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A Comprehensive Safety Assessment Methodology for Innovative Geometric Designs

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A COMPREHENSIVE SAFETY ASSESSMENT METHODOLOGY FOR INNOVATIVE GEOMETRIC DESIGNS

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ABSTRACT

As the population grows and travel demands increase, alternative interchange designs have become increasingly popular. The diverging diamond interchange is an alternative design that has been implemented in the United States. This design can accommodate higher and unbalanced flow and 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 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 future implementation of the diverging diamond interchanges.

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1. INTRODUCTION

1.1 Problem Statement

Innovative geometric designs are often considered a solution to the challenge of meeting increasing travel demands with limited recourses (FHWA, 2009). This study focuses on one such design—the diverging diamond interchange (DDI)—which aims to improve traffic flow and reduce congestion at highway junctions (Schroeder et al., 2014); but the methodology proposed is transferable to other geometric designs. Utah was among the first states to consider the DDI as a viable interchange option. The state opened its first DDI at the intersection of American Fork Main Street and I-15 in August 2010. Empirical studies have demonstrated the operational and cost benefits of DDIs (Bared et al., 2006; MoDOT, 2011; UDOT, 2012; Yang et al., 2014); however, the impact on safety remains inconclusive. Theoretically, the DDI design offers a safety benefit because it reduces the number of conflict points in comparison with other interchange options, which can lead to fewer crashes in general. Moreover, the lower design speeds in DDIs also may result in fewer, less severe crashes. One major safety concern with DDIs is that drivers may stay to the right at crossovers and accidentally enter the opposing lanes. Despite the theoretical safety benefits, little research has been done to quantify the safety impact of DDI using real-world crash data, primarily because of the limited accident history available. A preliminary safety study (MoDOT, 2011) directly compared crash rates before and after the construction of a DDI in Missouri and concluded that total crashes dropped by 46% in the first year of operation. However, the simple before-and-after method assumes that any changes to 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, also can affect the safety performance. Therefore, we propose an alternative approach to deal with possible confounding factors by comparing the safety performance of DDIs with that of a group of reference sites. Safety considerations for pedestrians and bicycle traffic will also be discussed.

1.2 Research Objectives

Utah has been a pioneer and leader in adopting innovative interchange and intersection designs. Currently, the state has six operating DDIs and more are under construction or have planned. Four DDIs in the state have been opened to traffic for more than two years, which provides sufficient accident data for a comprehensive safety study. This study will be one of the first independent studies in the nation to investigate the overall safety impact of DDIs. The results will be useful in evaluating DDI construction and retrofit projects in Utah and other states. The research is expected to have a broad and significant impact on the implementation of innovative interchange and intersection designs.

1.3 Scope

A literature review was performed on before-and-after study methodology and safety studies on DDIs. A comparison group of diamond interchanges that have not been converted to DDIs was compiled for the calibration of Utah's specific safety performance functions. Crash data provided by the UDOT Traffic and Safety Division were collected for operating DDIs and the comparison group sites for calculations in the Empirical Bayes analysis. Results from the Empirical Bayes analysis are presented for use in future consideration of DDI implementation.

1.4 Outline of the Report

This report contains a literature review and a review of past studies performed on the DDI. The data collection process is discussed followed by the methodology for the safety performance function (SPF) calibration. The Empirical Bayes approach is discussed in detail and results of this analysis are included. Pedestrian and cyclist design considerations also are presented. This report is based on Lloyd, 2016, Lloyd and Song, 2017 and Song et al., 2018.

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 section will provide a comprehensive review of DDI studies and before-after study methodology.

2.1 Diverging Diamond Interchange

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

2.1.1 Design Considerations

Due to the crossover of the lanes, there is no need for a left-turn phase in the signal timing for DDIs. Left-turn movements off 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 completely eliminated, resulting in shorter signal cycle times. Both 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, which 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 also can improve efficiency and allow 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 also must 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 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 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.

2.1.2 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 2.1 shows these two optional paths (UDOT, 2014).

