2020

Safety Evaluation of Innovative Intersection Designs: Diverging Diamond Interchanges and Displaced Left-turn Intersections

Ahmed Abdelrahman
University of Central Florida

SAFETY EVALUATION OF INNOVATIVE INTERSECTION DESIGNS: DIVERGING DIAMOND INTERCHANGES AND DISPLACED LEFT-TURN INTERSECTIONS

by

AHMED ABDELRAHMAN
B.S. Cairo University, 2012

A thesis submitted in partial fulfillment of the requirements
for the degree of Master of Science
in the Department of Civil, Environmental and Construction Engineering
in the College of Engineering and Computer Science
at the University of Central Florida
Orlando, Florida

Summer Term
2020

Major Professor: Mohamed Abdel-Aty

ABSTRACT

Diverging diamond interchanges (DDIs) and Displaced left-turn intersections (DLTs) are designed to enhance the operational performance of conventional intersections that are congested due to heavy left-turn traffic volumes. Since drivers are not familiar with these types of intersections, there is a need to evaluate their safety performance to validate their effect, and to estimate reliable and representative Crash Modification Factors (CMFs). The safety evaluation was conducted based on three common safety assessment methods, which are before-and-after study with comparison group, Empirical Bayes before-and-after method, and cross-sectional analysis. Furthermore, since DLTs showed poor safety performance, the study also investigated the operational performance of DLTs using a general linear model describing the relationship between traffic delay and other operational and geometric characteristics based on high-resolution traffic data. The DDI analysis included a sample size of 80 DDIs and 240 conventional diamond interchanges in 24 states, while the DLT analysis included 13 DLTs and 26 conventional intersections in 4 states. The analysis results indicated that converting conventional diamond interchanges to diverging diamond interchanges could significantly decrease the total, fatal-and-injury, rear-end and angle/left-turn crashes by 26%, 49%, 18%, and 68%, respectively. On the other hand, converting conventional intersections to displaced left-turn intersections could significantly increase the total number of crashes as well injury crashes and some other crash types (i.e., single vehicle, angle). However, the operational analysis implied that they have the potential to reduce the delay at intersections by 3.567 sec/veh. Consequently, the study quantified the costs and benefits associated with implementing DLTs. The results showed that this alternative design could provide much benefits in terms of its operational performance. However, its poor safety

performance could result in losses much higher than its benefits. The study concludes that DDIs could significantly decrease crash frequency, while DLTs could not provide safety benefits. However, DLTs might be more efficient for operational performance. It is recommended that appropriate safety countermeasures should be developed and implemented to enhance traffic safety at DLTs.

ACKNOWLEDGMENTS

I acknowledge all the appreciation to my advisor Professor Mohamed Abdel-Aty, for his support and advice which helped me completing my master's degree. I would also acknowledge Florida Department of Transportation for funding this study. I would like to acknowledge the support of my committee members, Dr. Naveen Eluru, and Dr. Yina Wu. I would also acknowledge the role of my parents who always provided me with a lot of support.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1: INTRODUCTION

Transportation professionals are challenged to achieve the mobility needs of high traffic demand with limited available resources. Drivers, pedestrians, and bicyclists have a bad experience at roadway at-grade intersections due to the increasing delay time and the exposure to safety risks. However, conventional intersections sometimes cannot mitigate these transportation problems. Consequently, many transportation engineers are investigating innovative intersection designs to enhance mobility and safety at intersections (Autey et al., 2013). These alternative intersections have different types and configurations (i.e., Continous Greens intersections, Median U-Turn intersections, etc.). However, all these configurations have a common important feature which is the elimination of left-turn movements at the main intersection. This results in reducing the number of potential conflict points and possible mobility and safety improvement. Two common alternative intersection designs are the diverging diamond interchange (DDI) and the displaced left-turn intersection (DLT) which is also known as continuous flow intersection (CFI).

The diverging diamond interchange is a popular alternative interchange design for improving traffic flow and reducing congestion. It is similar to the conventional diamond interchange except for how the left and through movements navigate between the ramp terminals. The purpose of this interchange design is to accommodate left-turning movements onto arterials and limited-access freeways while eliminating the need for a left-turn bay and a signal phase at the signalized ramp terminals. Figure 1 shows the typical movements that are accommodated in a DDI. The freeway is connected to the arterial by two on-ramps and two off-ramps in a manner similar to that of a conventional diamond interchange.

However, the main difference between a DDI and a conventional diamond interchange is the existence of crossovers on both sides of the interchange, which excludes the need for left-turning vehicles to cross the approaching through vehicles. This is achieved by shifting cross street traffic to the left side of the street between the signalized crossover intersections.

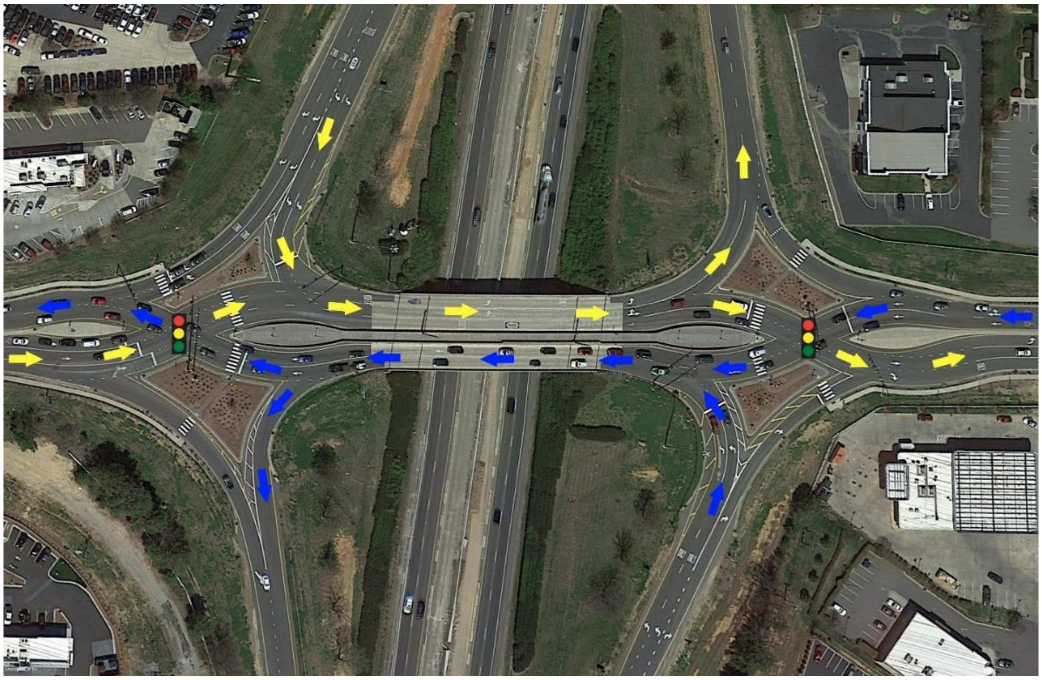


Figure 1: Different traffic movements at a typical DDI design (I-77 & Catawba Ave, Cornelius, North Carolina)

On the other hand, at displaced left-turn intersections, left-turn traffic is laterally displaced. In other words, left-turning traffic crosses over the opposing through movement at a location that is several hundred feet upstream of the major intersection. This upstream crossover location is typically controlled by a signal. The left-turning traffic then travels on a separated roadbed, which is on the outside of the opposing through lanes, as those vehicles proceed toward the major intersection. When these left-turning motorists reach the major intersection, they can proceed without conflict concurrently with the opposing through traffic.

The main feature of the DLTs is the relocation of the left-turn movement on an approach to the other side of the opposing roadway, which consequently eliminates the left-turn phase for this approach at the main intersection. As shown in Figure 2 (Hughes et al., 2010), traffic that would normally turn left at the main intersection first crosses the opposing through lanes at a signalized intersection, several hundred feet upstream of the main intersection.

Figure 2: Left-turn crossover movement at a three-legged partial DLT in Shirley, New York

Figure 3 (Hughes et al., 2010) shows a partial DLT where the DLT movement provisions have been implemented on two opposing approaches on the major road in this case. In most cases, the DLTs are on the major roadway. The left-turn movements of the minor road continue to take place at the main intersection.

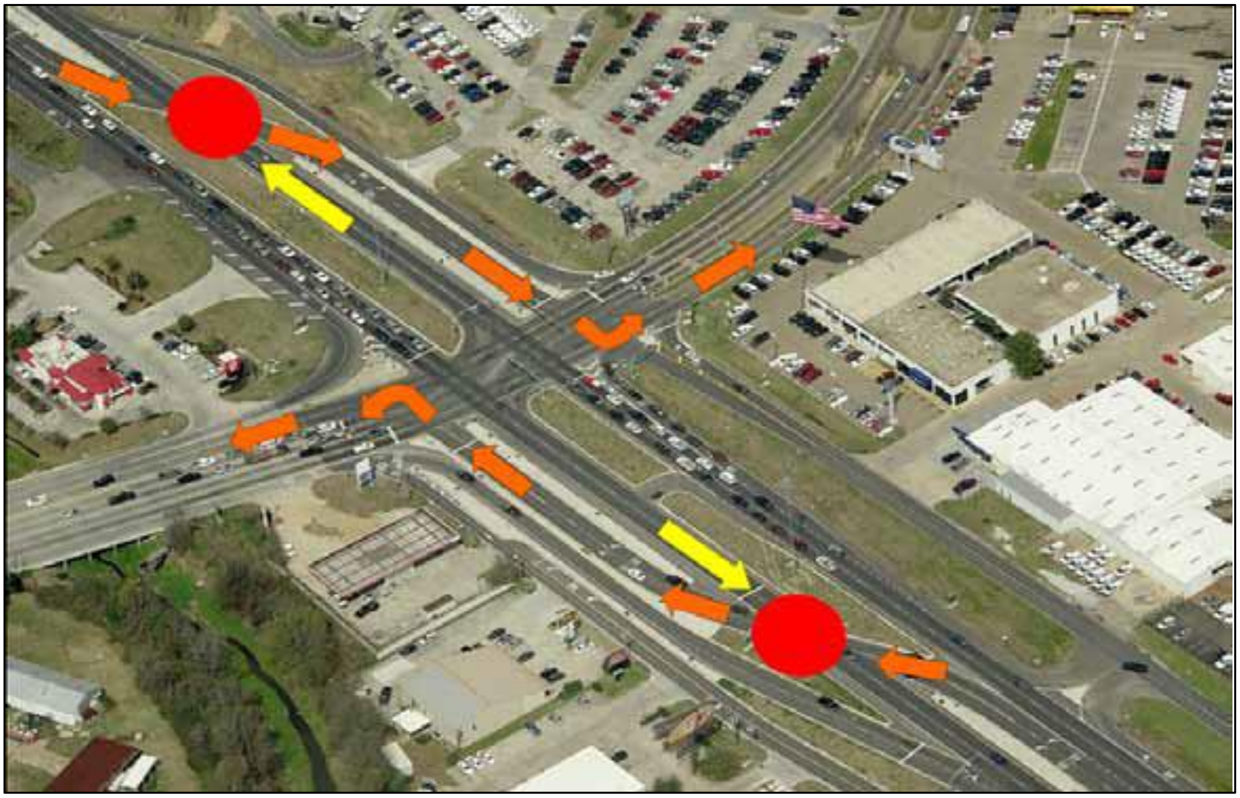


Figure 3: Left-turn crossover movement at a partial DLT in Baton Rouge, Louisiana

For the full DLT intersection, the left-turn movements are relocated to crossovers on all four approaches, as shown in Figure 4 (Hughes et al., 2010). In the figure, the red circle indicates a signal-controlled crossover, the orange arrows indicate left-turn crossover movements, and the yellow arrows indicate opposing through movements at a crossover controlled by a signal. There are five junctions with traffic signal control at a full CFI- the main intersection and the four left-turn crossovers. Furthermore, Figure 5 (Hughes et al., 2010) shows how the DLT intersection is operated under two phases

