Safety Evaluation of Diverging Diamond Interchanges in Missouri

Prepared by

Praveen Edara, Ph.D., P.E., PTOE
Carlos Sun, Ph.D., P.E., J.D.
Boris R. Claros, MSCE, Research Assistant
Henry Brown, MSCE, P.E., Research Engineer
Department of Civil and Environmental Engineering, University of Missouri-Columbia

Final Report Prepared for Missouri Department of Transportation
2015 January | Project TR201406 | Report cmr15-006

Safety Evaluation of Diverging Diamond Interchanges in Missouri

Praveen Edara, Ph.D., P.E., PTOE
Associate Professor
Department of Civil and Environmental Engineering
University of Missouri - Columbia

Carlos Sun, Ph.D., P.E., J.D.
Associate Professor
Department of Civil and Environmental Engineering
University of Missouri – Columbia

Boris R. Claros, MSCE
Research Assistant
Department of Civil and Environmental Engineering
University of Missouri – Columbia

Henry Brown, MSCE, P.E.
Research Engineer
Department of Civil and Environmental Engineering
University of Missouri - Columbia

A Report on Research Sponsored by

Mid-America Transportation Center

University of Nebraska-Lincoln

&

Missouri Department of Transportation

January 2015

Technical Report Documentation Page

1. Report No. cmr 15-006	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Safety Evaluation of Diverging Diamond Interchanges in Missouri		5. Report Date January 2015 6. Performing Organization Code
7. Author(s) Edara, P., Sun, C., Claros, B., Brown, H.		8. Performing Organization University of Missouri-Columbia
9. Performing Organization Name and Address University of Missouri-Columbia E 2509 Lafferre Hall, Columbia, MO 65211		10. Work Unit No. 11. Contract or Grant No. TR201406
12. Sponsoring Agency Name and Address Missouri Department of Transportation 105 W. Capitol Avenue Jefferson City, MO 65102		13. Type of Report and Period Covered 14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

The Diverging Diamond Interchange (DDI) has gained in popularity in the United States during the last decade. The operational benefits and lower costs of retrofitting a conventional diamond with a DDI have contributed to its increased use. Existing research on DDIs has focused primarily on the assessment of operational benefits. Unfortunately, formal safety evaluations of DDIs are lacking. This study filled the knowledge gap by conducting a safety evaluation at the project-level (interchange) and the site-specific level (ramp terminals) of DDIs using three types of before-after evaluation methods: Naïve, Empirical Bayes (EB), and Comparison Group (CG). Three evaluation methods were used since the methods involved different trade-offs, such as data requirements, complexity, and regression-to-the-mean. The safety evaluation at the project-level accounts for the influence of the DDI treatment in the entire footprint of the interchange. On the other hand, the site-specific approach focused on the influence at the ramp terminals only. All three methods showed that a DDI replacing a conventional diamond decreased crash frequency for all severities. At the project-level, the highest crash reduction was observed for fatal and injury (FI) crashes – 63.2% (Naïve), 62.6% (EB), and 60.6% (CG). Property damage only crashes were reduced by 33.9% (Naïve), 35.1% (EB), and 49.0% (CG). Total crash frequency also decreased by 41.7% (Naïve), 40.8% (EB), and 52.9% (CG). Similarly, in the site-specific analysis, the highest crash reduction was observed for fatal and injury (FI) crashes – 64.3% (Naïve), 67.8% (EB), and 67.7% (CG). Property damage only crashes were reduced by 35.6% (Naïve), 53.4% (EB), and 47.0% (CG). Total crash frequency also decreased by 43.2% (Naïve), 56.6% (EB), and 53.3% (CG). A collision type analysis revealed that the DDI, as compared to a diamond, traded high severity for lower severity crashes. While 34.3% of ramp terminal-related FI crashes in a diamond occurred due to the left turn angle crashes with oncoming traffic, the DDI eliminated this crash type. In summary, the DDI offers significant crash reduction benefits over conventional diamond interchanges.

17. Key Words Diverging Diamond, Safety, Alternative Designs, Empirical Bayes, DDI	18. Distribution Statement		
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 49	22. Price

Form DOT F 1700.7 (06/98)

Table of Contents

List of Figures

List of Tables

Acknowledgments

The authors are thankful for the assistance provided by MoDOT staff John Miller, Mike Curtit, Drew Williford, Ashley Reinkemeyer, Jen Harper, Myrna Tucker, Michelle Neuner, and other district-level MoDOT staff. The authors wish to acknowledge the contributions of Paige Martz, Calvin Fales, Dylan Hackman, and Lily Riehl for their assistance with the crash analysis.

Disclaimer

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the information presented herein. This document is disseminated under the sponsorship of the U.S. Department of Transportation's University Transportation Centers Program, in the interest of information exchange. The U.S. Government assumes no liability for the contents or use thereof.

Abstract

The Diverging Diamond Interchange (DDI) has gained in popularity in the United States during the last decade. The operational benefits and lower costs of retrofitting a conventional diamond with a DDI have contributed to its increased use. Existing research on DDIs has focused primarily on the assessment of operational benefits. Unfortunately, formal safety evaluations of DDIs are lacking. This study filled the knowledge gap by conducting a safety evaluation at the project-level (interchange) and the site-specific level (ramp terminals) of DDIs using three types of before-after evaluation methods: Naïve, Empirical Bayes (EB), and Comparison Group (CG). Three evaluation methods were used since the methods involved different trade-offs, such as data requirements, complexity, and regression-to-the-mean. The safety evaluation at the project-level accounts for the influence of the DDI treatment in the entire footprint of the interchange. On the other hand, the site-specific approach focused on the influence at the ramp terminals only. All three methods showed that a DDI replacing a conventional diamond decreased crash frequency for all severities. At the project-level, the highest crash reduction was observed for fatal and injury (FI) crashes – 63.2% (Naïve), 62.6% (EB), and 60.6% (CG). Property damage only crashes were reduced by 33.9% (Naïve), 35.1% (EB), and 49.0% (CG). Total crash frequency also decreased by 41.7% (Naïve), 40.8% (EB), and 52.9% (CG). Similarly, in the site-specific analysis, the highest crash reduction was observed for fatal and injury (FI) crashes – 64.3% (Naïve), 67.8% (EB), and 67.7% (CG). Property damage only crashes were reduced by 35.6% (Naïve), 53.4% (EB), and 47.0% (CG). Total crash frequency also decreased by 43.2% (Naïve), 56.6% (EB), and 53.3% (CG). A collision type analysis revealed that the DDI, as compared to a diamond, traded high severity for lower severity crashes. While 34.3% of ramp terminal-related FI crashes in a diamond occurred due to the left

turn angle crashes with oncoming traffic, the DDI eliminated this crash type. In summary, the DDI offers significant crash reduction benefits over conventional diamond interchanges.

Chapter 1 Introduction

Recently in the U.S., the Diverging Diamond Interchange (DDI) has become a popular alternative to other forms of interchange designs. Since the first DDI installation in Springfield, Missouri, in 2009, there have been more than 30 locations across the U.S. where DDIs have been installed. Three factors have contributed to this rapid adoption of the DDI in the U.S. First, the operational benefits of the DDI, including lower overall delay and higher left turn movement capacity compared to a conventional diamond, have made it an attractive alternative (Bared et al. 2006; Edara et al. 2005). Second, the lower costs of retrofitting an existing diamond interchange with a DDI have also played an important role in its adoption. For example, a cost comparison between the DDI and the Tight Urban Diamond Interchange (TUDI) in Kansas City, Missouri, showed the DDI cost approximately 50% less (Hughes et al. 2010; MoDOT 2014). Third, fewer conflict points compared to a conventional diamond, along with positive safety results from limited safety evaluations, have provided further encouragement about the merits of the design evaluations (Edara et al. 2005).

The operation of a DDI is shown in figure 1.1. In this figure, the freeway runs north and south, while the crossroad runs east and west. Two ramp terminals are shown – west crossover and east crossover. Proceeding from west to east, at the west crossover, the eastbound through and left turn traffic crisscrosses the westbound through traffic. At the east crossover the eastbound left turn movements diverge from the through traffic while the through traffic proceeds and crisscrosses the westbound through and left turn traffic. Thus, vehicles drive on the ‘wrong’ side on the crossroad between the two crossovers. The traffic that travels from east to west experiences the same crossovers. Four movement types are shown in figure 1.1. Eastbound and westbound through movements are shown in figure 1.1 (a), left turn movements from the crossroad onto the freeway are shown in figure 1.1 (b), left turn movements from freeway exit ramps to the crossroad are shown in figure 1.1 (c), and all right turn movements are shown in figure 1.1 (d).

