SAFETY EFFECTIVENESS OF ADDING BY-PASS LANES AT UNSIGNALIZED RURAL INTERSECTIONS IN KANSAS

by

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B.S., University of Isfahan, 2006
M.A., Shiraz University, 2011

A THESIS

submitted in partial fulfillment of the requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering
College of Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

2014

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Abstract

Construction of by-pass lanes at rural intersections typically has been considered a low-cost safety improvement. Safety analysis utilizes two common approaches to evaluate treatment effectiveness: before-and-after study and cross-sectional study. This research performed paired sample t-test statistical analysis to estimate changes in total of crash frequency, crash rates, EPDO crash frequency, and EPDO crash rates at intersections, three to five years after adding a by-pass lanes compared to identical time period before the by-pass lane was added. Crash data between 1990 and 2011 were obtained from Kansas Crash and Analysis Record System (KCARS) maintained by the Kansas Department of Transportation (KDOT). In order to perform a cross-sectional study, intersections with by-pass lanes were compared to intersections with no countermeasures; crash data were obtained for more than 1,100 intersections in the state of Kansas.

According to before-and-after study, addition of by-pass lanes improves safety at unsignalized rural intersections; crashes and their severities are reduced after adding by-pass lanes. But, these reductions are not statistically significant under 95% confidence level. However, when considering intersection related crashes, a statistically significant reduction in crash rates is happened after adding by-pass lanes at 3-legged intersections.

In cross-sectional study, crashes and their severities are lower at 3-legged intersections with the by-pass lanes versus 3-legged intersections without the by-pass lanes. However, these reductions are not statistically significant under 95% confidence level. When considering 300 feet intersection crash box, statistically significant reductions are happened at 4-legged intersection. In

contrast, crashes and their severities increased at 4-legged intersections with the by-pass lanes, but these changes are not statistically significant under 95% confidence level.

The Crash Modification Factors were calculated to evaluate safety effectiveness of adding by-pass lanes at unsignalized rural intersections. The calculated CMFs less than 1.0, indicate a reduction in crashes after implementation of by-pass lanes. Finally, this study concluded that expected crashes at intersections with by-pass lanes are lower than intersections without by-pass lanes.

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Acknowledgement

My appreciation goes to my advisor, Dr. Sunanda Dissanayake for providing the guidance throughout the whole process of my research. I would also like to thank my other two committee members, Dr. Robert Stokes and Dr. Eugene Russell, for their support on this research.

I would also like to thank Howard Lubliner, the project monitor, and Brian Gower for their opinions and guidance from beginning to end of this research.

My thanks also goes to the Kansas Department of Transportation, who supplied me with all the data I needed from databases.

Finally, I would like to thank my friends, family, and my wife, Niloufar, who have always supported me and encouraged me..

Chapter 1 - Introduction

1.1 Background

Due to the human density in urban areas and higher Annual Average Daily Traffic (AADT) of the urban roads, crashes occur more frequently in urban areas compared to rural areas. However, due to higher speed limits, lower signs, and traffic signals, crashes are more severe in rural areas compared to crashes on urban roadways. In 2010, 30,196 fatal crashes occurred in the United States, resulting in 32,885 fatalities. Fifty-four percent of fatal crashes and 55% of fatalities occurred in rural areas even though only 19% of the U.S. population lives in rural areas. Urban areas accounted for 45% of fatal crashes and 44% of fatalities. In 2010, the fatality rate per 100 million vehicle miles traveled was 2.5 times higher in rural areas than in urban areas (NHTSA, 2012).

According to 2010 census, 36% of crashes in Kansas occurred in rural areas; however, 69.7% of fatal crashes occurred in rural areas (KDOT, 2013), thereby demonstrating that crashes are more severe on rural roadways. Figures 1.1 and 1.2 show the proportion of rural and urban crashes compared to all crashes and fatal crashes in Kansas.

Figure 1.1 Proportion of urban and rural crashes in Kansas

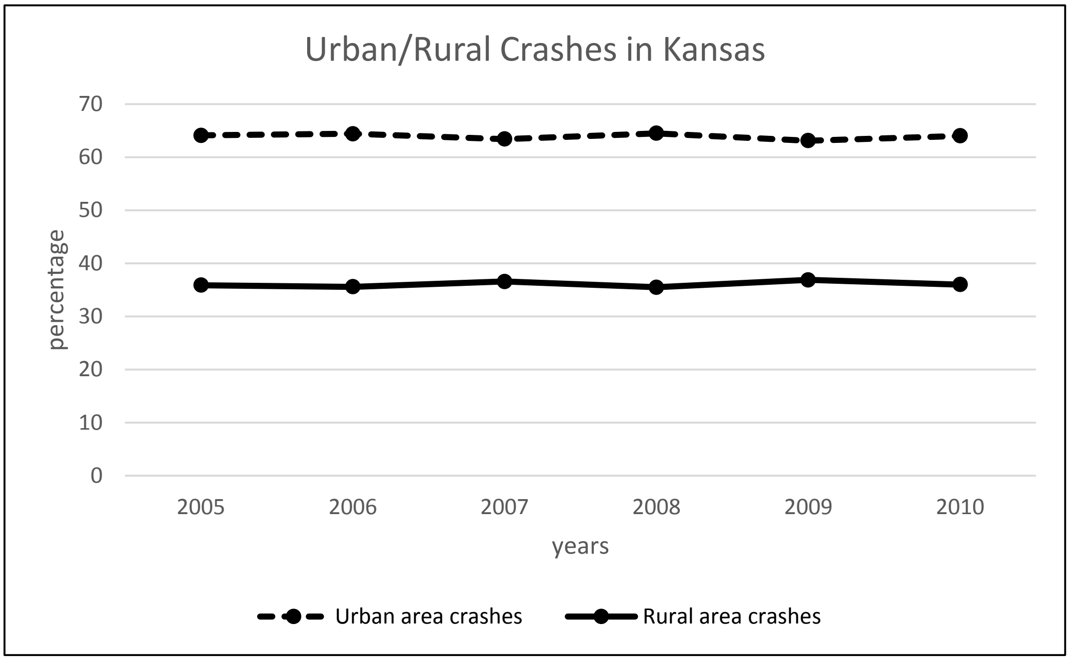
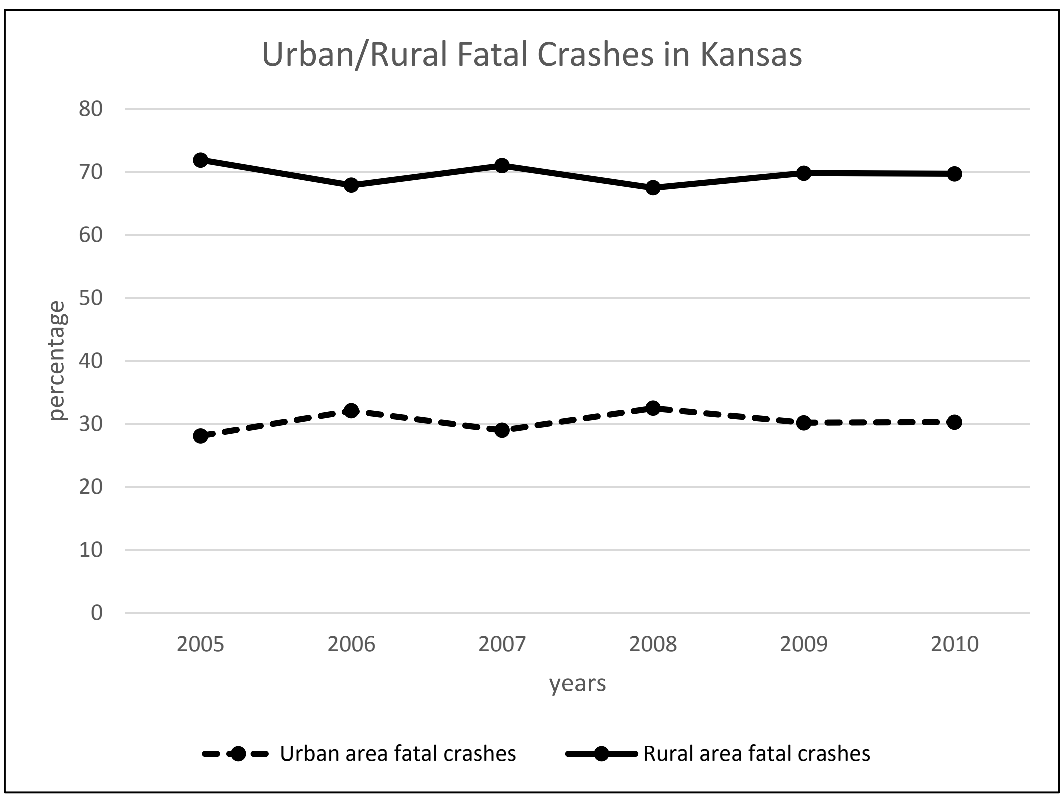
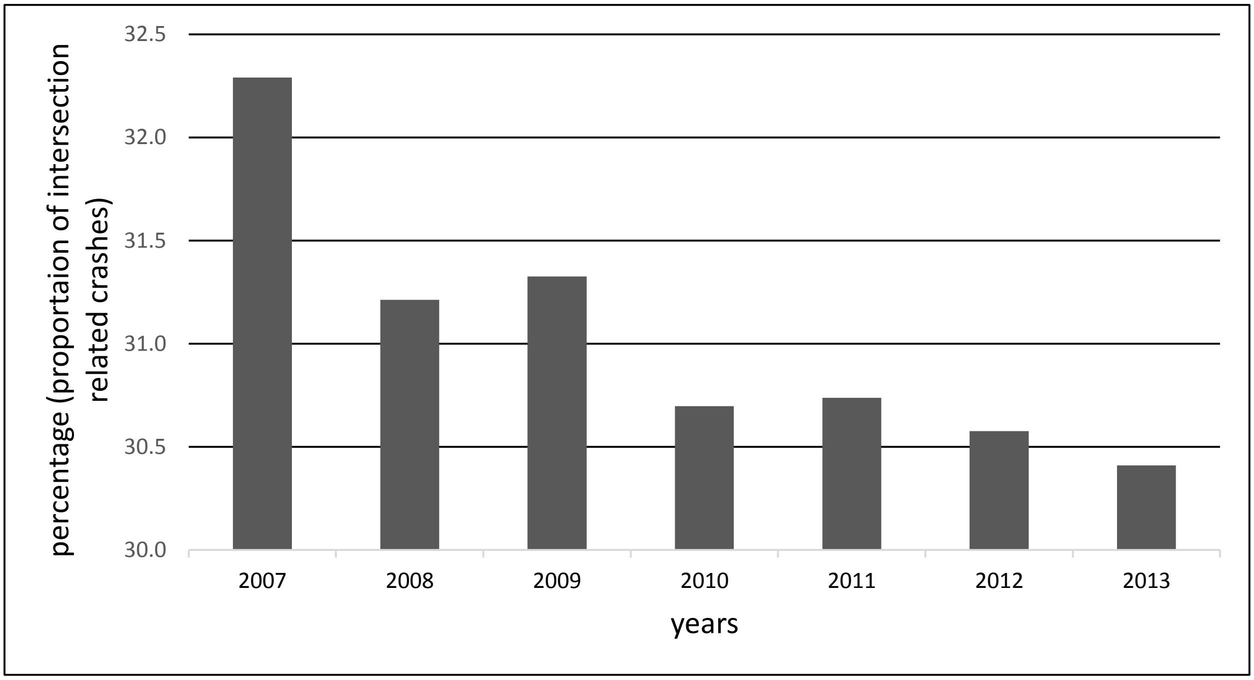


Figure 1.2 Proportion of urban and rural fatal crashes in Kansas



In concern of location of crashes almost 30% of crashes in Kansas happened at intersections or were intersection related (KDOT, 2013). Figure 1.3 shows the proportion of intersection related crashes compared to all crashes between years 2007 to 2013.

Figure 1.3 Proportion of intersection related crashes in compared to all



1.2 Overview

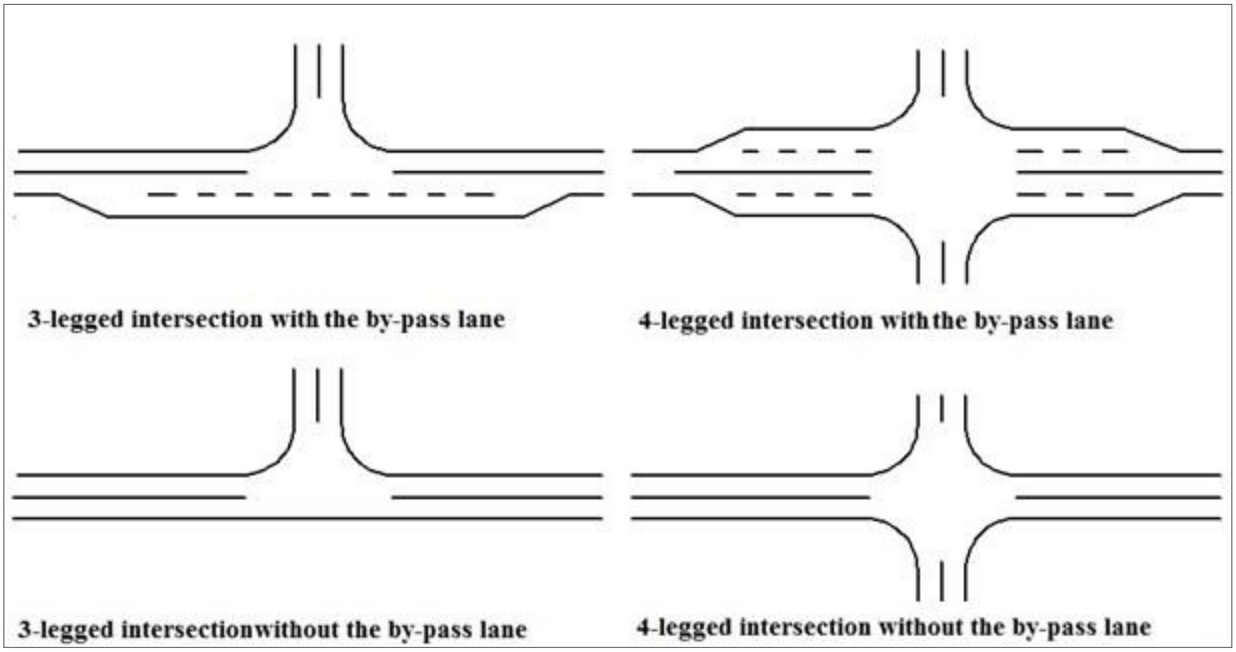
Safety should be defined before evaluating the level of safety of a transportation facility. An objective measure and subjective perception are commonly associated with road safety (ITE, 2009). The number of crashes and their severity indicate the objective measure of road safety, and the level of safety the driver feels when utilizing a transportation system shows subjective perception of road safety. Increased security level of a roadway does not necessarily translate into an enhanced level of road safety. In some cases, an increase in security may result in reduced safety because the road user feels safer and becomes less cautious. (ITE, 2009).

Due to lower AADT at rural roadways, the chance of crash is lower than urban roadways, so, drivers feel safer on rural roadways and have less expectation of crash occurrence in rural roadways. The opportunity for vehicle crashes increase at intersections, because vehicles approach the intersection from three or four directions.

This study focused on safety effectiveness of by-pass lane additions at rural unsignalized intersections. Urban intersections typically contain a specific lane for drivers turning left, but this lane commonly is not present at rural intersections. When a driver approaches an unsignalized intersection behind another left-turning vehicle, the driver must decrease vehicle speed and even stop. By-pass lane provides through traffic driving lane for following driver to bypass the slow or stopped left-turning vehicle. In concern of highway safety at 3-legged or 4-legged intersections where a portion of the paved shoulder may be marked as a lane for through traffic, installation of by-pass lanes at rural intersections have been identified as a low-cost safety improvement.

To address lack of operational and safety information related to the effects of by-pass lanes at rural intersections, Figure 1.4 shows a typical by-pass lane at 3-legged and 4-legged rural intersections on a two-lane highway.

Figure 1.4 Configuration of a typical by-pass lane



Kansas Department of Transportation (KDOT) has utilized by-pass lanes at rural intersections. If a vehicle stopped in the through travel lane is waiting to turn left, following vehicles are able to utilize the shoulder by-pass lane to avoid stopping (Fitzpatrick, et al., 2002). By-pass lanes are fairly prevalent on Kansas roadways, so a study was needed to determine whether it is beneficial for KDOT to continue adding by-pass lanes. Therefore, this study was expected to serve that purpose. Alternative evaluation methods are needed to provide safety effectiveness estimates for countermeasures such as by-pass lanes.

