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EFFECTIVENESS EVALUATION OF A MODIFIED RIGHT-TURN LANE DESIGN AT INTERSECTIONS

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16. Abstract

From 2006 to 2014, ten right-turn approaches in the Peoria, Illinois, area were selected and reconstructed with a modified right-turn lane design. The major purpose of the modified design was to improve the line of sight for passenger vehicles attempting to turn right, while also accommodating semi-tractor trailer trucks. While the actual changes varied among intersections, the final result at each was an improvement to the approach angle for right-turning vehicles stopped to view cross traffic. Bradley University researchers were involved in a two-part study in which they (1) performed site-specific effectiveness evaluations of the modified sites, and (2) conducted a crash causation analysis of right-turn crashes at 116 sites in Illinois to identify geometric design variables that correlate with right-turn crashes. In the site-specific evaluation, driver behavior analyses and traffic crash–based safety analyses were performed to assess the safety performance of the modified right-turn lane design. The field investigations of driver behavior, which included a comparative "test and control" analysis at ten sites, indicated that line of site and approach angle was improved; specifically, fewer drivers had to make exaggerated head turns to complete the right-turn, there were fewer roll-and-go stops, and more vehicles stopped at the stop bar when a stop was required. Additionally, the crash-based safety evaluation, using the empirical Bayes method, revealed statistically significant reductions in crashes at the subject approaches (59.0%) and the targeted crash type, right-turn crashes at the subject approaches (59.6%) after the modifications were made. Based on these results, it is recommended that improving the right-turn approach angle by reducing the skew of the right-turn approach be considered as a safety measure for sites experiencing right-turn crashes in Illinois. The results of the crash causation analyses revealed significantly higher right-turn crashes for approaches with head-turn angles greater than 140°, right-turn angles less than 45°, and acute intersection angle less than 75°. Recommendations on the characteristics of good candidate sites for the installation of the modified right-turn lane design in Illinois are presented in this report.

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EXECUTIVE SUMMARY

Right-turn lanes on state routes must be designed to accommodate semi-tractor trailer trucks as the design vehicle (WB-67). In many states, including Illinois, this results in a right-turn lane design with large turning radius and a channelized corner island, especially if the encroachment of trucks into other lanes is discouraged in the design. When the intersection angle is 90°, this radius may place passenger vehicles at an excessive approach angle to the intersection. If the intersection is skewed, the approach angle for right-turning passenger vehicles may pose further challenges related to reduced sight distance.

From 2006 to 2014, the Illinois Department of Transportation (IDOT) implemented modifications to ten right-turn approaches in the Peoria area. The changes were intended to improve the line of sight of right-turning passenger vehicles by reducing the skew of the approach angle, while allowing semi-tractor trailer trucks to make right turns without encroachment. Bradley University researchers were retained to conduct a two-part study; they (1) performed site-specific effectiveness evaluations of the modified sites, and (2) conducted an area-wide crash causation analysis of right-turn crashes. In the site-specific evaluation, driver behavior analyses and traffic crash-based safety analyses were performed to assess the safety performance of the modified right-turn lane design. The results of the crash causation analysis were used to identify the geometric variables that influence right-turn crashes to make recommendations on what types of sites (based on their characteristics) would make good candidates for the installation of the modified right-turn lane design in Illinois.

Driver Behavior Analysis

Field observations of driver behavior at right-turn lanes at ten test sites and ten control sites were collected to assess the modified right-turn lane design. The test and control comparisons revealed that drivers traveling through the modified right-turn lane design (test sites) used safer driving behaviors compared with the control sites with the traditional design. Drivers at the test sites:

- Used fewer exaggerated head turns → improved line of sight when checking for upstream traffic to the left
- Used fewer roll-and-go stops → minimized opportunity for traffic conflicts
- Stopped on or before the stop bar more frequently → drivers are able to stop at the proper location with adequate sight distance

Because drivers at the modified right-turn lane design use fewer exaggerated head turns, roll-and-go stops, and stops past the stop bar, they will essentially travel at slower speeds, have an improved line of sight, and approach the right-turn with less skew.