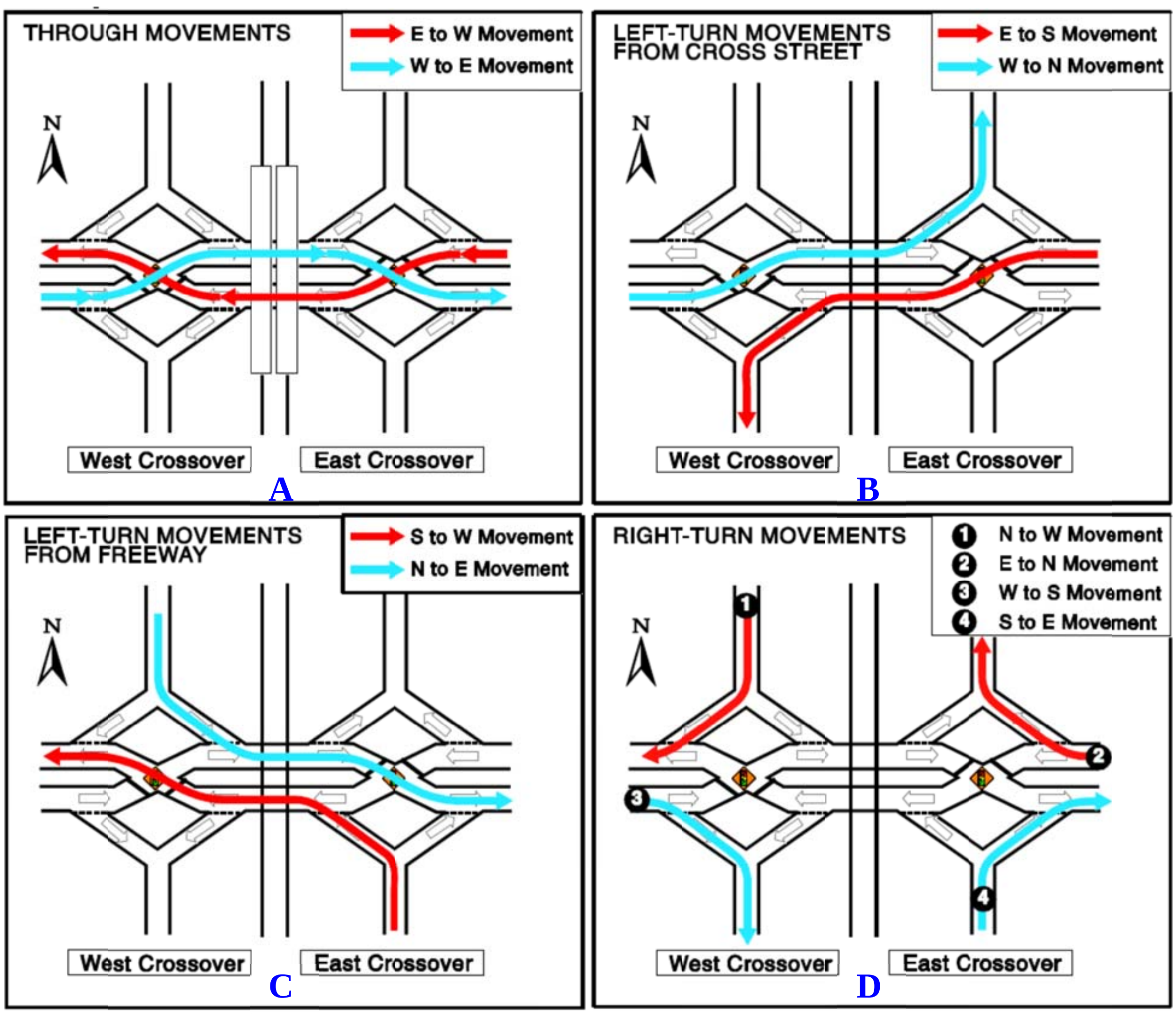


Figure 1.1 Traffic movements at a DDI (FHWA 2014)

The main impetus behind the initial research on DDI was to evaluate its operational benefits as compared to other designs. While the seminal study of Chlewicki (2003) illustrated the delay savings resulting from a DDI, the follow-up studies by Edara et al. (2005) and Bared et al. (2006) further confirmed its operational benefits, specifically the doubling of left turn movement capacity. Several subsequent studies have agreed with these early studies on the operational benefits of DDIs (Chlewicki 2013; Chilukuri et al. 2011). Because the motivation behind the initial research into the DDI was improving operational benefits, there has been a gap in the existing knowledge pertaining to the safety performance of the DDI. A preliminary assessment of the safety of an intersection or interchange design was examined using conflict points. Figure 1.2 shows the conflict points for both a DDI and a conventional diamond interchange. The DDI has 18 conflict points (2 crossing, 8 merging, and 8 diverging), while the conventional diamond interchange has 30 conflict points (10 crossing, 10 merging, and 10 diverging) (Hughes et al. 2010; Chlewicki 2013; FHWA 2004). Fewer conflict points across all conflict types reduce the exposure of traffic to crashes. More importantly, 8 out of 10 crossing conflict points are eliminated by the DDI design. Crossing conflicts typically result in right angle collisions that have a higher potential for injuries (Hughes et al. 2010).

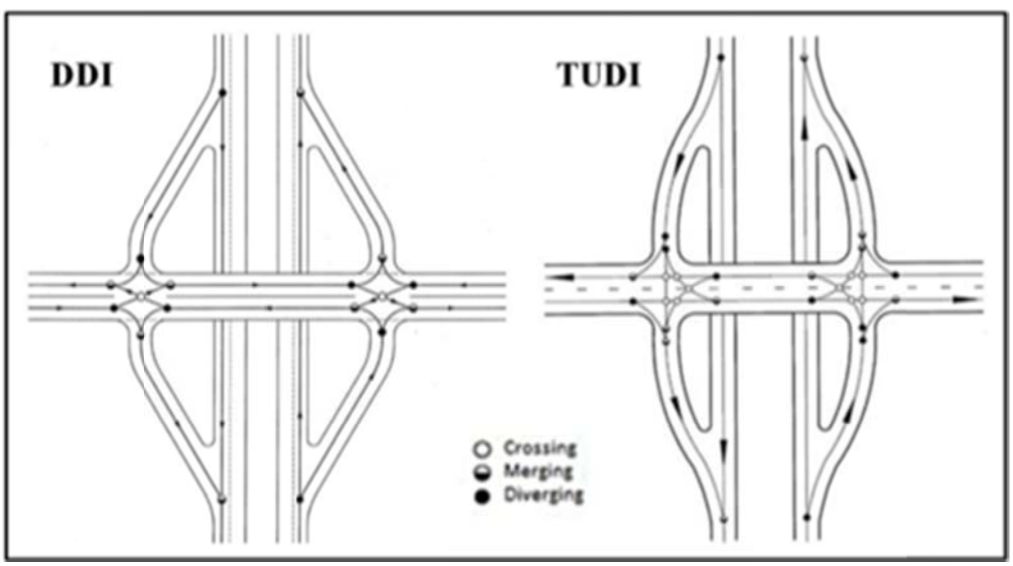


Figure 1.2 Conflict points at DDI and TUDI interchanges (FHWA 2004)

Typically, empirical safety evaluations of new alternative designs are not possible until a few years after they are introduced into practice due to the lack of sufficient crash data. One study (Chilukuri 2011) reviewed crash data for a one-year period after the first DDI was constructed in Springfield, Missouri. The study concluded that the DDI was operating safely based on a comparison of before and after crash frequencies. But the small sample size did not allow for a rigorous statistical safety evaluation.

Due to the crossover of traffic at the two ramp terminals in a DDI, there was some initial apprehension about the potential for wrong-way crashes (Chlewicki 2003). Some of these concerns were alleviated through human factor studies conducted by the Federal Highway Administration (FHWA). Using driver simulator studies, FHWA showed that wrong-way maneuvers were minimal and not statistically different from those at a conventional diamond interchange (Inman 2007). There are no empirical studies using real-world crash data either confirming or denying the higher frequency of wrong-way crashes at a DDI. There are also no empirical studies analyzing differences in the types and frequencies of crashes between a DDI and a conventional diamond.

The current study filled the knowledge gap in the safety of the DDI. Data from six sites in Missouri were used to conduct a before-after evaluation of the DDI. Missouri was the first state to have built a DDI and has the largest number of DDIs built or under construction (15 as of the writing of this report). Thus, Missouri offers a rich dataset for conducting a safety evaluation of DDIs. The safety evaluation consisted of three types of observational before-after evaluation methods: Naïve, Empirical Bayes (EB), and Comparison Group (CG). The approach with the three evaluation methods consisted of project-level analysis (complete interchange footprint crashes) and site-specific analysis (ramp terminal related crashes). Collision diagram analysis was also conducted to determine differences in crash types between a DDI and a conventional diamond interchange.

This study made a few key contributions to the body of literature on DDI performance. First, this was the first study to conduct a system-wide safety evaluation using multiple DDI sites. Second, this study presented the first extensive safety evaluation of DDIs, at both project and site-specific levels, using three before-after analysis methods. Third, crash modification factors (CMF) at both project and site-specific levels were developed for the first time for the DDI for total, fatal and injury, and property damage only crashes. The CMF values provide the expected reduction in crashes achieved by a DDI as compared to a conventional diamond interchange. Fourth, an extensive review of the collision diagrams was conducted to derive trends in the types of crashes before and after a DDI was installed at the study sites.

Chapter 2 Methodology

This chapter discusses the research methodology involving crash type and crash severity analysis, and three different before-after statistical methods. Figure 2.1 shows a schematic of the different steps involved in the methodology including site selection process, data collection, review of crash reports, analysis of crashes, and application of before-after statistical evaluation methods.

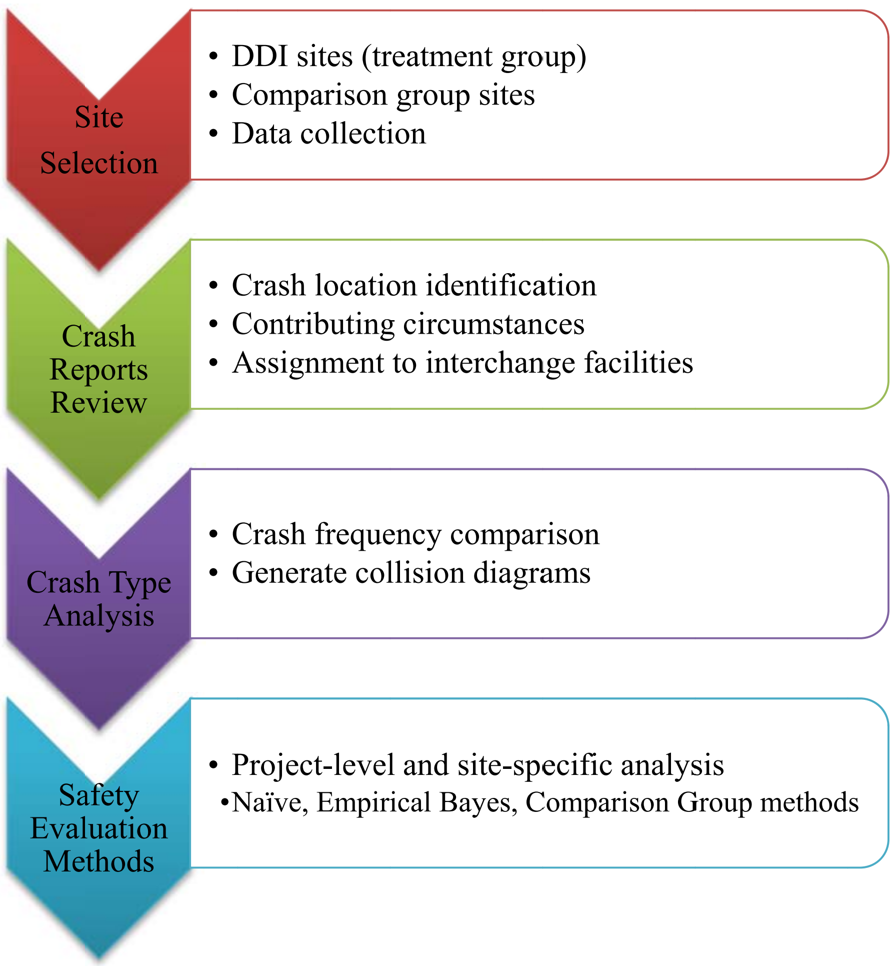


Figure 2.1 Steps involved in the research methodology

2.1 Site Selection and Data

The before-after safety analysis of DDI designs implemented in Missouri was conducted using data from six DDI sites (treatment sites). Six additional sites were used for comparison group analysis (comparison sites).