1.3 Objectives

The primary objective of this study was to determine a statistically reliable conclusion relative to a comparison of operational and safety characteristics of rural unsignalized intersections by specifically focusing on 3-legged and 4-legged intersection within rural areas in the state of

Kansas. This study discusses the results of two methods: before-and-after study and cross-sectional study. In before-and after study, crashes that occurred after adding by-pass lanes were compared to crashes occurring before the addition of by-pass lanes. Analysis was applied to estimate effectiveness of the countermeasure at identical intersections within two time periods. In cross-sectional study, intersections were divided into two categories: intersections with by-pass lanes and intersections without by-pass lanes. Statistical analysis was utilized to determine by-pass safety effectiveness at intersections with by-pass lanes compared to intersections without by-pass lanes.

Chapter 2 - Literature Review

Chapter 1 explained the definition of by-pass lanes at intersections and included national and state statistics which demonstrate the need for countermeasures at rural unsignalized intersections. As mentioned, by-pass lanes are fairly prevalent on Kansas roadways and it was necessary to conduct a study to identify whether it is beneficial for KDOT to continue adding by-pass lanes. Chapter 2 documents findings of previously published research regarding the safety evaluation of bypass lanes at intersections.

2.1 Studies Related to By-pass Lanes

A report published by Sebastian and Pusey (1982) investigated by-pass lanes after Delaware passed legislation in 1976 that allowed drivers to pass a stopped, left-turning car on the right, using the shoulder as necessary. This law did not designate a required shoulder width to be paved, so Delaware drivers utilized roadway shoulders to pass vehicles on the right on two-lane roads (Sebastian & Pusey, 1982). At that time, Delaware did not have standard widths of travel lanes, by-pass lane installation requirements, or pavement markings. Each by-pass lane was treated individually (Sebastian & Pusey, 1982). This study investigated the savings of user costs such as operating costs, time/delay, fuel consumption, and vehicle emissions and crash prevention to warrant the use of bypass lanes in designated left-turn lanes (Sebastian & Pusey, 1982).

Data was collected at 16 locations for three, two-hour peak periods: morning, noon, and evening. Average daily traffic was calculated using Delaware's Department of Transportation (DelDOT) annual summary report. Crashes were reviewed based on a three-year crash experience obtained from DelDOT's traffic crash records. Results indicated that rear-end crashes were the primary type of collision prevented by the use of by-pass lanes (Sebastian & Pusey, 1982). Conclusions of this report also included statistical proof of beneficial legalization of pass-on-the-right-lanes for reducing user operating costs, fuel consumption, travel delays, and emission and rear-end crashs (Sebastian & Pusey, 1982).

Minnesota Department of Transportation (MnDOT) funded a research project with BRW, Inc. to investigate the safety and use of rural intersections without turn lanes, with by-pass lanes, and with left-turn lanes in order to determine whether or not by-pass lanes should be used as a safety measure at unsignalized intersections (Preston & Schoenecker, 1999). Data on 3-legged intersections was collected using a survey sent to 212 government entities within Minnesota. Eighty-two completed surveys were returned. Another survey for 4-legged intersections was sent to 22 government entities and 14 were completed and returned. Results of this survey indicated that a majority of counties and cities did not reference MnDOT design guidelines. Most counties and cities also implemented inconsistent pavement markings that 3-legged bypass lanes had advantages in terms of delay and that 4-legged intersection bypass lanes should not be used (Preston & Schoenecker, 1999).

A legal review of by-pass lane implementation occurred because Minnesota revised highway design to include a required 10-ft paved shoulder. Consequently, users of rural roads began using the shoulder as a bypass lane to avoid turning vehicles although the intersection was not intended to include bypass lanes. Finally, it was stated that passing on the right is illegal in Minn. unless performed on a main-traveled portion of the roadway, thus requiring MnDOT to evaluate design regulations and implementation requirements for signage and marking (Preston & Schoenecker, 1999).

Safety analysis was conducted using crash data between 1995 and 1997 for the following areas (Preston & Schoenecker, 1999):

- Total and average number of intersection crashes
- Average crash rate using vehicle per day
    - 0-4000 vehicle per day
    - 4000-10000 vehicle per day
    - >10000 vehicle per day
- Distribution by severity and type

Intersections reviewed were 3 and 4-legged intersections categorized into (Preston & Schoenecker, 1999):

- No turn lanes
- Bypass lanes
- Left-turn lanes

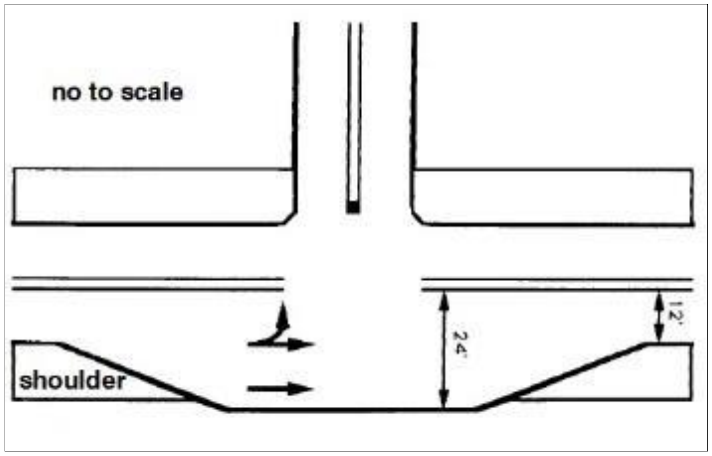
Additional before-and-after study was conducted which included a total of six years of crash data: three prior to installation and three post-installation. Sample size of the intersections was 69, and crash data used was between 1983 and 1994 (Preston & Schoenecker, 1999).

Safety summary of the 2700 reviewed intersections stated that 3-legged intersections had less vehicle crash occurrences than 4-legged intersections. The number of crashes did not appear to be a function of volume entering, but severity did. No statistical significance was evident between design types, and intersections with left-turn lanes had the lowest percentage of rear-end crashes (Preston & Schoenecker, 1999). Before-and-after study summary also showed no statistically significant differences, and intersections with by-pass lanes had a lower overall crash rate than the state average (Preston & Schoenecker, 1999). Safety analysis concluded that by-pass lanes are not statistically significant, thus suggesting that it is impossible to conclude that by-pass lanes should not be used as a safety device (Preston & Schoenecker, 1999).

Bruce & Hummer (1991) reviewed delay data to investigate the effectiveness of a left-turn by-pass lane on a two-lane rural T-intersection. Left-turn bypass lanes are defined as a paved area to the right of the lane on a major road, opposite the minor road at a T-intersection on a rural two-lane roadway (Bruce & Hummer, 1991). Design of the by-pass was designated as 300-ft taper out to 12-ft lane; 700-ft, 12-ft lane with 600-ft from end of run out taper to minor road centerline and then 100-ft past centerline; and 600-ft taper to a single-lane travel way (Bruce & Hummer, 1991).

The experiment relied on traffic simulation using TRAF-NETSIM, a detailed, stochastic, microscopic model developed by Federal Highway Administration (FHWA). Eight factors were identified for use in the simulation: volume of opposing traffic on major street, volume of right-turning traffic from minor street, left-turn volume, through volume, speed of vehicles, distance from T-intersection to nearest controlled intersection upstream/downstream, and the presence of a by-pass lane. Each factor was analyzed at LOS B and D. With eight factors, the experiment had a total of 256 combinations, but for efficiency, only 64 combinations were tested (Bruce & Hummer, 1991).

Figure 2.1 Left-turn by-pass concept



Significant variables found through analysis results included through volume, opposing volume, left-turn volume, speed upstream signal distance, and presence of the bypass lane. Average time saved was 0.50 s per vehicle (Bruce & Hummer, 1991).

2.2 Studies Related to Crash Modification Factor

A Crash Modification Factor (CMF) evaluates safety effectiveness of any given countermeasure. A CMF value less than one shows an expected reduction in vehicle crashes due to the countermeasure, but CMF greater than one indicates an increase in crashes after given countermeasure (Gross, et al., 2010).

Although a before-and-after study more commonly develops the CMF, an alternative method for CMF calculation needed to be explored. In before-and-after study, CMF is defined by comparing observed crash frequency after implementation of a countermeasure to crashes that did occurred before installation of that countermeasure. However, CMFs derived from cross-sectional data are based on a time period assuming that the ratio of average crash frequencies for sites with and without a feature is an estimate of CMF for implementing that feature (Gross & Donnell, 2011).

Gross and Donnell (2011) applied case control and cross-sectional method to developed CMF for roadway lightening and shoulder width. Four years (2001-2004) of data was used to estimate CMF for road lightening, including 6464 intersections in Minnesota. Only 13.7% of the intersections had signal control, while the remainder of the intersections operated with stop signs. Approximately 49% of the intersections were 4-legged, 40% were 3-legged, and 11% were 4-legged skewed intersections. The analysis database included 38,437 crash reports at intersections. This study obtained five years' data from 1997 through 2001 to evaluate safety effectiveness of shoulder and lane width. Total 21,688 segments which each was normally 0.8km from PennDOT. A total of 56,732 crashes occurred, and AADT of the segments ranged from 95 to 25,844 vehicles per day. Based on the case–control method, CMF for intersection lighting was 0.886, while according to the cross-sectional study, calculated CMF was 0.881. CMFs developed for lane and shoulder widths were also similar when the two methods were compared. This paper suggests that case–control and cross-sectional studies produce consistent results, especially when the before-and-after study was impractical (Gross & Donnell, 2011).

Gross and Jovanis (2007) applied case-control methods to evaluate safety effectiveness of lane and shoulder width. Their study estimated CMF as a common acceptable ratio to measure safety effectiveness by comparing the number of crashes with a countermeasure implementation and the number of crashes without a countermeasure. The study considered more than 28,000 rural two-lane undivided highways in Pennsylvania from the years 1997 to 2001. The paper provided a matched case-control design while adjusting for variables such as speed limit, AADT, and segment length. Finally, the CMF was provided for various ranges of shoulder width. Results showed that segments without shoulders are safer than segments with shoulder width from 0 to 1.83 m. However, CMF is lower than one for shoulder width greater than 1.83 m. Case-control estimation could advantageously estimate confidence levels, thereby conveying variability in safety effectiveness. Safety effectiveness range can be considered in economic analysis of alternative action.

2.3 Studies Related to Estimate Annual Average Daily Traffic (AADT)

Traffic volume of a road is identified by AADT. The ideal and most accurate method for estimating the AADT is traffic counting using permanent or temporary stations. However, this method is not applicable due to time and cost. Researchers tried to estimate AADT when the actual value is not available.

Mountain, et al. (1996) developed a model to predict crash rates on roads with minor junctions in which traffic counts on minor approaches are not available. This study was based on data for 3800 km of highway in the United Kingdom with more than 5000 minor junctions. Generalized linear model was used to develop regression estimates. Combined with crash counts, empirical Bayes procedure improved the estimates. The empirical Bayes model was utilized to remedy lack of AADT especially when traffic data were not available for minor roads. Data including information such as highway characteristics, crash counts, and traffic flow for 5-15 years. The study was limited to injury crashes only because property damage crashes were not reported in the U.K. Analysis did not include any major junction components because this study modeled minor junctions and links between minor junctions (Mountain, et al., 1996). Three methods were reviewed: crash count, predictive model, and empirical Bayes. Modeling results showed that crashes on highway links are not proportional to traffic flow and link length and crash frequencies are non-linear functions of traffic flow. Finally, the empirical Bayes method was superior to crash count followed by the predictive model as well as being the only method to produce unbiased estimates of high-risk sites (Mountain, et al., 1996).

Lubliner (2011) attempted to validate the Highway Safety Manual (HSM) prediction model for rural two-lane highway segments in Kansas. This study identified the difference between the Kansas highway system and how HSM recommends model application. A model was calibrated using HSM procedure and a new procedure according to 19 10-mile highway sections in Kansas. In order to select homogeneous segments, the study used IHSDM. The Control Section Analysis System (Kansas State Highway System Database) CANSYS database for 2007 was utilized to find the AADT at each homogeneous segment. Since AADT values varied over the analysis period, additional AADTS were gathered from KDOT historical traffic maps from 2005-2006. The study developed correlation between AADT and the observed/predicted (OP) crashes ratio for six districts in Kansas. The two highest OP ratios belonged to rural Districts 3 and 6 which had similar population density and travel demand.

Pan (2008) attempted to estimate AADT on all roads in Florida. This study used 26,721 traffic counts provided by FDOT to develop six AADT predictor models. Two different types of databases, including seven social economic and 14 independent variables were utilized to estimate the AADT. Pan used 10 years of social economic data, between the years of 1995-2005, collected for all 67 counties in Florida. Geometric road characteristics were gathered from various Geographical Information System (GIS) data layers provided by FDOT. The study used stepwise regression method on independent variables which were significant with 90% level of confidence. Finally, six linear regression models for highways in large metropolitan areas, local streets in large metropolitan areas, highways in small-medium urban areas, local streets in small-medium urban areas, highway models in rural areas, and local streets in rural areas were developed. R-square of the prediction models varied from 0.166 to 0.418.

Chapter 3 - Methodology

3.1 Background of Observational Studies

Researchers design an experiment or conduct an observational study to answer a specific questions. Experiments are studies that are implemented in a laboratory context; however, in observational studies, study parameters cannot be controlled entirely by researchers (ITE, 2009). Road safety studies are classified as observational studies because, in general, a crash is comprised of random circumstances and researchers are unable to control crashes. Observational studies can be categorized as before-and-after studies and cross-section studies.

In road safety studies, parameters that potentially influence safety may change in the before and after periods. For example, weather conditions and traffic regulations may change over traffic conditions in any given transportation system. Many attributes such as geometric designs of the road are constant. However, in cross-section observational studies, safety effects of one group of facilities are compared to another group. These two groups of facilities have similar features, but the safety effect of features that are not similar must be evaluated (ITE, 2009).

3.2 Before-and-After Studies

One common method for agencies to evaluate the safety effects of a specific roadway improvement is comparing the crash occurrence associated with the transportation facility before and after treatment implementation. Before-and-after designs include a treatment at some time periods and a comparison of safety performance before and after treatment for a site or group of sites (Gross, et al., 2010). However, these studies are challenging because crashes are random and change from year to year, unlike laboratory experiments in which the analyst controls extraneous conditions (ITE, 2009).

The before-and-after study is a commonly used method to measure safety effects of a specific treatment or a combination of treatments for highway safety (Hauer, 1997). In short controlled and fully randomized study design, a before-and-after study is deemed superior to cross-sectional studies since many attributes linked to converted sites where the treatment was implemented remain unchanged. Other parameters that affect the safety of a facility, such as traffic volume and weather conditions, change over time. Consequently, specific evaluation techniques are required to account for changes in order to estimate the true effects of safety improvements.

Although not perfect, the before-and-after study approach offers better control for estimating effects of a treatment. As the name suggests, the before-and-after implies that a change occurred between the "before" and "after" conditions (Hauer, 1997). In this section, an overview of four of the most commonly used methods in before-and-after studies is briefly explained (ITE, 2009).

3.2.1 Naïve Before-and-After Study

The naïve before-and-after study is the simplest technique for this kind of observational study. In naïve before-and-after study, future crashes are compared with before period crashes; so, the treatment effect can be considered as the difference between crash counts in the after period and before period crash counts (ITE, 2009).

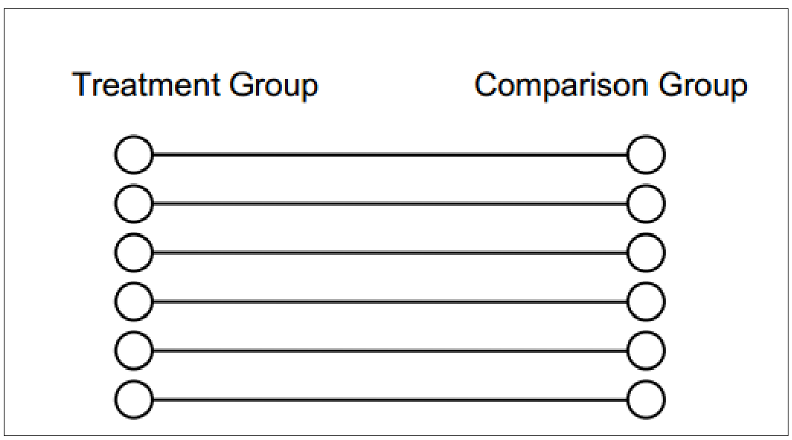
3.2.2 Before-and-After with Yoked Comparison

In yoked comparison, the evaluated treatment effect refer to the treatment site and comparison site, respectively (Griffin & Robert, 1997). The treatment group is similar to the comparison group with a one-to-one correspondence between each member of the comparison group and the treatment group. Therefore, a similar groups must be selected. For example, if the treatment facility is a roundabout, the comparison should be roundabout with respect to area type (urban, rural), number of roads lanes, geometric characteristics design and traffic volume.

The comparison site should not have undergone any geometric change or traffic control improvement during the before and after periods (Harwood, 2002).