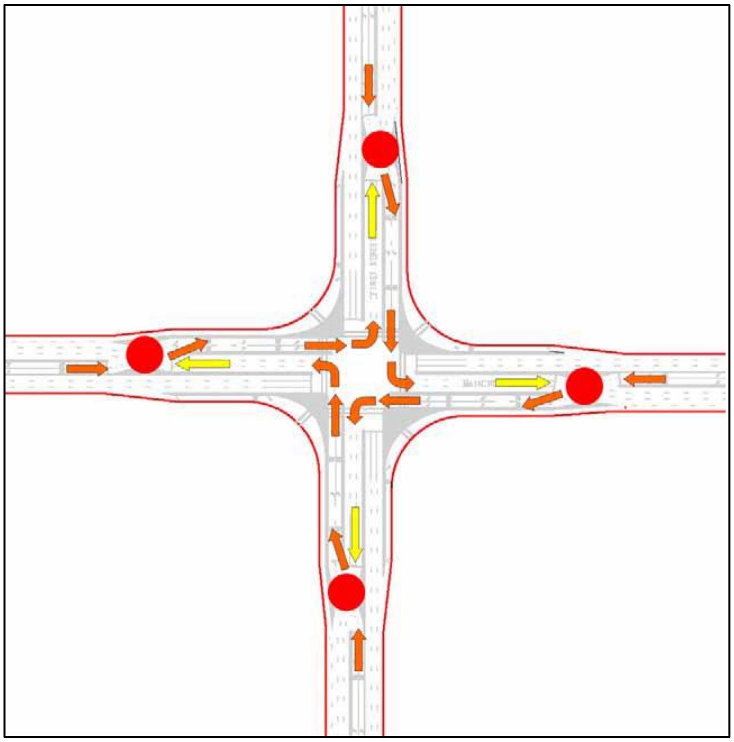


Figure 4: Illustration of left-turn cross movements at full DLT

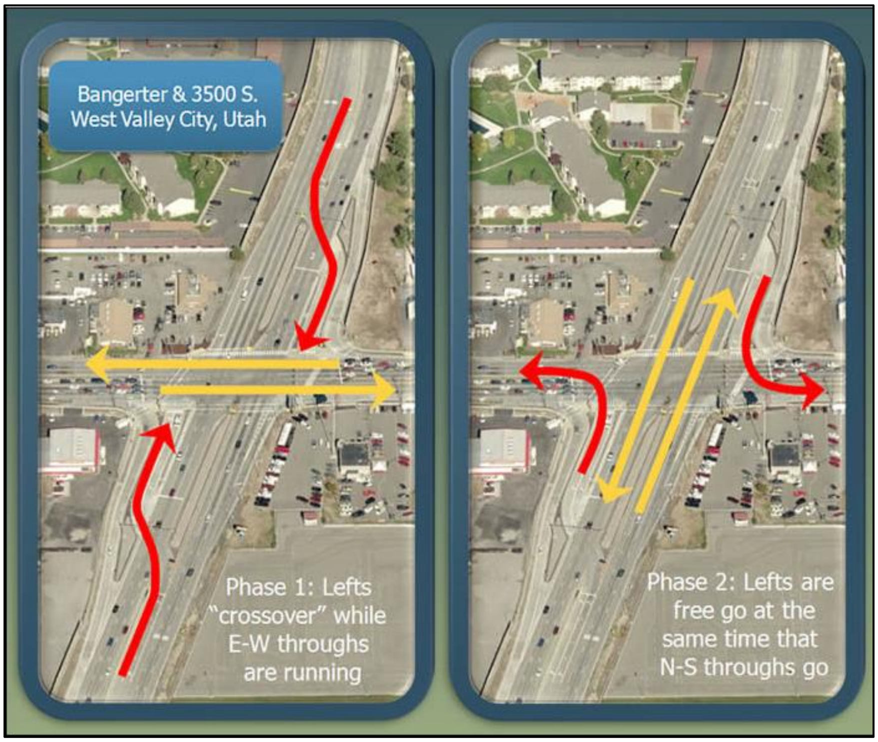


Figure 5: Explanation of how a DLT works

CHAPTER 2: LITERATURE REVIEW

Safety Performance of DDIs

One of the common concerns related to DDI is wrong-way driving. Vaughan et al. (2015) monitored five DDIs for 6 months using video camera footage data. The analysis showed that wrong-way maneuvers tended to occur more often when vehicles were first entering the DDI. Also, wrong-way maneuvers were found to occur more frequently at night than during the day. However, no crashes were identified to be associated with these wrong-way driving events.

The FHWA diverging diamond interchange informational guide (Schroeder et al., 2014) compared the number of conflict points of the DDI with the conventional diamond interchange. It was shown that the conventional diamond interchange has 26 conflict points, while the DDI has only 14. The DDI design is assumed to have safety benefits due to the reduced conflict points, especially crossing conflicts.

Many previous studies have discussed the safety performance of DDIs. Chilukuri et al. (2011) estimated CMFs for one DDI in Missouri using the before-after with comparison group method. They showed that the total and left-turn crashes were reduced by 46% and 72%, respectively. They claimed that the reduction in left-turn crashes is due to how left-turn movements are handled within the DDI.

B. R. Claros et al. (2015) estimated CMFs for 6 operational DDIs in Missouri using naïve, Empirical Bayes (EB), and comparison group (CG) methods. For the EB method, they used the calibrated safety performance functions which are provided by the Highway Safety Manual (not specific calibrated SPFs for Missouri). They found that converting conventional diamond interchanges to DDIs reduced total, fatal-and-injury, and PDO crashes by 45%, 61%, and 39%, respectively.

Hummer et al. (2016) evaluated the safety effectiveness of 6 DDIs in Missouri, Kentucky, New York, and Tennessee using the before-after with comparison group method. The results showed that the total crashes were reduced by 33%. The injury crashes were reduced by 41%.

B. Claros et al. (2017a) used the EB method to estimate the safety effect of the DDI on adjacent facilities. They showed that the DDI design has no effect on the crashes that occurred on the exit and entrance speed-change lanes. For signalized intersections near DDI ramp terminals, the EB analysis showed a 6.5% decrease in fatal-and-injury crashes, which was not statistically significant. The analysis also showed that total and PDO crashes increased by 12% and 19.5%, respectively. However, they concluded that there is no strong evidence that DDIs have positive or negative safety impact on the adjacent facilities.

B. Claros et al. (2017b) studied the safety performance of 9 DDIs in Missouri using the EB method. This study outperformed Claros et al. (4) by using location-specific safety performance functions not the HSM calibrated SPFs. They showed that the implementation of DDI reduced the total, fatal-and-injury, and PDO crashes by 37.5%, 55%, 31.4%, respectively.

More recently, Nye et al. (2019) evaluated the safety performance of DDIs based on 26 DDIs in 11 states by using the observational before-after with comparison group method. They recommended CMF values for the total, angle, and rear-end crashes of 0.633, 0.441, 0.549, respectively. They also found that fatal-and-injury crashes were reduced by 54%. However, they provided statistical significance measures for the total crashes only and did not prove the statistical significance of other crash types, which is one of the drawbacks of this study.

In summary, many previous studies have analyzed the safety performance of DDIs using different approaches. However, none of these studies considered a significant sample size representing most of the DDIs in the U.S. Even though the national-level study (Nye et al., 2019) was conducted based on 26 DDIs, the sample size is still relatively small given that there are 99 DDIs that are operational in the U.S. Moreover, no research has been conducted on the safety performance functions (SPFs), as well as the contributing factors of crash occurrence at DDIs. In this study, the author aims to evaluate the safety performance of DDI based on nationwide implementation data across 24 states. Multiple CMFs for different crash types are developed by using different approaches. Also, SPFs are developed for every crash type to investigate the safety impact of various geometric design attributes.

Safety Performance of DLTs

Hughes et al. (2010) explored the number of conflict points at DLTs. They found that the total number of conflict points at a DLT is 30 compared to the 32 conflict points at a conventional intersection. Inman (2009) analyzed the conflict points' diagram of a conventional four-leg at-grade intersection and a DLT. The results showed that a DLT has two fewer crossing points than the conventional four-leg at-grade intersection.

Steyn et al. (2014) compared the conflict points of a DLT (on major roads) to those of a typical four-leg intersection (Figure 6, Figure 7 and Figure 8) (Steyn et al., 2014). The results showed that there was a 6% to 12% decrease in conflict points for a four-leg signalized intersection.