2.1.1 Treatment Sites

Although there were ten operational DDI sites in Missouri at the time of this research, four sites were recently opened to traffic and did not have enough crash data for the after-implementation period. Therefore, these four sites were not included in the treatment group.

The duration of before and after periods, as shown in table 2.1, was determined by taking into account seasonality and construction effects. Five years of crash data were processed for the before period but some were unused in order to match the after period data. The after period duration varied depending on the opening date of the DDI. The after period ranged from one year to four years for the six sites. All six DDI designs replaced conventional diamond interchanges. Table 2.1 contains the following characteristics of the six DDI locations: traffic volume, date opened to traffic, the duration of before and after periods, geometric characteristics, pedestrian crossings, traffic control for left turn movements from the crossroad to the entrance ramp, and the right turn movements from the exit ramp to the crossroad. Figures 2.2 to 2.7 are aerial photographs of the six DDI sites.

Table 2.1 DDI site characteristics

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

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

Figure 2.2 RT-13 and I-44, Springfield, MO

Figure 2.3 I-270 and Dorsett Rd, Maryland Heights, MO

Figure 2.4 James River Exp. and National Ave., Springfield, MO

Figure 2.5 US 65 and MO 248, Branson, MO

Figure 2.6 I-435 and Front Street, Kansas City, MO

Figure 2.7 Chestnut Exp. and Route 65, Springfield, MO

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

Crash data was collected for the entire interchange footprint for the study periods reported in table 2.1. The footprint included the influence areas of all interchange components. For freeways, crashes were included from the beginning of speed change lanes to the end of speed change lanes in both directions of travel. For the crossroad, the influence area included 250 ft. (76 m) from the ramp terminals, and crashes were collected for the ramp terminals and the crossroad segment in between the terminals. This footprint is recommended in the Highway Safety Manual (HSM) (AASHTO 2010). Figure 2.8 illustrates in detail the footprint of the interchange and the facilities considered.

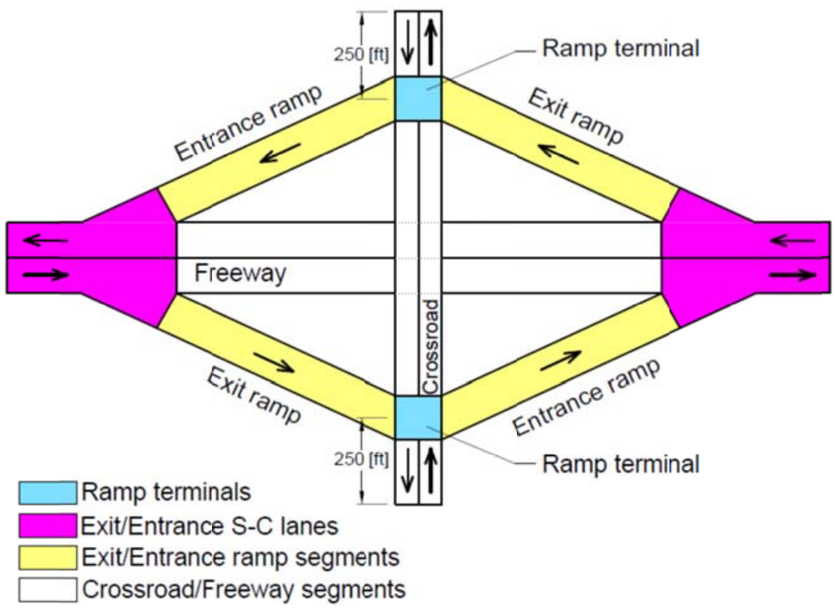


Figure 2.8 Interchange footprint

2.1.2 Comparison Sites

One comparison site was chosen for each treatment site. Each comparison site was carefully selected by matching the traffic, geometric characteristics, and crash frequency (during the before period) of the treatment site. The set of comparison sites is called the comparison group, and figures 2.9 to 2.14 are aerial photographs of these sites. Seasonality and construction effects were accounted for here just as in the collection of after treatment data as described in section 2.1.1.

Figure 2.9 US 60 and US 160, Springfield, MO

Figure 2.10 IS 170 and Page Ave., Overland, MO

Figure 2.11 US 65 and Division St., Springfield, MO

Figure 2.12 US 65 and Branson Hills Pkwy., Branson, MO

Figure 2.13 IS 435 and 23rd Trfy., Kansas City, MO

Figure 2.14 US 65 and Battlefield Rd. Springfield, MO

The basic characteristics of the comparison group sites are presented in table 2.2. The geometric features considered were the number of lanes, horizontal curves, left turn lanes on the crossroad, presence of a median, and signal control. The geometric features and the AADTs of the comparison facilities were tracked over the study period to ensure that they did not vary significantly or witness high traffic volume fluctuations over the years. The comparison samples were selected considering all the aforementioned features, but interchanges with identical features are difficult to find. Therefore, for each DDI site, the best matched interchange, one which was similar with respect to most of the important traffic and geometric features, was selected. A yoked comparison was employed, which is a special case of comparison group analysis where a single comparison site is matched to one treatment site based on similar conditions (Gross et al. 2010). For example, US 65 and Chestnut Exp. was yoked to US 65 and Battlefield Rd., both located in Springfield. As shown in the last column of tables 2.1 and 2.2, the AADTs for crossroad and freeway are similar, as are the speed limits and the number of lanes. However, one was an overpass, while the other was an underpass.

Table 2.2 Comparison group sites description

Site Location		US 60 and US 160 Springfield, Greene, MO	IS 170 and Page Ave. Overland, St. Louis, MO	US 65 and Division St. Springfield, Greene, MO	US 65 and Branson Hills Pkwy. Branson, Taney, MO	IS 435 and 23rd Trfy. Kansas City, Jackson, MO	US 65 and Battlefield Rd. Springfield, Greene, MO
Crossroad	Speed (mph)[1]	50	40	45	35	45	40
	AADT[2]	18461	34358	11178	16767	22497	22725
	Lanes[3]	5	6	4	5	6	4
Freeway	Speed (mph)[1]	70	60	60	60	65	60
	AADT[2]	23902	120770	58988	29562	79635	65260
	Lanes	4	6	6	4	6	6
Configuration Type		Overpass	Underpass	Overpass	Overpass	Underpass	Overpass
Spacing Ramp Terminals (ft.)		680	400	440	680	310	475
Distance to Public Road (ft.)		290/1000	530/550	220/440	430/430	890/225	575/800
Left Turn Signal IN[4]		PO/PO	PO/PO	PP/PP	PP/PP	PO/PO	PP/PP
Exit Ramp Right Turn Signal[5]		Y/Y	Y/Y	Y/Y	Y/Y	Y/Y	SC/SC

Notes: [1] Posted speed
[2] AADT of 2013 for reference purpose only
[3] Lanes between ramp terminals
[4] IN = Left turns on crossroad segment between ramp terminals, PP = Protective Permissive, PO = Protected Only
[5] Y = Yield, SC = Signal Control

2.2 Crash Reports Review

Crash reports in Missouri use the statewide Missouri Uniform Crash Report (MUCR) format. The Missouri State Highway Patrol is the state depository for traffic crash reports with the responsibility of training their officers to complete the reports following the Statewide Traffic Accident Records System standards (STARS 2012). All crashes within the footprint of the interchange were queried for both before and after periods.

Some inconsistencies were found in the crash data obtained from the electronic crash database. One inconsistency was the inaccurate placement of crashes occurring within the footprint of an interchange, the so-called crash landing problem. Crashes occurring on the freeway were sometimes placed on the crossroad and vice versa. Additionally, interchange terminal crashes were often placed between the two terminals when the crash occurred at one or the other terminal. Other inconsistencies included errors in the orientation or direction of travel. Crash reports had to be manually reviewed in order to correct the crash landing problem. This manual review of crashes ensured that accurate crash data was used for HSM site-specific analysis, collision diagram generation, and HSM calibration of ramp terminal facilities.

The project-level safety evaluation includes all facilities within the interchange footprint, while the site-specific analysis of DDI focuses only on ramp terminal related crashes. Ramp terminal related means that a crash occurred due to the ramp terminal geometric design, operational performance, and the influence of these factors on driver behavior. According to common crash reporting practices, crashes that are within 250 ft. on the roadways away from the center of the intersection in the approaching direction of the crossroad legs and exit ramp segment, are considered intersection-related crashes (Vogt 1999; Bonneson, Geedipally and Pratt 2012). However, there are some specific exceptions to this threshold. For instance, a crash that occurs beyond 250 ft. in the exit ramp segment or crossroad legs, that was caused by queuing at the ramp terminal, is ramp terminal related. Rear end and sideswipe crashes occurring on the freeway due to the accumulation of traffic from the ramp terminal are considered ramp terminal related crashes since the contributing circumstances were generated by the ramp terminal congestion (Bauer and Harwood 1998). Figure 2.15 shows the possible locations of ramp terminal related crashes (areas highlighted in blue). These locations include the ramp terminal itself, the crossroad approach legs, the exit ramps, part of the entrance ramps, and a small section of the freeway adjacent to the exit ramps. Thus, all crash reports were carefully reviewed to account for all ramp terminal related crashes for conducting the site-specific analysis.