Safety Evaluation and Crash Modification Factors

The traffic crash–based evaluation of seven test approaches with three years of before- and three years of after-modification data also provides insight into the safety impact of the modified right-turn lane design. The results of the empirical Bayes (EB) method indicated statistically significant reductions in subject approach crashes (59.0%), and right-turn crashes at the subject approaches

(59.6%) at 95% level of confidence (LOC). Additional analyses were conducted to determine the impact of the modified design on two subsets of drivers, older (age 65+) and younger (age 16 to 21 years). The findings from the older- and younger-driver analyses were based on the naïve before and after method because the needed safety performance functions (SPFs) for the EB method are not available for these age categories. The following are the statistically significant reductions observed:

- Older-driver right-turn crashes at the subject approach were significantly reduced by 70%
- Younger-driver crash analysis revealed significant reductions of 43% for intersection crashes, 63% for approach crashes, and 66% for right-turn crashes at the subject approach

Using the procedures outlined in the *Highway Safety Manual* (AASHTO 2010), crash modification factors (CMFs) were developed for each crash type. The resulting CMFs, along with their confidence intervals (CI), are as follows:

- Subject approach crashes CMF = 0.410 with 95% CI = 0.224 to 0.596
- Right-turn crashes at the subject approach CMF = 0.404 with 95% CI = 0.183 to 0.625

Overall, the safety evaluation provides evidence that modifying right-turn lanes to sharpen the approach angle of right-turn vehicles improves safety by reducing the frequency of right-turn crashes at the approaches, as studied in the Peoria, Illinois, area.

An analysis was conducted to determine the economic effectiveness of the modified right-turn lane design at seven test sites in the Peoria area. Economic costs and benefits of the modified design were calculated and annualized to determine the benefit to cost ratio of the modified right-turn lane design. The resulting benefit to cost ratio indicates that the accrued benefits in dollar value exceeds the annualized cost of the modified right-turn lane design over a period of 15 years by a factor of 13.8.

Based on the overall finding of the study, both the driver behavior and safety evaluations, the modification of right-turn lanes to improve the approach angle of right-turning vehicles has significant benefits. It is recommended that similar modifications to improve the right-turn approach angle/position be considered as potential improvements for intersection approaches that experience a pattern of right-turn crashes.

Crash Causation Analysis

In total, 116 right-turn approaches located throughout Illinois, with 3,174 right-turn crashes over a four-year period were included in the crash causation analysis. Crash rates for right-turn crashes and injury crashes were calculated using the traditional definition of crash rate (crashes per million entering vehicles) and an alternate one (crashes per million entering right-turn approach and intersecting approach vehicles). This resulted in four crash rates, with two rates for right-turn crashes and two for right-turn injury crashes for each of the 116 study approaches.

Once the crash and injury rates were calculated, statistical analyses were performed to determine whether there was a significant difference in crashes among a series of two or three comparison groups for the following variables at 95% LOC:

- Right-turn approach location (acute quadrant vs. 90° intersection angle vs. obtuse quadrant)
- Intersection angle (intersection angle ≥ 75° vs. intersection angle < 75°)
- Right-turn angle (right-turn angle ≥ 45° vs. right-turn angle < 45°)
- Head-turn angle (head-turn angle ≤ 140° vs. head-turn angle > 140°)
- Length of right-turn radius (radius ≤ 100 feet vs. radius > 100 feet)
- Channelized right-turn approach (channelized vs. unchannelized)

Based on the analysis of right-turn crashes, two characteristics were found to be significant from these comparisons. Approaches with right-turn angles less than 45° and head-turn angles greater than 140° were associated with significantly higher crash rates. Based on the analysis of right-turn injury crashes, three of the geometric variables were found to be significant. Right-turn injury crash rates were significantly higher for approaches with intersection angles less than 75°, right-turn angles less than 45°, and head-turn angles greater than 140°.

Correlations between right-turn crashes and the geometric characteristics and traffic volume data were investigated through regression analyses using a negative binomial underlying distribution. It was found that head-turn angle and right-turn radius had a statistically significant impact on right-turn crashes.