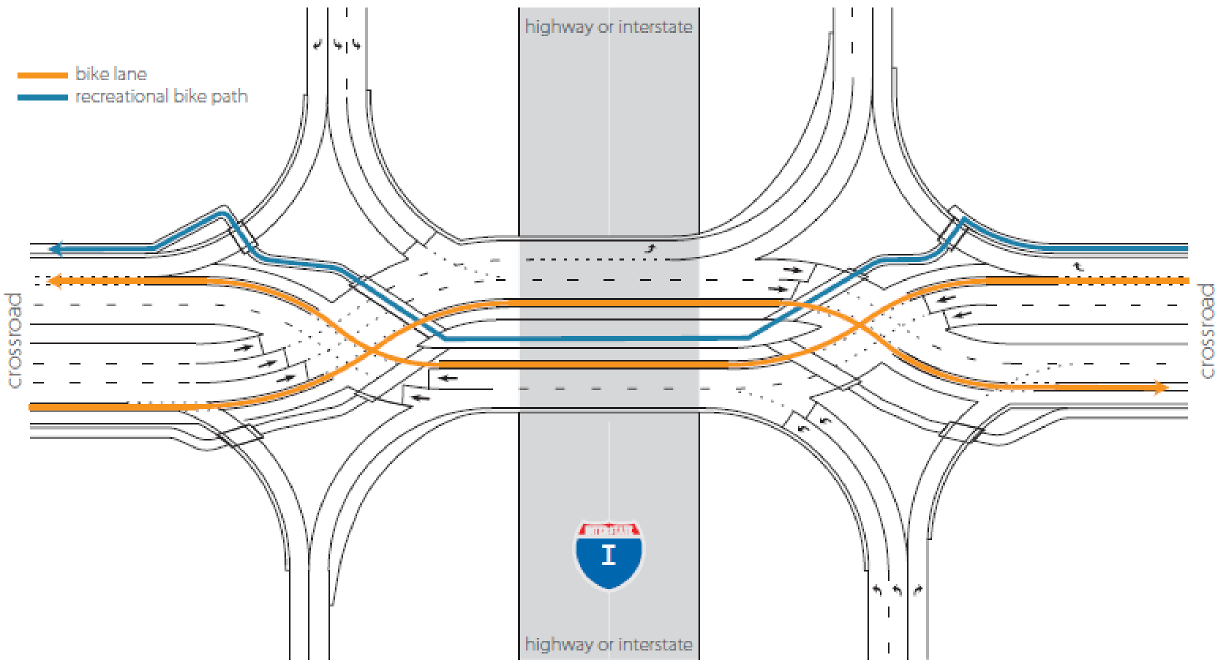

Figure 2.1 DDI Bicycle Paths (UDOT, 2014)

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 placement of the 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 and the through traffic at the crossover. If the walkway is located outside of the interchange, pedestrians and cyclists cross the path of the vehicles turning right from the freeway off-ramp and the path of the vehicles turning left onto the freeway on-ramp. Vehicles on the ramps could be traveling fast 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 Section 6.

2.1.3 Operational Performance

Using a VISSIM simulation, an MoDOT study found a decrease in average delay time per vehicle during times with higher volumes in the total DDI network configuration. MoDOT also observed decreased back-ups from traffic due to Friday night tourists and PM peak periods when compared with back-up levels of up to a mile or more before the DDI was implemented. However, morning commute back-ups were found at the DDI in Springfield, Missouri. 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. 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 with the traditional diamond interchange, Chlewicki (2003) observed the following improvements:

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

These simulations support the theoretical expectation that the DDI will improve capacity and flow when compared with 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, with other interchange configurations, to determine if the DDI is a good fit or if another option would better serve 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, which 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.

2.1.4 Safety

Safety is also a large concern when introducing a new interchange configuration such as the DDI. As reviewed in Section 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 determine if implementation of the DDI can improve the safety at a given location. As the DDI gains popularity, more studies are being performed on this matter. At this time, there are only a few conclusive studies. Table 2.1 shows a compilation of the study summary and results of recent DDI safety studies.

The VISSIM simulation study performed by the FHWA, listed first in Table 2.1, analyzed 74 licensed drivers in the Washington, D.C., 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 five years after implementing the DDI. This is a large decrease when compared with the average 23 fatal and injury crashes at diamond interchanges in the United States (Poorbaugh and Houston, 2006).

Most studies summarized in Table 2.1 use the naïve before-after method. 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. More before-and-after crash data will be available in the future, allowing for more accurate study results. Employing more advanced before-after study methods also will provide more reliable results that account for changes in input variables from the before period to the after period, and from the regression-to-the-mean tendency. This study aims to us increased data in after periods and EB analysis to provide safety analysis methodology and results.

Table 2.1 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

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 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 Section 5.

2.2 Before-After Study Methodology

Safety studies generally employ a before-after study method 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.

2.2.1 Naïve Before-After Method

Before-after studies are used frequently in safety studies in the transportation field. Table 2.1 shows a common approach to measure effectiveness of implemented roadway improvements/changes in 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 if the treatment not been implemented, as shown in equation 2.1. The data is 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 2.2, with the percent improvement and percent effectiveness shown in equations 2.3 and 2.4, respectively.

$$N_{obs-b} = N_{exp-a} \tag{2.1}$$

$$\Delta_{cr} = N_{exp-a} - N_{obs-a} \tag{2.2}$$

$$\% \, \Delta_{cr} = \frac{N_{obs-a}}{N_{obs-b}} \times 100 \tag{2.3}$$

$$\% \, Effectiveness = (1 - \% \, \Delta_{cr}) * 100 \tag{2.4}$$

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

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 into 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 2.2 (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).

2.2.2 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 because 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, and used to calculate the expected annual average crash rate for the after period if the treatment had not been implemented. This number is then compared with the actual observed crash rate to measure the increased or decreased safety of the study site.