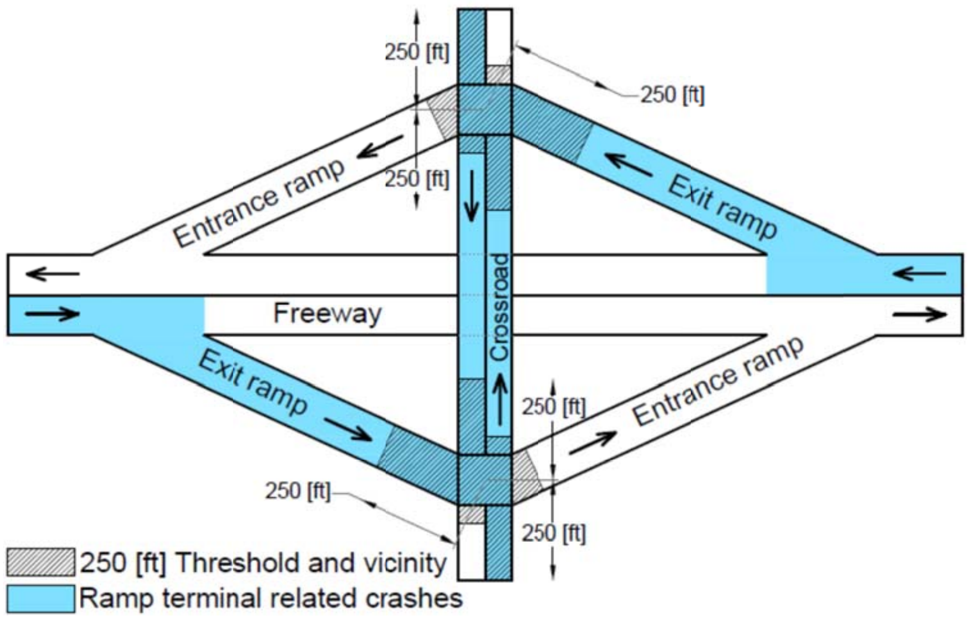


Figure 2.15 Area of interest for ramp terminal related crashes

2.3 Crash Type Analysis

Fatal and injury crash reports occurring at ramp terminals were reviewed to identify differences in the types of crashes occurring at a conventional diamond versus a DDI. The reports consisted of identical before and after period durations. A total of 356 months of crash data, 178 months for the conventional diamond interchanges (before period), and another 178 months for the Diverging Diamond Interchanges (after period), were analyzed.

Although crashes occurring at all interchange facilities were reviewed, only the crashes occurring at the ramp terminals or related to the ramp terminals were analyzed using the collision diagrams. This focus on ramp terminals was due to the fact that the primary difference between the conventional diamond and a DDI is the configuration of ramp terminals and the interaction between traffic movements at the terminals.

A collision diagram showing the location, type of crash, and relative frequency of occurrence was created using before and after data. The use of a collision diagram to analyze crash types is often used for analyzing intersection safety, including roundabout safety (FHWA 2010). The collision diagram facilitated the identification and visualization of trends, locations, geometric influence, and vehicle trajectories before the collision. As a result, collision diagrams with the crash type and frequencies at both conventional diamond interchanges and DDIs were developed.

2.4 Safety Effectiveness Evaluation

Safety effectiveness evaluations use quantitative estimates of how a treatment, project, or a group of projects affected crash frequencies or severities. The effectiveness estimate is useful for future decision-making and policy development (AASHTO 2010). The observational before and after evaluation methods used in this study compared the anticipated safety of a site without the treatment in the after period to the actual safety of the entity with the treatment in the after period (Hauer 1997). Two approaches, project-level and site-specific level, with three different methods, were selected to evaluate the safety effectiveness of the DDI. The three methods were Naive, Empirical Bayes (EB), and Comparison Group (CG). These methods were selected due to their different approach and use in previous safety research (AASHTO 2010; Hummer et al. 2010). With project-level analysis, an interchange is considered the entire facility or project by aggregating the various facilities within its footprint. The facilities within the interchange footprint include ramp terminals, ramp segments, speed-change lanes, crossroad, and freeway segment, as previously shown in figure 2.8. On the other hand, site-specific analysis focuses on an individual facility type such as the ramp terminals as in this study. When a DDI replaces an existing diamond interchange, the ramp terminals and crossroads undergo the most significant changes. The project-level analysis produced the safety effect over the entire interchange footprint while the site-specific analysis produced the effects at the ramp terminals of a DDI.

2.4.1 Naive Method

The main impetus behind the Naïve before-after evaluation method is that the change in safety from the before period to the after period is the result of all the changes that may have occurred at the site, including the effect of treatment. The treatment may not be the only change that occurs at a site and thus attributing the change in safety to the applied treatment alone may not be accurate. Instead, the Naïve method assumes that the change in safety is caused by all factors that may have changed from the before period to the after period (Hauer 1997). The safety effectiveness is calculated using the expected number of crashes and the actual observed number of crashes for the after period as discussed in Hauer (1997).

The key steps of the procedure are presented here. The expected number of crashes for the after period (π) is calculated as

$$\pi = \sum r_d(j)K(j) \qquad (2.1)$$

where

π = expected crashes in the after period;

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

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

$$r_d(j) = \frac{Duration\ of\ after\ period\ (j)}{Duration\ of\ before\ period\ (j)} \qquad (2.2)$$

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

$$SE = 100 \times (1 - \theta) \qquad (2.3)$$

where

$$\theta = \frac{\frac{\lambda}{\pi}}{1 + \frac{var(\pi)}{\pi^2}} \qquad (2.4)$$

The variance is used to express the precision or statistical significance of the odds ratio. Thus, highly variable data is less precise.

2.4.2 Empirical Bayes Method

The second before-after method, Empirical Bayes (EB), has been used in previous studies to evaluate the safety effectiveness of alternative intersection designs (Hummer et al. 2010). The EB method is also recommended by the HSM (2010) for conducting safety evaluations. HSM discusses many safety effectiveness performance measures, such as percent reduction of crashes, shift in crash type and severity, and crash modification factors (CMF) (HSM 2010). For observational before-after studies, it is important to understand the underlying reasons for implementing a certain treatment. Sites chosen for implementing a DDI typically have either congestion or safety problems, or both. Thus, a selection bias is introduced into the sample. To account for this bias and the resulting regression to the mean, the HSM (AASHTO 2010) recommends using the EB method.

The EB method utilizes safety performance functions (SPF) to estimate the average crash frequency for treated sites during the after period as though the treatment had not been applied

(AASHTO 2010). This estimated average crash frequency is then compared with the actual crash frequency during the after period. The expected crash frequency is calculated as the weighted average of the observed crash frequency and the SPF-predicted crash frequency. The weights are determined using the overdispersion parameter of the SPF and are not dependent on the observed crash frequency. The comparison of the expected crash frequency and the observed crash frequency for the after period forms the basis for deriving safety effectiveness (AASHTO 2010).

The SPF and the associated CMFs for each facility type were used to predict crashes for each year of the study periods. The general form of the SPF used is shown in equation 2.5 as

$$N_{predicted} = N_{SPF} \times C_i \times (CMF_1 \times CMF_2 \times \ldots \times CMF_i) \quad (2.5)$$

where

$N_{predicted}$ = predicted crash frequency for a specific year of a site type (crashes/year);

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

C_i = calibration factor;

CMF_i = crash modification factor specific to a site type characteristic *i*.

For the site-specific analysis, the calibration factors for ramp terminals were developed following the HSM approach. Developing calibration factors for other interchange facilities such as speed change lanes and ramp segments was outside the scope of this study. Thus, for the project-level analysis, a calibration factor of 1.0 was used for all interchange facilities.

2.4.2.1 EB Project-Level

There are some differences in the way the EB method is applied at the project-level and site-specific level, which are described in this section and the next section. For project-level EB analysis, the predicted crash frequency for the whole interchange was obtained by summing the predicted values for all interchange facilities as shown in equation 2.6 (AASHTO 2010). Equation 2.6 is formatted as

$$N_{inter} = \sum_{years} \left(\sum_{i=1}^{2} N_{ramp\ ter,i} + \sum_{i=1}^{4} N_{ramp,i} + N_{freeway} + \sum_{i=1}^{4} N_{scl,i} \right) \quad (2.6)$$

where

N_{inter} = predicted crash frequency for all years of an interchange (crashes/year);

$N_{ramp\ ter,i}$ = predicted crash frequency for ramp terminal i (crashes/year);

$N_{ramp,i}$ = predicted crash frequency for ramp segment i (crashes/year);

$N_{freeway}$ = predicted crash frequency for freeway segment (crashes/year);

$N_{scl,i}$ = predicted crash frequency for speed-change lane i (crashes/year).

The expected crash frequency was calculated using a weighted average of all the facilities of an interchange, taking into account correlations among the facilities as recommended by the HSM (AASHTO 2010). According to Hauer et al. *(*1997, 2002*),* there are two bounds of correlation: perfectly correlated and independent facilities. The weight adjustment factors and expected crashes following the bounds of correlation are:

$$w_I = \frac{1.0}{1.0 + \frac{\sum_i^{all} k_i \times \left(N_{pred,i}\right)^2}{N_{pred,all}}} \quad (2.7)$$

$$w_C = \frac{1.0}{1.0 + \frac{\left[\sum_i^{all} \sqrt{k_i \times \left(N_{pred,i}\right)^2}\right]^2}{N_{pred,all}}} \quad (2.8)$$

$$N_{exp,I,all} = w_I \times N_{pre,all} + (1 - w_I) \times N_{obs,all} \quad (2.9)$$

$$N_{exp,C,all} = w_C \times N_{pre,all} + (1 - w_C) \times N_{obs,all} \quad (2.10)$$

where

w_I = weighted adjustment factor assuming independence for all sites;

w_C = weighted adjustment factor assuming perfect correlation for all sites;

k_i = overdispersion parameter for facility i;

$N_{pred,i}$ = predicted crash frequency for facility i;

$N_{pred,all}$ = total predicted crash frequency for all sites;

$N_{exp,I,all}$ = total expected crashes with independent correlation for all sites;

$N_{exp,C,all}$ = total expected crashes with perfect correlation for all sites;

$N_{obs,all}$ = total observed crashes for all sites.

For partial correlation conditions, Bonneson et al. (2012) recommend averaging the expected crash estimate of the perfect correlation and independent conditions. The average expected crash frequency of partial correlation conditions is:

$$N_{e,PC} = \frac{N_{exp,I} + N_{exp,C}}{2} \quad (2.11)$$

where

$N_{exp,PC}$ = expected number of crashes assuming partial correlation among interchange facilities;

$N_{exp,I}$ = expected number of crashes assuming independence among interchange facilities;

$N_{exp,C}$ = expected number of crashes assuming perfect correlation among interchange facilities.

