnL	784	-1.83258146	-.478035801	-1.13902205	.2289975964

Goodness of Fit[a]

	Value	df	Value/df
Deviance	739.616	781	.947
Scaled Deviance	739.616	781	1.239
Pearson Chi-Square	967.378	781	
Scaled Pearson Chi-Square	967.378	781	
Log Likelihood[b]	-937.091		
Akaike's Information	1880.182		
Criterion (AIC)	1880.212		
Finite Sample Corrected AIC (AICC)	1894.175		
Bayesian Information Criterion (BIC)	1897.175		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnAADT, lnL
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
74.076	2	.000

Dependent Variable: crash
Model: (Intercept), lnAADT, lnL
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III		
	Wald Chi-Square	df	Sig.
(Intercept)	66.031	1	.000
lnAADT	63.914	1	.000
lnL	.029	1	.864

Dependent Variable: crash
Model: (Intercept), lnAADT, lnL

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-8.703	1.0710	-10.802	-6.604	66.031	1	.000
lnAADT	.936	.1171	.707	1.166	63.914	1	.000
lnL	-.042	.2435	-.519	.436	.029	1	.864
(Scale)	1[a]						
(Negative binomial)	1[a]						

Dependent Variable: crash

Model: (Intercept), lnAADT, lnL

a. Fixed at the displayed value.

ISAT Terminal Property Damage Only

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (MLE)
Link Function	Log

Case Processing Summary

	N	Percent
Included	391	95.4%
Excluded	19	4.6%
Total	410	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	391	.0	33.0	6.343	4.7647
Covariate	lnaadtcr	391	6.851184927	10.76363112	9.901702093	.6716411313
	lnaadt off	391	6.525029658	10.14854914	8.896382325	.5897026589

Goodness of Fit[a]

	Value	df	Value/df
Deviance	421.905	387	1.090
Scaled Deviance	421.905	387	.997
Pearson Chi-Square	385.760	387	
Scaled Pearson Chi-Square	385.760	387	
Log Likelihood[b]	-1065.960		
Akaike's Information	2139.920		
Criterion (AIC)	2140.023		
Finite Sample Corrected AIC (AICC)	2155.795		
Bayesian Information Criterion (BIC)	2159.795		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
63.107	2	.000

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III Wald Chi-Square	df	Sig.
(Intercept)	28.818	1	.000
lnaadtcr	41.346	1	.000
lnaadtoff	13.286	1	.000

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-3.833	.7140	-5.233	-2.434	28.818	1	.000
lnaadtcr	.351	.0546	.244	.459	41.346	1	.000
lnaadtoff	.243	.0666	.112	.373	13.286	1	.000
(Scale)	1a	.0345	.242	.378			
(Negative binomial)	.302						

Dependent Variable: crash

Model: (Intercept), lnaadtcr, lnaadtoff

a. Fixed at the displayed value.

HSM Ramp Injury/Fatality

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (1)
Link Function	Log lnL
Offset Variable	

Case Processing Summary

	N	Percent
Included	784	95.1%
Excluded	40	4.9%
Total	824	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	784	.0	8.0	.291	.7408
Covariate	lnAADT	784	6.525029658	10.14854914	8.889134805	.5736535637
Offset	aadt lnL	784	682.0	25554.0	8339.147	4103.4205
		784	-1.83258146	-.478035801	-1.13902205	.2289975964

Goodness of Fit[a]

	Value	df	Value/df
Deviance	544.291	781	.697
Scaled Deviance	544.291	781	1.329
Pearson Chi-Square	1037.688	781	
Scaled Pearson Chi-Square	1037.688	781	
Log Likelihood[b]	-521.644		
Akaike's Information	1049.287		
Criterion (AIC)	1049.318		
Finite Sample Corrected AIC (AICC)	1063.281		
Bayesian Information Criterion (BIC)	1066.281		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnAADT, aadt, offset = lnL
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
54.704	2	.000

Dependent Variable: crash
Model: (Intercept), lnAADT, aadt, offset
= lnL
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III		
	Wald Chi-Square	df	Sig.
(Intercept)	10.659	1	.001
lnAADT	9.294	1	.002
aadt	1.579	1	.209

Dependent Variable: crash
Model: (Intercept), lnAADT, aadt, offset = lnL

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-15.896	4.8688	-25.439	-6.353	10.659	1	.001
lnAADT	1.832	.6011	.654	3.011	9.294	1	.002
aadt	-8.155E-5	6.4896E-5	.000	4.565E-5	1.579	1	.209
(Scale)	1[a]						
(Negative binomial)	1[a]						

Dependent Variable: crash

Model: (Intercept), lnAADT, aadt, offset = lnL

a. Fixed at the displayed value.