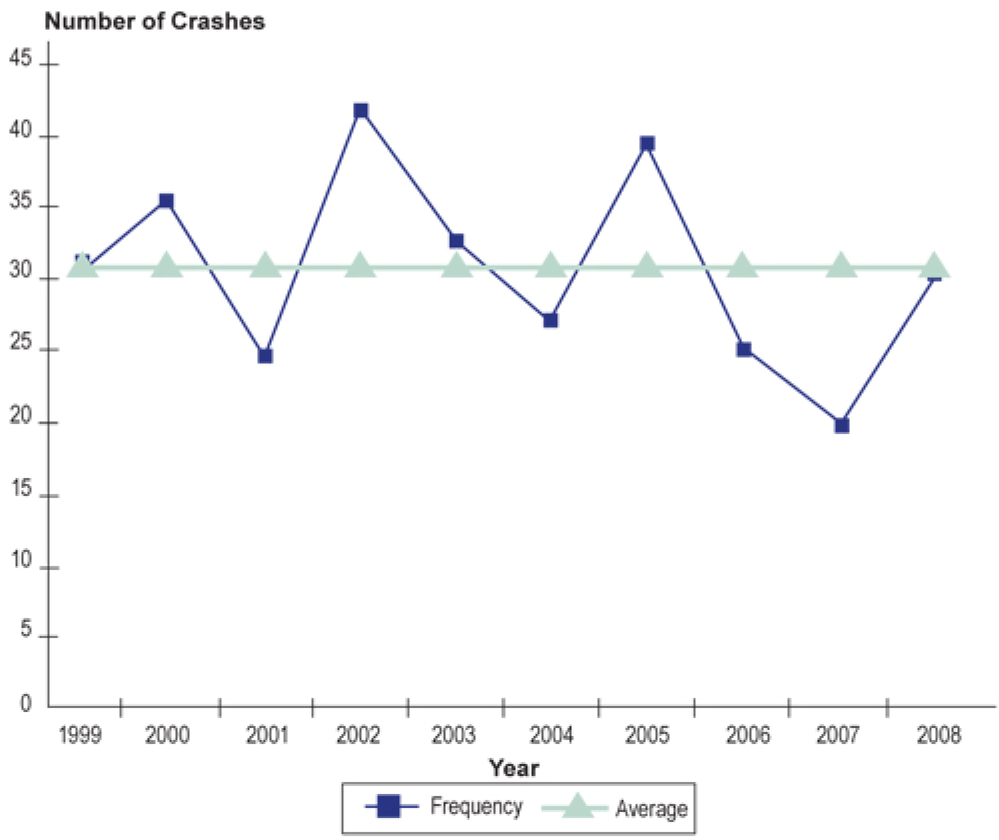

Figure 2.2 Regression-to-the-Mean Illustration (FHWA, 2010) Highway Safety Improvement Program Manual (Section 2.3)

Hauer (1997) indicates 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 difficult 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 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 2.5.

$$N_{exp-a-t} = \frac{N_{obs-a-u}}{N_{obs-b-u}} \times N_{obs-b-t} \tag{2.5}$$

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 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.

2.2.3 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, including the traits of 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 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. 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 also will account for the factors that are likely to change over time, including traffic patterns, AADT, weather and driver behaviors, as previously mentioned. 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 frequently used in before-after studies, is a regression approach, as suggested by Hauer. Data collected from the reference group sites can be analyzed and a regression fit to the data, which 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. 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 are over-dispersed. The negative binomial regression accounts for this over-dispersion 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 use SPFs for specific roadway types.

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 must 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 calibration of the appropriate SPF. The reference group used, discussed in Section 3, is a much broader group of sites than a comparison group. The reference sites 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, but 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 are available in the before period for all comparison group and study site locations. Section 5 will discuss the calculations necessary for this method.

3. DATA COLLECTION

Two forms of data, i.e., AADT and crash counts, were used in this study and obtained from UDOT. Details regarding the selection of study sites, and the collection process for crash counts and AADT will be discussed in this section.

3.1 Study Site Selection

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 3.1. 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. This 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 3.1 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

3.2 Comparison Group Site Selection

The EB before and after method involves the use of SPFs in the beginning calculations. Section 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 and dditional diamond interchange sites were pulled from SR-201, I-80 and I-215, totaling 26 sites. These sites will be used in calibrating the SPFs employed in the EB analysis. The selected diamond interchange sites are listed in Table 3.2.

Table 3.2 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

Comparing Table 3.1 and Table 3.2, 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. Only 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.

3.3 Crash Count Data Collection

The crash count data was provided by the UDOT Traffic and Safety Division. Using the provided data, the appropriate route numbers, and latitude and longitude coordinate ranges were selected for the interchanges 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 (AASHTO, 2010). This definition was applied in this project as shown in Figure 3.1. Traditionally, the crossroad section more than 250 ft. beyond the ramp terminal/intersection would not be included in the terminal, but, this study is concerned with all areas affected by implementation of the DDI and 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.

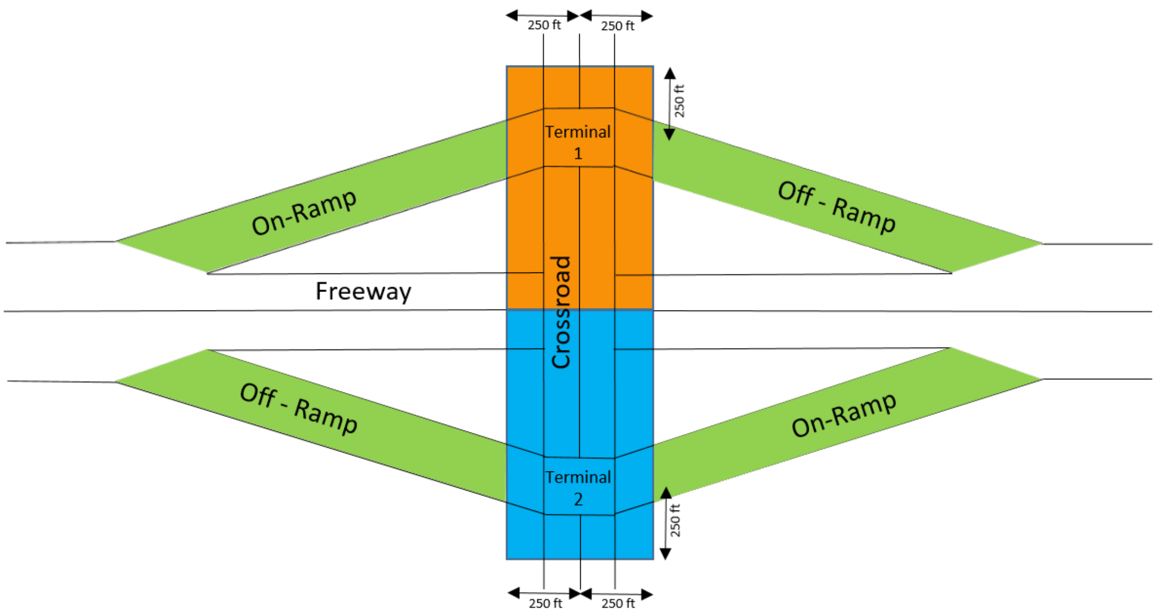

Figure 3.1 Crash Site Assignment Diagram

The route number and coordinate range sort was adequate enough for selecting 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 on which type of roadway the accident occurred. All crashes in the desired route and coordinate range with an "R" roadway type designation were selected for the study data set. These selections were mapped in ArcMap to verify that the crashes were in 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, data were ready for regression analysis in SPSS as discussed in Section 3. This data collection process is shown in the flow chart in Figure 3.2.