2.4.2.2 EB Site-Specific Analysis

For site-specific EB analysis, the predicted crash frequency for each ramp terminal was obtained by predicting crashes using the prediction methodology described in the HSM (AASHTO 2010). All ramp terminals prior to the treatment were full diamond interchanges (D4 type in HSM). The prediction model for D4 ramp terminals is formatted in equation 2.12 as

$$N_{D4,i} = N_{SPF\ D4,i} \times C_{D4,i} \times (CMF_1 \times CMF_2 \times \ldots \times CMF_i) \quad (2.12)$$

where

$N_{D4,i}$ = predicted crash frequency for a specific year of a D4 ramp terminal with *i* lanes (crashes/year);

$N_{SPF\ D4,i}$ = predicted crash frequency for base SPF of a D4 ramp terminal with i lanes (crashes/year)

$C_{D4,i}$ = calibration factor for D4 ramp terminal with *i* lanes;

CMF_i = crash modification factor specific to a site type characteristic i.

The expected crashes were calculated based on the weighted value of each facility. The weight is a function of the predicted crashes and the SPF overdispersion parameter (k), and it is formatted in equation 2.13 as

$$w_{D4,i} = \frac{1}{1+k_{D4,i} \times \sum N_{D4,i}} \quad (2.13)$$

$$N_{exp\ D4,i} = w_{D4,i} \times N_{D4,i} + (1 - w_{D4,i}) \times N_{obs,i} \quad (2.14)$$

where

$w_{D4,i}$ = weighed value for D4 ramp terminal;

$k_{D4,i}$ = overdispersion parameter for D4 ramp terminal with *i* lanes;

$N_{D4,i}$ = predicted crash frequency for a specific year of a D4 ramp terminal with *i* lanes (crashes/year);

$N_{exp\ D4,i}$ = expected crashes at D4 ramp terminal type with *i* lanes;

$N_{obs,i}$ = observed crashes at ramp terminal *i*.

2.4.3 Comparison Group

A before and after comparison group method compares the after period crash frequency of treatment sites (DDI) with the crash frequency of a set of control (or comparison) sites. One comparison site was chosen for each treatment site. Each comparison site was carefully selected by examining the traffic, geometric characteristics, and crash frequency (during the before period) of the treatment site. The same procedure applies to both project-level and site-specific analysis.

The suitability of the comparison group was verified using the sample odds ratio test presented by Hauer (1997). This test compares crashes over a specified time period for the comparison and treatment groups during a period before the treatment was implemented. If the mean of the sequence of odds ratios is sufficiently close to 1.0 and the confidence interval includes the value of 1.0, then the candidate comparison group is considered a good candidate (Gross et al. 2010; Hauer 1997). The sample odds ratio is calculated as

$$Sample\ Odds\ Ratio(SOR) = \frac{\frac{T_1 \times C_2}{T_2 \times C_1}}{1 + \frac{1}{T_2} + \frac{1}{C_1}} \qquad (2.15)$$

where

T_1 = total crashes for treatment group in year 1;

T_2 = total crashes for treatment group in year 2;

C_1 = total crashes for comparison group in year 1;

C_2 = total crashes for comparison group in year 2.

The CG safety effectiveness is calculated using both observed crash data and predicted values. In the first step, SPFs are used to determine the predicted crashes for both before and after periods, and for treated and comparison sites. An adjustment factor by severity for each period is then calculated for each pair of treatment and comparison sites by dividing the total number of predicted crashes for the treatment site and the total number of predicted crashes for the comparison site. Each treated site is compared to all the comparison sites, thus there are adjustment factors for each pair of treatment and comparison sites. The expected crashes for comparison and treatment sites are then calculated using the adjustment factors and observed crashes. The safety effectiveness values for each site and for the entire treatment group are computed using the expected and observed crashes. The HSM provides the necessary equations and an illustrative example for computing the adjustment factors, expected crashes, and the safety for the CG method (AASHTO 2010).

Chapter 3 Results

3.1 Introduction

This chapter presents the results for the crash type analysis and the safety effectiveness evaluation discussed in the previous chapter. The final collision diagram is presented, describing the crash types for conventional diamond interchanges and Diverging Diamond Interchanges. The safety effectiveness evaluation results are presented, showing crash frequencies by severity, and the results from the other three methods: 1) Naïve, 2) Empirical Bayes (EB), and Comparison Group (CG).

3.2 Crash Type Analysis

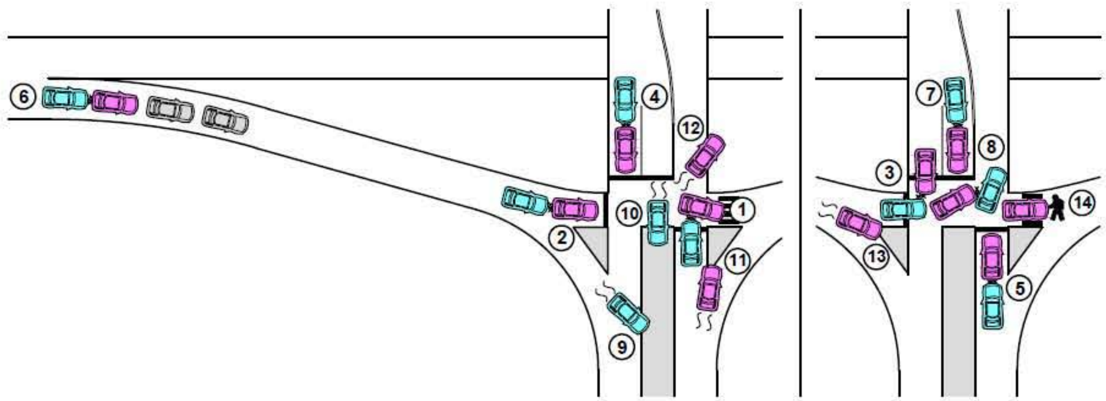
Since crash type analysis examines the total number of crashes in addition to percentages, the same data duration, before and after DDI implementation, was used for each site. Sites 1, 2, and 3 had the same duration of before and after periods. However, for sites 4, 5, and 6, the duration of the after period was shorter than the before period. Thus, the duration of the before period for sites 4, 5, and 6 was reduced to match the shorter after period. It is important to note that this adjustment in duration was only performed for the collision diagram analysis. The crash frequency analysis and the safety evaluation (Naïve, EB, CG) procedures used the actual durations listed in table 3.1. The collision diagrams for the before and after period are shown in figure 3.1. Crashes were classified into 14 different types for the before and after periods. Although the total number of crash types was 14 in both periods, the distribution and ranking of the types of crashes was different.

Table 3.1 DDI before/after duration

Site Location		RT-13 and I-44 Springfield, MO	I-270 and Dorsett Rd Maryland Heights, MO	James River Exp. and National Ave. Springfield, MO	US 65 and MO248 Branson, MO	I-435 and Front Street Kansas City, MO	Chestnut Exp. and Route 65 Springfield, MO
Opening Date		6/21/2009	10/17/2010	7/12/2010	11/20/2011	11/6/2011	11/10/2012
Periods (Months)	Before	51	35	38	44	44	40
	After	51	35	38	22	22	10

BEFORE PERIOD

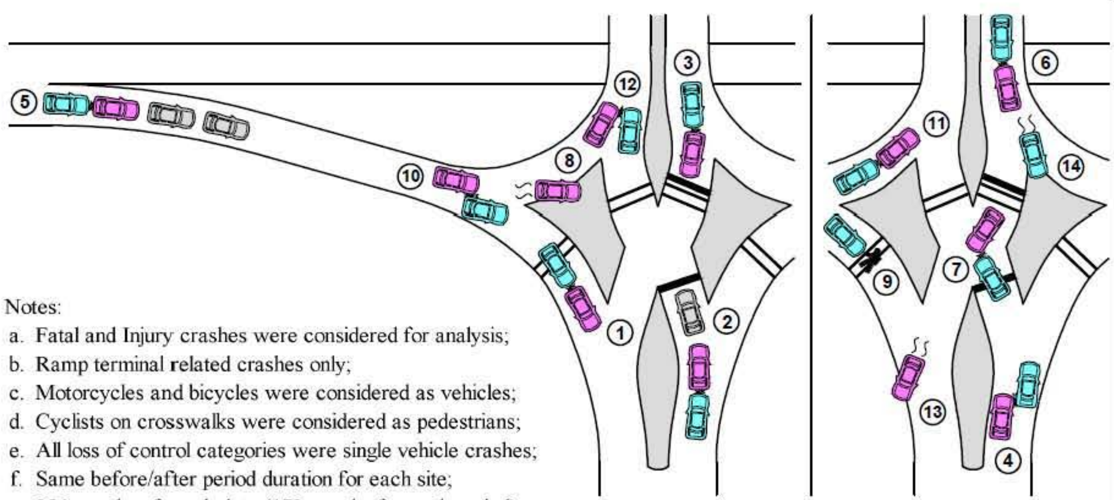
Conventional Diamond Interchange (D4 Ramp Terminals)



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

AFTER PERIOD

Diverging Diamond Interchange (DDI)



Notes:

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

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

Figure 3.1 Before / After collision diagrams for fatal and injury crashes

As seen in figure 3.1, the top two crash types in the before period at the conventional diamond ramp terminals were: 1) collision of left turn movements from inside the crossroad and the oncoming through movement, and 2) rear end collisions on the exit ramp at the intersection. In the after period for the DDI design, the top two crash types were: 1) rear end collisions between right turning movements on the exit ramp at the intersection, and 2) rear end collisions on the outside crossroad approach leg to the ramp terminal. It was also observed that some other types of crashes distributed across the different legs of the DDI ramp terminal increased, but all these crashes were of lower severity. For instance, sideswipes at the different merging and diverging locations, and the loss of control in the bays while making turning movements, increased with the DDI; however, none of these types of crashes resulted in any severe injuries. Thus, the DDI design traded a severe crash type, a right angle left turn crash, with less severe rear end, sideswipe, and loss of control crash types. The wrong way crashes inside the crossroad between the two ramp terminals accounted for 4.8% of the crashes occurring at the DDI ramp terminals.

3.3 Safety Effectiveness Evaluation

3.3.1 Crash Severity Analysis

The severity of crashes was studied during the before and after periods. The annual crash frequency was calculated for each treated facility, and it was classified into three severity categories: total crashes (TOT), fatal and injury (FI), and property damage only (PDO). Figure 3.2 shows the results of the calculations. The crash frequency for most of the facilities decreased for all severity categories.