Figure 3.1 represents the one–to-one correspondence between each member of the comparison group and the treatment group.

Figure 3.1 Relationship between treatment and comparison groups (ITE, 2009)

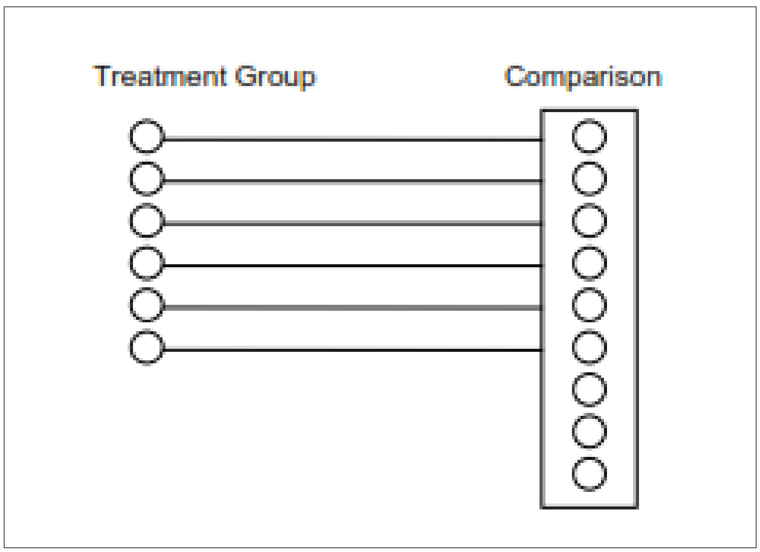


Unknown casual factors are a critical issue, so in this method, it is hoped that unknown casual factors have the same effect on the comparison group and treatment group. Although crashes during the after period may change without any improvement, based on the crash change in comparison site the after to before crash ratio will be calculated in this method. The crash frequency during after period is calculated by crash frequency during before period multiplied by the after to before crash ratio. This is the crash frequency during after period with no improvement. The difference between predicted after frequency crashes and actual after period crash frequency demonstrate effect of the treatment (ITE, 2009).

3.2.3 Before-and-After Study with Comparison Group

This approach follows the same rational as the yoked comparison method, but the comparison group and treatment group have different sizes. The comparison group has a bigger sample size, and no one-to-one matching is present between them (ITE, 2009). Figure 3.2 is a graphical representation of the treatment and comparison groups.

Figure 3.2 Relationship between treatment group and comparison group - before and after study with comparison group (ITE, 2009)



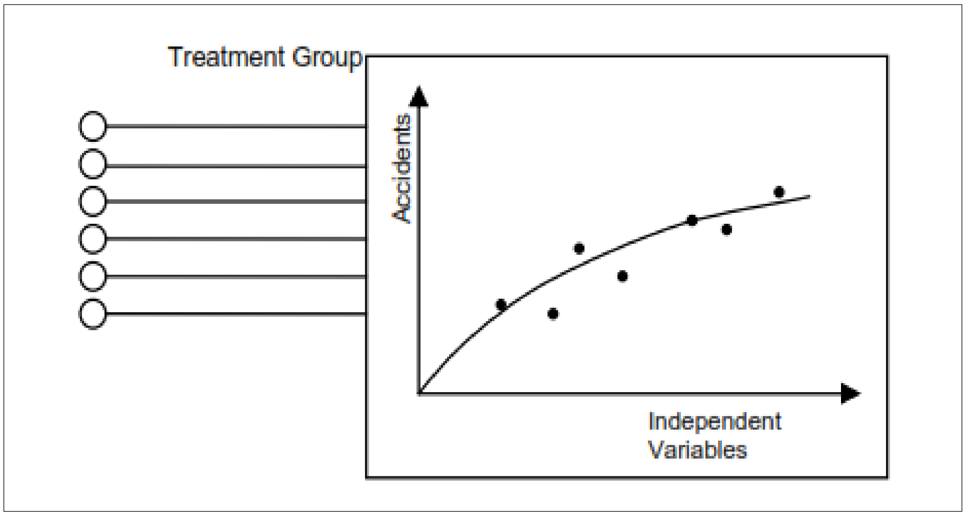
In this approach, however, the comparison group does not have to be identical to the facilities in the treatment group, but it is important that treatment and comparison groups have a similar crash history during before period. In addition, this technique is similar to the yoked comparison approach in that it cannot determine treatment effectiveness if crash counts in the before or after period in the comparison group equal zero. This situation is unlikely to occur due to having a group of comparison sites rather than only one single comparison site for each specific treatment site (ITE, 2009).

3.2.4 Before-and-After Study with the Empirical Bayes Approach

In general, the safety treatments were applied for high crash rates facilities. However, if the selection during before period was selected within a short-term high occurrence of crashes, a lower crash rate would be expected in the after period, even if no improvement had been implemented. In statistics, this approach is known as regression-to-the-mean in which a regression line with the appropriate coefficient of each relevant factor is determined to predict the crash rate for treatment group (ITE, 2009). Safety performance functions (SPFs) are used to estimate crash frequencies. SPFs are regression models that explain the relationship between crash frequency and explanatory variables, such as traffic volume of the facility (ITE, 2009). Figure 3.3 shows a graphical representation of the treatment and comparison function for this method.

In this approach, crash frequency in the after period, with no treatment, can be estimated based on observed frequency in the before period and the SPF function developed for the comparison group (ITE, 2009). Therefore, the difference between expected future crash and actual crashes in the after period reveals effect of the treatment

Figure 3.3 Relationship between treatment group and comparison group – Empirical Bayes approach (ITE, 2009)



3.3 Cross-sectional Studies

Cross-sectional study is observational study in transportation safety. Cross-sectional study approach compares safety performance of a site or group of sites with the treatment of interest to similar sites without treatment at a single point in time (Gross, et al., 2010). Cross-sectional studies divide intersections in two major groups:

- Intersections with the by-pass lanes
- Intersections without the by-pass lanes

As mentioned earlier, challenge inherent in observational studies is that crashes are random and change from year to year (ITE, 2009). In addition, other parameters that affect facility safety, such as traffic volume and weather conditions, are also different at each intersection. In order to evaluate the safety effectiveness of specific treatment, the HSM recommends a period of three to five years comparison between crash data at sites where the treatment was implemented versus sites without any countermeasure (AASHTO, 2010).

3.4 Statistical Analysis Using t-test

The t distribution is a symmetric distribution like normal distribution, which is ticker tails (Martz & Paret, 2013). This distribution is useful for analyzing the mean of an approximately normal population when the population standard deviation is unknown (Martz & Paret, 2013).

Considering crash frequency at intersections with by-pass lanes is the case. If the average crash frequency per intersection before and after adding the by-pass lane is μ_1 and μ_2 respectively, the t-test can be used to determine whether a significant change occurs between average crash frequency per intersection in the before and after period. Therefore, the null hypothesis is

$$H_0 : \mu_1 = \mu_2$$

Depending on the issue that is being analyzed, the alternative hypothesis can take one of the following forms:

$$H_1 : \mu_1 > \mu_2 \ (one - tail\ test)$$

$$H_1 : \mu_1 < \ \mu_2 \ (one - tail\ test)$$

$$H_1 : \mu_1 \neq \mu_2 \ (two - tail\ test)$$

When the critical area of the distribution is one side, either greater than or less than certain value, it called one tailed test. And two tailed test would be used to see if two means are different. The t-value is computed from Eq.3.1 (Ruxton, 2006).

$$T = \frac{\bar{X}_1 - \bar{X}_2}{S_p^2\sqrt{\frac{1}{n_1} + \frac{1}{n_2}}} \tag{3.1}$$

Where,

$\bar{X}_1 and\ \bar{X}_2$ = Sample means

$n_1\ and\ n_2$ = Sample size

S_p = Square root of the pooled variance given by (Ruxton, 2006)

$$S_p{}^2 = \frac{(n_1-1)S_1{}^2+(n_2-1)S_2{}^2}{n_1+n_2} \tag{3.2}$$

Where,

$S_1\ and\ S_2$ = Variance of the population

The degree of freedom and the level of significance (α) affect the value of t. The degree of freedom for t-distribution is $(n_1 + n_2 - 2)$, and the level of significance is the probability of rejecting the null hypothesis. When the null hypothesis is true and rejected, it is typically referred to as Type 1 error. If the null hypothesis is not true and it is accepted, error Type 2 is said to be happened. The most commonly used "α" value in traffic safety studies is 5%, although 10% is sometimes used. When the t-test is one-tail, the "*t"* value is selected for "α"; when the test is two-tail, the *t* value is selected for "α/2" Rejection of the null hypothesis is shown in Table3.1.

Table 3.1 Rejection of null hypothesis based on t-value.

Alternative hypothesis	Rejection region for H_0
$H_1 : \mu_1 > \mu_2\ (one-tail\ test)$	$T > t_\alpha$
$H_1 : \mu_1 < \mu_2\ (one-tail\ test)$	$T > t_\alpha$
$H_1 : \mu_1 \neq \mu_2\ (two-tail\ test)$	$\lvert T \rvert > t_{\propto/2}$

The null hypothesis is rejected if the sample t-value is more than the critical t-value, meaning that the probability of obtaining a t-value at least as critical t-value is less than 5% (or whatever α is). Therefore, the null hypothesis is not true. In other words, there is a significant reduction between two sample means (Ruxton, 2006). The null hypothesis is not be rejected if the sample t-value is less than the critical t-value, meaning that the probability of obtaining a t-value at least as critical t-value is greater than 5% (or whatever α is). Therefore, the null hypothesis could be true, or there is no significant difference between the populations means (Ruxton, 2006).

3.4.1 The p-value vs. α value

The standard level is known as alpha (α), usually is set at 0.05. Assuming that the null hypothesis is true, the null hypothesis may be rejected only if observed data are so unusual that they occurred by chance at most 5 % of the time. Each statistic has an associated probability value (p-value), which is, the likelihood of an observed statistic occurring due to chance, given sampling distribution. Instead of comparing t-critical and t-statistical values to determine significant difference, p-value may be used to compare to the significance level (Martz & Paret, 2013). A large t-value means a large difference between sample means, so a larger t-value is associated with a smaller p-value. Table 3.2 shows the rejection regions of the null hypothesis.

Table 3.2 Rejection of null hypothesis based on p-value.

Alternative hypothesis	Rejection region for H_0
$H_1: \mu_1 > \mu_2\ (one - tail\ test)$	$\alpha\ > p - value$
$H_1: \mu_1 < \ \mu_2\ (one - tail\ test)$	$\alpha\ > p - value$
$H_1: \mu_1 \neq \mu_2\ (two - tail\ test)$	$^{\propto}/_{2}\ > p - value$

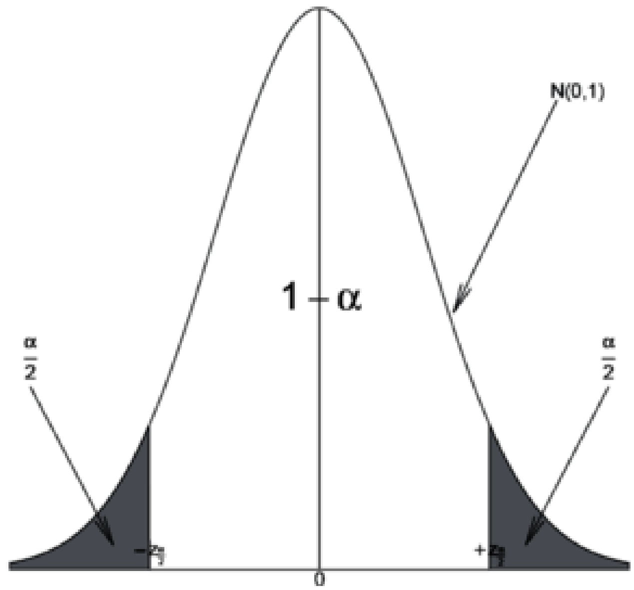
Significance level sets the standard for how extreme data must be before rejecting the null hypothesis. The p-value indicates how extreme the data are (Martz & Paret, 2013). The p-value is compared to significance level to determine whether the observed data are statistically significantly different from the null hypothesis:

- If the p-value is less than or equal to the alpha (p-value ≤ α), then the null hypothesis is rejected, or there is a significant difference between samples means.
- If the p-value is greater than alpha (p-value > α), the null hypothesis is not rejected, or there is no significant reduction between samples means.

3.4.2 Confidence Interval

Confidence interval (CI) is an interval estimation of the population to indicate reliability of the estimation. A CI gives an estimated range of values likely to include an unknown population parameter; the estimated range is calculated from a given set of sample data. Confidence level, known as (1 - α), is associated with CI are calculated as 95%, but sometime 90%, 99% or whatever CI can be used (Sharabati.W, 2009).

Figure 3.4 Confidence interval representation (Sharabati.W, 2009)



3.5 Crash Modification Factor

Crash Modification Factor (CMF) can be used by transportation professionals, such as traffic engineers, transportation planners, and designers (Gross, et al., 2010). CMF can be used to evaluate the effectiveness of a given countermeasure, to find the cost-benefit treatment. It can be used to select the reasonable evaluation, compare the new analysis with the existing CMFs. CMF application can be used for all crashes and locations, or for specific crashes and locations, such as in collision with animals at rural two lane rural highways. In general, CMF application may change of different crash characteristics, such as, crash severity, crash type, crash frequency, and crash location in rural or urban area.

CMF is used to estimate the safety effectiveness of specific countermeasure. It is used to compute the number of crashes after implementation of a countermeasure to compute the effect of that countermeasure at specific site locations (Gross, et al., 2010) . A CMF greater than 1.0

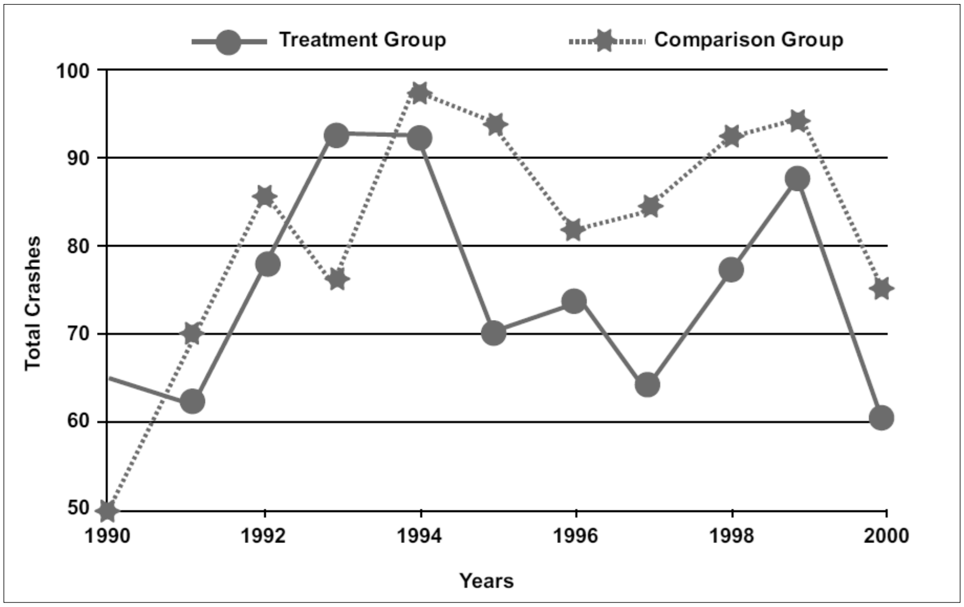
indicates an expected increase in crashes, demonstrating that the countermeasure decrease safety in that location. In contrast, a CMF less than 1.0, indicates a reduction in crashes after implementation of where the given countermeasure has been introduce, demonstrating the countermeasure increases the safety in that location (Gross, et al., 2010).

CMF function is a formula used to compute CMF for each site. Based on site characteristics, a different CMF should be assumed for each site. A countermeasure may have several levels, so different CMF formulas offer accurate ratios to estimate safety effectiveness of each of the steps (Gross, et al., 2010).

3.5.1 Before–and-After with Comparison Group Study to Measure the CMF

In this approach, an untreated comparison group similar to treated groups is used to account for crash changes which are not relevant to countermeasures. The unrelated effect calculated by changes in crash frequency in the after period compared to the before period in comparison group. Then the observed crash frequency multiplied by the comparison ratio provides the expected number of crashes in the after period without treatment implementation. The difference between expected number of crashes in the after period actual number demonstrates safety effectiveness of the specific treatment (Gross, et al., 2010). It is difficult to achieve a perfect comparison group, since the change in crashes at the treatment sites without treatment cannot be known (Hauer, 1997). Figure 3.5 illustrates the similarity and suitability of a comparison group. In this example, the treatment implied after 2000.