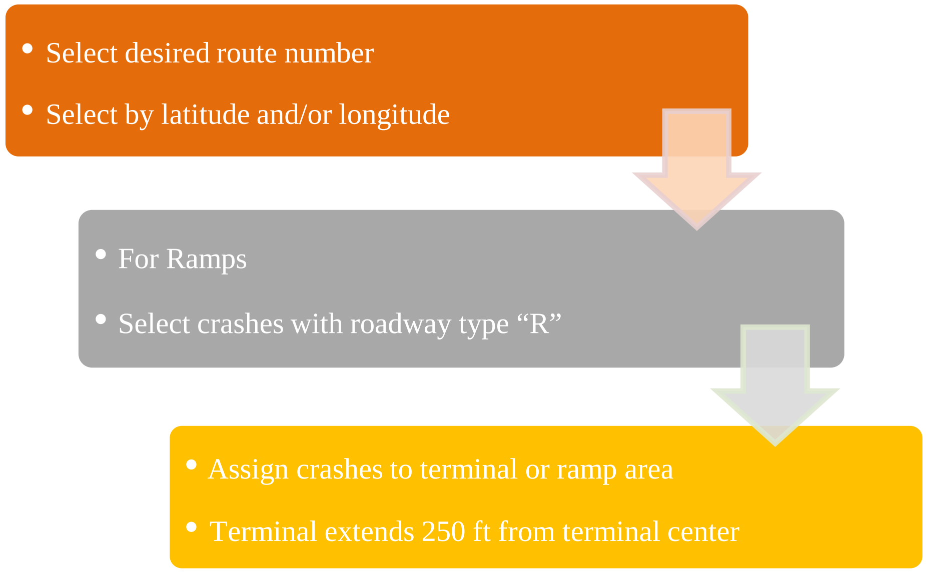

Figure 3.2 Comparison Group Data Scrubbing Process

4. SAFETY PERFORMANCE FUNCTION

In the transportation industry, it has become commonplace for the negative binomial regression to be used to model 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, thus causing the Poisson distribution to be inoperative. In the case where the variance exceeds the mean, also known as being over-dispersed, 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 over-dispersed-Poisson distribution, lending itself to the negative binomial distribution. SPFs for the study site and comparison group sites are used in the EB calculations, which will be discussed in Section 5.

4.1 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.

4.1.1 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 4.1 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 4.2. The calibration factor calculation is shown in equation 4.3. The resulting value of equation 4.2 is the number of predicted crashes for the before period. It is important to note that the use of CMFs, which 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).

$$N_{spf,int,1\,way\,stop} = exp(a + b \times ln[c \times AADT_{xrd}] + d \times ln[c \times AADT_{ex} + c \times AADT_{en}]) \qquad (4.1)$$

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

$$N_{pred-b} = N_{spf-b} \times \left(CMF_{1x} \times CMF_{2x} \times \ldots \times CMF_{yx}\right) \times C_x \tag{4.2}$$

$$C_x = \frac{\sum N_{obs-b}}{\sum N_{spf-b}} \tag{4.3}$$

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

4.1.2 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 is to look only at the most recent year in which all the crash data are available. This would cause the SPFs to directly model after only the year of data used. The second is to use up to 10 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 have an impact on 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 4.4 and applied to the SPF as shown in equation 4.5.

$$C = \frac{N_{obs-b}}{N_{spf-b}} \tag{4.4}$$

$$N_{pred-b} = N_{spf-b} \times C \tag{4.5}$$

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

4.1.3 SPF for Signalized Diamond Interchanges

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, as given in equations 4.6 - 4.9.

$$N_{spf-ramp\ terminal} = a \times VE^b \times exp\left(c \times Y_{dif} + d \times AR_{dif} + eRT_{dummy} + f \times S_{terminal}\right) \quad (4.6)$$

$$VE = AADT_{ramp1} \times AADT_{crd1} + AADT_{ramp2} \times AADT_{crd2} \quad (4.7)$$

$$Y_{dif} = Y_{obs} - Y_{ITE} = Y_{obs} - \left(T_{pr} + \frac{V_a}{2d_r + 2gG_r}\right) \quad (4.8)$$

$$AR_{dif} = AR_{obs} - AR_{ITE} = AR_{obs} - \left(\frac{S+L}{V_a}\right) \quad (4.9)$$

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. he 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, the SPF includes the signal timing data, which differ 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.

4.2 SPF Calibration Analysis

As discussed previously in this section, the HSM and ISAT provide base SPFs and predetermined parameters specific to different roadway configurations and various characteristics specific to a study site. The HSM and ISAT prescribe that the appropriate coefficients be selected to match characteristics of the site being studied. For this study, 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 Section 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 4.10-4.11 and 4.12-4.13 respectively (AASHTO, 2010; FHWA, 2007).

$$N_{HSM\ ramp} = L \times exp\left(a + b \times ln\left[c \times AADT_{ramp}\right] + d \times \left[c \times AADT_{ramp}\right]\right) \tag{4.10}$$

$$N_{HSM\ terminal} = \exp[\mathrm{a} + \mathrm{b} \times \ln(\mathrm{c} \times AADT_{crossroad}) + d \times \ln(c \times AADT_{exit} + c \times AADT_{entrance})] \tag{4.11}$$