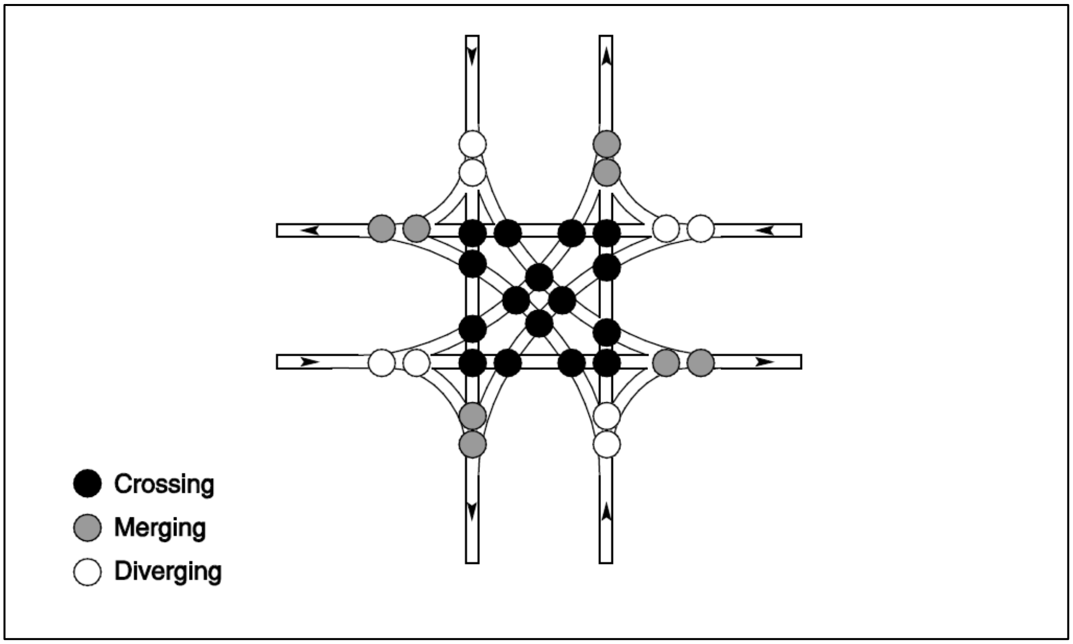


Figure 6: Conflict points for a conventional intersection

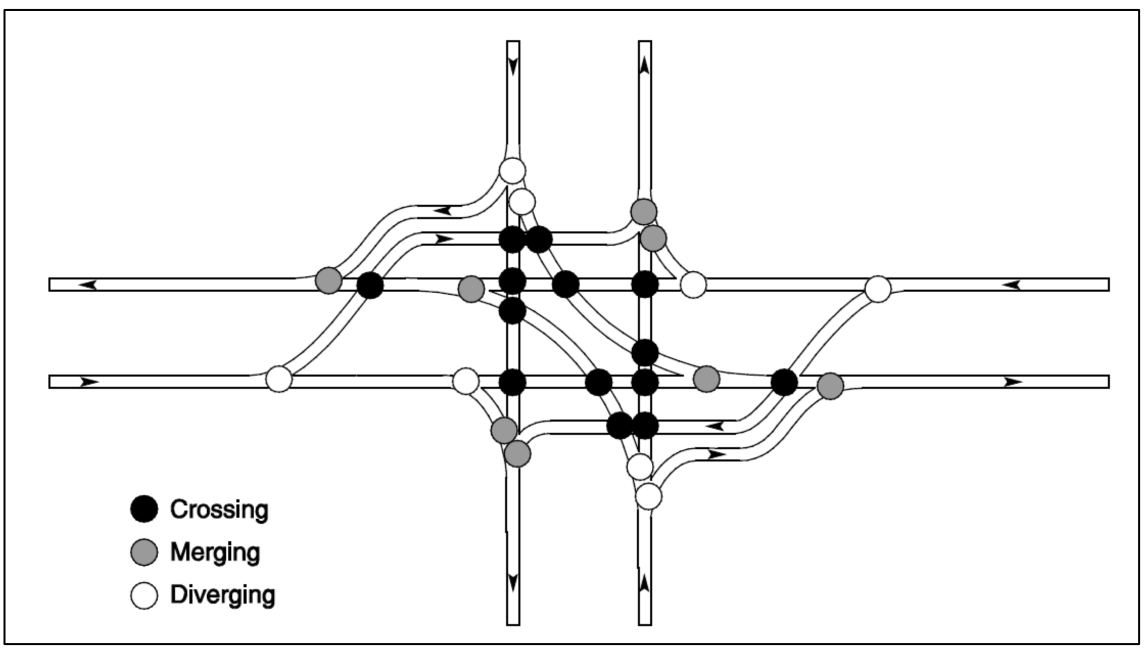


Figure 7: Conflict points for a DLT with two displaced left-turns

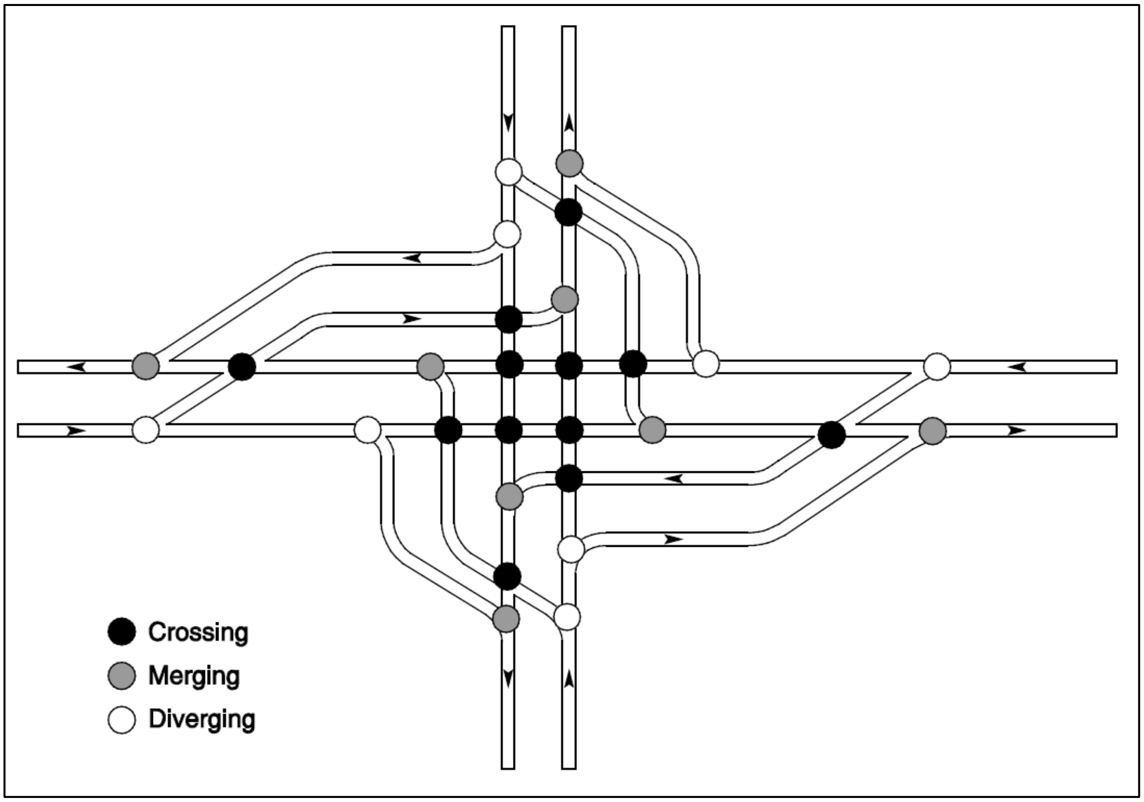


Figure 8: Conflict points for a DLT with four displaced left-turns

Table 1 (Steyn et al., 2014) compares the number of conflict points of DLT and conventional intersections. In case of three-legged intersections, the number of conflict points are nine in both types. On the other hand, CFIs have less conflict points compared with conventional intersections at four-legged intersections.

Table 1: Comparison of conflict points: DLT vs. conventional intersections

Number of Intersection Legs	Number of Crossovers on a DLT	Conflict Points	
		Conventional	DLT
3	1	9	9
4	2	32	30
4	4	32	28

Previous safety studies have analyzed a limited number of DLT intersections to determine if there is significant safety benefits from them. However, more research is needed to consider an additional number of intersections with more years of available crash data.

Park and Rakha (2010) analyzed the safety impacts of this design based on video analysis for two DLT intersections in Utah and Louisiana. They found that the installation of this design resulted in a large number of hazardous maneuvers although the number of conflict events decreased by 50%. They claimed that these hazardous maneuvers are due to driver unfamiliarity with this type of intersections.

Another study (Louisiana Department of Transportation, 2007) explored the impact of the implementation of a DLT intersection at US-61 and LA-3246, Baton Rouge. They used a simple before-after study and concluded that the number of crashes decreased by around 25%. However, they considered a small crash interval (only 18 months) and a limited sample size (only one intersection).

The Federal Highway Administration report (Steyn et al., 2014) stated that DLT intersections resulted in 6% to 12% reduction in conflict points compared to a conventional four-leg intersection. Tarko et al. (2008) evaluated the safety impacts of this design on the basis of conflict points analysis. It stated that this design may result in some safety benefits as it reduces the potential conflict points.

Zlatkovic (2015) used the Empirical Bayes before-and-after method to evaluate the safety performance of 8 DLTs in Utah. He concluded that the conversion of conventional intersections to DLTs reduced the total crashes by 12%. However, the analyzed crash data was not sufficient enough to provide representative safety measures. The reason is that the crash database included the years from 2008 to 2013 and most of the analyzed DLTs (5 out of 8) were constructed in 2013, which means that there is less than one year of crash data after their implementation.Operational Performance of DLTs

Many studies have analyzed the operational performance of DLT intersections in comparison to conventional intersections for different traffic volumes. In most cases, the alternative design has shown better performance in terms of the operational measures of effectiveness (i.e., delay, throughput, etc.).

Esawey and Sayed (2007) compared both the DLT and the upstream signal crossover design to a conventional intersection using VISSIM. The authors stated that the alternative design showed a reduction in delay time in comparison to conventional intersections. They attributed this reduction to the greater left-turn storage space available in the DLT design.

Dhatrak et al. (2010) compared the operational performance of a DLT intersection to a similar alternative design called the parallel flow intersection (PFI) using VISSIM. This study analyzed the maximum throughput for both left-turn and through movements. The results showed that DLT can serve more left-turn movements of up to 177 vehicles per hour. The authors attributed this to the fact that DLT has fewer stops for both left-turn and through movements.

Another study Olarte and Kaisar (2011) compared the operational performance of three alternative designs, the DLT, the diverging flow design, and the left-turn bypass intersection. It stated that the DLT showed an average delay less than 20 seconds per vehicle while the other alternatives did not operate well.

In addition, a study (Jagannathan & Bared, 2004) evaluated the operational performance of the DLT in comparison to conventional intersections using VISSIM. It concluded that the reduced number of phases on approaches with DLT crossovers resulted in delay savings up to 48%.

Another study (Yang et al., 2013) claimed that DLT can decrease the average delay per vehicle, total travel time and the average number of stops per vehicle by 35%, 15%, and 7%, respectively. Zhao et al. (2015) stated that the DLT outperforms the conventional intersection in terms of increasing the intersection capacity up to 75%.

In summary, many studies have been conducted to address the operational performance of DLT intersections. However, the safety performance of this alternative design has not been analyzed sufficiently due to the limited number of sites and available crash data years after implementation. Consequently, this study aims to investigate the safety effects of converting conventional four-leg signalized intersections to DLTs. In addition, it aims at providing reliable crash modification factors to be included in the database of CMF Clearinghouse and used as a reference for transportation authorities that are interested in implementing this type of alternative intersection design. Moreover, the operational performance of DLTs is investigated based on high-resolution traffic data. Furthermore, evaluation of the different aspects of costs and benefits are conducted to decide if this alternative design is appropriate for implementation.

CHAPTER 3: METHODOLOGY

To investigate the safety effects of converting conventional intersections to treated ones (i.e., DLT and DDI), crash modification factors (CMFs) were estimated. According to the Highway Safety Manual (2010), a CMF is defined as the change in the number of crashes at any location due to a change in one condition in case of all other characteristics are the same. If the calculated CMF is significantly greater that one, this means that the proposed solution caused an increase in the number of crashes. On the other hand, if it is lower than one, this indicates the number of crashes decreased due to the proposed change. However, the CMF value may be approximately one which means that the change has no significant effect on the number of crashes.

Before-and-After with Comparison Group

Two approaches were employed to evaluate the safety effectiveness of DDIs and DLTs. The first method is a before-and-after study with the comparison group. This method evaluates the safety performance of the alternative design not only based on the treatment sites' number of crashes but also use the crash data of the comparison sites (conventional intersections) which did not experience any change. This approach accounts for other factors (i.e., traffic volume trends, time) that could affect crash reduction or increase rather than the proposed treatment (Hauer, 1997). However, the comparison group should be similar to the treatment group in terms of operational and geometric characteristics. A suitable comparison group should have a ratio of crash counts in the after period to those in the before period similar to it in the treatment group (Gross et al., 2010). Hauer (1997) suggested using a series of sample odds ratios to evaluate the suitability of the comparison group using Equation (1). Based on these sample odds ratios, the sample mean and standard error are calculated. The selected comparison group is the one that has a sample mean significantly close to one. After selecting the appropriate comparison group, the CMF could be calculated using Equations 2-4. Significance measures (i.e., confidence interval) can be calculated to assess the reliability of the calculated CMF (Gross et al., 2010).

$$sample\ odds\ ratio = \frac{(Treat._{before} * Comp._{after})/(Treat._{after} * Comp._{after})}{1 + \frac{1}{Treat._{after}} + \frac{1}{Comp._{before}}} \tag{1}$$

Where,

$Treat._{before}$ = the number of crashes at the treatment group in year i

$Comp._{before}$ = the number of crashes at the comparison group in year i

$Treat._{after}$ = the number of crashes at the treatment group in year j

$Comp._{after}$ = the number of crashes at the comparison group in year j

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

$$N_{expected,T,A} = N_{observed,T,B} * \frac{N_{observed,C,A}}{N_{observed,C,B}} \tag{3}$$

$$Var(N_{expected,T,A}) = (N_{expected,T,A})^2 \left(\frac{1}{N_{observed,T,B}} + \frac{1}{N_{observed,C,B}} + \frac{1}{N_{observed,C,A}}\right) \tag{4}$$

Where $N_{observed,T,B}$, $N_{observed,T,A}$, $N_{observed,C,B}$ and $N_{observed,C,A}$ are the observed number of crashes in the before period and after period for the treatment group and the comparison group

Empirical Bayes Before-After Approach

For the Empirical Bayes before-after method, henceforth referred to as EB method, the expected number of crashes for a treated site in the 'after' period is estimated based on the crash history of the treated site and the crash history of a group of reference sites with similar yearly traffic trend, physical characteristics, and land use. One of the main advantages of the EB method is that it accurately accounts for the changes in crash frequencies in the 'before' and 'after' periods at the treatment sites that may be due to regression-to-the-mean bias. The EB method introduces an estimate for the expected crash frequency of similar untreated sites using safety performance functions, which relates the crash frequency of the sites to their traffic and geometric characteristics.