Figure 3.5 Time series plot of crashes in treatment and comparison group



Hauer (1997) proposed a ratio to assess the suitability of comparison groups to treatment groups. Sample odds ratios were computed for each before-and-after pair in the time series before the treatment was implemented. From this sequence of sample odds ratios, the sample mean and standard error were determined. If this sample mean was sufficiently close to 1.0 (i.e., subjectively close to 1.0 and the CI includes the value of 1.0) then the candidate reference group was deemed suitable (Hauer, 1997)

$$sample\ odds\ ratio = \frac{(treatment_{before} \times comparison_{after}) \times (treatmenttreatment_{after} \times comparison_{before})}{1 + \frac{1}{treatment_{after}} + \frac{1}{comparison_{before}}} \qquad (3.3)$$

Where,

Treatment before = total crashes for the treatment group in year i.

Treatment after = total crashes for the treatment group in year j.

Comparison before = total crashes for the comparison group in year i.

Comparison after = total crashes for the comparison group in year j.

Additional requirements of a suitable comparison group, as outlined by (Hauer, 1997), include:

1. Before and after periods for the treatment and comparison group should be identical.
2. Reason should be evident for the change in factors, such as traffic volume changes, which influence safety rather than the studies treatment are the same in the treatment and comparison groups.
3. Crash counts must be sufficiently large.

Table 3.3 Before –and-After with Comparison Group Study

Risk Factor	Number of Cases	Number of Controls
Before	No. observed, T, B	No. observed, C, B
Absence	No. observed, T, A	No. observed, C, A

Where,

No.observed,T,B = the observed number of crashes in the before period for the treatment group.

No.observed,T,A = the observed number of crashes in the after period for the treatment group.

No.observed,C,B = the observed number of crashes in the before period in the comparison group.

No.observed,C,A = the observed number of crashes in the after period in the comparison group.

The comparison ratio ($N_{observed,C,A}$ / $N_{observed,C,B}$) indicates how crash counts are expected to change in the absence of treatment. CMF can be derived from Equations 3.4 to 3.7, which shows safety effectiveness of the specific treatment.

$$No.Expected_{TA} = No.Observed_{TB} \times \frac{No.Observed_{CA}}{No.Observed_{CB}} \tag{3.4}$$

$$VAR\ (No.Expected_{TA}) = No.Expected_{TA}{}^{2} \times (\frac{1}{No.Observed_{TB}} + \frac{1}{No.Observed_{CB}} + \frac{1}{No.Observed_{CA}}) \qquad (3.5)$$

$$CMF = (\frac{No.Observed_{TA}}{No.Expected_{TA}})/(\ 1 + (\frac{VAR\ (No.expected_{TA})}{No.Expected_{TA}{}^{2}})) \qquad (3.6)$$

$$VAR(CMF) = CMF^{2} \times \left[\left(\frac{1}{No.Observed_{TA}}\right) + \left(\frac{VAR\ (No.expected_{TA})}{No.Expected_{TA}{}^{2}}\right)\right]/[\ 1 + \left(\frac{VAR\ (No.expected_{TA})}{No.Expected_{TA}{}^{2}}\right)]^{2} \qquad (3.7)$$

3.5.2 Case-Control Studies to Measure the CMF

Several studies have been carried out in deferent aspects of highway safety, but few studies have been performed on geometric design. Recently case-control studies have been employed on geometric design elements (Gross & Jovanis, 2007). In case-control studies, samples are selected based on their status (crash or not crash) and then treatment is determined. Cases defined as intersections with crash and control sites were identified as intersections without a crash during the study period.

Table 3.4 Tribulation for Case-Control Analysis

Risk Factors	Number of Cases	Number of Controls
Present	A	B
Absence	C	D

$$Odds\ Ratio(CMF) = \frac{A/B}{C/D} = \frac{A \times D}{B \times C} \qquad (3.5)$$

Where,

A = number of cases with risk factor present

B = number of controls with risk factor present

C = number of cases with risk factor absent

D = number of controls with risk factor absent

Case-Control studies cannot be used to measure exact probability of an event, such as crash or severe injury, in terms of expected frequency. Instead, these studies are often used to demonstrate relative effects of treatments (Gross, et al., 2010).

3.6 Data Collection

This section discusses all data elements collected for this study and the data source and data collection procedure. The following sections include additional discussion that explicitly demonstrates the need for each data elements.

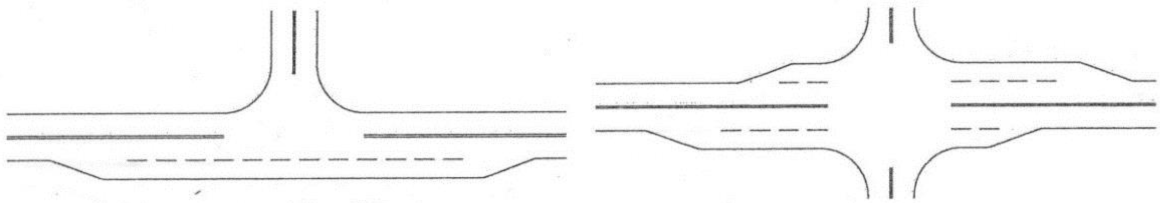
3.6.1 Survey Forms

Kansas Department of Transportation (KDOT) has utilized by-pass lanes as a rural unsignalized intersections in whole state of Kansas. KDOT has geographically divided the state, starting at the highest level of the district, for which there are six in the state. Each district has its own areas, for which there are 26 areas in Kansas (KDOT, 2013). Every district and area has its own area or district engineers. In order to find the location and the characteristics of rural unsignalized intersections which have by-pass lanes, survey forms were sent to area engineers. Survey form includes several questions, such as name of the roads, AADT, speed limits, pavement markings, and question regarding date of adding the by-pass lanes. The sample of survey form is shown in Figure 3.6. For the survey responses, 563 completed survey forms were received. Figure 3.7 shows the number of received survey forms by districts. Splitting the received surveys by districts were used primarily to ensure proper distribution of data geographically throughout the state.

Figure 3.6 Completed survey form

Bypass Lanes at Unsignalized Rural Intersections Survey

Section 2: Intersection Inventory (Please make additional copies as required)



X Three-Legged *Intersection Bypass ____ Four-Legged *Intersection Bypass

(Please modify the sketch as necessary)

*Intersection bypass lanes are described as any additional pavement added to the shoulder of an intersection that is not designated or marked as a lane.

Please indicate if intersection information below is for a three or four-legged design

<u>Main Road Information</u>

Name: US-281

<u>Minor (Cross) Road Information</u>

Name: NW Elm Mills Barber County

If additional information is available, please assist with any of the following:

<u>Main Road Information cont.</u>

ADT: 1460
Speed: 65

<u>Minor (Cross) Road Information cont.</u>

ADT: N/A
Speed: 55

What date/year was this intersection bypass lane added? 1994

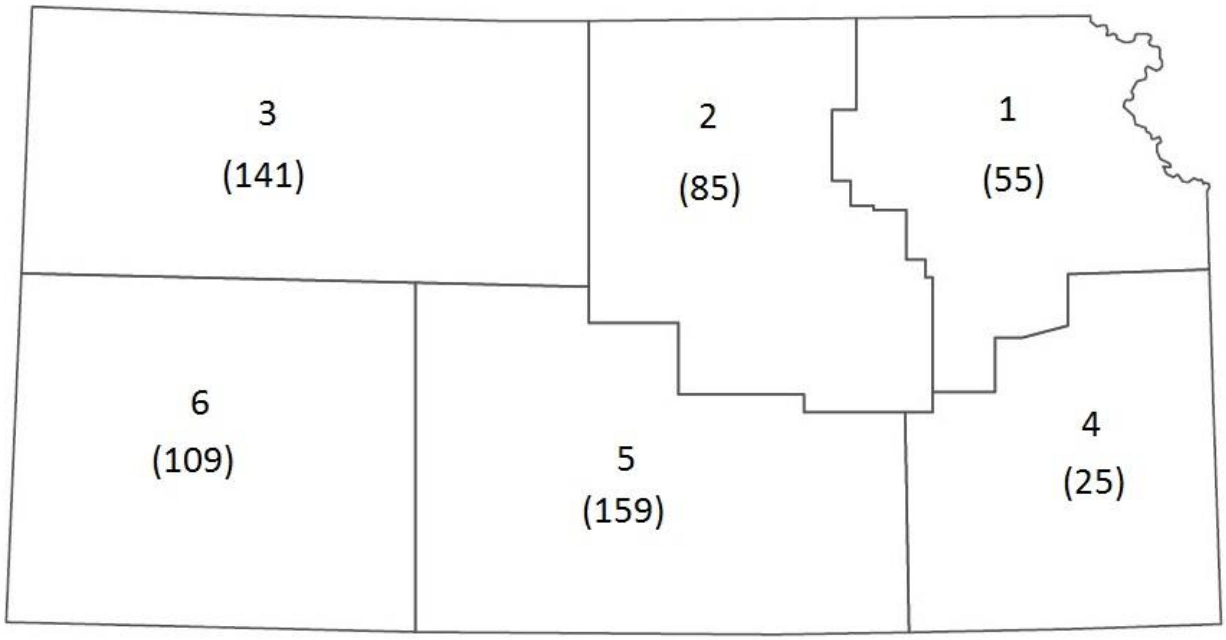
What kind of signing is used on the main road?
Pass With Care Road Narrows None Other (Please Describe)

What type of pavement marking is used in delineating the shoulder bypass?
Solid White Skip White None Other (Please Describe)

What kinds of pavement messages are used?
Thru Arrow "BYPASS ONLY" None Other (Please Describe)

What kind of pavement design do you require for the shoulder?
Standard Reinforced Road Lane Design Other (Please Describe)

Figure 3.7 Distribution of completed survey forms by districts



3.6.2 Kansas Crash and Analysis Record System (KCARS)

Although past statistics indicated that most traffic crashes resulted from drivers' errors (behavioral factors), but with a better understanding of non-behavioral factors, transportation engineers will be able to design freeways with higher safety standards. The safety effectiveness of any countermeasure are shown by the reduction in number of the crashes or their severities cause by treatment implementation. In order to find the crashes at each intersection, the KCARS, which maintain by KDOT, was utilized. The KDOT maintains a database of all crashes on the Kansas highway system. This database is coded in accordance with the Kansas Motor Vehicle Crash Report (850A). A report is filled out for every incident involving the Kansas Highway Patrol (KHP). For this study, every crash report filed from 1990-2011 were gathered. When performing data collection, the HSM recommends a period of three to five years be utilized, because periods shorter than three years are subject to high variability due to randomness of crashes. Longer periods than five years are subject to introduction of bias due to changes in reporting standards or the physical changes to the roadway features (AASHTO, 2010).

3.6.2.1 Crash ID

The KCARS contains a field that identifies the location and specific identification number of each crash. Crash ID is a unique value for each crash, so it can be used to combine crash characteristics from KCARS to other databases, such as Control Section Analysis System (Kansas State Highway System Database) CANSYS, in order to add information about highway geometric characteristics.

3.6.2.2 Location of Crash

Several fields in the KCARS represent where a crash occurs, including the county milepost and distance from a named intersection. Because incident responders do not typically have precise positioning equipment to determine the specific milepost of an incident, this value can contain inaccuracies. Two more columns in KCARS provide longitude and latitude of the crash location.

3.6.2.3 Crash Severity

The KCARS contains three main types of crash severity, where injury severity could again be subdivided into three level as follows (KDOT, 2005):

1. Fatal crashes
2. Injury crashes:
    - Possible injury
    - Injury, non-incapacitating

- Disable, incapacitating

3. Property damage only (PDO).

Multiple vehicle crashes can varying severity levels, based on personal injury severities. Each crash is assigned to the most severe level experienced by persons who are involved.

Fatal injury

Fatal injury is defined as any injury that results in death to a person within 30 days of the crash. If a person dies of a medical condition or after the 30 day limit, the injury checkbox is marked in crash reports (not fatal), and the injury severity is shown as possible injury (KDOT, 2005).

Possible injury

A possible injury is defined as any reported or claimed injury which is not fatal, incapacitating, or non-incapacitating, including momentary unconsciousness, claim of injuries not evident, limping, complaint of pain, nausea or hysteria (KDOT, 2005).

Injury (non-incapacitating)

A non-incapacitating injury is defined as any injury, other than a fatal injury or incapacitating injury, which is evident to observers at the scene of the crash at which the injury occurred (KDOT, 2005).

Disabled (incapacitating)

An incapacitating injury is defined as any injury, other than fatal, which prevents the injured person from walking, driving, or normally continuing activities he/she was capable before the injury occurred, including severe lacerations, broken or distorted limbs, skull or chest injuries, abdominal injuries, unconsciousness at or when taken from the crash scene, or inability to leave the crash scene without assistance (KDOT, 2005).

Property Damage Only (PDO)

Property or under the $1,000 property damage threshold with no injuries are not submitted to KDOT. Multiple vehicle crashes can have varying severity levels for each vehicle involved in the crash (KDOT, 2005).

3.6.3 Equivalent number of Property Damage Only crashes (EPDO):

To compare and ranking the severity of each location, the severity of individual crashes can be expressed in terms of Equivalent number of Property Damage Only (EPDO) crashes. In this approach a weight is assigned to each fatal or injury crashes to represent the severity of the location (Knapp, 2005).

$$EPDO = no.PDO\ Crashes + W_1 \times no.Injury\ Crashes + W_2 \times no.Fatal\ Crashes \qquad (3.6)$$

Where,

$$w_1 = weight\ factor\ to\ convert\ injury\ crashes\ to\ PDO\ crashes = \frac{Average\ Injury\ crash\ cost}{Average\ PDO\ crash\ cost}$$

$$w_2 = weight\ factor\ to\ convert\ fatal\ crashes\ to\ PDO\ crashes = \frac{Average\ Fatal\ crash\ cost}{Average\ PDO\ crash\ cost}$$

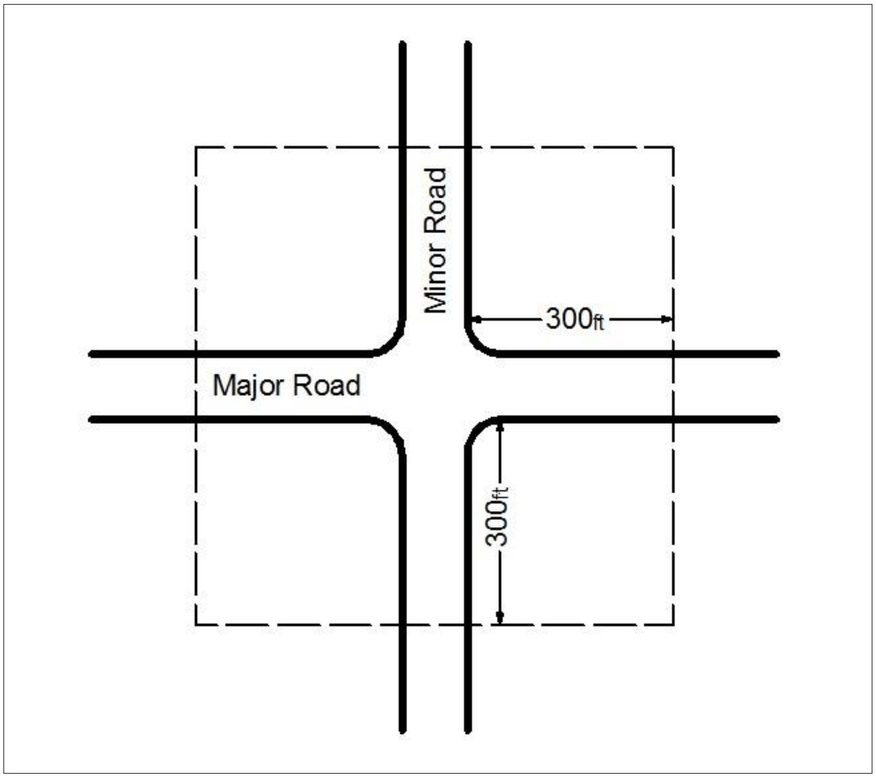
In Kansas: $W_1 = W_2 = 15$

3.6.4 Relevant Crashes

The focus of this study is unsignalized rural, 3-legged and 4-legged intersections in Kansas. In order to determine relevant crashes include in the study, KDOT follows two methods:

1. A fixed distance of 300-ft along each approach leading to the intersections, regardless of whether or not crashes are intersection related.

Figure 3.8 Intersection related crash box



2. Consideration of the column in KCARS which distinguishes whether or not crashes are intersection related, no matter crash distance from named intersections.

3.7 KDOT Traffic Count Maps

Crash rates, can be an effective parameter to evaluate the safety of allocation. The combination of crash frequency and traffic volume results in crash rates. Crash rates can be used to comparison between the relatively safety at intersections. The traffic volume for each approach at intersections were needed to calculate the crash rates at intersections (Green & Agent, 2003). Traffic counts shown on figure 3.9 represent the Annual Average Daily Traffic (AADT) for the year 2012. These AADT's were primarily derived from 24-hour volume recorded traffic counters.