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

$$N_{ISAT\ ramp} = e^{a} \times {AADT_{ramp}}^{b} \times RL^{e} \tag{4.12}$$

$$N_{ISAT\ terminal} = e^{a} \times {AADT_{crossroad}}^{b} \times {AADT_{exit}}^{c} \tag{4.13}$$

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. Crash numbers 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 4.14,

$$Y_i = \beta_0 + \beta_1 x_{i1} + \beta_2 x_{i2} + \cdots + \beta_{ik} x_{ik} + \varepsilon_i \tag{4.14}$$

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 4.15 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 4.1. 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 also be checked. 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 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 4.15-4.26. As a cross-check, the data also were analyzed using SAS, another statistical analysis program, with similar results with negligible differences in parameter estimation. This comparison simply reflects an accurate fit using the SPSS or SAS programs.

$$N_{HSM\ ramp,total} = L \times exp\big(-11.477 + 1.466 \times ln\big[1 \times AADT_{ramp}\big] - (5.442 \times 10^{-5}) \times \big[1 \times AADT_{ramp}\big]\big) \tag{4.15}$$

$$N_{HSM\ ramp,pdo} = L \times exp\big(-13.311 + 1.66 \times ln\big[1 \times AADT_{ramp}\big] - (8.161 \times 10^{-5} \times \big[1 \times AADT_{ramp}\big]\big) \tag{4.16}$$

$$N_{HSM\ ramp,injury/fatality} = L \times exp\big(-15.896 + 1.832 \times ln\big[1 \times AADT_{ramp}\big] - (8.155 \times 10^{-5}) \times \big[1 \times AADT_{ramp}\big]\big) \tag{4.17}$$

$$N_{HSM\ terminal,total} = \exp[-6.062 + .391 \times \ln(1 \times AADT_{crossroad}) + .451 \times \ln(1 \times AADT_{exit} + 1 \times AADT_{entrance})] \tag{4.18}$$

$$N_{HSM\ terminal,pdo} = \exp[-5.387 + .325 \times \ln(1 \times AADT_{crossroad}) + .411 \times \ln(1 \times AADT_{exit} + 1 \times AADT_{entrance})] \tag{4.19}$$

$$N_{HSM\ terminal,injury/fatality} = \exp[-9.866 + .692 \times \ln(1 \times AADT_{crossroad}) + .409 \times \ln(1 \times AADT_{exit} + 1 \times AADT_{entrance})] \tag{4.20}$$

$$N_{ISAT\ ramp,total} = e^{-8.875} \times AADT_{ramp}{}^{.979} \times RL^{-.117} \tag{4.21}$$

$$N_{ISAT\ ramp,pdo} = e^{-8.703} \times AADT_{ramp}{}^{.936} \times RL^{-.042} \tag{4.22}$$

$$N_{ISAT\ ramp,injury/fatality} = e^{-11.058} \times AADT_{ramp}{}^{1.061} \times RL^{-.208} \tag{4.23}$$

$$N_{ISAT\ terminal,total} = e^{-4.604} \times AADT_{crossroad}{}^{.414} \times AADT_{exit}{}^{.299} \tag{4.24}$$

$$N_{ISAT\ terminal,pdo} = e^{-3.833} \times AADT_{crossroad}{}^{.351} \times AADT_{exit}{}^{.243} \tag{4.25}$$

$$N_{ISAT\ terminal,injury/fatality} = e^{a} \times AADT_{crossroad}{}^{b} \times AADT_{exit}{}^{c} \tag{4.26}$$

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.

Table 4.1 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	--	--

5. BEFORE-AFTER SAFETY ANALYSIS METHODOLOGY

The Empirical Bayes before-after method involves a series of calculations to 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 compared with 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.

5.1 Empirical Bayes Analysis

When performing the 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, which has no exits, entrances, or intersections, is being studied, 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 affects the selection of SPFs and data required. At the site-specific level, crash data, AADT, and other included factors must be detailed enough to assign each reported accident to the appropriate site type in 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 also must 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 must be the same length for each study site. Periods should not include times when construction was being performed at the selected study sites.

The EB analysis 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 5.1 followed by a description of each step.

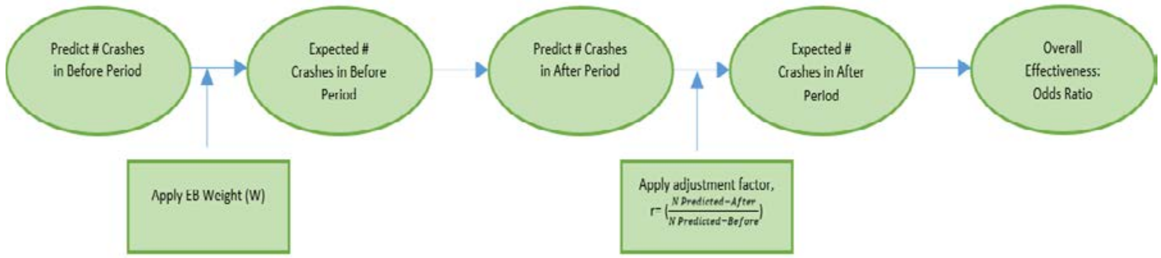

Figure 5.1 Empirical Bayes Method Flow Chart

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

As previously mentioned in this section, the SPF is used as the base point in the EB method. Once site types are determined, the SPFs can be selected. The SPF is applied to 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. To account for these differences, the SPFs must 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 Section 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 to obtain the total number of predicted crashes for each period, respectively.

5.1.2 Step 2 – Overdispersion Parameter

When using the HSM SPF, the over-dispersion 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 over-dispersion parameter. It is common in the field of statistics to use the Pearson Chi-Square/degrees of freedom as the over-dispersion parameter; therefore, this value will be used in this study.