The estimation of the expected crashes at treatment sites is based on a weighted average of information from treatment and reference sites, as shown in Equation (5) (Hauer, 1997):

$$N_{expected,\,T,\,B} = SPF\ weight * N_{predicted,\,T,B} + (1 - SPF\ weight) * N_{observed,T,B} \qquad (5)$$

Where SPF weight is a weight factor estimated from the over-dispersion parameter of the SPF. The evidence from the reference sites is the output from the SPF, which is a regression model that provides an estimated crash frequency of a given roadway facility. In this study, the negative binomial model was used to develop SPFs, which fit the crash data of the reference sites with their geometric and traffic parameters. A typical SPF will be in the following form:

$$N_{predicted} = e^{(\beta_0 + \beta_1 x_1 + \beta_2 x_2 + \cdots + \beta_n x_n)} \qquad (6)$$

Where,
β_i: Regression parameters,

x_1: The logarithmic value of AADT, and x_i (i > 1) represent other traffic and geometric parameters of interest.

It should be noted that the estimates from Equation (5) are the number of crashes in the 'before' period. Since it is required to get the estimated number of crashes at the treatment site in the 'after' period; the estimates obtained from Equation (5) are adjusted by multiplying the ratio of the predicted number of crashes in the 'after' period to that in the 'before' period, as shown in Equation 7 (Hauer, 1997).

$$N_{expected,\,T,\,A} = N_{expected,\,T,B} * (\frac{N_{predicted,\,T,A}}{N_{predicted,\,T,B}}) \tag{7}$$

Then the variance of $N_{expected,\ T,A}$ and the CMF can be calculated similarly to the before-after with CG method using Equations (8), (9):

$$Var\left(N_{expected,\,T,\,A}\right) = N_{expected,\,T,\,A} * \left(\frac{N_{predicted,\,T,A}}{N_{predicted,\,T,B}}\right) * (1 - SPF\ weight) \tag{8}$$

$$CMF = (\frac{N_{observed,T,A}}{N_{expected,\,T,\,A}})/(1 + (\frac{Var\left(N_{expected,\,T,\,A}\right)}{{N_{expected,\,T,\,A}}^{2}}) \tag{9}$$

Cross-Sectional Analysis

The second method is a cross-sectional analysis which was conducted for two reasons. First, to compare the safety performance between DLTs and conventional intersections. Second, to determine factors that affect the safety performance of DDIs and DLTs (i.e., operational and geometric characteristics). This method is based on developing a safety performance function which represents the relation between the response variable (crash frequency) and predictors (i.e., intersection type, traffic volume, etc.). Since the response variable is considered as count data, Poisson and negative binomial distributions are the most common distributions that can model this type of data. However, the negative binomial model was selected in this study because it allows for overdispersion. The proposed model and its parameters are shown in Equation (10). A CMF could be calculated by exponentiating the parameter of the variable related to the proposed change (e.g., 1 if DLT and 0 if conventional). Furthermore, the significance level of the CMF could be inferred based on the significance of the associated parameter.

$$Y_i = exp[\beta_0 + \beta X_i + log_e(year) + \varepsilon_i] \tag{10}$$

Where,

Y_i = predicted number of crashes at intersection i

β_o = model intercept

β = set of parameters associated with the independent variables

X_i = set of independent variables

year = the number of crash-years

ε_i = a gamma-distributed error with mean 1 and variance α which allows for overdispersion

Operational Performance Analysis

To evaluate the operational performance of DLTs, a statistical model was developed using high-resolution traffic data to describe the factors affecting congestion at this type of intersection. There are several measures of effectiveness that are used to assess the operational performance of intersections (i.e., throughput, queue length, delay, etc.). In this study, the average delay per vehicle was selected as a response variable and other geometric and operational variables were used as independent variables.

CHAPTER 4: DATA PREPARATION

Data Preparation for DDI Analysis

As of August 2019, there are 99 DDIs across the U.S. with different years of implementation. However, not all of these DDIs are valid for the analysis because 10 DDIs were recently implemented in 2019 or 2018 with not enough crash data after their implementation. Moreover, 4 DDIs were designed to be different from the regular DDI (e.g., partial or 3-leg DDIs). As a result, the remaining number of DDIs is 85, which are located in 27 states. Consequently, we contacted the DOTs of the 27 states asking for multi-year crash and traffic data. Since not all of the DOTs were able to provide access for the requested crash data, we ended up with considering 80 DDIs in 24 states including Missouri (18), North Carolina (11), Utah (8), Minnesota (6), Georgia (5), Kansas (4), Indiana (3), Colorado (3), Texas (3), Virginia (2), Nevada (2), Michigan (2), Tennessee (2), Florida (1), Idaho (1), Iowa (1), Kentucky (1), New Mexico (1), New York (1), Ohio (1), Oregon (1), Pennsylvania (1), Wisconsin (1), and Wyoming (1).

For every treatment site, several comparison sites were selected. Since most of the DDIs were conventional diamond interchange before being converted, the comparison sites were also selected from the conventional diamond interchanges. For each DDI, three comparison sites that have similar AADT values were selected from the same state where the DDI is located to ensure that the treatment site (DDI) and its comparison sites have similar driver behavior patterns. In total, 240 comparison diamond interchanges were selected for the 80 DDIs.

It should be noted that this sample is not valid for all types of analysis methods that are proposed in this study. The full sample is valid only for the cross-sectional analysis, which only focuses on the treatment sites after their implementation, regardless of what they were before that. On the other hand, the before-and-after approaches look at the crash frequencies before and after the treatment implementation. In our case, not all the DDIs were diamond Interchanges before converting them to DDIs. The majority (65 out of 80) were diamond interchanges, while some of them were other types (i.e., cloverleaf interchange, intersection) or not even a junction. As a result, different numbers of DDIs were utilized for different analyses. Specifically, 80 DDIs were used for the cross-sectional analysis, while 65 DDIs were used for the before-after analysis.

In order to calculate the crash frequency at the designated interchanges, a crash influence area should be determined. Since the purpose of this study is to address the safety effects of converting the diamond interchange to DDI, the research team only focused on the crash frequencies at the crossovers/ramp terminals, which are the main differences between DI and DDI. Three different scenarios were proposed for the crash influence area based on the literature review, as shown in Figure 9:

1) 250 feet buffer from the center of each crossover/ramp terminal (Bonneson et al., 2012);
2) 250 feet buffer from the center of each crossover/ramp terminal in addition to the segment between the crossovers;
3) A large buffer covering 800 feet along the arterial from the freeway centerline in both directions (Nye et al., 2019).

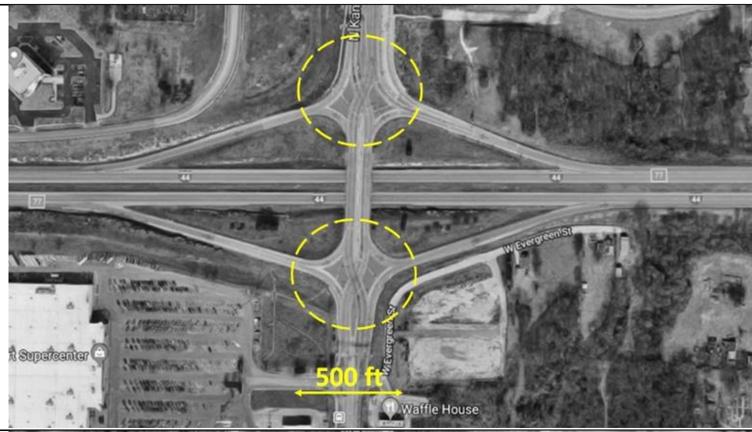
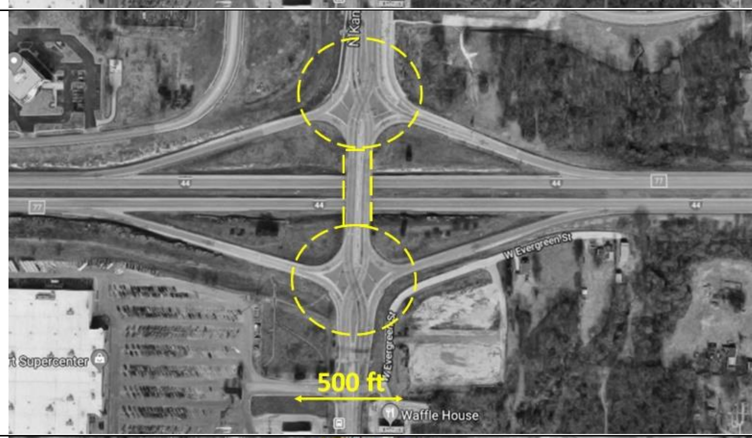
Scenario Number	Crash Influence Area
1	
2	
3	

Figure 9: Different proposed crash influence areas

The first scenario is based on the NCHRP project No. 17-45 (Bonneson et al., 2012), while the third scenario is based on Nye et al. (2019). It should be noted that the second scenario is the same as the first one but include the roadway segment between the crossovers/ramp terminals, which may have a significant effect on crash frequency. To select the most appropriate scenario, statistical significance tests were conducted to compare the average crash frequencies of each scenario by crash type, as shown in Table 2: Comparison between the different scenarios of crash influence area. The null hypothesis of the t-test assumes that there is no difference between the two scenarios. The table shows that there is no strong evidence to reject the null hypothesis when comparing the 1st and 2nd scenarios. On the other hand, there is a significant difference between the crash frequencies of the 1st and the 3rd scenarios for most crash types and severities.

Table 2: Comparison between the different scenarios of crash influence area

Crash Type	Scenario 1 Avg. Crash Frequency	Scenario 2 Avg. Crash Frequency	Scenario 3 Avg. Crash Frequency	P-value of t-test (1) vs. (2)	P-value of t-test (1) vs. (3)
Total	19.855	20.396	25.361	0.642	0.021**
Fatal	0.035	0.042	0.049	0.315	0.963
Injury	4.435	4.489	6.632	0.723	0.047**
PDO	15.404	17.523	19.102	0.932	0.038**
Rear-end	9.991	10.214	13.521	0.423	0.087*
Angle/Left-turn	4.551	5.634	7.301	0.842	0.067*
Sideswipe	2.121	2.642	2.932	0.963	0.253
Head-on	0.363	0.389	0.399	0.421	0.975
Non-motorized	0.051	0.069	0.091	0.652	0.042**
Single-vehicle	2.188	3.301	3.964	0.512	0.083*

*** significant at 99%, ** significant at 95%, and * significant at 90%.

Based on the statistical significance tests, the 1st scenario was selected for calculating the crash frequencies. Although the 3rd scenario has a significant difference from the 1st scenario, the authors believe that it may be not appropriate in this study because the distance 1600 feet could cover the adjacent intersections in case of the crossovers' distance is relatively short.

Based on the selected crash influence area, the yearly number of crashes was calculated at the DDIs and the comparison diamond interchanges by crash type. The descriptive statistics of the crash data are shown in Table 3. It should be noted that the average crash frequency was calculated by averaging over the years and the locations. For most of DDIs, the crash data was available for at least 5 years, however, for a few recently implemented DDIs, the crash data was available for two or three years only after the implementation. As shown in Table 3, the average crash frequencies of the DDIs are lower than that of the comparison diamond interchanges for most crash types, which may imply that the DDIs are safer than the conventional diamond interchanges. However, this is not strong evidence, and more reliable statistical analyses should be conducted.