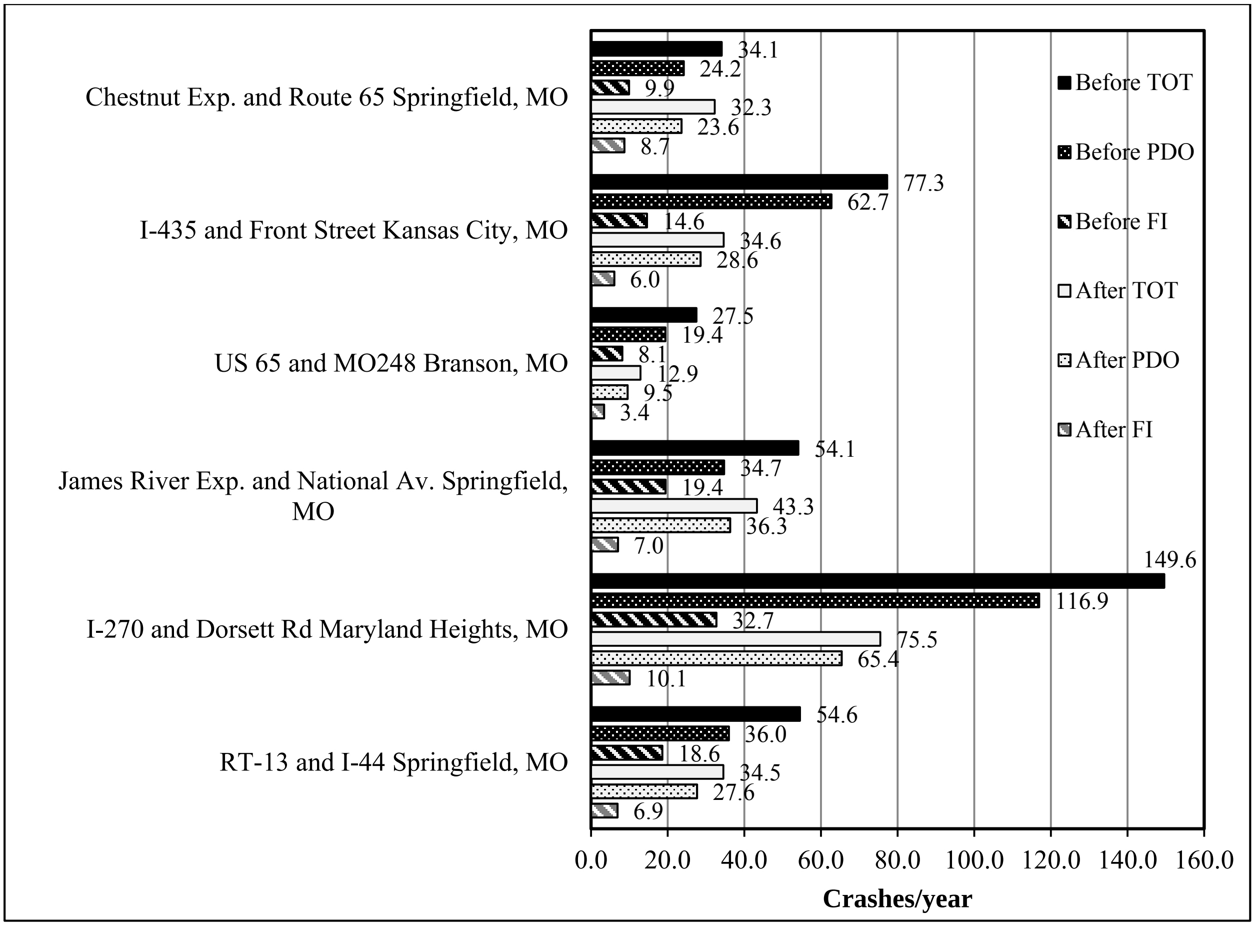


Figure 3.2 Crash frequencies before/after DDI implementation by facility

The crash data was aggregated across all six sites by severity type, and the annual crash frequency was calculated as shown in figure 3.3. The crash data was classified into four severity categories: minor injury, disabling injury, fatal, fatal and injury (FI), property damage only (PDO), and total crashes (TOT). Figure 3.3 shows that the crash frequency decreased for minor injury, disabling injury, and PDO crashes. There were no fatal crashes at any of the six sites before the installation of DDI. There was one pedestrian fatality that occurred during the after period at one site, but the details of that crash were unknown since it was a hit and run that occurred late at night. Since the fatal crash occurred within the footprint of the DDI, it was still included in the safety evaluation in this study. Figure 3.3 also presents the aggregate crash frequency of all injury crashes denoted by FI (fatal and injury) and the total number of crashes denoted by TOT. The percentage reductions in crash frequency after DDI implementation were 57.7% for FI (16.8 to 7.1), 26.5% for PDO (47.1 to 34.6), and 34.7% for TOT (63.9 to 41.7).

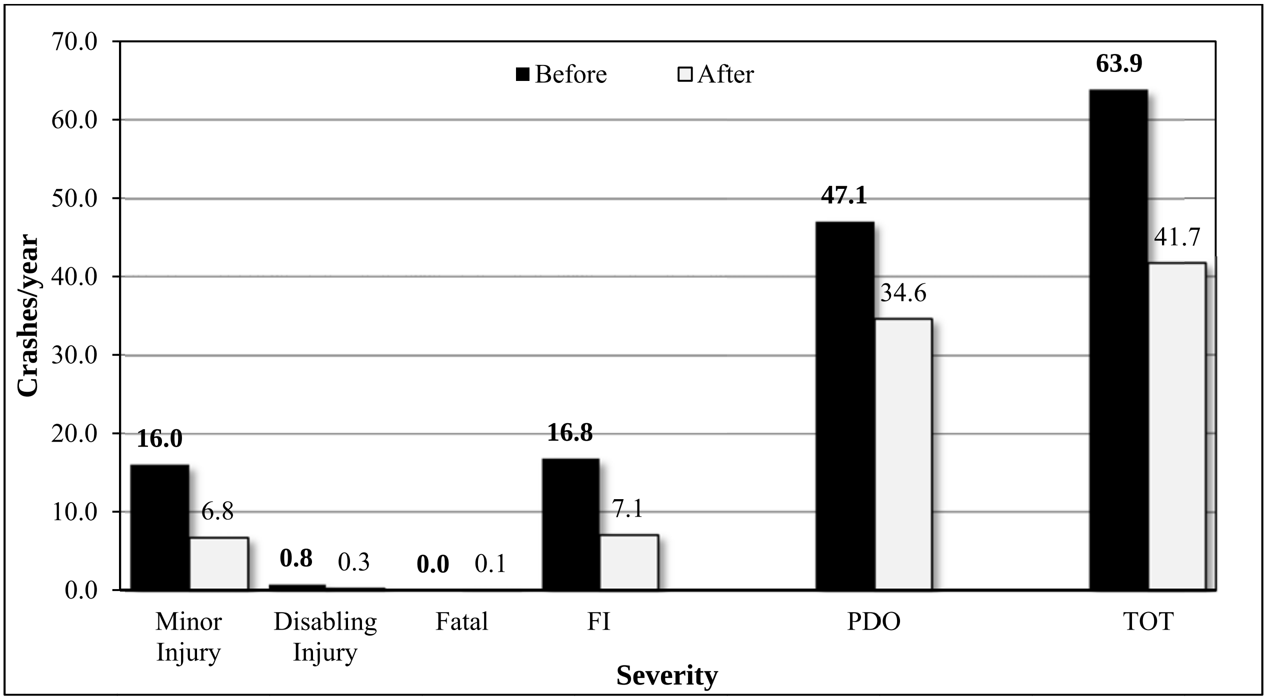


Figure 3.3 Crash frequencies before/after DDI implementation all facilities

3.3.2 Naïve Method

The odds ratio and safety effectiveness were computed for three categories of crashes – fatal and injury crashes (FI), property damage only crashes (PDO), and total crashes (TOT). As shown in table 3.2, the safety effectiveness results showed a 41.7% (2.9%) reduction in total crash frequency after DDI implementation. The value in the parenthesis denotes the standard error of the estimated safety effectiveness. The FI crash frequency experienced the greatest reduction of 63.2% (4.1%), while the PDO crash frequency decreased by 33.9% (3.7%). All reductions were statistically significant at the 95% confidence level.

Table 3.2 Naïve method results: project-level

Parameters	Estimates			St. Error		
	FI	PDO	TOT	FI	PDO	TOT
Observed Crashes After Period	104.00	506.00	610.00	10.20	22.49	24.70
Expected Crashes After Period	282.02	764.10	1046.12	15.99	25.99	30.51
Expected and Observed Difference	178.02	258.10	436.12	18.96	34.37	39.25
Odds Ratio	0.37	0.66	0.58	0.04	0.04	0.03
Safety Effectiveness (%)	63.2	33.9	41.7	4.1	3.7	2.9
95% Statistical Significance	Yes	Yes	Yes			

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

3.3.3 Empirical Bayes Method

The project-level EB method involved three choices for correlations previously discussed: independent, fully correlated, and partially correlated. The results for the three crash severity categories are shown in table 3.3. In table 3.3, the observed crashes, the EB expected crashes, and the safety effectiveness values for each site are reported in different rows. The standard error values are also reported in parenthesis next to each safety effectiveness value. The right-most column provides the results for the entire treatment group (combination of all six sites).

Since the actual correlation among the interchange facilities is not known, the safety effectiveness values obtained assume partial correlation can be used for determining the crash modification factors for the DDI (Bonneson 2012). The safety effectiveness values for partial correlation are highlighted in red bold text in table 3.3, although the results from the three correlation choices did not differ very much. For the entire treatment group ('All Sites' column in table 3.3), the percentage reduction in crashes was the greatest for FI crashes at 62.6% compared to the 35.1% for PDO and 40.8% for TOT crashes. These findings are consistent with the results of the crash severity analysis and the Naïve method. The left turn angle crashes that were predominant in the traditional diamond design (before period) were completely eliminated in the DDI design (after period), which accounts for the reduction in severe crashes.

The EB results for individual sites (see table 3.3) showed that the DDI was effective at decreasing the FI crashes at all six sites, although the reduction at the sixth site was not statistically significant at the 95% confidence level. The PDO crashes also decreased at all six sites with the reductions being statistically significant except for sites 3 and 6. The TOT crashes also decreased at all six sites, and all of the reductions were statistically significant except for site 6. The lack of statistical significance of the EB results for site 6 was due to two reasons. First, the duration of the after period for site 6 was the smallest among all six sites at 10 months. Thus, the lack of statistical significance can simply be the result of the small sample size. Second, the observed crash frequencies per year before DDI (10 FI, 24 PDO, 34 TOT) and after DDI (9 FI, 24 PDO, 32 TOT) were not considerably different.