Design Flexibility

Based on the findings of the effectiveness evaluation and the crash causation study, flexibility in design may be needed for intersections and right-turn approaches when one or more of the following characteristics exist because higher right-turn crash rates may result:

- Channelized island present with large right-turn radius
- Intersection angle < 75°
- Right-turn angle 25° to 30°, or < 45°
- Head-turn angle > 140°
- Right-turning volume > 250 vph, or right-turn approach ADT > 3,125 vpd
- Moderate truck volumes
- Five or more right-turn crashes at an approach per year

Intersections and right-turn approaches that exhibit a combination of these characteristics may experience higher right-turn crash rates. The modified right-turn lane design should be considered as a safety countermeasure at approaches with these noted characteristics.

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LIST OF ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
AADT	annual average daily traffic
ADT	average daily traffic
ASCE	American Society of Civil Engineers
BU	Bradley University
CI	confidence interval
CMF	crash modification factor
CN	condition numbers
EB	empirical Bayes
EUAB	equivalent uniform annual benefit
EUAC	equivalent uniform annual cost
FHWA	Federal Highway Administration
HSM	*Highway Safety Manual*
IDOT	Illinois Department of Transportation
ITE	Institute of Transportation Engineers
LOC	level of confidence
LOS	level of service
MUTCD	*Manual on Uniform Traffic Control Devices*
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
PDO	property damage only
RITA	Research and Innovative Technology Administration
RTM	regression to the mean
RTOR	right turn on red
SPF	safety performance function
TRB	Transportation Research Board
USDOT	United States Department of Transportation

CHAPTER 1: INTRODUCTION

According to the National Highway Traffic Safety Administration (NHTSA), intersection-related crashes account for approximately 47% of all crashes and 27% of fatal crashes in the United States (NHTSA 2013). Within Illinois, they accounted for approximately 39% and 26% of total and fatal crashes, respectively in 2011 (IDOT 2013). Beyond this, the Federal Highway Administration (FHWA) estimates that intersection-related crashes add up to $40 billion annually in societal costs (FHWA 2011).

Transportation professionals commonly conduct studies to examine intersection designs and determine how they correlate to specific crash types. Of recent concern is the exclusive right-turn lane design and right-turn crash experience. Historically, exclusive right-turn lanes have been associated with improved safety because they separate turning vehicles from through traffic. There are different types of geometric configurations for right-turn lanes based on the design elements used and method of control of the right-turn movement. Two of the most common methods include right-turn lane with pavement markings and right-turn lane with channelized island. Some engineers think that right-turn lanes with certain configurations may not provide the safety benefit they once believed, especially for right-turn lanes with channelized islands. Some hypothesize that when channelized islands and turning roadways are used in the design, rear-end crashes may increase as a result of "collisions with vehicles stopping at the end of the turning roadway, or to drivers being more focused on looking upstream in preparation of the merging task rather than scanning to the right for potential pedestrians" (Fitzpatrick et al. 2006a).

Because state routes are often designated Class I and II truck routes regardless of truck volume and percentage, right-turn lanes for those routes must be designed to accommodate semi-tractor trailer trucks as the design vehicle (WB-67). In many states, including Illinois, and per the American Association of State and Highway Transportation Official's (AASHTO) *A Policy on Geometric Design of Highways and Streets* (AASHTO 2011), this results in a right-turn lane design with large turning radius (200-50-600 foot, three-centered curve) and a channelized corner island, especially if trucks encroaching into other lanes is discouraged in the design. When the intersection angle is 90°, this radius may place passenger vehicles at a flat approach angle to the intersection. If the intersection is skewed, the approach angle for right-turning passenger vehicles may pose further challenges caused by reduced sight distance.

In the summer of 2006, the Illinois Department of Transportation (IDOT) reconstructed the first intersection included in this study with a modified right-turn lane design. Then, beginning in 2009, nine more intersections in the Peoria area were reconstructed with a similar right-turn lane design. In 2011, Bradley University was retained to perform a two-part investigation of the modified right-turn lane design. The first part included an effectiveness evaluation of the modified right-turn lane design at the intersections where they were implemented. The second part included a crash causation analysis to determine which, if any, geometric characteristics lead to right-turn crashes. The effectiveness evaluation consisted of a driver behavior analysis as well as a traffic crash-based safety analysis. The crash causation analysis was used to identify the geometric variables that influence right-turn crashes in order to make recommendations on what types of sites (based on their

characteristics) would be good candidates for installation of the modified right-turn lane design in Illinois.