Table 3: Crash data descriptive statistics

Variable	**Diamond Interchange (N=240)**				**DDI (N=80)**			
	Mean	**Stdev**	**Min**	**Max**	**Mean**	**Stdev**	**Min**	**Max**
Total	21.744	24.450	0.154	107.307	19.855	22.459	0.231	82
Fatal	0.026	0.047	0	0.154	0.035	0.051	0	0.151
Injury	5.093	5.405	0.077	18.923	4.435	4.612	0.013	15.54
PDO	16.625	19.421	0.077	90.154	15.404	18.072	0.154	66.464
Rear-end	10.332	13.042	0.145	53.462	9.991	12.442	0.211	51
Angle/Left-turn	5.378	6.323	0.154	27.615	4.551	4.902	0.114	13.701
Sideswipe	1.923	2.775	0.113	14.231	2.121	3.012	0.012	10.85
Head-on	0.509	0.715	0	3.769	0.363	0.391	0	1.231
Non-motorized	0.043	0.070	0	0.231	0.051	0.074	0	0.232
Single-vehicle	2.764	3.314	0	14.769	2.188	2.252	0.077	7.462

Moreover, many explanatory variables were identified and collected, including the AADTs of the freeway, the arterial and ramps if available, speed limits, the number of lanes for each traffic movement, skew angle, and lighting. It should be noted that arterial AADTs were available for all the 80 DDIs and their comparison sites, while only 47 DDIs and their comparison sites were provided with freeway ramp AADTs. To balance the effects of sample size and the completeness of AADT, two modeling strategies were considered in developing SPFs. The first strategy includes all the 80 DDIs and their comparison sites with only arterial AADTs. The second strategy includes 47 DDIs and their comparison sites with the consideration of total vehicles entering the DDI (TEV), which is the summation of the AADTs of the freeway exit ramps and the arterial. Other important factors that are related to the geometric configuration of DDIs were also considered, such as crossovers' distance and configuration type. The crossovers' distance indicates the distance between crossovers in the case of DDI and the distance between ramp terminals in the case of the conventional diamond interchange. The configuration type indicates whether the interchange is overpass or underpass, which means the arterial passes over or under the freeway. Table 4 shows the descriptive statistics of all the collected explanatory variables.

Table 4: Explanatory variables descriptive statistics

Variable	Diamond interchange (N=240)				DDI (N=80)			
	Mean	Stdev	Min	Max	Mean	Stdev	Min	Max
Freeway Exit Ramp AADT*	6086.8	4097.12	488	21060	6049.19	3870.80	503	18000
Arterial AADT	18934.93	10088.23	1489	46783	21224.08	13287.98	1295	76100
Distance between crossovers/ramp terminals (ft)	667.96	251.65	228.60	1656.07	731.92	244.38	364.23	1651.51
Freeway Exit Speed Limit	36.22	8.2	25	40	39.71	4.13	25	45
Arterial Speed Limit	43.25	3.22	40	55	48.89	4.16	35	55
Distance to the nearest intersection (ft)	954.68	712.33	291	1863	845.32	413.52	176	1147
Configuration Type(overpass=1, underpass=0)	0.63	0.49	0	1	0.70	0.46	0	1
Skew Angle (˚)	12.65	9.63	0	38	15.52	13.52	0	45
Lighting	0.71	0.13	0	1	0.85	0.15	0	1

Table 4 (Continued)

Variable	Diamond interchange (N=240)				DDI (N=80)			
	Mean	**Stdev**	**Min**	**Max**	**Mean**	**Stdev**	**Min**	**Max**
Pedestrian Facility type (median=1, sidewalk=0)	0.23	0.15	0	1	0.62	0.32	0	1
Freeway Exit Right Turn Control Type(signalized=1, unsignalized=0)	0.34	0.05	0	1	0.74	0.38	0	1
Freeway Exit Left Turn Lanes	1.13	0.14	1	2	1.22	0.05	1	2
Arterial Left Turn Lanes	0.89	0.09	0	1	0	0	0	0
Freeway Exit Right Turn Lanes	1.05	0.08	0	2	1.12	0.32	1	2
Arterial Right Turn Lanes	0.78	0.12	0	1	0.65	0.08	0	1
Arterial Through Lanes	2.17	0.28	1	3	2.45	0.11	1	3

Data Preparation for DLT Analysis

Two types of data were collected for this analysis. First, historical crash data was acquired from different states to assess the safety performance of these intersections. Although there are more than 30 DLTs in the US, only 13 intersections were considered in this study due to limited data availability. The reason is that some DLTs were implemented before 2009 and there is no available historical crash data for years before their implementation date. The crash data before implementation is necessary for conducting the before-after analysis which is the first method used in this study. The studied DLTs are located in four states which are Utah, Colorado, Louisiana, and Ohio. However, most of them (10 out of 13) are located in Salt Lake City metropolitan area, UT. For each DLT, two conventional intersections were selected as a part of the comparison group. The conventional intersections were chosen considering some constrains (i.e., same number of legs, same control type, comparable traffic volumes, etc.). The total sample size was 39 intersections (13 DLTs and 26 conventional). Statewide historical crash data was acquired from the transportation authorities in the previously mentioned states since year 2008 up to the latest available year. However, the statewide crash data should be manipulated to prepare the crash frequency at each studied intersection. Since DLTs consist of both main intersection and crossover left-turn locations, different effectiveness regions of intersections were considered:

1) 250 feet buffer from the center of the main intersection.
2) Large buffer covering all the left-turn crossovers and the main intersection
3) 250 feet buffer from the center of the main intersection and 50 feet buffer from the center of each crossover point

These different scenarios resulted in different crash frequencies at each intersection. However, preliminary analysis showed that the third scenario is the most realistic. This is predictable because the first scenario does not consider crashes related to the crossover and the second scenario considers crashes that may be related to neither the main intersection nor the crossover. In addition to crash data, operational and geometric characteristics (i.e., AADT, DVMT, skew angle, speed limit, etc.) were collected for each intersection.

Table 5 and Table 6 show the descriptive statistics for DLTs and conventional intersections, respectively. The crash data is summarized for 5 years after each DLT implementation (not all DLTs were constructed in the same year). The descriptive statistics show that DLTs have an average crash frequency of 168.17 crashes per intersection and the conventional intersections have 141.26 crashes per intersection. This indicates that DLTs might not have safety benefits. However, a more detailed analysis should be considered.

Table 5: Descriptive Statistics of DLTs

Variables	Mean	Stdev.	Min.	Max.
Crash Type				
Total crashes	168.17	77.10	53	365
Fatal-and-Injury	52.88	31.47	15	145
PDO	115.29	49.77	38	220
Single-vehicle	9.94	5.15	3	20
Multi-vehicle	141.29	65.88	44	304
Non-motorized	1.64	2.34	0	8
Explanatory Variables				
Major AADT (vehicles/day)	49827.24	14220.42	20288	70000
Minor AADT	23883.06	13094.50	6075	43000
Total Entering Vehicles (vehicles/day)	73710.29	23433.65	28223	104000
Major DVMT (vehicle miles/day)	6707.51	1914.29	2731.08	9423.08
Minor DVMT	2296.45	1259.09	584.13	4134.62
Total DVMT	7835.74	2406.89	3056.88	11041.67
Skew Angle (˚)	6.235	10.317	0	32
Skewed (yes=1, no=0)	0.294	0.470	0	1
Major Speed Limit (mph)	48.235	6.359	40	60
Minor Speed Limit	39.706	4.832	30	45
Lighting (yes=1, no=0)	0.941	0.243	0	1
Pedestrian Crossing (yes=1, no=0)	0.882	0.332	0	1

AADT refers to the Annual Average Daily Traffic and DVMT refers to the Daily Vehicle Miles Traveled

Table 6: Descriptive Statistics for the Conventional Intersections

Variables	Mean	Stdev.	Min.	Max.
Crash Type				
Total crashes	141.26	69.24	34	313
Fatal-and-Injury	48.47	24.49	11	120
PDO	92.85	46.56	23	220
Single-vehicle	7.588	5.02	2	22
Multi-vehicle	119.05	57.96	29	260
Non-motorized	3.44	3.01	0	15
Explanatory Variables				
Major AADT (vehicles/day)	40985.38	8278.54	17652	54000
Minor AADT	16923.00	9968.77	2116	38000
Total Entering Vehicles (vehicles/day)	57908.38	14214.71	21467	92000
Major DVMT (vehicle miles/day)	3881.19	783.95	1671.59	5113.64
Minor DVMT	1602.56	944.01	200.38	3598.48
Total DVMT	5483.75	1346.09	2032.86	8712.12
Skew Angle (˚)	3.35	7.746	0	25
Skewed (yes=1, no=0)	0.14	0.35	0	1
Major Speed Limit (mph)	41.91	6.74	35	60
Minor Speed Limit	36.02	6.71	20	50
Lighting (yes=1, no=0)	0.82	0.38	0	1
Pedestrian Crossing (yes=1, no=0)	0.94	0.23	0	1

AADT refers to the Annual Average Daily Traffic and DVMT refers to the Daily Vehicle Miles Traveled

The second type of data considered for this study is high-resolution traffic data which has been proposed to evaluate the operational performance of DLTs. The source of this data is the ATSPM which is a traffic signal management system (FHWA). This data describes various traffic signal events (i.e., green phase start, detector on/off) provided for every second. Although the DLT intersections are implemented in different states, we were able to obtain high-resolution traffic data only from Utah. Consequently, this operational analysis considered only the DLTs located in UT and their comparison intersections. The data was collected for 7 days from 04/01/2019 to 04/08/2019. This time interval was selected to consider all the traffic volume fluctuations by day and night throughout the week. This data provided the operational measure of performance used in this analysis which is intersection delay and other traffic measures like through and left-turn volumes. These traffic volumes are acquired from the responses of the advance and stop detectors located at the intersection. On the other hand, the intersection delay was provided using Equation 11 which depends on the number of arrivals and departures during the signal green and red times. Please refer to (Day et al., 2014) for further details regarding the method of calculating the delay.

Furthermore, other operational and geometric characteristics (i.e., speed limit, skew angle) were collected to check if they affect the intersection operational performance. Table 7 shows the descriptive statistics for the most important variables related to the intersection operation. It indicates that DLTs can accommodate higher through and left-turn traffic volumes than the conventional intersections. They also provide lower delay time for the users. This is consistent with the previous studies that claimed DLTs have great operational benefits. Nevertheless, a detailed analysis should be conducted to check the validity of this conclusion.

$$d = \int_{t_o}^{t_o+T} [q(t_o) + A(t) - D(t)]dt \tag{11}$$

Where,

$q(t0)$ = queue length at time t_o (number of vehicles)

$A(t)$ = arrival rate (vehicles per unit time)

$D(t)$ = service, or departure, rate (vehicles per unit time)

t_o = beginning of analysis period

T = duration of analysis period

Table 7: Operational Measures Descriptive Statistics

Variable	**Mean**		**Stdev.**		**Min.**		**Max.**	
	DLT	Conv.	DLT	Conv.	DLT	Conv.	DLT	Conv.
Delay (Sec/Veh)	18.27	20.34	4.78	2.96	5.75	13	22.75	26.50
Through Volume (vph)	3701.20	3074.05	926.54	596.70	2021	1958	4748	4321
Left Turn Volume (vph)	1064.90	852.80	383.55	231.50	594	578	1886	1456

CHAPTER 5: SAFETY PERFORMANCE OF DDIs

Before-After Analysis

Two before-after approaches, before-after with comparison group (CG) and Empirical Bayes before-after (EB), were conducted to evaluate the safety performance of DDIs. For the EB method, two modeling strategies were considered for the analysis. The first strategy included 65 DDIs and their reference sites with only arterial AADTs. The other strategy included 37 DDIs with their reference sites considering all vehicles entering the DDI (TEV). The key difference between the two methods is how to calculate the expected number of crashes after the DDI implementation.

In the CG method, the expected number of crashes is calculated based on the observed crash frequencies at the comparison sites before and after the treatment in addition to the observed crash frequency at the treated sites before the implementation. On the other hand, the EB method calculates this expected number based on the predicted crash frequency at the treated sites before and after the implementation. These predictions were conducted based on specific safety performance functions, which were developed using a reference group. The selected comparison group was used as a reference group for the EB method. It should be noted that, for the EB method, there was not much difference between the two proposed modeling strategies. However, the results of the strategy considering partial sample size were discarded since the full sample size strategy provided more statistically significant SPFs' parameters.

Table 8 shows the developed SPFs that were used to calculate the predicted and then the expected crash frequencies in case of the full sample size. These SPFs were developed in terms of the arterial volume. The table shows significant positive effects of either the arterial AADT or the TEV on the crash frequencies for most crash types.