5.1.3 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 over-dispersion parameter from the negative binomial regression model. This calculation is shown in equation 5.1. 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 that the expected number of crashes will be close to the observed number of crashes in the before period (Hauer, 1997).

$$w_b = \frac{1}{1+k\sum N_{pred-b}} \tag{5.1}$$

where

w_b = weight used in the Empirical Bayes method

k = dispersion parameter

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

5.1.4 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 5.2.

$$N_{exp-b} = w_b \times N_{pred-b} + (1 - w_b) \times N_{obs-b} \quad (5.2)$$

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

5.1.5 Step 5 – Adjustment Factor

A ratio is used to adjust for variance between the predicted number of crashes in the before and after periods, as shown in equation 5.3. This will account for differences in period duration and AADT between the periods (AASHTO, 2010).

$$r = \frac{\sum N_{pred-a}}{\sum N_{pred-b}} \quad (5.3)$$

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

5.1.6 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 calculated for the before period, as shown in equation 5.4. 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.

$$N_{exp-a} = N_{exp-b} \times r \quad (5.4)$$

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

5.1.7 Step 7 – Estimated Effectiveness of Treatment for Each Site

The calculated expected number of crashes in the after period, if the treatment had not been implemented, is compared with 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 5.5 for each site individually. This value shows the effectiveness of each site individually.

$$OR_i = \frac{N_{obs-a,i}}{N_{exp-a},i} \tag{5.5}$$

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

5.1.8 Step 8 – Safety Effectiveness

Using equation 5.6, the effectiveness of the total location can be measured.

$$OR = \frac{\sum N_{obs-a}}{\sum N_{exp-a}} \tag{5.6}$$

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

5.1.9 Step 9 – Adjusted Odds Ratio: Unbiased Safety Effectiveness

The HSM shows that the value found in equation 5.6 could be bias and must be adjusted to result in an unbiased effectiveness value for the treated site. Equations 5.7 and 5.8 show this calculation.

$$OR_{adj} = \frac{OR}{1+\frac{Var\left[\sum N_{exp-a}\right]}{\left(\sum N_{exp-a}\right)^2}} \tag{5.7}$$

$$Var\left[\sum N_{exp-a}\right] = \sum\left[(r)^2 \times N_{exp-b} \times (1-w_b)\right] \tag{5.8}$$

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 5.6

$\sum N_{exp-a}$ = 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

5.1.10 Step 10 – Safety Effectiveness as a Percent

The calculation in equation 5.9 returns the percent improvement in number of crashes for each study location.

$$Safety\ Effectiveness = 100\ \times \left(1 - OR_{adj}\right) \tag{5.9}$$

where

OR_{adj} = adjusted odds ratio, from equation 5.7

The variance and standard error of the odds ratio from equation 5.7 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.

5.2 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, and other study circumstances. Results from the above EB analysis will be used in conjunction with the FHWA guide to develop the DDI-specific CMF. Equation 5.10 exhibits the required calculation for creating the CMF (FHWA, 2010). Equations 5.11 through 5.13 show the CMF variance, standard error, and confidence interval calculations, respectively.

$$CMF = \left(\frac{N_{observed,A}}{N_{expected,A}}\right)/\left(1 + \left(\frac{Var\left(N_{expected,A}\right)}{{N_{expected,A}}^2}\right)\right) \tag{5.10}$$

$$CMF\ Variance = \left(CMF^2 * \left[\left(\frac{1}{N_{observed,A}}\right) + \left(\frac{Var\left(N_{expected,A}\right)}{{N_{expected,A}}^2}\right)\right]\right)/\left(1 + \frac{Var\left(N_{expected,A}\right)}{{N_{expected,A}}^2}\right) \tag{5.11}$$

$$CMF\ Standard\ Error = \sqrt{CMF\ Variance} \tag{5.12}$$

$$CMF\ 95\%\ Confidence\ Interval = CMF \pm 1.96 * CMF\ Standard\ Error \tag{5.13}$$

6. RESULTS and DISCUSSION

A site-specific EB before-after analysis was applied to collected data for the selected DDIs in Utah. as specified in the Study Site Selection section. Analysis results are shown in Table 6.1. The effectiveness shows the percent of change that resulted after 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, thus 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 6.1.

The data were analyzed on three different levels including total crashes, property damage only (PDO) crashes, and injury and fatality crashes. In each level, the HSM and ISAT SPFs were applied to each individual terminal and ramp at each study site. The data also were summed across all study locations for each road type at the three levels with results showing in the "all sites combined" column in Table 6.1. The terminal results returned positive safety effectiveness values, with a large number of the results being significant. Overall, the ramps 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; however, quite a few ramps did return this result. It is important to note that all negative results seen in Table 6.1 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; but, some locations do differ more than the others.

When comparing the road type results at each study location, and 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 highly 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 also was conducted on the data. In the event that crash data are 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 Section 5. The results of the project level analysis are presented in Table 6.2. These results show 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 shown in Table 6.2. 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 Section 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, also could be studied. For example, in this study, the EB analysis concluded that Exit 284 had 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? Was it due to incorrect or ineffective geometric designs at the DDI, rapid increase in AADT due to increased businesses in the area or 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 also were calculated as discussed in Section 5. The site-specific and project-level crash modification factors are reported in Table 6.3 and Table 6.4, 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 6.1 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 6.2 Project Level Empirical Bayes 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 6.3 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 6.4 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

7. EVALUATING PEDESTRIAN & CYCLIST SAFETY IN DDIS

The DDI is an effective tool for increasing capacity at unbalanced interchanges and decreasing crossing points resulting in increased safety for vehicles traveling through the interchange. While vehicles will compose the majority of 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 bicycle walkways can be placed in one of two different locations in 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 7.1(UDOT, 2014).