Short-term counts were adjusted for day-of-week and seasonal variations and axle correction factor was applied to each short-term count. Heavy commercial volumes were derived from short-term vehicle classification counts (ITE, 2009). The focus of this study is rural intersections, so it includes many minor local roads not mentioned in traffic flow maps the Kansas state highway system.

In addition to traffic count state maps, AADT values of county major collector rural roads are available on KDOT website, which provide minor road AADT in some cases. Roads are labeled their Road Secondary (RS) numbers. RS number are different from the name of the roads, so it had to match up the RS route with Google Maps to find the road name of each RS number. After getting the RS route from the district map, then Google Map was check simultaneously. A city along the route can be chosen on the county map, and then the side roads can be counted to match them on the county map and Google Map. Figure 3.10 shows the matchup between Road Secondary map and Google Map. It shows RS 1924 is Anderson Avenue which is close to Manhattan, KS.

Figure 3.9 Traffic flow map, a part of the Kansas highway system

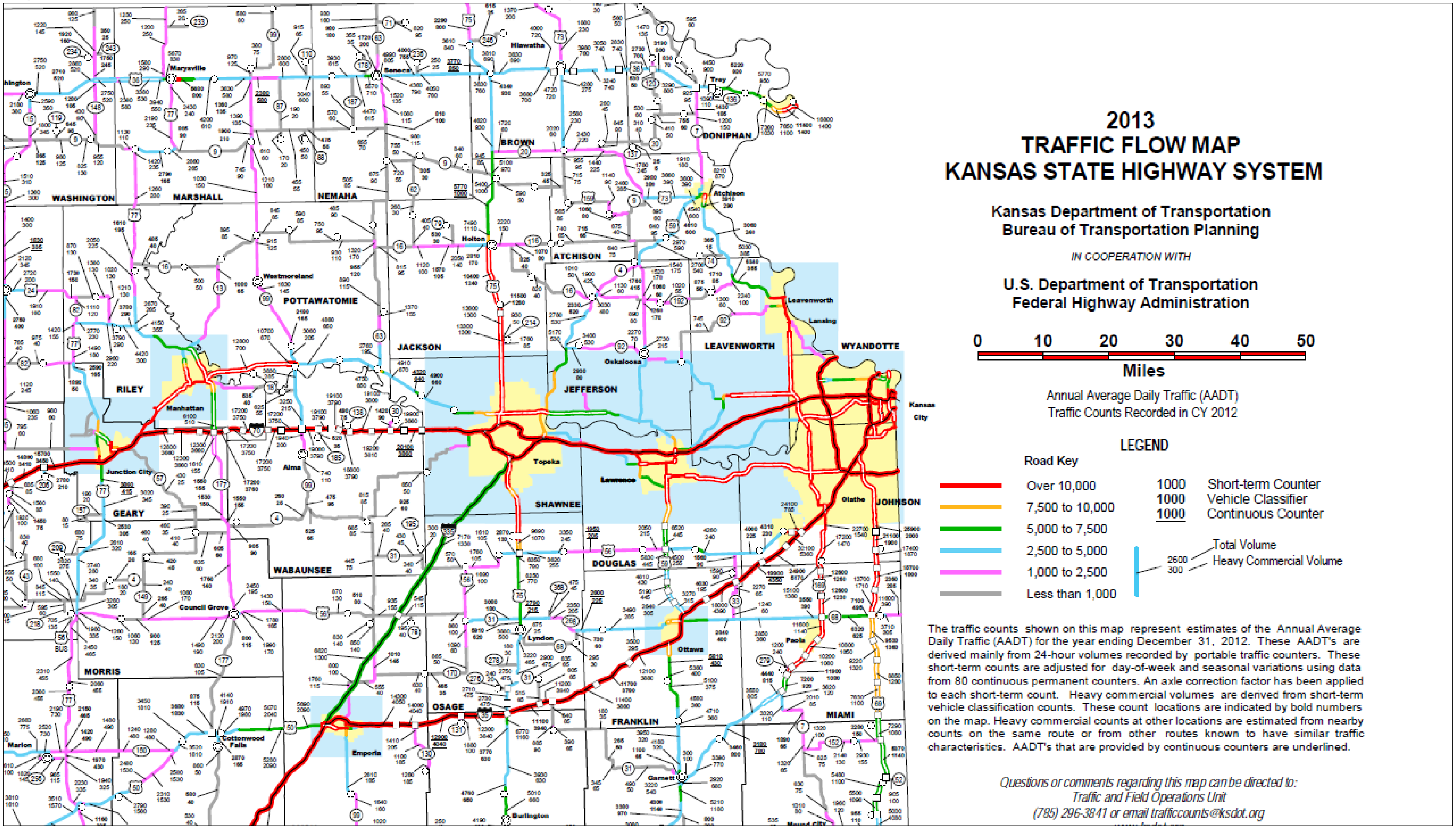
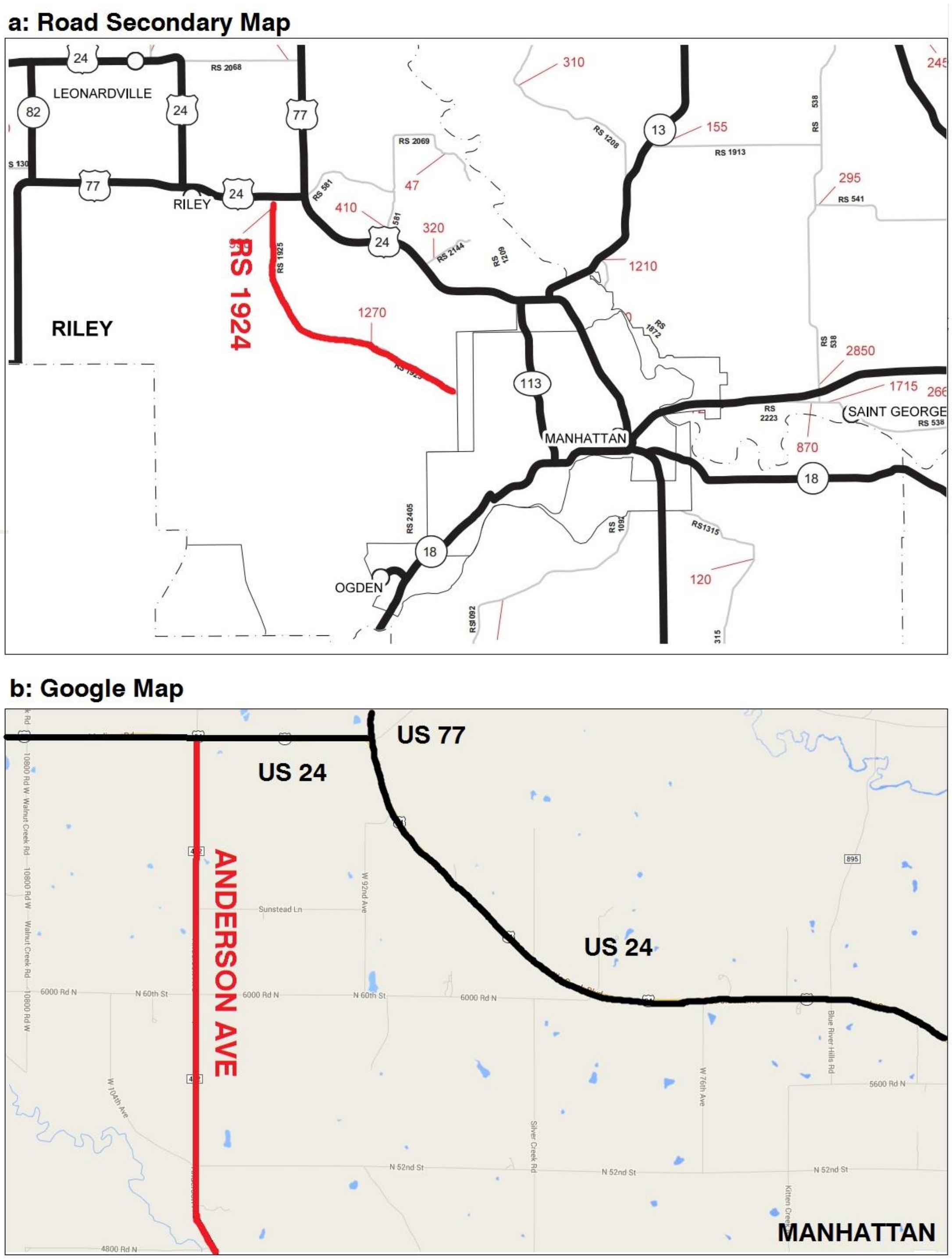


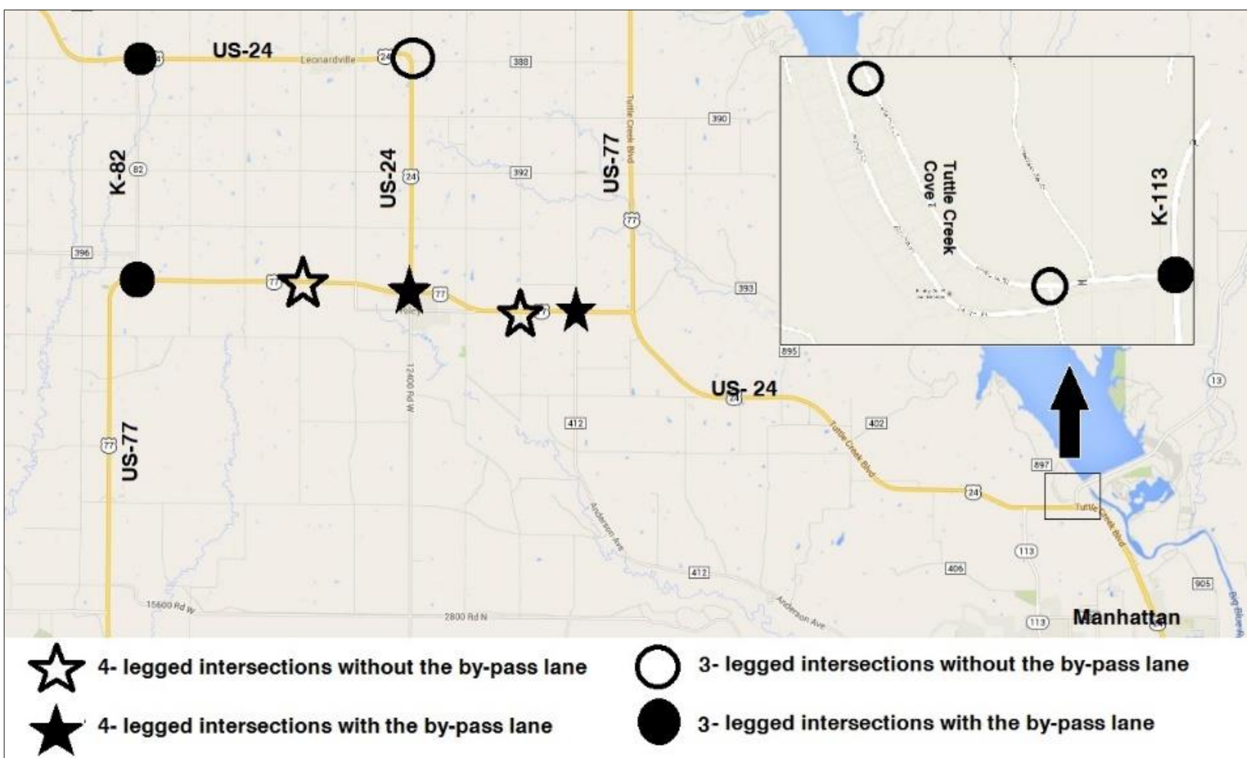
Figure 3.10 Traffic flow maps and Google maps



3.8 Video Recording

If stopped vehicle in the through travel lane is waiting to make a left turn, following vehicles can use the shoulder by-pass lane to avoid having stop. In order to quantitatively calculate the speed reduction and delays caused by the absence of by-pass lanes at intersections, video capturing has been implemented. However, traffic counting at intersections is challenging, especially when speed reduction and delay must be recorded. In order to capture the maneuver of drivers in areas around Manhattan, 10 different locations had been selected among intersection with and without by-pass lanes as shown in Figure 3.11.

Figure 3.11 Intersection locations



Locations were selected near Manhattan, so that would be convenient to go there and install the video camera. Locations were selected in proximity to have similar traffic volume and driver behaviors. In order to record traffic movements, a video camera was installed on a pole, sign, or on tripod near the intersections. Figure 3.12 shows an example of the installed camera, at one of the sites.

Figure 3.12 Installed Go-pro camera on pole

3.9 Calibrating a Prediction Model to Estimate Minor Roads AADT

The need of the traffic volume for each approach leading to an intersection, to calculate the total entering volume, was discussed in section 3.7. State traffic count maps, Road Secondary maps, KCARS, and survey forms were used to find the AADT of the roads, but the AADT of 35% minor roads remained unknown after using all those resources. According to many studies, one of the feasible method to estimate the AADT is calibrating a prediction model to estimate the AADT (Pan, 2008). AADT is one of the most important traffic variables needed for analysis of traffic crash rates and is widely used in almost all transportation fields (Pan, 2008). AADT prediction models are classified into two major types (Pan, 2008):

- Time series models
- Linear regression models.

Based on available historical AADT data, time series estimates AADT growth; however, AADT values on such roads can be estimated using multiple linear regression models or other transportation demand estimating models (Pan, 2008).

In order to calibrate a linear regression model data collection is conducted to cover most possible factors that have impact on AADT. An efforts were made to compile and process these data. Two different types of data were collected from different sources, including social-economic data and intersection characteristics data.

Most intersections characteristics data used in this study were the different types of the intersection, whether there are the by-pass lanes at intersections or not. Then, intersections were categorized in several groups in term of number of legs, whether 3-legged or 4-legged intersection. Additionally, there is a hierarchical highway system in Kansas. There are interstate roads, US roads, Kansas roads, RS roads and local roads in Kansas. So, based on the rank of the approaching lanes, each intersection was belong to different category. All these categories are listed from X_1 to X_{12} in Table 3.5.

Table 3.5 Input variables to calibrate a AADT prediction model

x_1	Intersection with the bypass= 1, intersection without the bypass = 0	x_{15}	Total road miles within the county
x_2	4-legged intersections = 1, 3-legged intersections = 0	x_{16}	Per capital personal, who lives in the county, income in a year
x_3	If minor road crosses minor roads = 1, Otherwise =0	x_{17}	Median age of residence in the county
x_4	If US highway crosses US highway = 1, Otherwise =0	x_{18}	Number of households in the county
x_5	If US highway crosses K highway = 1, Otherwise =0	x_{19}	Number of people per household
x_6	If K highway crosses K highway = 1, Otherwise =0	x_{20}	Labor force
x_7	If US highway crosses RS road = 1, Otherwise =0	x_{21}	Number of employed within the county
x_8	If K highway crosses RS road = 1, Otherwise =0	x_{22}	Number of unemployed within the county
x_9	If RS road crosses RS road = 1, Otherwise =0	x_{23}	Area of the county in square mile
x_{10}	If US highway crosses minor road = 1, Otherwise =0	x_{24}	Urban proportion in percent
x_{11}	If K highway crosses minor road = 1, Otherwise =0	x_{25}	Rural proportion in percent
x_{12}	If RS road crosses minor road = 1, Otherwise = 0	x_{26}	Urban area in square mile
x_{13}	County population	x_{27}	Rural area in square mile
x_{14}	Number of registered cars within the county	Y	Total Entering Volume at intersection

Social economic data, for all 105 counties in Kansas were collected from Kansas Statistical Abstract 2012. Several categories of social economic data, including population, number of registered cars, the income of the person, median age of residence, number of households, labor force, number of people per households, number of employed/unemployed, area, and urban/rural proportion were selected in the scale of county. All social economic variables are listed in Table 3.5 from X_{13} to X_{27}.

One of the methods to select the actual set of predictors in the final model is backward regression model. Backward regression removes nonsignificant variables from the regression model for the purpose of identifying a useful subset of the predictors. In this method, the initial model starts with all variables. At each step, the variable that is least significant is removed. In other words, those variables with p-values greater than the significance level (α) will be removed. This process continues until no nonsignificant variable remains.

Chapter 4 - Results

This chapter documents a comparative crash analysis of unsignalized rural intersections in Kansas. In order to evaluate safety effectiveness of adding by-pass lanes, two approaches were utilized:

- Before-and-after study
- Cross-sectional study

In addition, CMF was estimated to evaluate safety effectiveness of adding by-pass lanes. A comparison crash analysis was conducted to determine basic crash characteristics for two categories of intersections:

- 3-legged intersections
- 4-legged intersections

Moreover, the results of video recording that shows the drivers maneuvers and the delay caused by absence of by-pass lanes at intersection are shown in following section.