Table 8: SPFs for Empirical Bayes' expected crash frequency calculation (full sample size)

Crash Type		Intercept	LnAADT_Arterial	Dispersion
Total	**Coef**	3.0458	0.0132*	0.6137
	P-value	<.0001	0.0862	
Fatal&Injury	**Coef**	1.118	0.047*	0.5701
	P-value	0.1392	0.0540	
PDO	**Coef**	2.919	0.0312*	0.6346
	P-value	0.0001	0.0689	
Rear-end	**Coef**	2.6995	0.0631**	0.7424
	P-value	0.0012	0.0457	
Angle/Left-turn	**Coef**	1.8336	0.0193*	0.5447
	P-value	0.0143	0.0800	
Sideswipe	**Coef**	0.3125	0.0249*	0.8625
	P-value	0.7445	0.0798	
Head-on	**Coef**	0.2512	0.0875	0.9813
	P-value	0.8323	0.4684	
Non-motorized	**Coef**	-9.1573	0.6431**	1.3365
	P-value	0.0037	0.0406	
Single-vehicle	**Coef**	-0.9155	0.1801**	0.5662
	P-value	0.2653	0.0307	

** significant at 95%, and * significant at 90%.

Table 9 shows the crash modification factors (CMFs) associated with converting the conventional diamond interchange to DDI. The CG method shows that the DDI can *decrease* the crash frequency of the total, fatal-and-injury, PDO, rear-end, and angle/left-turn crashes by 26%, 49%, 19%, 18%, and 68%, respectively. On the other hand, the EB method shows that it can *decrease* them by 14%, 44%, 8%, 11%, and 55%, respectively. It is clearly shown that the two methods concluded similar trends, while the CMF values of the EB method are slightly higher than those of the CG method. This may be due to the regression to the mean effect. In other words, the CG method showed a higher crash reduction. However, a proportion of this reduction may be due to the regression to the mean effect that the EB approach can successfully account for. It should be noted that the reduction in rear-end crashes makes sense because left-turn freeway traffic volumes do not have to stop immediately at the end of the exit ramp as in the conventional diamond interchange. Regarding the large reduction in angle/left-turn crashes, it can be explained in that the number of crossing conflict points at the DDI is lower than it at the conventional diamond interchange.

Table 9: CMFs for DDIs resulting from the before-after methods

Crash Type	B-A with CG		EB B-A (full sample size)	
	CMF	P-value	CMF	P-value
Total	0.736***	<0.001	0.858***	<0.001
Fatal&Injury	0.515***	<0.001	0.558***	<0.001
PDO	0.812***	0.006	0.920***	<0.001
Rear-end	0.824**	0.039	0.887***	0.002
Angle/Left-turn	0.319***	<0.001	0.448***	<0.001
Sideswipe	1.156	0.538	1.241	0.475
Head-on	0.378	0.478	0.643	0.412
Non-motorized	1.232	0.726	1.762	0.394
Single-vehicle	1.166	0.488	0.845	0.213

*** significant at 99%, ** significant at 95%.

Cross-Sectional Method

Using the Cross-Sectional analysis, safety performance functions were developed for each crash type based on the collected crash data and explanatory variables for the two modeling approaches. The first, includes 80 DDIs and their comparison sites, while the second includes 47 DDIs and their comparison sites. Similar to the EB method, using the full sample size using the arterial AADT provided more significant parameters, thus the results of the other approach using the TEV is not shown here. These SPFs included all the significant explanatory variables along with the natural logarithm of the traffic volume variable (arterial AADT) and the dummy variable DDI (1 if the interchange is DDI and 0 if it is a diamond interchange).

Table 10 presents the developed SPFs for the total crashes and each crash type. It shows that the variable "LnAADT$_{arterial}$" has positive effect on crash frequency for the total number of crashes, as well as other crash types (i.e., fatal-and-injury, PDO, angle/LT, non-motorized and single-vehicle). Moreover, the attribute "DDI=1" has a negative effect on the crash frequencies of the total, fatal-and-injury, PDO, rear-end, and angle/LT crashes, which means that DDIs have lower crash numbers than the conventional diamond interchanges. This finding is consistent with the results of the before-after methods.

The SPFs also showed that the speed limit variables, which are "Arterial Speed Limit" and "Freeway Exit Speed Limit", have positive effects on the crash frequency. The increase of the arterial's speed limit can significantly increase the total crashes, while the increase of the freeway exit's speed limit can significantly increase the total crashes as well as the angle crashes. The developed SPF for PDO crashes shows that signalizing the freeway right-turn exit has a negative effect on the frequency of PDO crashes. The variables of "Distance to Adjacent intersection" and "Adjacent Intersection Control Type" did not show any significant effects on safety performance.

Table 10: Safety Performance Functions from the cross-sectional analysis (full sample size)

Crash Type		Intercept	$LnAADT_{arterial}$	DDI	Distance Between Crossovers	Config. Type	Distance To adjacent	Adjacent Intersect. Cont.Type	Freeway Exit Sp. Limit	Arterial Speed Limit	Fr Ex Rt Ct Type
Total	**Coef**	3.6846	0.0530**	-0.2722***	-0.0005***	0.1343	-0.0001	0.0154	0.0063**	0.0214*	
	P-value	<.0001	0.0312	0.0037	0.0029	0.1086	0.1333	0.8465	0.0305	0.0721	
Fatal&Injury	**Coef**	0.8986	0.0970*	-0.4816***	-0.0004**	0.1462	-0.0001	0.0320		0.0543	
	P-value	0.0921	0.0614	<.0001	0.0196	0.8484	0.3372	0.6897		0.2415	
PDO	**Coef**	2.7615	0.0256*	-0.2008***	-0.0006***						-0.8912*
	P-value	<.0001	0.0625	0.0317	<.0001						0.0817
Rear-end	**Coef**	2.4541	0.0741	-0.0220**	-0.0006***						
	P-value	<.0001	0.2143	0.0416	0.0012						
Angle/Left-turn	**Coef**	1.8766	0.0208*	-0.8098***	-0.0004**	0.0180	-0.0002	-0.0304	0.2144*	0.7316	
	P-value	0.0007	0.0697	<.0001	0.0297	0.8336	0.5321	0.7077	0.0632	0.2422	
Sideswipe	**Coef**	0.9158	0.0517	-0.1156	-0.0006***						
	P-value	0.4266	0.4560	0.3625	0.0097						
Head-on	**Coef**	1.2411	-0.0348	-0.3293							
	P-value	0.3739	0.6891	0.7481							
Non-motorized	**Coef**	-7.3772	0.7416***	0.5558		0.6417***					
	P-value	0.0121	0.0008	0.4174		0.0088					
Single-vehicle	**Coef**	0.1970	0.1366***	0.1812	-0.0008***	0.2098**					
	P-value	0.8092	0.0096	0.5274	<.0001	0.0104					

*** significant at 99%, ** significant at 95%, and * significant at 90%.

DDI (DDI=1, conventional diamond interchange=0)

Configuration Type (underpass=1, overpass=0)

Adjacent Intersection Control Type (signalized=1, unsignalized=0)

Freeway Exit Right-turn Control Type (signalized=1, unsignalized=0)

Furthermore, the variable of "Distance between Crossovers/Ramp Terminals" has a negative effect on the crash frequency of the total crashes as well as the fatal-and-injury, PDO, rear-end, angle/LT, and single-vehicle crashes, which means that the longer distance between crossovers/ramp terminals is associated with lower crash frequencies. For more clarification of the safety effect of the distance between crossovers/ramp terminals, Figure 10 shows the relation between the average crash frequency and the distance between crossovers/ramp terminals in case of all other variables are constant. For instance, if the crossovers' distance of an interchange increases from 600 to 800 feet, the average total crash frequency could decrease from 12 to 8 crashes per year, which means around 33% decrease.

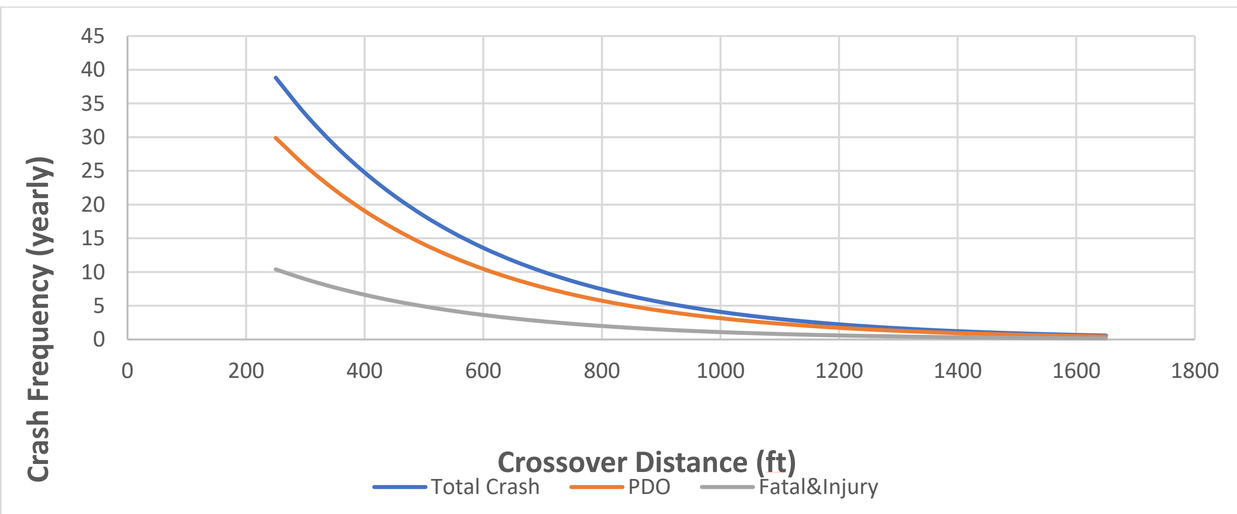


Figure 10: Effect of crossovers' distance on average crash frequency

In addition, the attribute of configuration type "underpass" has a positive effect on the non-motorized and single-vehicle crashes, which means that the interchanges with the underpass configuration have more crashes than those of the interchanges with the overpass configuration.

This may be because the overpass configuration provides more space for the non-motorized users (Schroeder et al., 2014) and so better accommodate them.

CHAPTER 6: SAFETY AND OPERATIONAL PERFORMANCE OF DLTs

Safety Analysis

Table 11 summarizes the results of the safety analysis using the first proposed safety analysis method for this analysis (before-and-after method with the comparison group). It shows that DLTs can significantly increase the total number of crashes. In addition, they can increase particularly fatal-and-injury, single-vehicle crashes. However, they have the potential to reduce non-motorized crashes (p-value=0.103). Overall, about 11% of the total crashes have increased, and 22% and 7% fatal-injury and PDO crashes, respectively, have also increased after the implementation of DLTs. The most significant increasing crash type is single-vehicle which has increased by 52%. However, no significant change was observed in other types like rear-end, head-on and sideswipe.

Table 11: CMFs for the Implementation of DLTs by Crash Type (B-A study with CG)

Crash Type	**CMF**	**S.E.**	**P-value**
Total crashes	1.112**	0.046	0.015
Fatal-and-Injury	1.224**	0.090	0.013
PDO	1.069**	0.034	0.045
Single-vehicle	1.519**	0.221	0.019
Non-motorized	0.612	0.238	0.103
Angle	1.244	0.149	0.102
Rear-end	0.946	0.095	0.570
Head-on	0.713	0.288	0.318
Sideswipe same direction	0.967	0.220	0.882

*** significant at 99% confidence level, ** significant at 95% confidence level, and * significant at 90% confidence level.

To sum up, the before-after method showed that DLTs can significantly increase total, angle and single-vehicle crashes. The crash increase may be due to the drivers' confusion with the non-traditional left-turn maneuver. On the other hand, the potential decrease of non-motorized crashes may be due to excluding the left-turn maneuver at the main intersection. Consequently, this may reduce the conflict between the vehicles turning left and the crossing pedestrians.