This report contains the following chapters:

Chapter 2—Literature Review

Chapter 3—Study Purpose and Methodology

Chapter 4—Site Description of Test Sites

Chapter 5—Field Investigation of Driver Behavior

Chapter 6—Traffic Crash–Based Safety Evaluation

Chapter 7—Economic Analysis

Chapter 8—Crash Causation Study

Chapter 9—Conclusions and Recommendations

CHAPTER 2: LITERATURE REVIEW

A comprehensive literature review was conducted to assess the state-of-the-art of right-turn lane designs and their impact on safety and operations. This search was conducted through Web-based queries, as well as queries through specific search engines such as the U.S. Department of Transportation (USDOT), Federal Highway Administration (FHWA), Research and Innovative Technology Administration (RITA), National Transportation Library (NTL), the American Society of Civil Engineers (ASCE), the Institute of Transportation Engineers (ITE), and others. Numerous journal papers, reports, and other published documents were reviewed. These sources reported findings on the following:

- Design of right-turn lanes
- Traffic crash–based studies
- Traffic operations–based studies
- Pedestrian and older-driver concerns

2.1 DESIGN OF RIGHT-TURN LANES

A number of factors, including speed, traffic and pedestrian volumes, percentage of trucks, capacity and type of highway, and the arrangement and frequency of intersections, enter into making the decision on whether right-turn lanes should be used (Fitzpatrick et al. 2006a). Some of the key variables considered when designing a right-turn lane include length, radius, corner islands, and turning roadway widths. Several right-turn lane designs are shown in Figure 2.1. The right-turn lane designs are a right-turn lane with a lane-line pavement marking, shared lane with island, right-turn lane with island, and right-turn lane with island and dedicated downstream lane.

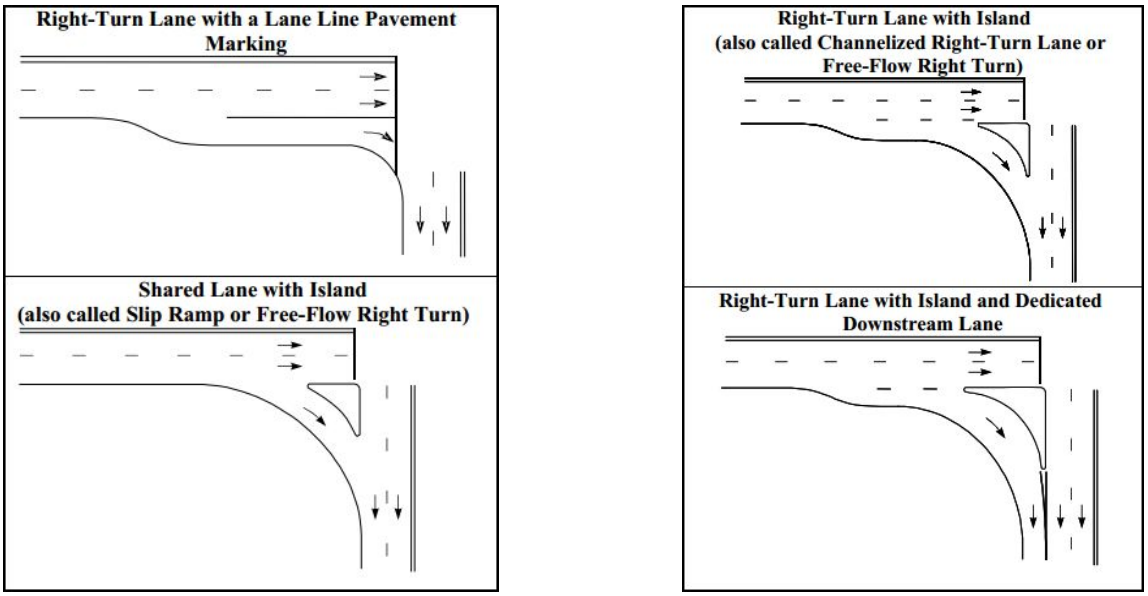


Figure 2.1 Right-turn lane designs (Source: Fitzpatrick et al. 2006a).