4.1 Video Recording

Videos were taken during morning peak hours (8:00-10:00 a.m.) and evening peak hours (4:00-6:00 p.m.) in order to capture maximum traffic flow and increased use of by-pass lanes. However, AADT of the selected roads were greater than 1000 vpd; few circumstances, a car reached the intersection when another car was waiting to turn left. Figure 4.1 shows a following driver who utilized the by-pass lane when the lead car decreased speed to turn left at the intersection.

Figure 4.1 Use of by-pass lane

Table 4.1 Results of video capturing

Intersection types	K13 -Tuttle Cove (3-legged intersection with bypass lane)	Tuttle Cove Rd - Freeman (3-legged intersection without bypass lane)	US 24 – Falcon (4-legged intersection with bypass lane)
Travel time (in second)			
Drivers go straight when there is no car ahead	9.3	3.0	16
Drivers used bypass lane	9	-	-
Drivers did not use bypass lane	11.5	4.72	17.1
Distance (ft)	480	180	920
No. of drivers who did not use by-pass lane	7	5	5
No. of drivers who used by-pass lane	7	-	-

According to Table 4.1, drivers at 3-legged intersection with by-pass lane at K-13 and Tuttle Cove road needed 9.3 s to pass 480ft along K-13. During video capturing, seven drivers used by-pass lane to pass a stopped car ahead and seven drivers did not use the by-pass lane. The average time to pass the fixed distance was 9 and 11.5 s, respectively. Therefore, absence of the by-pass lane caused a 2.2 seconds delay. Delay times at Tuttle Cove road and Freeman road and Main road-Falcon road were 1.7 and 1.1 seconds, respectively. Video showed that use of the by-pass lane rarely occurred.

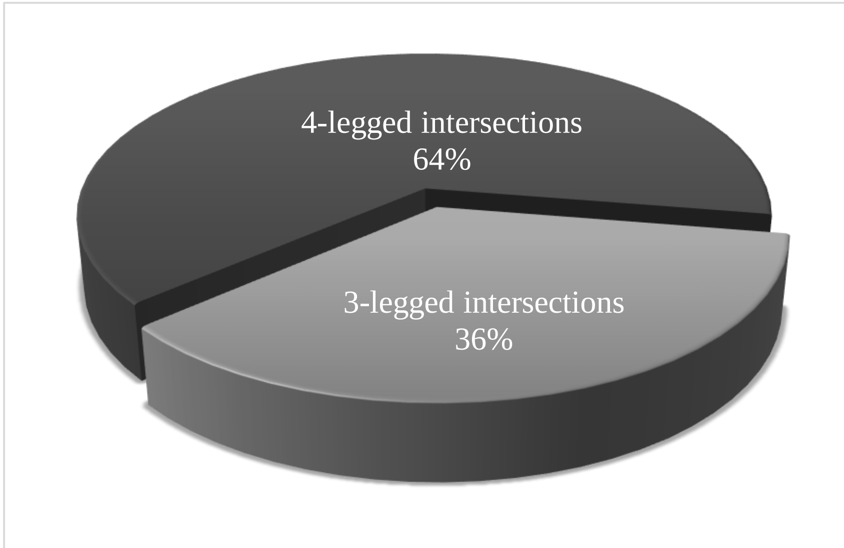
4.2 Before –and- After Study

A before versus after crash analysis was conducted to evaluate safety effectiveness of adding by-pass lanes. The HSM recommends a period of three to five years be utilized (AASHTO, 2010). Shorter periods than three years are subject to high variability due to the randomness of crashes. The periods longer than five years are more subject to introduction of bias due changes in reporting standards or the physical changes to the roadway features. Crash data for before-and-after study were extracted from KCARS from 1990-2011.

4.2.1 Five Years Consideration

This section documents data during five years before construction of the by-pass lane and five years after by-pass construction (not including the year in which by-pass lanes were constructed). Crash data was collected for a total of 61 intersections (22 3-legged intersections and 39 4-legged intersections) where by-pass lanes had been constructed between1990-2011. Figure 4.2 shows the proportion of intersection types during five years consideration.

Figure 4.2 Proportion of intersection types during five years consideration



4.2.1.1 Comparison of Crash Frequency

Within the consideration of the 300 feet intersection box, a total of 20 crashes or 0.328 crashes per intersection, occurred before adding the by-pass lanes and 13 crashes or 0.213 crashes per intersection occurred after adding by-pass lanes. When considering intersection related crashes, a total of 21 crashes or 0.344 crashes per intersection, occurred before adding by-pass lanes and 18 crashes or 0.295 crashes per intersection, occurred after adding by-pass lanes. The paired t-test under 95% confidence level was conducted on a number of crashes at each intersection. Table 4.2 shows the statistical analysis of the crash frequency when considering a five years period before and after by-pass lanes installation.

Table 4.2 Statistical analysis of reduction in crash frequency within five years range

Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean crash frequency (before)	0.500	0.636	0.231	0.179
Mean crash frequency (after)	0.409	0.318	0.103	0.282
Mean crash frequency difference	0.091	0.318	0.128	-0.103
t-value	0.460	1.670	1.300	-0.750
p-value	0.324	0.055	0.100	0.772

The positive values of the mean difference show a reduction of crash frequency after adding by-pass lanes. In contrast, the negative value of mean difference shows an increase in crash frequency. Furthermore, the *p*-values are greater than 0.05, which indicates that there is no statistically significant difference under 95% confidence level in crash frequency, after adding by-pass lanes. Adding by-pass lanes at 3–legged intersections caused higher safety improvement. This is supported by the p-value which is close to 0.05.

4.2.1.2 Comparison of EPDO Crash Frequency

Within the consideration 300 feet intersection box, the total EPDO crash frequency was equal to 146 or 2.393 per intersection, before adding the by-pass lanes and the EPDO crash frequency after adding by-pass lanes was 55 or 0.902 per intersection. When considering intersection related crashes, the total EPDO crash frequency after adding by-pass lanes and before construction were 105 and 130 respectively or 1.721 and 2.131 per intersection. The paired t-test under 95% confidence level was conducted on the number of EPDO crash frequency at each intersection. Table 4.3 shows the statistical analysis on EPDO crash frequency when considering a five years period before and after by-pass lanes installation.

Table 4.3 Statistical analysis of reduction in EPDO crash frequency within five years range

Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean EPDO crash frequency (before)	3.680	3.18	1.667	0.897
Mean EPDO crash frequency (after)	1.680	2.86	0.462	1.718
Mean difference in EPDO crash frequency	2.000	0.320	1.205	-0.821
t-value	0.890	0.150	1.380	-1.04
p-value	0.192	0.442	0.088	0.847

The positive value of the mean difference shows a reduction of EPDO crashes after adding by-pass lanes. In contrast, the negative value of mean difference shows an increase in EPDO crashes. Furthermore, the p-values are greater than 0.05, which indicates that there is no statistically significant difference under 95% confidence level in EPDO crash frequency, after adding by-pass lanes.

4.2.1.3 Comparison in Crash Rates

Crash rates are a factor to identify the road safety. They enable the comparison between the relatively safety at intersections. Crash rates are calculated for rural intersection in terms of crashes per Million Entering Vehicle (MEV) (Green & Agent, 2003).

$$Crash\,rate = \frac{Average\ no.of\ Crashes\ per\ year \times 10^6}{\sum AADT \times 365} \tag{4.1}$$

Within the consideration 300 feet intersection box, the total crash rate per million entering vehicle was 3.69 or 0.061 per intersection, before adding the by-pass lanes and 1.79 or 0.294 per intersection, after adding the by-pass lanes. When considering intersection related crashes the crash rates after adding by-pass lanes and before that were 3.82, and 3.7 respectively or 0.0626 and 0.061 per intersection. The paired t-test under 95% confidence level was conducted on the crash rates at each intersection. Table 4.4 shows the statistical analysis on crash rates when considering five years period before and after by-pass lanes installation.

Table 4.4 Statistical analysis of reduction in crash rates within five years range

Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean crash rates (before)	0.060	0.079	0.061	0.055
Mean crash rates (after)	0.044	0.046	0.021	0.069
Mean difference in crash rates	0.016	0.033	0.040	-0.016
t-value	0.650	1.870	1.380	-0.460
p-value	0.262	0.038	0.087	0.675

The positive values of the mean difference show a reduction of crash rates after adding a by-pass lanes. In contrast, the negative value of mean difference shows an increase in crash frequency. Furthermore, the p-values are greater than 0.05, which indicates that there is no statistically difference under 95% confidence level in crash rates, after adding by-pass lanes. However, when considering intersection related crashes, a significant reduction is happened at 3-legged intersection.

4.2.1.4 Comparison in EPDO crash Rates

EPDO crash rates shows the rate of the severity of the crashes to traffic volumes. EPDO crash rates are calculated for rural intersection in terms of crashes per Million Entering Vehicle (MEV).

$$EPDO\ rate = \frac{Average\ EPDO\ per\ year \times 10^6}{\sum AADT \times 365} \tag{4.2}$$

Within the consideration 300 feet intersection box, the total EPDO crash rate per million entering vehicle was 29.108 or 0.477 per intersection, before adding the by-pass lanes and the total EPDO crash rate was 8.455 or 0.139 per intersection, after adding by-pass lanes. When considering intersection related crashes the EPDO crash rates before and after adding the by-pass lanes were 24.848, and 34.136 respectively or 0.407 and 0.56 per intersection. The paired t-test under 95% confidence level was conducted on the EPDO crash rates at each intersection. Table 4.5 shows the statistical analysis on EPDO crash rates when considering five years period before and after by-pass lanes installation.

Table 4.5 Statistical analysis of reduction in EPDO crash rates within five years range

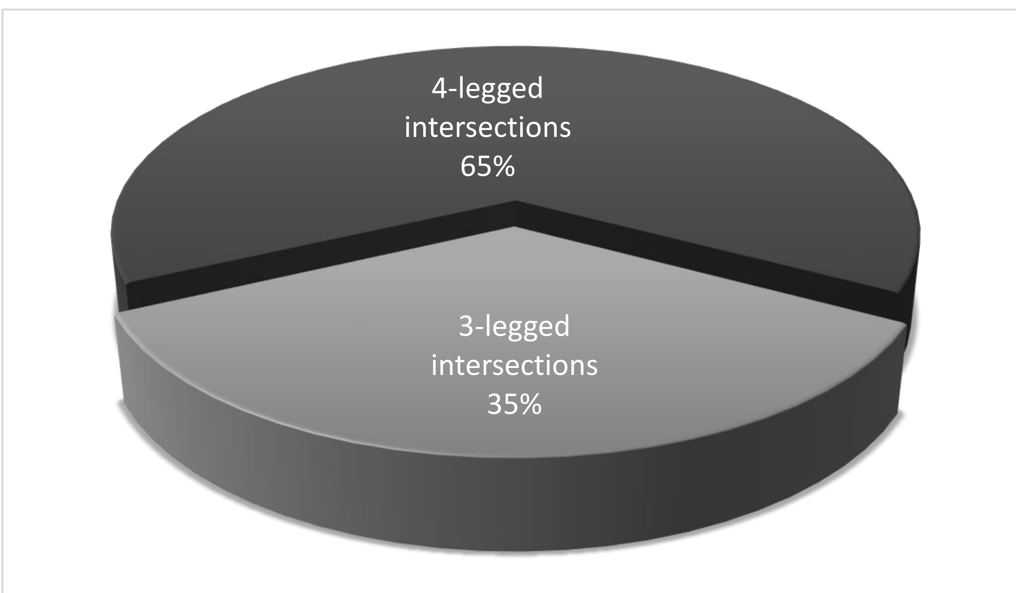
Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean EPDO crash rates (before)	0.407	0.451	0.517	0.383
Mean EPDO crash rates (after)	0.224	0.561	0.09	0.559
Mean difference in EPDO crash rates	0.182	-0.11	0.040	-0.176
t-value	0.96	-0.3	1.48	-0.85
p-value	0.174	0.617	0.074	0.799

The positive values of the mean difference show a reduction of EPDO crash rates after adding a by-pass lanes. In contrast, the negative value of mean difference shows an increase in EPDO crash rates. Furthermore, the p-values are greater than 0.05, which indicates that there is no statistically significant difference under 95% confidence level in EPDO crash rates, before and after adding by-pass lanes.

4.2.2 Four Years Consideration

This section documents data during four years before the construction of the by-pass lanes and four years after by-pass lanes construction (not including the year by-pass lanes were constructed). Crash data was collected for a total of 68 intersections (24 3-legged intersections and 44 4-legged intersections) where by-pass lanes were constructed between 1990 -2011. Figure 4.3 shows the proportion of intersection types during four years consideration

Figure 4.3 Proportion of intersection types during four years consideration



4.2.2.1 Comparison of Crash Frequency

Within the consideration of the 300 feet intersection box, a total of 20 crashes or 0.294 crashes per intersection, occurred before adding the by-pass lanes and 15 crashes or 0.221 crashes per intersection occurred after adding by-pass lanes. When considering intersection related crashes, a total of 26 crashes or 0.382 crashes per intersection, occurred before adding by-pass lanes and 18 crashes or 0.265 crashes per intersection, occurred after adding by-pass lanes. The paired t-test under 95% confidence level was conducted on a number of crashes at each intersection. Table 4.6 shows the statistical analysis of the crash frequency when considering a four years period before and after by-pass lanes installation.

Table 4.6 Statistical analysis of reduction in crash frequency within four years range

Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean crash frequency (before)	0.417	0.500	0.227	0.318
Mean crash frequency (after)	0.375	0.250	0.136	0.273
Mean crash frequency difference	0.042	0.250	0.091	0.045
t-value	0.200	1.370	0.810	0.360
p-value	0.420	0.093	0.210	0.360

The positive values of the difference show the reduction in crash frequency after adding by-pass lanes. Due to the *p*-values are greater than the 0.05, the reductions are not statistically significant under 95% confidence level.

4.2.2.2 Comparison in EPDO Crash Frequency

Within the consideration 300 feet intersection box, the total EPDO crash frequency was equal to 174 or 2.559 per intersection, before adding the by-pass lanes and the EPDO crash frequency after adding by-pass lanes was 71 or 1.044 per intersection. When considering intersection related crashes, the total EPDO crash frequency after adding by-pass lanes and before construction were 180 and 130 respectively or 2.647 and 1.912 per intersection. The paired t-test under 95% confidence level was conducted on EPDO crash frequency at each intersection. Table 4.7 shows the statistical analysis on EPDO crash frequency when considering a four years period before and after by-pass lanes installation.

Table 4.7 Statistical analysis of reduction in EPDO crash frequency within four years range

Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean EPDO crash frequency (before)	3.330	3.420	2.136	2.230
Mean EPDO crash frequency (after)	1.540	2.000	0.773	1.860
Mean difference in EPDO crash frequency	1.790	1.420	1.360	0.360
t-value	0.870	0.710	1.240	0.350
p-value	0.196	0.242	0.111	0.366

The positive values of the difference show the reduction in EPDO crash frequency after adding by-pass lanes. Whereas, the *p*-values are greater than the 0.05; which indicates that there is no statistically difference under 95% confidence level in EPDO crash frequency, after adding by-pass lanes.

4.2.2.3 Comparison in Crash Rates

Within the consideration 300 feet intersection box, the total crash rate per million entering vehicle was 4.712 or 0.069 per intersection, before adding the by-pass lanes and 3.029 or 0.101 per intersection, after adding the by-pass lanes. When considering intersection related crashes the crash rates after adding by-pass lanes and before that were 6.895 and 4.809 respectively or 0.045 and 0.071 per intersection. The paired t-test under 95% confidence level was conducted on the crash rates at each intersection. Table 4.8 shows the statistical analysis on crash rates when considering four years period before and after by-pass lanes installation.

Table 4.8 Statistical analysis of reduction in crash rates within four years range

Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersectio n related
Mean crash rates (before)	0.056	0.084	0.076	0.111
Mean crash rates (after)	0.051	0.040	0.041	0.087
Mean difference in crash rates	0.005	0.044	0.035	0.24
t-value	0.180	1.55	0.860	0.590
p-value	0.429	0.067	0.198	0.281

The positive values of the difference shows a reduction in crash rate after adding by-pass lanes. Whereas, the p-values are greater than the 0.05, which indicates that there is no statistically significant difference under 95% confidence level in crash rate, after adding by-pass lanes.

4.2.2.4 Comparison in EPDO Crash Rates

Within the consideration 300 feet intersection box, the total EPDO crash rate per million entering vehicle was 37.845 or 0.557 per intersection, before adding the by-pass lanes and the total

EPDO crash rate was 14.772 or 0.217 per intersection, after adding by-pass lanes. When considering intersection related crashes the EPDO crash rates before and after adding the by-pass lanes were 54.439, and 35.833 respectively or 0.801 and 0.527 per intersection. The paired t-test under 95% confidence level was conducted on the EPDO crash rates at each intersection. Table 4.9 shows the statistical analysis on EPDO crash rates when considering four years period before and after by-pass lanes installation.