The cross-sectional analysis was conducted to validate the results of the before-after method and investigate if there are operational or geometric characteristics that affect the crash frequency. Table 12 and Table 13 show the safety performance functions (SPFs) and the crash modification factors (CMFs) resulting from the cross-sectional analysis. First, the safety performance functions of implementing DLTs by each crash type are shown in Table 12. They show that the variable 'DLT*Ln(DVMT)' has a positive effect on most of the crash types except the non-motorized crashes. This implies that DLTs could increase the crash frequency in comparison to conventional intersections. It is possible that drivers may be confused about the new operation rules of DLTs and this resulted in more crashes. In contrast, DLTs tend to have a smaller number of crashes involving pedestrians and bicyclists. This may be due to prohibiting left-turn vehicle movements at the main intersection. Moreover, the variable IRI (International Roughness Index) significantly increases the total and PDO crashes, which makes sense because higher IRI values indicate poor pavement conditions. Furthermore, the variable 'Speed difference' significantly decreases most of the crash types which implies that the crash frequency decreases when the minor street has a low speed limit. This is predictable because the lower speeds at the minor street the lower likelihood of crash events at the intersection.

Table 12: SPFs of DLTs Implementation (Cross-Sectional Analysis)

Crash type		Interception	Ln(DVMT)	Ln(DVMT) * DLT	Speed dif.	IRI	Dispersion
Total crashes	Coef.	-2.248	0.586**	0.050***		0.002*	0.145
	S.E.	2.696	0.2954	0.018		0.001	0.042
Fatal-and-Injury	Coef.	-0.952	0.384	0.040***	-0.033***		0.129
	S.E.	2.106	0.244	0.019	0.012		0.043
PDO	Coef.	-2.830	0.623**	0.067***	-0.028***	0.002**	0.108
	S.E.	2.398	0.264	0.017	0.010	0.001	0.034
Single vehicle	Coef.	3.534	-0.334	0.064***	-0.022		0.092
	S.E.	2.478	0.288	0.023	0.014		0.056
Non-motorized	Coef.	5.646	-0.714	-0.062*			0.313
	S.E.	4.017	0.468	0.0387			0.208
Angle	Coef.	-1.213	0.424	0.039*	-0.042***	0.001	0.206
	S.E.	3.647	0.3961	0.024	0.0149	0.001	0.065
Rear-end	Coef.	-4.709	0.798***	0.051***	-0.020*	0.001	0.156
	S.E.	2.840	0.310	0.174	0.011	0.000	0.048
Head-on	Coef.	-2.510	0.407	0.070	-0.081**	0.001	0.401
	S.E.	6.571	0.729	0.052	0.033	0.002	0.200
Sideswipe same direction	Coef.	-3.880	0.569	0.081	-0.034***	0.002***	0.055
	S.E.	2.302	0.249	0.018	0.012	0.001	0.044

*** significant at 99% confidence level, ** significant at 95% confidence level, and * significant at 90% confidence level.
Speed dif. refers to the difference between the major and minor roads' speeds

Using the developed safety performance functions, various CMFs for DLT were estimated. Since the derived SPFs present the CMFs as a function of traffic exposure, Table 13 shows the crash modification functions and the associated CMFs for different traffic demand levels. It shows that DLTs have negative safety impacts in comparison to conventional intersections for most of the crash types except non-motorized crashes as discussed before. For instance, total and rear-end crashes could be increased by up to 59% in the case of high traffic demand. The results of the cross-sectional analysis are quite consistent with the before-after analysis results which validate the conclusion that DLTs could have negative safety impacts.

Table 13: CMFs of DLTs Implementation (Cross-Sectional Analysis)

Crash type	**Crash Modification Functions**	**Crash Modification Factors (CMFs)**		
		Low traffic volumes (DVMT=3000)	**Moderate traffic volumes (DVMT=6000)**	**High traffic volumes (DVMT=9000)**
Total crashes	$DVMT^{0.050***}$	1.492^{***}	1.545^{***}	1.577^{***}
Fatal-and-Injury	$DVMT^{0.040***}$	1.377^{***}	1.416^{***}	1.439^{***}
PDO	$DVMT^{0.067***}$	1.710^{***}	1.791^{***}	1.840^{***}
Single vehicle	$DVMT^{0.064***}$	1.669^{***}	1.745^{***}	1.791^{***}
Non-motorized	$DVMT^{-0.062*}$	0.609^{*}	0.583^{*}	0.569^{*}
Angle	$DVMT^{0.039*}$	1.366^{*}	1.404^{*}	1.426^{*}
Rear-end	$DVMT^{0.051***}$	1.504^{***}	1.558^{***}	1.591^{***}
Head-on	$DVMT^{0.070}$	1.751	1.839	1.891
Sideswipe same direction	$DVMT^{0.081}$	1.913	2.023	2.091

*** significant at 99% confidence level, ** significant at 95% confidence level, and * significant at 90% confidence level.

Operational Analysis

Most of the previous studies regarding DLTs claimed that they have operational effectiveness. However, this study used a new approach to analyze the operational benefits of DLTs as discussed before. The high-resolution data that was used in this study could be more meaningful and realistic than the microsimulation approach. The measure of performance used in this analysis is the intersection delay which has been acquired from the performance charts provided by Utah DOT (UDOT, 2019). Table 14 shows a general linear model that was developed to describe the relation between the measure of effectiveness (delay) and other traffic measures. Other operational and geometric parameters (i.e., skew angle and speed limit) were tested in the model, but they did not show any significant effect on the intersection delay. The model results show that converting the conventional signalized intersection to DLT could increase the delay by 3.567 sec/veh in case of they are exposed to the same left-turn volume. Furthermore, the results show that intersection delay increases with the left-turn volume regardless intersection type. This is expected because heavy left-turn volumes result in increasing the intersection delay. However, if DLTs and conventional intersections are exposed to the same left-turn volumes, DLTs will show better performance. The model has an adjusted R^2 value of 0.58 which is a goodness of fit measure. It indicates the model's predicted delay values fit the actual values reasonably well. However, the model parameters have a high level of significance and this is strong evidence that the relation exists between the delay and the predictors.

Table 14: Operational Performance General Linear Model

Variable	Estimate	S.E.	P-value
Intercept	-35.834	13.190	0.011**
Type (1 if DLT, 0 if conventional)	-3.567	1.230	0.007***
Log Left-turn Volume	7.833	1.905	<0.001***

CHAPTER 7: COSTS AND BENEFITS OF DLT IMPLEMENTATION

This section presents the costs and benefits associated with the implementation of DLT intersections to be used as a reference for agencies which are interested in this alternative design. These costs and benefits could be summarized in three main components:

1) The initial construction cost and the annual maintenance cost
2) The annual benefits of the safety performance
3) The annual benefits of the operational performance

Construction and Maintenance Cost

FHWA defines the project costs as the implementation and operation cost of project alternatives (Beatty, 2002). The construction cost of a DLT intersection is supposed to be greater than a conventional intersection due to the increased associated right-of-way requirements (Hughes et al., 2010). The costs of right-of-way will increase the cost of a DLT intersection beyond that of a conventional intersection. However, the grade separation could be an alternative solution in case of high traffic volumes, the DLT may provide similar operational efficiency with less implementation costs. Table 15 shows the cost of three existing DLT intersections to provide an approximate range of costs (Steyn et al., 2014).

Table 15: Construction Costs of Existing DLT Intersections

Location	Opening Year	Cost
Airline Highway / Siegen Lane Intersection Baton Rouge, Louisiana	2006	$4.4 million[1]
Bangerter Highway / 3500 South Intersection Salt Lake City, Utah	2007	$7.5 million[2]
Route 30 / Summit Drive Intersection Fenton, Missouri	2007	$4.5 million[1]

[1] Cost represents the construction bid price of the project only
[2] Cost includes all costs associated with the project (e.g., planning/environmental, engineering, and right-of-way)

The maintenance of a DLT intersection is similar to a conventional signalized intersection. However, there are more medians compared to the conventional intersection. Since there is no reference providing an estimate of the maintenance cost of the DLT intersection, the maintenance cost of the conventional signalized intersection will be proposed for this study which is $8,000 according to Chandler et al. (2013).

Safety Benefits

Since DLT intersections have CMF values greater than 1, it is expected that implementing these intersections will result in increasing the crash frequency which means that there are no expected safety benefits associated with them. This section discusses how to quantify this increase in terms of monetary values. These calculations were conducted based on the CMF values, crash frequencies at the base condition and the crash cost values which are provided by Harmon et al. (2018). As shown in Table 16, the estimated annual safety values are calculated for each crash severity level. The average annual crash frequencies are assumed based on the descriptive statistics of the crash data in this study (Table 5). The CMF values are the ones developed in this study based on the before-after with CG (Table 11). Table 16 shows that converting the conventional signalized intersection to DLT can increase the fatal-and-injury and PDO crashes by 2.171 and 1.281 Crashes per Intersection per year, respectively. Consequently, based on the crash cost values, this could result in annual losses of $13,383,323.

Table 16: The Monetary Value of Safety Effect

Crash Severity	Fatal-and-Injury	PDO
Annual Crashes for Base Condition (per Intersection)	9.694	18.570
CMF	1.224	1.069
Annual Crashes after implementing DLT	11.865	19.851
Annual Increase in Crashes	2.171	1.281
Crash Value	$6,156,150	$12,108
Annual Safety Values	$13,367,809	$15,514
Total Annual Safety Values	$13,383,323	

Operational Benefits

The benefits associated with the operational performance can be quantified based on the reduction in the travel time that people spend in their trips and the value of time. This study showed that converting the conventional signalized intersection to DLT can decrease the travel delay by 3.567 sec/veh (Table 14). To calculate the annual reduction in travel delay, this number should be multiplied by the annual average daily number of vehicles entering the intersection and the number of working days in a year. The descriptive statistics of the explanatory variables (Table 6) shows that DLT intersection have a daily “Total Entering Vehicle” number of 73710 veh/day. The value of time was assumed to be $26 per hour per person according to Blincoe et al. (2015). The average occupancy rate should be used to quantify the travel delay in terms of person-hours. It was assumed to be 1.7 person/veh according to Santos et al. (2011). Equation 12 shows how the annual operational benefits could be quantified. The calculations showed that implementing the DLT intersection could result in annual time savings with a value of $865,284.

$$Annual\ Operational\ Benefits = \frac{Delay\ Reduction * TEV * O.R. * Time\ Value * 262}{3600} \tag{12}$$

Where,

Delay Reduction = the reduction in travel time delay (sec/veh)

TEV = Total Entering Vehicles (veh/day)

O.R. = average occupancy rate of personal vehicles

Time Value = the value of person hour in USD

Table 17 shows the annual monetary values of maintenance costs, safety and operational benefits associated with implementing the DLT intersections taking into account a discount rate of 3%. The analysis period is assumed to be 20 years since this is a common practice in transportation projects. Equation 8 shows how to calculate the present value of the amount of cost or benefit in any year (Lawrence et al., 2018). It should be noted that the construction time is assumed to be one year. In other words, when substituting in Equation 13 to calculate the PV of an amount of money after year 1, the value t should be 2 years (considering one year for construction).

$$PV = \left(\frac{1}{(1+r)^t}\right) A_t \tag{13}$$

Where,

PV = present value at time zero (base year)

r = discount rate

t = time (year)

A_t = amount of cost or benefit in year t

Table 17: Present Value of Annual Costs and Benefits

Years after construction	Maintenance Cost	Crash Values	Time Savings
1	$7,541	$12,615,066	$815,613
2	$7,321	$12,247,636	$791,857
3	$7,108	$11,890,909	$768,794
4	$6,901	$11,544,572	$746,402
5	$6,700	$11,208,322	$724,662
6	$6,505	$10,881,866	$703,555
7	$6,315	$10,564,919	$683,063
8	$6,131	$10,257,203	$663,168
9	$5,953	$9,958,449	$643,853
10	$5,779	$9,668,397	$625,100
11	$5,611	$9,386,793	$606,893
12	$5,448	$9,113,392	$589,216
13	$5,289	$8,847,953	$572,055
14	$5,135	$8,590,246	$555,393
15	$4,985	$8,340,044	$539,216
16	$4,840	$8,097,131	$523,511
17	$4,699	$7,861,292	$508,263
18	$4,562	$7,632,322	$493,459
19	$4,429	$7,410,021	$479,087
20	$4,300	$7,194,196	$465,133
Total Present Value	$115,553	$193,310,730	$12,498,292

To decide if implementing the DLT intersection is appropriate, the total present value of costs (initial construction cost + annual maintenance cost) should be compared versus those of benefits. It is clearly shown that this design has much benefits in terms of the operational performance. However, the increased crashes associated with it could results in losses which are much higher than its benefits with a benefit-cost ratio of around 1:16.