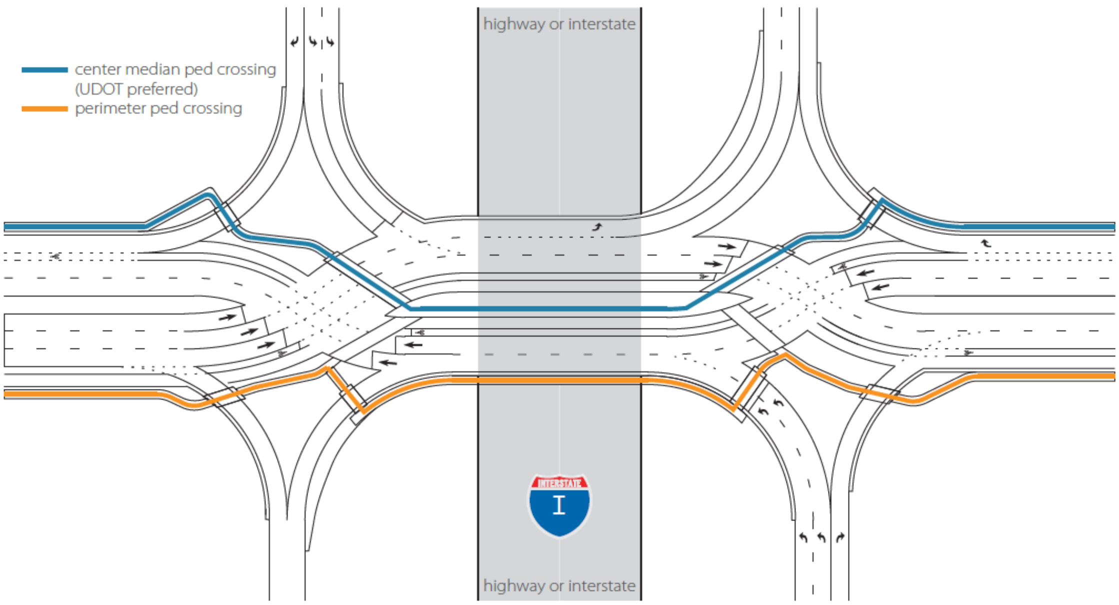

Figure 7.1 Center and Outside Pedestrian and Bicycle Walkways - UDOT DDI Guideline (UDOT, 2014)

In either the center or outside walkway configurations, if the right-turning lanes have no signals, precautions should be taken to increase the safety of the pedestrians at these crossing points. FHWA recommends that 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 7.1 and Table 7.2 respectively (FHWA, 2014).

Table 7.1 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

Table 7.2 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, 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 crossing 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)	

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 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 and 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 do not always have signals, which could create safety concerns.

Channeling of the center walkway has an increased safety level. Table 7.3 shows the before and after existence of pedestrian and bicycle walkways at the DDI locations selected for this study. Figure 7.2 through Figure 7.6 show images of center and outside walkways at Utah DDIs.

Edara et al. (2003) performed a simulation using VISSIM to analyze performance of the DDI in regard 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 a pedestrian, 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.

Table 7.3 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 and South)
278	Yes (North side)	Yes (outside - North and South)
284	No	Yes (outside - South side only)
13	No	No

Figure 7.2 St. George (Exit 8) DDI Center Walkway Aerial (ESRI ArcMap Imagery Basemap)

Figure 7.3 St. George (Exit 8) DDI Center Walkway Crossing Point (Google Maps)

Figure 7.4 St. George (Exit 8) DDI Center Walkway (Google Maps)


Figure 7.5 American Fork Main Street (Exit 278) DDI Outside Walkway Aerial (Google Maps)

Figure 7.6 American Fork Main Street (Exit 278) DDI Outside Walkway (Google Maps)

With the introduction of new DDIs, pedestrians and cyclists may elect a different route from origin to destination 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 effects of pedestrians and cyclists cannot be analyzed in this report due to lack of adequate data. It would be beneficial for future studies t 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.

8. CONCLUSIONS

This study analyzed crash data at five locations along the I-15 corridor and I-215 belt route, 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 to provide a statistical analysis of the increase or decrease of crashes at the location since the DDI conversion. The crash data were 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 Section 7, another major safety concern in the DDI involved 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 to be conducted, including detailed pedestrian and cyclist data to be collected including AADT, crashes involving vehicles and crashes not involving motorized vehicles.

Other future studies are also recommended to continue the analysis of the safety effects of the DDI. Further studies could be conducted with more data available after more DDIs in operation. Additional after data at DDIs across the United States also will provide more comprehensive safety improvement performance measures.

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APPENDIX A: SPSS REGRESSION ANALYSIS RESULTS

HSM Ramp

Model Information

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

Case Processing Summary

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

Continuous Variable Information

		N	Minimum	Maximum	Mean	Std. Deviation
Dependent Variable	crash	784	.0	29.0	1.098	2.0202
Covariate	lnAAD	784	6.525029658	10.14854914	8.889134805	.5736535637
Offset	T aadt	784	682.0	25554.0	8339.147	4103.4205
	lnL	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	
Pearson Chi-Square	1122.392	781	1.437
Scaled Pearson Chi-Square	1122.392 -	781	
Log Likelihood[b]	1101.768		
Akaike's Information Criterion (AIC)	2209.536		
Finite Sample Corrected AIC (AICC)	2209.567		
Bayesian Information Criterion (BIC)	2223.529		
Consistent AIC (CAIC)	2226.529		

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)	1a						
(Negative binomial)	1a						

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

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) lnaadtcr	-6.062	1.0267	-8.075 .288	-4.050 .494	34.863	1	.000
lnaadtoffon	.391	.0525	.232	.670	55.390	1	.000
(Scale)	.451	.1118			16.249	1	.000
(Negative binomial)	1a		.240	.356			
	.292	.0293					

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 .2289975964
	lnL	784	-1.83258146	-.478035801	-1.13902205	

Goodness of Fit[a]

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

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 .000 .634
lnAADT	.979	.1148	.754	1.204 .364	72.734	1	
lnL	-.117	.2455	-.598		.227	1	
(Scale)	1a			1.569			
(Negative binomial)	1.272	.1362	1.031				

Dependent Variable: crash
Model: (Intercept), lnAADT, lnL
a. Fixed at the displayed value.