HSM Terminal Injury/Fatality

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (MLE)
Link Function	Log

Case Processing Summary

	N	Percent
Included	391	95.4%
Excluded	19	4.6%
Total	410	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	391	.0	15.0	2.872	2.8472
Covariate	lnaadtcr	391	6.851184927	10.76363112	9.901702093	.6716411313
	lnaadtoffon	391	8.713088868	10.54599912	9.663815558	.3509873226

Goodness of Fit[a]

	Value	df	Value/df
Deviance	440.209	387	1.137
Scaled Deviance	440.209	387	1.033
Pearson Chi-Square	399.906	387	
Scaled Pearson Chi-Square	399.906	387	
Log Likelihood[b]	-810.223		
Akaike's Information	1628.445		
Criterion (AIC)	1628.549		
Finite Sample Corrected AIC (AICC)	1644.320		
Bayesian Information Criterion (BIC)	1648.320		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoffon
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
91.231	2	.000

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoffon
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III Wald Chi-Square	df	Sig.
(Intercept)	49.051	1	.000
lnaadtcr	54.845	1	.000
lnaadtoffon	7.638	1	.006

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoffon

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-9.866	1.4087	-12.627	-7.105	49.051	1	.000
lnaadtcr	.692	.0934	.509	.875	54.845	1	.000
lnaadtoffon	.409	.1480	.119	.699	7.638	1	.006
(Scale)	1[a]						
(Negative binomial)	.372	.0567	.276	.502			

Dependent Variable: crash

Model: (Intercept), lnaadtcr, lnaadtoffon

a. Fixed at the displayed value.

ISAT Ramp Injury/Fatality

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (1)
Link Function	Log

Case Processing Summary

	N	Percent
Included	784	95.1%
Excluded	40	4.9%
Total	824	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	784	.0	8.0	.291	.7408
Covariate	lnAADT	784	6.525029658	10.14854914	8.889134805	.5736535637
	lnL	784	-1.83258146	-.478035801	-1.13902205	.2289975964

Goodness of Fit[a]

	Value	df	Value/df
Deviance	533.734	781	.683
Scaled Deviance	533.734	781	1.211
Pearson Chi-Square	945.827	781	
Scaled Pearson Chi-Square	945.827	781	
Log Likelihood[b]	-516.365		
Akaike's Information	1038.730		
Criterion (AIC)	1038.761		
Finite Sample Corrected AIC (AICC)	1052.723		
Bayesian Information Criterion (BIC)	1055.723		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnAADT, lnL
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
47.135	2	.000

Dependent Variable: crash
Model: (Intercept), lnAADT, lnL
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III Wald Chi-Square	df	Sig.
(Intercept)	49.429	1	.000
lnAADT	38.621	1	.000
lnL	.360	1	.549

Dependent Variable: crash
Model: (Intercept), lnAADT, lnL

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-11.058	1.5728	-14.140	-7.975	49.429	1	.000
lnAADT	1.061	.1707	.726	1.395	38.621	1	.000
lnL	-.208	.3469	-.888	.472	.360	1	.549
(Scale)	1[a]						
(Negative binomial)	1[a]						

Dependent Variable: crash

Model: (Intercept), lnAADT, lnL

a. Fixed at the displayed value.

ISAT Terminal Injury/Fatality

Model Information

Dependent Variable	crash
Probability Distribution	Negative binomial (MLE)
Link Function	Log

Case Processing Summary

	N	Percent
Included	391	95.4%
Excluded	19	4.6%
Total	410	100.0%

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	391	.0	15.0	2.872	2.8472
Covariate	lnaadtcr	391	6.851184927	10.76363112	9.901702093	.6716411313
	lnaadt off	391	6.525029658	10.14854914	8.896382325	.5897026589

Goodness of Fit[a]

	Value	df	Value/df
Deviance	437.620	387	1.131
Scaled Deviance	437.620	387	1.025
Pearson Chi-Square	396.672	387	
Scaled Pearson Chi-Square	396.672	387	
Log Likelihood[b]	-804.366		
Akaike's Information	1616.733		
Criterion (AIC)	1616.837		
Finite Sample Corrected AIC (AICC)	1632.608		
Bayesian Information Criterion (BIC)	1636.608		
Consistent AIC (CAIC)			

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff
a. Information criteria are in smaller-is-better form.

b. The full log likelihood function is displayed and used in computing information criteria.

Omnibus Test[a]

Likelihood Ratio Chi-Square	df	Sig.
102.943	2	.000

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff
a. Compares the fitted model against the intercept-only model.

Tests of Model Effects

Source	Type III Wald Chi-Square	df	Sig.
(Intercept)	80.573	1	.000
lnaadtcr	60.907	1	.000
lnaadtoff	18.906	1	.000

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff

Parameter Estimates

Parameter	B	Std. Error	95% Wald Confidence Interval		Hypothesis Test		
			Lower	Upper	Wald Chi-Square	df	Sig.
(Intercept)	-9.269	1.0326	-11.293	-7.245	80.573	1	.000
lnaadtcr	.693	.0888	.519	.867	60.907	1	.000
lnaadtoff	.375	.0862	.206	.544	18.906	1	.000
(Scale)	1[a]						
(Negative binomial)	.353	.0549	.260	.478			

Dependent Variable: crash

Model: (Intercept), lnaadtcr, lnaadtoff

a. Fixed at the displayed value.

APPENDIX B: STUDY LOCATION BEFORE & AFTER PICTURES

I-15 Exit 8

Before

ESRI Basemap

After

Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

I-15 Exit 276

Before

2009 Utah Imagery - AGRC

After

Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

I-15 Exit 278

Before

2009 Utah Imagery - AGRC

After

Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

I-15 Exit 284

Before

2009 Utah Imagery - AGRC

After

Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

SR-201 Exit 13

Before

2009 Utah Imagery - AGRC

After

Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community