CHAPTER 8: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This study evaluated the safety benefits of diverging diamond interchanges (DDIs) , in addition to the safety and operational benefits of displaced left-turn intersections (DLTs). For the safety analysis of DDIs, three methods were adopted to estimate the CMFs, which are before-after with comparison group, Empirical Bayes before-after, and the cross-sectional analysis. The studied sample included 80 DDIs and 240 conventional diamond interchanges as comparison sites located in 24 states. Different data types were collected to conduct the analysis. First, multi-year crash data were acquired from the designated states. Then, traffic and geometric features were collected, including AADT, speed limits, and the distance between crossovers/ramp terminals. Since the AADT of the freeway exit ramp was not available for all interchanges, two modeling approaches were considered for the EB method and the cross-sectional analysis. The first included all DDIs and their comparison sites, while the second one included the DDIs with available ramp traffic volumes only and their comparison sites.

The before-and-after analysis with CG showed that converting the conventional diamond interchange to DDI can decrease (1 – CMF) the total, fatal-and-injury, PDO, rear-end and angle/LT crashes by 26%, 49%, 19%, 18%, and 68%, respectively. On the other hand, the Empirical Bayes method showed that it could decrease them by 14%, 44%, 8%, 11%, and 55%, respectively. It is obvious that the two methods provided similar trends; however, the CMFs of the Empirical Bayes method are slightly higher than those of the Before-After with CG method. This difference may be due to the regression to the mean effect that was considered in the Empirical Bayes approach.

The cross-sectional method was used to develop safety performance functions that describe the relationship between crash frequency and various explanatory variables. The developed SPFs showed that converting the diamond interchange to DDI can decrease the total, fatal-and-injury, PDO, rear-end, and angle/LT crashes, which is consistent with the before-and-after methods. Moreover, the distance between crossover/ramp terminals was found to have a negative effect on the crash frequency, which means that the higher distance, the lower crash frequency. Furthermore, the interchanges with the underpass configuration were found to have more non-motorized and single-vehicle crashes than those of the interchanges with the overpass configuration. In addition, both variables of “Arterial Speed Limit” and “Freeway Exit Speed Limit” were found to have positive effects on the crash frequency. In other words, increasing the speed limit of the freeway exit ramp can significantly increase the total crashes as well the angle crashes, while the increase of the arterial’s speed limit can significantly increase the total crashes. The SPFs also revealed that the variable of “Freeway Exit Right-turn Control Type” is significantly associated with the safety performance of DDI, where the signalized exit has significantly lower frequency of PDO crashes.

Regarding the analysis of DLTs, although this innovative design has been implemented in different states, this study considered only 13 DLTs in four states, Utah, Louisiana, Colorado, and Ohio due to limited historical crash data availability.

For the safety analysis of DLTs, two analysis methods were conducted which are the before-and-after with comparison group and cross-sectional analysis. Both results showed similar safety effects of DLTs. They showed that DLTs can significantly increase the total number of crashes as well injury crashes and some other crash types (i.e., single vehicle, angle). However, they have the potential to decrease the non-motorized crashes. This may be due to the exclusion of left-turn movements at the main intersection. Moreover, the safety performance functions showed that other factors (i.e., International Roughness Index) have a significant effect on the crash frequency.

The study also investigated the operational performance of this innovative intersection design using high-resolution traffic data. The results showed that DLTs have a lower average delay than conventional intersections. This is consistent with most of the previous studies that claimed DLTs have potential operational benefits.

Furthermore, the study quantified the costs and benefits associated with implementing DLTs. The results showed that this alternative design could provide much benefits in terms of its operational performance. However, its poor safety performance could result in losses much higher than its benefits.

The study concludes that converting conventional diamond interchanges to DDIs is a countermeasure which can significantly reduce the crash frequency at this type of junctions. On the other hand, converting conventional intersections to DLTs could have negative safety impacts. However, this design seems to have potential operational benefits which make it a good design for implementation after addressing the associated safety issues.

This study presents a reliable reference for the web-based repository, CMF Clearinghouse, since it is the first study to evaluate the safety performance of the displaced left-turn intersections based on a relatively large sample size and sufficient years of crash data before and after their implementation.

The author recommends that transportation authorities should pay more attention to the safety problems associated with displaced left-turn intersections as they have a significant ability to reduce congestion. The solution may be providing intensive awareness for the users that use this new type of intersection design.

Future research efforts should be directed to some important issues:

1) Considering a greater number of DLTs with appropriate before and after crash data years which may lead to more statistically reliable results.

2) Using driving simulation to evaluate the safety performance of DLTs. This will mimic the users' behavior and may address the safety problems associated with this alternative design.

3) Checking the temporal change in safety effects of DDIs. This effect (CMF) might change over time due to some driving behavior attributes (Mannering, 2018)

4) Addressing the effect of implementing DDIs on the crash patterns at the adjacent upstream and downstream intersections. This could help the transportation agencies prevent any potential crash migration effects, associated with implementing DDI, on the adjacent intersections.

REFERENCES

AASHTO. (2010). Highway Safety Manual. In (1st Edition ed.).

Autey, J., Sayed, T., & El Esawey, M. (2013). Operational performance comparison of four unconventional intersection designs using micro-simulation. *Journal of Advanced Transportation, 47*(5), 536-552.

Beatty, T. L. (2002). Life-cycle cost analysis primer. *US Department of Transportation: Federal Highway Administration*.

Blincoe, L., Miller, T. R., Zaloshnja, E., & Lawrence, B. A. (2015). *The economic and societal impact of motor vehicle crashes, 2010 (Revised)*. Retrieved from

Bonneson, J., Geedipally, S., Pratt, M., & Lord, D. (2012). Safety prediction methodology and analysis tool for freeways and interchanges. *National Cooperative Highway Research, 7*, 17-45.

Chandler, B. E., Myers, M., Atkinson, J. E., Bryer, T., Retting, R., Smithline, J., . . . Venglar, S. P. (2013). *Signalized Intersections Informational Guide*. Retrieved from

Chilukuri, V., Siromaskul, S., Trueblood, M., & Ryan, T. (2011). *Diverging diamond interchange performance evaluation (I-44 and Route 13)*. Retrieved from

Claros, B., Edara, P., & Sun, C. (2017a). Safety Effect of Diverging Diamond Interchanges on Adjacent Roadway Facilities. *Transportation Research Record, 2618*(1), 78-90.

Claros, B., Edara, P., & Sun, C. (2017b). When driving on the left side is safe: Safety of the diverging diamond interchange ramp terminals. *Accident Analysis & Prevention, 100*, 133-142.

Claros, B. R., Edara, P., Sun, C., & Brown, H. (2015). Safety evaluation of diverging diamond interchanges in Missouri. *Transportation Research Record, 2486*(1), 1-10.

Day, C. M., Bullock, D. M., Li, H., Remias, S. M., Hainen, A. M., Freije, R. S., . . . Brennan, T. M. (2014). *Performance measures for traffic signal systems: An outcome-oriented approach* (162260296X). Retrieved from

Dhatrak, A., Edara, P., & Bared, J. G. (2010). Performance analysis of parallel flow intersection and displaced left-turn intersection designs. *Transportation Research Record, 2171*(1), 33-43.

DOT, L. (2007). *Continuous Flow Intersection (CFI) Report US 61 (Airline Highway) @ LA 3246 (Siegen Lana)*. Retrieved from

Esawey, M. E., & Sayed, T. (2007). Comparison of two unconventional intersection schemes: crossover displaced left-turn and upstream signalized crossover intersections. *Transportation Research Record, 2023*(1), 10-19.

FHWA. (01/31/2019). EDC-4: Automated Traffic Signal Performance Measures (ATSPMs). Retrieved from https://www.fhwa.dot.gov/innovation/everydaycounts/edc_4/atspm.cfm

Gross, F., Persaud, B. N., & Lyon, C. (2010). *A guide to developing quality crash modification factors*. Retrieved from

Hauer, E. (1997). *Observational before/after studies in road safety. Estimating the effect of highway and traffic engineering measures on road safety*.

Hughes, W., Jagannathan, R., Sengupta, D., & Hummer, J. (2010). *Alternative intersections/interchanges: informational report (AIIR)*. Retrieved from

Hummer, J. E., Cunningham, C. M., Srinivasan, R., Warchol, S., Claros, B., Edara, P., & Sun, C. (2016). Safety evaluation of seven of the earliest diverging diamond interchanges installed in the United States. *Transportation Research Record, 2583*(1), 25-33.

Inman, V. W. (2009). Evaluation of sign and marking alternatives for displaced left-turn lane intersections.

Jagannathan, R., & Bared, J. G. (2004). Design and operational performance of crossover displaced left-turn intersections. *Transportation Research Record, 1881*(1), 1-10.

Lawrence, M., Hachey, A., Bahar, G. B., & Gross, F. B. (2018). *Highway Safety Benefit-Cost Analysis Guide*. Retrieved from

Mannering, F. (2018). Temporal instability and the analysis of highway accident data. *Analytic methods in accident research, 17*, 1-13.

Nye, T. S., Cunningham, C. M., & Byrom, E. (2019). National-level safety evaluation of diverging diamond interchanges. *Transportation Research Record, 2673*(7), 696-708.

Olarte, C. L., & Kaisar, E. I. (2011). *Operational Performance Comparison between Three Unconventional Intersection Designs: Left-turn Bypass, Diverging Flow and Displaced Left-turn.* Paper presented at the The 9th Latin American and Caribbean Conference. Medellin: LACCEI.

Park, S., & Rakha, H. (2010). *Continuous flow intersections: A safety and environmental perspective.* Paper presented at the 13th International IEEE Conference on Intelligent Transportation Systems.

Santos, A., McGuckin, N., Nakamoto, H. Y., Gray, D., & Liss, S. (2011). *Summary of travel trends: 2009 national household travel survey*. Retrieved from

Schroeder, B., Cunningham, C., Ray, B., Daleiden, A., Jenior, P., & Knudsen, J. (2014). *Diverging diamond interchange: informational guide*. Retrieved from

Steyn, H., Bugg, Z., Ray, B., Daleiden, A., Jenior, P., & Knudsen, J. (2014). *Displaced left turn intersection: informational guide*. Retrieved from

Tarko, A. P., Inerowicz, M., & Lang, B. (2008). Safety and operational impacts of alternative intersections.

UDOT. (2019). Automated Traffic Signal Performance Measure. Retrieved from https://udottraffic.utah.gov/atspm

Vaughan, C., Jagadish, C., Bharadwaj, S., Cunningham, C. M., Schroeder, B. J., Hummer, J. E., . . . Rouphail, N. M. (2015). Long-term monitoring of wrong-way maneuvers at diverging diamond interchanges. *Transportation Research Record, 2484*(1), 129-139.

Yang, X., Chang, G.-L., Rahwanji, S., & Lu, Y. (2013). Development of planning-stage models for analyzing continuous flow intersections. *Journal of Transportation Engineering, 139*(11), 1124-1132.

Zhao, J., Ma, W., Head, K. L., & Yang, X. (2015). Optimal operation of displaced left-turn intersections: A lane-based approach. *Transportation Research Part C: Emerging Technologies, 61*, 29-48.

Zlatkovic, M. (2015). Development of performance matrices for evaluating innovative intersections and interchanges (No. UT-15.13). Utah. Dept. of Transportation. Research Division.