ISAT Terminal

Model Information

Dependent Variable Probability Distribution Link Function	crash Negative binomial (MLE) 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	
Pearson Chi-Square	391.976	387	1.013
Scaled Pearson Chi-Square	391.976	387	
Log Likelihood[b]	-1179.456		
Akaike's Information Criterion (AIC)	2366.912		
Finite Sample Corrected AIC (AICC)	2367.015		
Bayesian Information Criterion (BIC)	2382.786		
Consistent AIC (CAIC)	2386.786		

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) lnaadtcr	-4.604 .414	.6643 .0502	-5.906 .316	-3.302 .513	48.045	1	.000
lnaadtoff	.299	.0620	.178	.421	68.221	1	.000
(Scale)	1a				23.288	1	.000
(Negative binomial)	.285	.0289	.234	.348			

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
	aadt	784	682.0	25554.0	8339.147	4103.4205
Offset	lnL	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	
Pearson Chi-Square	1779.270	780	2.281
Scaled Pearson Chi-Square	1779.270	780	
Log Likelihood[b]	-1024.522		
Akaike's Information Criterion (AIC)	2057.043		
Finite Sample Corrected AIC (AICC)	2057.095		
Bayesian Information Criterion (BIC)	2075.701		
Consistent AIC (CAIC)	2079.701		

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 .000
lnAADT	1.660	.3180	1.037	2.284	27.252	1	.018
aadt	-8.161E-5	3.4539E-5	.000	-1.391E-5	5.583	1	
(Scale)	1a .106[b]	.	.	.			
(Negative binomial)							

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

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) lnaadtcr	-5.387 .325	1.0887	-7.521 .213	-3.253 .437	24.481	1	.000
lnaadtoffon	.411	.0570	.179	.644	32.526	1	.000
(Scale)	1a	.1187			12.002	1	.001
(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 Probability Distribution Link Function	crash Negative binomial (1) 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	
Pearson Chi-Square	967.378	781	1.239
Scaled Pearson Chi-Square	967.378	781	
Log Likelihood[b]	-937.091		
Akaike's Information Criterion (AIC)	1880.182		
Finite Sample Corrected AIC (AICC)	1880.212		
Bayesian Information Criterion (BIC)	1894.175		
Consistent AIC (CAIC)	1897.175		

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 Lower	 Upper	Hypothesis Test Wald Chi-Square	 df	 Sig.
(Intercept) lnAADT	-8.703	1.0710	-10.802	-6.604	66.031	1	.000 .000 .864
lnL	.936	.1171	.707 -.519	1.166	63.914	1	
(Scale)	-.042 1a	.2435		.436	.029	1	
(Negative binomial)	1a						

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

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) lnaadtcr	-3.833 .351	.7140 .0546	-5.233 .244	-2.434 .459	28.818	1	.000
lnaadtoff	.243	.0666	.112	.373	41.346	1	.000
(Scale)	1a				13.286	1	.000
(Negative binomial)	.302	.0345	.242	.378			

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff
a. Fixed at the displayed value.

HSM Ramp Injury/Fatality

Model Information

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

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
	aadt	784	682.0	25554.0	8339.147	4103.4205
Offset	lnL	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	
Pearson Chi-Square	1037.688	781	1.329
Scaled Pearson Chi-Square	1037.688	781	
Log Likelihood[b]	-521.644		
Akaike's Information Criterion (AIC)	1049.287		
Finite Sample Corrected AIC (AICC)	1049.318		
Bayesian Information Criterion (BIC)	1063.281		
Consistent AIC (CAIC)	1066.281		

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)	1a						
(Negative binomial)	1a						

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

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) lnaadtcr	-9.866	1.4087	-12.627	-7.105	49.051	1	.000 .000
lnaadtoffon	.692	.0934	.509	.875	54.845	1	.006
(Scale)	.409	.1480	.119	.699	7.638	1	
(Negative	1a	.0567	.276	.502			
binomial)	.372						

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoffon
a. Fixed at the displayed value.

ISAT Ramp Injury/Fatality

Model Information

Dependent Variable Probability Distribution Link Function	crash Negative binomial (1) 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 6.525029658	8.0	.291	.7408
Covariate	lnAADT	784	-1.83258146	10.14854914	8.889134805	.5736535637 .2289975964
	lnL	784		-.478035801	-1.13902205	

Goodness of Fit[a]

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

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 .000 .549
lnAADT	1.061	.1707	.726 -.888	1.395	38.621	1	
lnL	-.208	.3469		.472	.360	1	
(Scale)	1a						
(Negative binomial)	1a						

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

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 .000
lnaadtcr lnaadtoff	.693	.0888	.519	.867	60.907	1	.000
(Scale)	.375	.0862	.206	.544	18.906	1	
(Negative	1a	.0549	.260	.478			
binomial)	.353						

Dependent Variable: crash
Model: (Intercept), lnaadtcr, lnaadtoff
a. Fixed at the displayed value.

APPENDIX B: STUDY LOCATIONI 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