Table 4.9 Statistical analysis of reduction in EPDO crash rates within four years range

Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean EPDO crash rates (before)	0.36	0.68	0.664	0.866
Mean EPDO crash rates (after)	0.206	0.335	0.224	0.632
Mean difference in EPDO crash rates	0.154	0.346	0.44	0.234
t-value	0.91	0.84	1.21	0.68
p-value	0.187	0.206	0.117	0.251

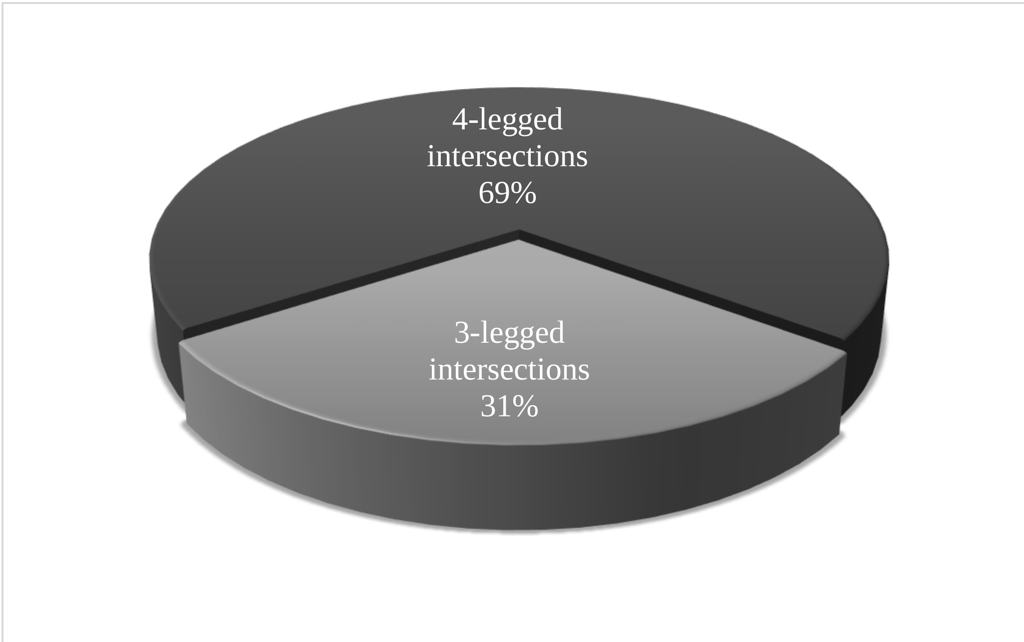
The positive value in mean difference show a reduction in EPDO crash rates after adding by-pass lanes. However, due to the *p*-values are greater than 0.05, the reductions are not statistically significant under 95 % confidence level.

4.2.3 Three Years Consideration

This section documents data during three years before construction of the by-pass lanes and three years after by-pass lanes construction (not including the year that the by-pass lanes were constructed). Crash data was collected for a total of 88 intersections (27 3-legged intersections and

61 4-legged intersections), where by-pass lanes were constructed between the years 1990-2011. Figure 4.4 shows the proportion of intersection types during three years consideration.

Figure 4.4 Proportion of intersection types during three years consideration



4.2.3.1 Comparison of Crash Frequency

Within the consideration of the 300 feet intersection box, a total of 16 crashes or 0.182 crashes per intersection, occurred before adding the by-pass lanes and 14 crashes or 0.159 crashes per intersection, occurred after adding by-pass lanes. When considering intersection related crashes, a total of 22 crashes or 0.25 crashes per intersection, occurred before adding by-pass lanes and 13 crashes or 0.148 crashes per intersection, occurred after adding by-pass lanes. The paired t-test under 95% confidence level was conducted on a number of crashes at each intersection. Table 4.10 shows the statistical analysis of the crash frequency when considering a three years period before and after by-pass lanes installation.

Table 4.10 Statistical analysis of reduction in crash frequency within three years range

Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean crash frequency (before)	0.259	0.370	0.148	0.197
Mean crash frequency (after)	0.259	0.111	0.115	0.164
Mean crash frequency difference	0.000	0.259	0.033	0.033
t-value	0.000	1.370	0.420	0.390
p-value	0.500	0.091	0.337	0.349

The positive values of the difference clearly show the reduction in crash frequency after adding by-pass lanes. Whereas, the *p*-values are greater than the 0.05, which indicates that there is no statistically significant difference under 95% confidence level in crash frequency, after adding by-pass lanes.

4.2.3.2 Comparison of EPDO Crash Frequency

Within the consideration 300 feet intersection box, the total EPDO crash frequency was equal to 142 or 1.614 per intersection, before adding the by-pass lanes and the EPDO crash frequency after adding by-pass lanes was 70 or 0.795 per intersection. When considering intersection related crashes, the total EPDO crash frequency after adding by-pass lanes and before construction were 162 and 111 respectively or 1.841 and 1.261 per intersection. The paired t-test under 95% confidence level was conducted on EPDO crash frequency at each intersection. Table 4.11 shows the statistical analysis on EPDO crash frequency when considering a three years period before and after by-pass lanes installation.

Table 4.11 Statistical analysis of reduction in EPDO crash frequency within 3 years range

Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean EPDO crash frequency (before)	0.850	2.960	1.066	1.344
Mean EPDO crash frequency (after)	1.300	0.150	0.574	1.311
Mean difference in EPDO crash frequency	1.550	0.810	0.492	0.033
t-value	0.850	1.060	0.680	0.050
p-value	0.201	0.150	0.249	0.482

The positive values of the difference shows a reduction in EPDO crashes after adding by-pass lanes. Whereas, the *p*-values are greater than the 0.05; which indicates that there is no statistically significant difference under 95% confidence level in EPDO Statistical parameters, after adding by-pass lanes.

4.2.3.3 Comparison of Crash Rates

Within the consideration 300 feet intersection box, the total crash rate per million entering vehicle was 5.162 or 0.059 per intersection, before adding the by-pass lanes and 3.889 or 0.044 per intersection, after adding the by-pass lanes. When considering intersection related crashes the crash rates after adding by-pass lanes and before that were 7.958 and 4.625 respectively or 0.09 and 0.053 per intersection. The paired t-test under 95% confidence level was conducted on the crash rates at each intersection. Table 4.12 shows the statistical analysis on crash rates when considering three years period before and after by-pass lanes installation.

Table 4.12 Statistical analysis of reduction in crash rates within 3 years range

Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean crash rates (before)	0.043	0.072	0.066	0.099
Mean crash rates (after)	0.049	0.026	0.042	0.064
Mean difference in crash rates	-0.006	0.045	0.023	0.035
t-value	-0.150	1.170	0.600	0.920
p-value	0.559	0.127	0.275	0.181

The positive values of the mean difference show a reduction of crash rates after adding a by-pass lanes. In contrast, the negative value of mean difference shows an increase in crash rates. Furthermore, the p-values are greater than 0.05, which indicates that there is no statistically difference under 95% confidence level in crash rates, after adding by-pass lanes.

4.2.3.4 Comparison in EPDO Crash Rates

Within the consideration 300 feet intersection box, the total EPDO crash rate per million entering vehicle was 5.162 or 0.059 per intersection, before adding the by-pass lanes and the total EPDO crash rate was 3.889 or 0.044 per intersection, after adding by-pass lanes. When considering intersection related crashes the EPDO crash rates before and after adding the by-pass lanes were 7.958, and 4.625 respectively or 0.09 and 0.053 per intersection. The paired t-test under 95% confidence level was conducted on the EPDO crash rates at each intersection. Table 4.13 shows the statistical analysis on EPDO crash rates when considering three years period before and after by-pass lanes installation.

Table 4.13 Statistical analysis of reduction in EPDO crash rates within 3 years range

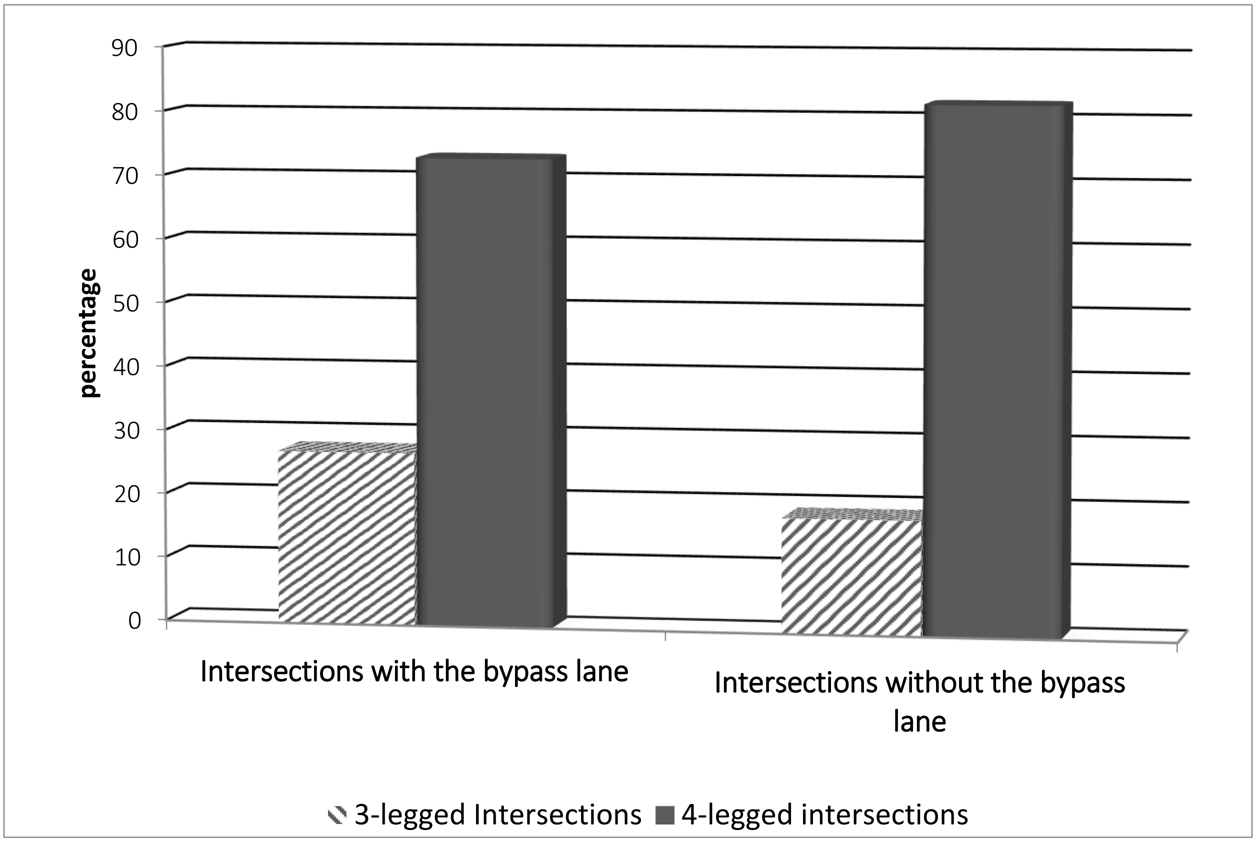
Statistical parameters	3-legged Intersections		4-legged Intersections	
	Crash selection criteria		Crash selection criteria	
	300feet	Intersection related	300feet	Intersection related
Mean EPDO crash rates (before)	0.403	0.779	0.483	0.736
Mean EPDO crash rates (after)	0.232	0.283	0.218	0.588
Mean difference in EPDO crash rates	0.171	0.496	0.265	0.149
t-value	0.84	1.02	0.79	0.46
p-value	0.204	0.159	0.216	0.325

The positive value in the mean difference shows a reduction in EPDO crash rate after adding by-pass lanes. Due to the p-values are greater than the 0.05, the reductions are not statistically significant under 95 percent confidence level.

4.3 Cross-Sectional Study

Analysis was conducted to determine safety effectiveness of by-pass lanes by comparing crash statistics at intersections with by-pass lanes and intersections with no by-pass lanes and no left-turn lane. Intersections with the by-pass lanes were derived from the received survey forms. Due to lack of information provided by area engineers, 558 intersections were taken account in analysis. In opposition, 579 intersections without the by-pass lanes were selected. The second group of intersections were located in proximity of intersections with the by-pass lanes to have similar traffic volume and driver behaviors. Figure 4.5 shows the proportion of 3-legged and 4-legged intersections in two sample.

Figure 4.5 Proportion of intersection types in cross-sectional study



Crash data were derived from KCARS from 2009 - 2011, then a two sample t-test was conducted to evaluate significance of reductions in the number of crashes, EPDO crashes, crash rates, and EPDO crash rates. A comparison crash analysis was conducted to determine basic crash characteristics for two different categories of intersections:

- 3-legged intersections
- 4-legged intersections

Each of these categories was subdivided into intersections with the by-pass lanes and without the by-pass lanes.

4.3.1 Comparison of Crash Frequency

A two samples t-test under 95% confidence level was conducted on a crash frequency at each intersection. Table 4.14 shows statistical analysis of crash frequency reduction within 200 ft, 300 ft. along each approach leading to the intersections, and intersection related crashes as well.

Table 4.14 Comparison of crash frequency

Statistical parameters	3-legged Intersections			4-legged Intersections		
	Crash selection criteria			Crash selection criteria		
	200ft	300ft	Intersection related	200ft	300ft	Intersection related
Mean crash frequency (before)	0.319	0.67	0.521	0.65	0.87	0.503
Mean crash frequency (after)	0.460	0.493	0.42	0.397	0.463	0.51
Mean crash frequency difference	-0.141	0.177	0.101	0.253	0.407	- 0.007
t-value	- 1.2	1.3	0.82	4.11	5.71	-0.13
p-value	0.885	0.098	0.207	0.001	0.001	0.55

Although the crash frequency is increased within 200 feet at 3-legged intersection with the by-pass lanes, but the positive values of the mean difference show a reduction of crash frequency within 300 feet along each approaches leading to 3-legged intersection and intersection related crashes. However, according to p-values greater than 0.05, none of the changes are significant. According to *p*-values less than 0.05 at 4-legged intersections, reduction in the number of crashes at intersections with by-pass lanes are significant. In contrast, when considering intersection related crashes, a nonsignificant growth of crash frequency is happened at 4-legged intersections with by-pass lanes versus 4-legged intersections without by-pass lanes.

4.3.2 Comparison of EPDO Crash Frequency

A two samples t-test under 95% confidence level was conducted on EPDO crash frequency at each intersection. Table 4.15 shows statistical analysis of EPDO crashes reduction within 200 ft., 300 ft. along each approach leading to the intersections, and intersection related crashes as well.

Table 4.15 Comparison of EPDO crash frequency

Statistical parameters	3-legged Intersections			4-legged Intersections		
	Crash selection criteria			Crash selection criteria		
	200ft	300ft	Intersection related	200ft	300ft	Intersection related
Mean EPDO crash frequency (before)	0.91	2.16	3.35	2.72	3.87	3.71
Mean EPDO crash frequency (after)	1.67	1.89	3.03	2.25	2.45	4.0
Mean difference in EPDO crash frequency	-0.758	0.266	0.318	0.474	1.423	-0.305
t-value	-1.47	0.37	0.33	1.08	2.85	-0.43
p-value	0.929	0.358	0.372	0.139	0.002	0.667

Although the EPDO crash frequency is increase within 200 feet at 3-legged intersection with the by-pass lanes, but the positive values of the mean difference show a reduction of EPDO crash frequency within 300 feet along each approaches leading to 3-legged intersections and intersection related crashes. However, according to p-values greater than 0.05, none of changes are significant. When considering 300 feet along each approach leading to 4-legged intersections, the *p*-values less than 0.05 shows a significant reduction in the EPDO crash frequency at intersections with by-pass lanes. However, this reduction is not significant within 200 feet. In contrast, when considering intersection related crashes, a nonsignificant growth of EPDO crash frequency is happened at 4-legged intersections with by-pass lanes versus 4-legged intersections without by-pass lanes.

4.3.3 Comparison in Crash Rates

As mentioned, for 35% of intersection the actual AADT of the minor roads are unknown. A two samples t-test under 95% confidence level was conducted on crash rates at each intersection which their total entering volumes are identified. Table 4.16 shows statistical analysis of crash rates reduction within 200 ft., 300 ft. along each approach leading to the intersections, and intersection related crashes as well.