Table 3.3 Project-level EB results

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

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

3.3.4 Comparison Group Method

The sample odds ratio is used for determining the suitability of the comparison group. For computing the sample odds ratio, a time frame of five years was chosen (2004 to 2009) before any DDIs in the treatment group were implemented. The mean, the standard error, and the 95% confidence interval of the sample odds ratio were computed, and the results are shown in table 3.4. The mean value for FI, PDO, and TOT crashes were 0.97 (0.31 standard error), 1.01 (0.20), and 1.00 (0.22), respectively, all close to 1.0. All 95% confidence intervals also included 1.0. Based on the sample odds ratio results and confidence intervals, the comparison group was deemed to be suitable for comparison with the treatment group following the FHWA guidelines for developing crash modification factors (Gross et al. 2010).

Table 3.4 Comparison of treatment and control sites

Sample Odds Ratio	Severity		
	FI	PDO	TOT
Mean	0.97	1.01	1.00
Standard Error	0.31	0.20	0.22
95% Confidence Interval	[0.36 - 1.58]	[0.62 - 1.40]	[0.57 - 1.43]

The safety effectiveness was then calculated using the comparison group (CG) method previously discussed. The CG method produced safety effectiveness values (and standard errors) of 60.6% (4.6%) reduction in FI crashes, 49.0% (3.0%) reduction in PDO crashes, and 52.9% (2.5%) reduction in TOT crashes, all significant at the 95% confidence level.

The project-level safety effectiveness results from the Naïve, EB, and CG methods are compared in table 3.5. The safety effectiveness values for each category (FI, PDO, TOT) are shown in different rows for the three methods. Again, the standard error values are reported in parenthesis next to each safety effectiveness value. The overall safety effectiveness values for the entire treatment group are also shown in the right-most column. The Naïve results for individual sites shown in table 3.5 revealed that the DDI was effective at decreasing FI crashes at all six sites, PDO crashes at five out of six sites (one site witnessed an increase that was not statistically significant), and total crashes at all six sites. The variation in the safety effectiveness values for FI crashes across the sites was not high. However, PDO and TOT crashes showed higher variation across the six sites. The EB results for individual sites were previously discussed. The CG results for individual sites, shown in table 3.5, indicated statistically significant reductions in FI crashes for sites 1, 2, and 3 only. Site 6 actually showed an increase in FI crashes, although it was not statistically significant. For the CG method, statistically significant reduction in PDO and TOT crashes were observed for the first five sites. Again, site 6 showed increases in PDO and TOT crashes that were statistically significant. In addition to the short duration of the after period and the lack of considerable variation in the observed crash frequency before and after DDI for site 6, one additional reason may have contributed to the CG results for site 6. The comparison site used for site 6 witnessed higher crash reductions for FI and TOT crashes. For comparison site 6, the observed crash frequencies per year in the before period were: 12 FI, 31 PDO, 42 TOT and in the after period were: 2 FI, 34 PDO, 36 TOT crashes. Is summary, able 3.5 shows that DDI decreased FI, PDO, and TOT crashes, and the results are similar across all three methods.

Table 3.5 Project-level safety effectiveness

Severity	Method	RT-13 and I-44 Springfield, MO (Site 1)	I-270 and Dorsett Rd. Maryland Heights, MO (Site 2)	James River Exp. and National Ave. Springfield, MO (Site 3)	US 65 and MO248 Branson, MO (Site 4)	I-435 and Front Street Kansas City, MO (Site 5)	Chestnut Exp. And Route 65 Springfield, MO (Site 6)	**All Sites (in %)**
FI	Naïve	63.3 (7.9)	69.5 (6.4)	64.5 (8.7)	60.0 (17.3)	59.3 (13.2)	15.1 (34.4) [1]	**63.2 (4.1)**
	EB	63.2 (7.8)	65.9 (7.0)	64.7 (8.4)	62.1 (15.8)	58.6 (12.8)	19.3 (30.9) [1]	**62.6 (4.1)**
	CG	70.7 (6.6)	71.6 (6.3)	69.9 (7.8)	37.9 (29.2) [1]	22.6 (26.7) [1]	-195.7 (142.5) [1]	**60.6 (4.6)**
PDO	Naïve	23.7 (9.4)	44.2 (5.1)	-3.6 (13.8) [1]	51.5 (13.0)	54.6 (6.9)	3.7 (24.3) [1]	**33.9 (3.7)**
	EB	36.0 (8.3)	40.2 (5.4)	7.1 (12.4) [1]	56.3 (11.2)	49.2 (7.4)	0.1 (23.5) [1]	**35.1 (3.7)**
	CG	60.9 (5.1)	60.6 (3.8)	37.4 (8.7)	44.0 (15.7)	32.4 (10.9)	-169.4 (76.7)	**49.0 (3.0)**
TOT	Naïve	37.0 (6.7)	49.7 (4.2)	20.5 (9.1)	53.6 (10.6)	55.4 (6.2)	6.2 (20.3) [1]	**41.7 (2.9)**
	EB	42.9 (6.2)	45.4 (4.5)	18.8 (9.0)	57.8 (9.2)	51.1 (6.4)	6.2 (18.8) [1]	**40.8 (2.9)**
	CG	64.0 (4.0)	63.3 (3.3)	49.2 (6.1)	44.7 (13.3)	32.3 (9.9)	-163.8 (64.5)	**52.9 (2.5)**

Notes: Standard error values are shown in the parenthesis next to the safety effectiveness
[1] Not significant at the 95% confidence level

3.4 Results of Site-Specific Analysis

The statistical sample size for conducting site-specific analysis was two times the one used for project-level analysis since each interchange has two ramp terminals. Thus, a total of 12 ramp terminals from the DDI sites and another 12 ramp terminals from the CG sites were included in the analysis. As previously discussed, the Empirical Bayes method uses a different set of equations for the site-specific analysis. The Naïve and CG approaches for site-specific analysis are identical to those used for project-level analysis.

3.4.1 Naive Method

The safety effectiveness results of the Naïve method are shown in table 3.6. The FI crash frequency experienced the greatest reduction of 64.3% (5.4%), while the PDO crash frequency decreased by 35.6% (4.8%), and the total crash frequency decreased by 43.2% (3.8%) after DDI implementation. The values in the parenthesis denote the standard error of the estimated safety effectiveness. All reductions were statistically significant at the 95% confidence level.

Table 3.6 Naïve method results: site-specific analysis

Parameters	Estimates			St. Error		
	FI	PDO	TOT	FI	PDO	TOT
Observed Crashes After Period	57.00	280.00	337.00	7.55	16.73	18.36
Expected Crashes After Period	158.76	433.79	592.55	12.15	19.95	23.36
Expected and Observed Difference	101.76	153.79	255.55	14.31	26.04	29.71
Odds Ratio	0.36	0.64	0.57	0.05	0.05	0.04
Safety Effectiveness (%)	64.3	35.6	43.2	5.4	4.8	3.8
95% Statistical Significance	Yes	Yes	Yes			

3.4.2 Empirical Bayes Method

The safety effectiveness was calculated following the site-specific EB methodology previously described. The results of EB method for individual sites and all sites combined are shown in Table 3.7. The EB method produced safety effectiveness values (and standard errors) of 67.8% (4.7%) reduction in FI crashes, 53.4% (3.5%) reduction in PDO crashes, and 56.6% (2.8%) reduction in total crashes, all significant at the 95% confidence level. Additionally, the results for individual sites (ramp terminals) showed significant reduction at a majority of the sites. The ramp terminal of the DDI at Chestnut Exp. and Route 65, in Springfield, Missouri, had negative safety effectiveness results for PDO and TOT, but the results were *not significant*.

Table 3.7 Site-specific EB results

Severity		FI			PDO			TOT		
Location	Ramp Terminal[1]	Observed Crashes	EB Expected Crashes[2]	SE (St.E.)[3]	Observed Crashes	EB Expected Crashes	SE (St.E.)	Observed Crashes	EB Expected Crashes	SE (St.E.)
RT-13 and I-44 Springfield, MO	S	10	22	54.6 (15.8)	52	65	19.9 (15.3)[4]	62	87	28.7 (11.7)
	N	9	29	68.8 (11.2)	32	103	69.1 (6.6)	41	132	69.0 (5.6)
I-270 and Dorsett Rd Maryland Heights, MO	E	5	21	76.5 (11.1)	32	81	60.6 (8.4)	37	102	63.9 (7.0)
	W	9	34	73.7 (9.6)	47	166	71.8 (4.8)	56	201	72.1 (4.3)
James R. Exp. and Nat. Ave. Springfield, MO	S	6	18	67 (14.3)	44	36	-22.6 (26.6)[4]	50	54	7.5 (17.8)[4]
	N	12	26	54.1 (14.6)	43	67	36 (12.4)	55	93	41.1 (9.6)
US 65 and MO248 Branson, MO	E	2	5	58.0 (29.7)[4]	1	9	88.8 (10.9)	3	14	78.1 (12.8)
	W	1	7	85.6 (14.1)	9	22	59.7 (14.6)	10	29	65.9 (11.6)
I-435 and Front Street Kansas City, MO	E	1	5	79.8 (19.8)	5	23	78.0 (10.1)	6	28	78.3 (9.1)
	W	0	5	100.0 (0.3)	4	17	76.6 (12.0)	4	22	82.1 (9.1)
Chestnut Exp. and RT 65 Springfield, MO	E	0	2	100.0 (0.0)	2	5	55.9 (31.1)	2	6	67.5 (23.0)
	W	2	2	14.1 (60.7)[4]	9	4	-100.5 (75.8)[4]	11	7	-61.4 (54.1)[4]
All Sites Analysis		**57**	**177**	**67.8 (4.7)**	**280**	**599**	**53.4 (3.5)**	**337**	**776**	**56.6 (2.8)**

Notes: The expected crash values are rounded (up) to facilitate comparison with the observed crash values increase in crashes

[1] The ramp terminal designation was according to the location with respect to the center of the interchange and the orientation. N = North, S = South, E = East, and W = West

[2] SE denotes Safety Effectiveness (%). ST.E. denotes Standard Error (%).