Table 4.16 Comparison of crash rates

Statistical parameters	3-legged Intersections			4-legged Intersections		
	Crash selection criteria			Crash selection criteria		
	200ft	300ft	Intersection related	200ft	300ft	Intersection related
Mean crash rates (before)	0.231	0.276	0.188	0.249	0.310	0.123
Mean crash rates (after)	0.187	0.194	0.131	0.133	0.157	0.153
Mean difference in crash rates	0.044	0.082	0.056	0.116	0.153	-0.03
t-value	0.58	1.04	0.78	3.85	4.78	-1.12
p-value	0.282	0.151	0.218	0.001	0.001	0.869

The positive values of the mean difference show a reduction of crash rates within 200, feet, 300 feet along each approach leading to 3-legged intersection and intersection related crashes.

However, according to p-values greater than 0.05, none of reductions are significant. According to *p*-values less than 0.05, reduction of crash rates along 200 feet and 300 feet along each approach leading to 4-legged intersection with a by-pass lanes are significant. In contrast, when considering intersection related crashes, a nonsignificant growth of crash rates is happened at 4-legged intersections with by-pass lanes versus 4-legged intersections without by-pass lanes.

4.3.4 Comparison of EPDO Crash Rates

Similar to crash rates analysis, a two samples t-test under 95% confidence level was conducted on EPDO crash rates at each intersection which their total entering volumes are identified. Table 4.17 shows statistical analysis of EPDO crash rates reduction within 200 ft., 300 ft. along each approach leading to the intersections, and intersection related crashes as well.

Table 4.17 Comparison in EPDO crash rates

Statistical parameters	3-legged Intersections			4-legged Intersections		
	Crash selection criteria			Crash selection criteria		
	200ft	300ft	Intersection related	200ft	300ft	Intersection related
Mean EPDO crash rates (before)	0.54	0.84	0.131	0.83	1.09	0.75
Mean EPDO crash rates (after)	0.89	0.93	0.147	0.71	0.77	0.99
Mean difference in EPDO crash rates	-0.344	-0.097	-0.016	0.117	0.32	-0.242
t-value	-1.06	-0.25	-0.66	0.67	1.69	-1.29
p-value	0.854	0.6	0.744	0.253	0.046	0.901

The negative values of the mean difference show a growth of EPDO crash rates within 200, feet, 300 feet along each approach leading to 3-legged intersections and intersection related crashes. However, according to p-values greater than 0.05, none of them are significant. When considering 300 feet along each approach leading to 4-legged intersections, the *p*-values less than 0.05 shows a significant reduction of the EPDO crash rates at 4-legged intersections with by-pass lanes. However, this reduction is not significant within 200 feet. In contrast, when considering intersection related crashes, a nonsignificant growth of EPDO crash rates is happened at 4-legged intersections with by-pass lanes versus 4-legged intersections without by-pass lanes.

4.3.5 Model Calibration

In order to estimate the total entering volume for 35% of intersection, a predictor model should be calibrated. So, two different types of data were collected from different sources, including social-economic data and intersection characteristics data which were listed in table 3.5. Before using all input data, the test of independency applied on input data to find the independent ones. According to that, four variable were dropped, so following parameters were set to zero in the final model. The dependent variables were a linear combination of other variables, as shown in Table 4.18.

Table 4.18 Results of dependency test

X_{12} =	Intercept - X_3 - X_4 - X_5 - X_6 - X_7 - X_8 - X_9 - X_{10} - X_{11} - 531E-19 × X_{14}
X_{22} =	217E-14 × X_{13} + 534E-15 × X_{14} + X_{20} - X_{21}
X_{25} =	100 × Intercept - 685E-17 × X_{13} + 38E-16 × X_{14} - 699E-14 × X_{17} + 539E-16 × X_{20} - 508E-16 × X_{21} - X_{24}
X_{27} =	172E-15 × X_{13} - 192E-14 × X_{20} + 17E-13 × X_{21} + X_{23} - X_{26}

After dropping the dependent variable, variables which have a significant effect on the output should be found by using the test of p-value. Results are shown in Table 3.8. According to the confidence level, which is 95%, variables with related p-value greater than 0.05 were dropped. A total of 693 intersections with known roads AADT provided to estimate the total entering volume of intersections. Initially an attempt was made to estimate the actual Total Entering Volume (TEV), which is sum of AADTs of the roads, but the test of normality showed that residuals distribution did not follow normal distribution. However, when an estimation of $\log_{10} TEV$ was attempted, residuals followed normal distribution. Therefore, $\log_{10} TEV$ was estimated instead of actual TEV value.

Table 4.19 Test of p-values of the inputs

Variable	p-value	Variable	p-value	Variable	p-value
X_1	<.0001	X_{10}	<.0001	X_{19}	0.161
X_2	0.0107	X_{11}	<.0001	X_{20}	<.0001
X_3	<.0001	X_{12}	dropped	X_{21}	<.0001
X_4	<.0001	X_{13}	<.0001	X_{22}	dropped
X_5	<.0001	X_{14}	0.0071	X_{23}	0.0001
X_6	<.0001	X_{15}	<.0001	X_{24}	0.0377
X_7	<.0001	X_{16}	0.0485	X_{25}	dropped
X_8	<.0001	X_{17}	<.0001	X_{26}	0.291
X_9	<.0001	X_{18}	0.4466	X_{27}	dropped

Finally, regression results of AADT prediction model are given in Equation 4.3. The R-square value of the model was 0.69.

$$\log_{10} TEV = 3.768 + 0.095x_1 + 0.062x_2 + 0.832x_3 + 0.788x_4 + 0.63x_5 + 0.442x_6 + 0.6x_7 + 0.38\,x_8 - 0.339x_9 + 0.64x_{10} + 0.432x_{11} - 4 \times 10^{-5}x_{13} + 9 \times 10^{-7}x_{14} + 1.6 \times 10^{-5}x_{15} - 4 \times 10^{-6}x_{16} - 0.022x_{17} + 3 \times 10^{-4}x_{20} - 3 \times 10^{-4}x_{21} - 3 \times 10^{-4}x_{23} + 0.028x_{24} \quad (4.3)$$

Where,

TEV= Total Entering Volume at intersection

4.3.6 Comparison of Estimated Crash Rates

After estimating the unknown total entering volume at 35% remaining intersections, a two samples t-test under 95% confidence level was conducted on crash rates at each intersection. Table 4.20 shows statistical analysis of estimated crash rates reduction within 200 ft., 300 ft. along each approach leading to the intersections, and intersection related crashes as well.

Table 4.20 Comparison of estimated crash rates

Statistical parameters	3-legged Intersections			4-legged Intersections		
	Crash selection criteria			Crash selection criteria		
	200ft	300ft	Intersection related	200ft	300ft	Intersection related
Mean crash rates (before)	0.129	0.276	0.188	0.233	0.299	0.138
Mean crash rates (after)	0.174	0.194	0.131	0.140	0.151	0.147
Mean difference in crash rates	-0.045	0.0821	0.057	0.093	0.148	-0.009
t-value	-0.92	1.04	0.78	3.67	5.02	-0.36
p-value	0.821	0.151	0.218	0.001	0.001	0.639

Although the crash rates is increased within 200 feet at 3-legged intersection with the by-pass lanes, but the positive values of the mean difference show a reduction of crash rates within 300 feet along each approaches leading to 3-legged intersection and intersection related crashes. However, according to p-values greater than 0.05, none of the changes are significant. When considering an intersection crash box, according to *p*-values less than 0.05 at 4-legged intersections, reduction in the crash rates at intersections with a by-pass lanes are significant. In contrast, when considering intersection related crashes, a nonsignificant growth of crash rates is happened at 4-legged intersections with by-pass lanes versus 4-legged intersections without by-pass lanes.

4.3.7 Comparison of Estimated EPDO Crash Rates

Similar to statistical analysis of estimated crash rates, a two samples t-test under 95% confidence level was conducted on EPDO crash rates at each intersection. Table 4.21 shows statistical analysis of estimated EPDO crash rates reduction within 200 ft., 300 ft. along each approach leading to the intersections, and intersection related crashes as well.

Table 4.21 Comparison of estimated EPDO crash rates

Statistical parameters	3-Legged Intersections			4-Legged Intersections		
	Crash selection criteria			Crash selection criteria		
	200ft	300ft	Intersection related	200ft	300ft	Intersection related
Mean EPDO crash rates (before)	0.281	0.58	0.88	0.8	1.08	0.84
Mean EPDO crash rates (after)	0.83	0.87	1.03	0.69	0.74	0.95
Mean difference in EPDO crash rates	-0.546	-0.289	-0.155	0.11	0.346	-0.114
t-value	-2.05	-0.99	-0.48	0.74	2.04	-0.67
p-value	0.979	0.839	0.684	0.228	0.021	0.749

The negative values of the mean difference and p-values greater than 0.05 show nonsignificant growth of EPDO crash rates within, 300 feet along each approach leading to 3-legged intersection and intersection related crashes. However, this growth is significant within 200 feet intersection crash box. When considering 300 feet along each approach leading to 4-legged intersections, the *p*-values less than 0.05 shows a significant reduction of the EPDO crash rates at intersections with by-pass lanes. However, this reduction is not significant within 200 feet. In contrast, when considering intersection related crashes, a nonsignificant growth of EPDO crash rates is happened at 4-legged intersections with by-pass lanes versus 4-legged intersections without by-pass lanes.

4.4 Crash Modification Factor

CMF is used to compute the expected number of crashes after implementing a given countermeasure at a specific site. A CMF greater than 1.0 indicates an expected increase in crashes, while a value less than 1.0 indicates an expected reduction in crashes after implementation of a given countermeasure. For example, a CMF of 0.9 indicates an expected safety benefit, specifically a 10% expected reduction in crashes. A CMF of 1.1 indicates an expected degradation in safety, specifically a 10% expected increase in crashes. Table 4.22 shows the results of case-control study to calculate the CMF.

Table 4.22 Case-control CMF ratio from 2009-2011

Risk Factors	Intersections types	Case		Control		CMF
		With by-pass lane	Without by-pass lane	With by-pass lane	Without by-pass lane	
		A	C	B	D	
Crashes within 300 feet from intersection	3-legged intersections	46	35	104	59	0.75
	4-legged intersections	123	225	285	260	0.50
Intersection related crashes	3-legged intersections	35	34	115	60	0.54
	4-legged intersections	112	157	296	328	0.79

According to case-control method, which was utilized for cross-sectional study, all calculated CMF values are less than one, so future crashes are expected to reduce. This reduction is associated with the addition of by-pass lanes at rural intersections.

CMF ratios were calculated based on before-and-after study, and results are shown in Table 4.23. The only CMF greater than one is for intersection related crashes at 4-legged intersections with a five years consideration before and after adding a by-pass lanes. However, even in this case, when the sample size increased to three and four years considration the CMF became less than one. Otherwise, all calculated CMF are less than one, so future crashes are expected to reduce after adding by-pass lanes at rural unsignalized intersections.

Table 4.23 Before-and-after CMF ratio

Categories			Treatment before	Treatment after	Comparison before	Comparison after	CMF
3 year Consideration	3-legged intersection	Crashes within 300ft	7	7	2	2	0.88
		Intersection related crashes	5	3	2	3	0.22
	4-legged intersection	Crashes within 300ft	9	7	11	9	0.83
		Intersection related crashes	12	10	24	19	0.96
4 year Consideration	3-legged intersection	Crashes within 300ft	10	9	3	6	0.36
		Intersection related crashes	7	6	1	3	0.18
	4-legged intersection	Crashes within 300ft	10	6	11	15	0.32
		Intersection related crashes	14	12	27	22	0.97
5 year Consideration	3-legged intersection	Crashes within 300ft	11	9	4	6	0.45
		Intersection related crashes	14	7	2	2	0.39
	4-legged intersection	Crashes within 300ft	9	4	9	10	0.25
		Intersection related crashes	7	11	25	31	1.18

Chapter 5 - Summary and Conclusions

The primary objective of this study was to present a statistically reliable conclusion for a comparison of operational and safety characteristics of rural unsignalized intersections with by-pass lanes to rural unsignalized intersections without by-pass lanes or turning lanes.

To measure the delay caused the lack of the by-pass lane, video capturing was performed at 10 different locations near to Manhattan. Videos were taken during morning peak hours (8:00-10:00 a.m.) and evening peak hours (4:00-6:00 p.m.) to capture maximum traffic flow and increased use of by-pass lanes. Due to low traffic volume, the need of by-pass lane was not that much, and few drivers utilized by-pass lanes. According to captured videos, lack of a by-pass lane at intersections caused 1.1 to 2.2 seconds delay when speed limits were 55 and 35 mph, respectively.

A before-and-after study was conducted within three, four, and five years before and after construction of by-pass lanes at unsignalized rural intersections, to evaluate the safety effectiveness of by-pass lanes. The summary of results are shown in table 5.1. When considering three and four years before-and-after study, by-pass lane construction reduces crash frequency, EPDO crash frequency, crash rates, and EPDO crash rates; but these reductions are not statistically significant under 95% confidence level. However, when considering 300 feet intersection box at 3-legged intersections, the crash rates are increased after adding by-pass lanes. But this growth is not statistically significant under 95% confidence level.

In five years before-and-after study, when considering 300 feet intersection crash box, a reduction are happened in crashes and their severities; but the reductions are not statistically significant under 95% confidence level. Considering intersection related crashes, the same results are happened in crash frequency and EPDO crash frequency at 3-legged intersections. However, not statistically significant growth in crashes and their severities are happened at 4-legged intersection. Although, at 3-legged intersection crash rates a statistically significant reduction is happened under 95% confidence level. The EPDO crash rates at 3-legged intersections is increased, but that is not statistically significant under 95% confidence level. The calculated CMFs less than one also demonstrate the expected reduction in crashes after adding by-pass lanes at unsignalized rural intersections.

Moreover, a cross-sectional study was performed on crash data which are extracted from KCARS from 2009-2011. The results of analysis are summarized in Table 5.2. A modest decrease in crash frequency, EPDO crash frequency, and crash rates occurred at 3-legged intersections with the by-pass lanes, but these reduction are not statistically significant under 95% confidence level. The EPDO crash rates at 3-legged intersections are increased, but that is not statistically significant under 95% confidence interval.

Table 5.1 Summary of before-and-after study results

Parameters	Crash types	5 years consideration				4 years consideration				3 years consideration			
		3-legged intersection		4-legged intersection		3-legged intersection		4-legged intersection		3-legged intersection		4-legged intersection	
		Reduction	Significant	Reduction	Significant	Reduction	Significant	Reduction	Significant	Reduction	Significant	Reduction	Significant
Crash frequency	300 ft	YES	NO	YES	NO	YES	NO	YES	NO	YES	NO	YES	NO
	Int-related	YES	NO	NO	NO	YES	NO	YES	NO	YES	NO	YES	NO
EPDO crash frequency	300 ft	YES	NO	YES	NO	YES	NO	YES	NO	YES	NO	YES	NO
	Int-related	YES	NO	NO	NO	YES	NO	YES	NO	YES	NO	YES	NO
Crash rates	300 ft	YES	NO	YES	NO	YES	NO	YES	NO	NO	NO	YES	NO
	Int-related	YES	YES	NO	NO	YES	NO	YES	NO	YES	NO	YES	NO
EPDO crash rates	300 ft	YES	NO	YES	NO	YES	NO	YES	NO	YES	NO	YES	NO
	Int-related	NO	NO	NO	NO	YES	NO	YES	NO	YES	NO	YES	NO

Table 5.2 Summary of cross-sectional study results

Intersections types	Crash types	Crash frequency		EPDO crash frequency		Crash rates		EPDO crash rates		Est. crash rates		Est. EPDO rates	
		Reduction	Significant	Reduction	Significant	Reduction	Significant	Reduction	Significant	Reduction	Significant	Reduction	Significant
3-legged intersections	200 ft.	NO	NO	NO	NO	YES	NO	NO	NO	NO	NO	NO	YES
	300 ft.	YES	NO	YES	NO	YES	NO	NO	NO	YES	NO	NO	NO
	Int-related	YES	NO	YES	NO	YES	NO	NO	NO	YES	NO	NO	NO
4-legged intersections	200 ft.	YES	YES	YES	NO	YES	YES	YES	NO	YES	YES	YES	NO
	300 ft.	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
	Int-related	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO

In contrast, when considering 300 feet intersection crash box, at 4-legged intersections, significant reductions occurred in crash frequency, EPDO crash frequency, crash rates, and EPDO crash rate. However, when considering intersection related crashes, the presence of a by-pass lanes caused not significant increase of crash frequency, EPDO crash frequency, crash rates, and EPDO crash rates. In addition, according to case-control study the CMF were calculated to estimate the changes in crashes which is associated with adding by-pass lanes at intersections. CMF indicates an expected reduction in crashes after adding by-pass lanes. However, when considering 300 feet intersection box, lower value of CMF demonstrate higher reduction of crashes at 4-legged intersections. Though, when considering intersection related crashes, the crash reduction is higher at 3-legged intersections.

The low AADT of rural roads resulting in lower crash frequency intensify the need of higher sample size to lead to robust result. Therefore, with a development the sample size, there would be a strong probability of more significant results.

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