[3] Not significant at the 95% confidence level

3.4.3 Comparison Group

The mean value of odds ratio for FI, PDO, and TOT crashes were computed and found to be close to 1.0. All 95% confidence intervals also included 1.0. These values mean that the CG was suitable. The safety effectiveness for site-specific analysis was then calculated using the CG method previously discussed. The CG method produced safety effectiveness values (and standard errors) of 67.7% (5.39%) reduction in FI crashes, 47.0% (4.52%) reduction in PDO crashes, and 53.3% (3.5%) reduction in TOT crashes, all significant at the 95% confidence level. A comparison of the results of the Naïve, EB, CG methods for the site-specific analysis is presented in table 3.8. Again, similar to table 3.5 (project-level), the site-specific analysis also shows that all three methods (Naïve, EB, and CG) resulted in similar improvements in interchange safety due to DDI implementation.

Table 3.8 Site-specific safety effectiveness

Severity		FI			PDO			TOT		
DDI Site	Ramp Terminal[1]	Naïve	EB	CG	Naïve	EB	CG	Naïve	EB	CG
1	S	60.0 (14.5)	54.6 (15.8)	63.2 (14.0)	-10.6 (21.9)[2]	19.9 (15.3)[2]	22.1 (15.9)[2]	12.7 (15.0)[2]	28.7 (11.7)	36.5 (11.2)
	N	70.0 (11.1)	68.8 (11.2)	76.5 (9.1)	46.7 (11.5)	69.1 (6.6)	72.3 (6.2)	53.9 (8.6)	69.0 (5.6)	73.7 (5.0)
2	E	61.5 (18.9)	76.5 (11.1)	63.0 (19.9)	45.8 (11.7)	60.6 (8.4)	54.2 (10.3)	47.9 (10.4)	63.9 (7.0)	56.0 (9.1)
	W	60.9 (14.8)	73.7 (9.6)	63.2 (14.8)	60.2 (6.8)	71.8 (4.8)	65.4 (6.1)	60.0 (6.3)	72.1 (4.3)	65.6 (5.6)
3	S	70.0 (13.3)	67 (14.3)	70.7 (13.9)	-41.9 (32.5)[2]	-22.6 (26.6)[2]	-35.3 (32.3)[2]	0.0 (19.7)[2]	7.5 (17.8)[2]	9.0 (18.5)[2]
	N	63.6 (11.9)	54.1 (14.6)	66.1 (11.8)	24.6 (15.0)[2]	36 (12.4)	37.1 (12.9)	38.2 (10.5)	41.1 (9.6)	49.2 (8.9)
4	E	63.7 (25.6)	58.0 (29.7)[2]	33.4 (52.6)[2]	85.7 (13.8)	88.8 (10.9)	71.3 (29.9)	75.0 (14.7)	78.1 (12.8)	54.2 (28.2)[2]
	W	83.3 (15.9)	85.6 (14.1)	73.2 (28.2)	42.0 (21.3)	59.7 (14.6)	13.8 (33.0)[2]	52.4 (16.4)	65.9 (11.6)	32.2 (24.1)[2]
5	E	81.8 (17.3)	79.8 (19.8)	61.3 (41.0)[2]	79.2 (9.6)	78.0 (10.1)	55.6 (21.0)	79.3 (8.7)	78.3 (9.1)	58.4 (18.0)
	W	100.0 (0.0)	100.0 (0.3)	100.0 (0.0)	75.0 (12.9)	76.6 (12.0)	46.1 (28.7)[2]	78.4 (11.1)	82.1 (9.1)	56.1 (23.2)
6	E	100.0 (0.0)	100.0 (0.0)	100.0 (0.0)	46.6 (37.7)[2]	55.9 (31.1)[2]	-75.0 (135.0)[2]	55.5 (31.4)[2]	67.5 (23.0)	-34.4 (102.3)[2]
	W	11.0 (62.5)[2]	14.1 (60.7)[2]	-284.6 (320.5)[2]	-125.2 (89.0)[2]	-100.5 (75.8)[2]	-899.8 (435.3)	-83.5 (64.5)[2]	-61.4 (54.1)[2]	-654.5 (287.7)
All Sites		**64.3 (5.4)**	**67.8 (4.7)**	**67.7 (5.39)**	**35.6 (4.8)**	**53.4 (3.5)**	**47.0 (4.52)**	**43.2 (3.8)**	**56.6 (2.8)**	**53.3 (3.5)**

Notes: Standard error values are shown in the parenthesis next to the safety effectiveness

[1] Ramp terminal designation was according to the location with respect to the center of the interchange and the orientation. N = North, S = South, E = East, and W = West

[2] Not significant at the 95% confidence level

Chapter 4 Conclusions

This research project addressed the safety evaluation of the Diverging Diamond Interchanges using crash and operational data from 6 interchanges in Missouri. Missouri was ideal for such a study because it was the first state to implement DDIs in the U.S., thus significant after treatment data was available. Collision diagram analysis was conducted to determine the crash type and frequency at conventional diamond interchanges and DDIs. A safety effectiveness evaluation was performed using crash rates by severity, and three observational before and after studies were used at the interchange (project-level) and ramp terminal (site-specific) levels: Naïve, Empirical Bayes (EB), and Comparison Group (CG).

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

The three before-after safety evaluation methods produced consistent results. The DDI design replacing a conventional diamond decreased crash frequency for all severities. At the project-level, the most significant crash reduction was observed for fatal and injury crashes – 63.2% (Naïve), 62.6% (EB) and, 60.6% (CG). Property damage only crashes reduced by 33.9% (Naïve), 35.1% (EB), and 49.0% (CG). The total crash frequency also decreased by 41.7%

(Naïve), 40.8% (EB), and 52.9% (CG). Similarly, in the site-specific analysis, the highest crash reduction was observed for fatal and injury (FI) crashes – 64.3% (Naïve), 67.8% (EB), and 67.7% (CG). Property damage only crashes were reduced by 35.6% (Naïve), 53.4% (EB), and 47.0% (CG). Total crash frequency also decreased by 43.2% (Naïve), 56.6% (EB), and 53.3% (CG). The safety effectiveness results for the individual sites also demonstrated that FI, PDO, and TOT crashes decreased at most sites after DDI implementation. This study documented the safety benefits of DDI, which complements the existing knowledge on the operational benefits of DDI.

References

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

Bared, J. G., P. Edara, and R. Jagannathan. Design and Operational Performance of Double Crossover Intersection and Diverging Diamond Interchange. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1912*, Transportation Research Board of National Academies, Washington, D.C., 2006, pp 31-38.

Bauer, K., and D. Harwood. *Statistical Models of Accidents on Interchange Ramps and Speed-Change Lanes*. FHWA-RD-97-106. Federal Highway Administration, Washington, D.C., 1998.

Bonneson J.A., S. Geedipally, and M. P. Pratt. *Safety Prediction Methodology and Analysis Tool for Freeways and Interchanges*. National Cooperative Highway Research Program, Project 17-45, National Cooperative Highway Research Program Transportation Research Board of The National Academies, Washington, D.C., 2012.

Chilukuri, V., S. Smith, T. Michael, and T. Ryan, 2011. *Diverging Diamond Interchange Performance Evaluation (I-44 and Route 13).* Report No. OR11-012, U.S. Department of Transportation, Federal Highway Administration, Missouri Department of Transportation.

Chlewicki, G. New Interchange and Intersection Designs: The Synchronized Split-Phasing Intersection and the Diverging Diamond Interchange. Presented at the 2nd Urban Street Symposium, Anaheim, California, 2003.

Chlewicki, G., 2013. Learning from Eight Operational Diverging Diamond Interchanges in the United States. Presented at the 92nd Annual Meeting of the Transportation Research Board, Washington, D.C.

Edara, P., J.G. Bared, and R. Jagannathan. Diverging Diamond Interchange and Double Crossover Intersection: Vehicle and Pedestrian Performance. Presented at the 3rd International Symposium of Highway Geometric Design, 2005 Chicago, IL.

FHWA, *Signalized Intersections: Informational* Guide, Publication No. FHWA-HTR-04-091U.S. Department of Transportation. Federal Highway Administration., Washington, DC, 2004.

FHWA, *Roundabouts: An Informational Guide (Second Edition)*. NCHRP Report 673, U.S. Department of Transportation, Federal Highway Administration, Transportation Research Board, 2010.

FHWA, *Diverging Diamond Interchange Informational Guide. Report No. FHWA-SA-14-067,* U.S. Department of Transportation, Federal Highway Administration, Washington, DC, 2014.

Gross, F., B. Persaud, and C. Lyon. A Guide to Develop Quality Crash Modification Factors. Report No. FHWA-SA-10-032. FHWA, U.S. Department of Transportation, 2010.

Hauer, E. *Observational Before-After Studies in Road Safety: Estimating the Effects of Highway and Traffic Engineering Measures on Road Safety*. Department of Civil Engineering, University of Toronto, Emerald Group Publishing Limited, United Kingdom, 1997.

Hauer, E., D.W. Harwood, F. Council, and M. Griffith. Estimating Safety by the Empirical Bayes Method: A Tutorial. In *Transportation Research Record, No. 1784*, TRB, National Research Council, Washington, D.C., 2002, pp. 126-131.

Hughes, W., R. Jagannathan, D. Sengupta, and J. Hummer. *Alternative Intersections and Interchanges: Informational Report*. Publication FHWA-HRT-09-060. FHWA, U.S. Department of Transportation, 2010.

Hummer, S. E., J. E. Haley, R. L. Ott, R. S. Foyleand, and C. M. Cunningham. *Superstreet Benefits and Capacities*. Report No. FHWA/NC/2009-06. North Carolina Department of Transportation, U.S. Department of Transportation, Federal Highway Administration, 2010.

Inman, V.W., 2007. *Driver's Evaluation of the Diverging Diamond Interchange*. Report No. FHWA-HRT-07-048. Federal Highway Administration, U.S. Department of Transportation.

MoDOT, *Interstate 435 and Front Street Interchange Improvements*. Missouri Department of Transportation.http://www.modot.org/kansascity/major_projects/Front_Street_DDI.htm. Accessed: May 19, 2014.

STARS, *Missouri Uniform Crash Report Preparation Manual (Version 2.0).* The Statewide Traffic Accident Record System (STARS), 2012.

Vogt, A. *Crash Models for Rural Intersections: Four-Lane by Two-Lane Stop-Controlled and Two-Lane by Two-Lane Signalized*. Report No. FHWA-RD-99-128. Federal Highway Administration, McLean, Virginia, 1999.