The corner radius is a crucial design element in right-turn lanes. It affects how drivers traverse the intersection in terms of speed and turning path. It also affects the provision of islands and pedestrian crossing time. Typically, the design is a compromise between pedestrian and driver needs. There are different benefits associated with larger and smaller corner radii. The benefits of larger turning radii include the following (Fitzpatrick et al. 2006a):

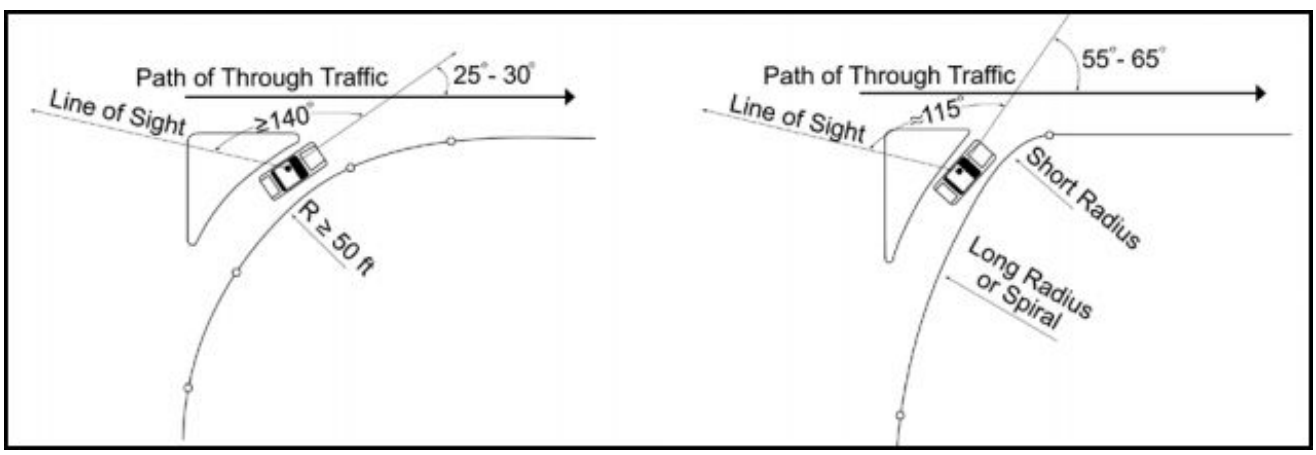
- Accommodation of larger vehicles without encroachment
- Higher vehicle turning speeds in free-flow situations, which can produce smaller speed differentials that result in less severe rear-end crashes
- Possible allowance for the presence of islands for traffic control devices and areas for pedestrian refuge

The benefits of smaller corner radii include these:

- Reductions in vehicle crossing time
- Reduced pedestrian crossing times
- Slower vehicle turning speeds
- Less pavement area

Corner islands are frequently designed to be triangular, with one side curved to match the alignment of the roadway. Islands are effective in reducing conflicts where large corner radii result in large areas of pavement. They also effectively delineate the path of through and turning vehicles clearly. Lastly, they provide a safe refuge area for pedestrians and an area for placement of traffic control devices (Fitzpatrick et al. 2006a).

An example of a traditional right-turn channelized island design compared with an alternative design is shown in Figure 2.2.



Traditional Design ***Alternative Design***

Figure 2.2 Traditional vs. alternative right-turn lane designs (Source: Gattis et al. 2009).

The traditional design has a flatter entry-angle combined with a larger radius, which may increase the speed of right-turning vehicles. The flatter entry-angle also requires a large degree of head rotation by the driver to screen for oncoming traffic from the left. The alternative design has several advantages compared with the traditional. It results in slower speeds that provide more safety for other vehicles and pedestrians, and it requires a significantly smaller degree of head rotation by drivers. This reduced head rotation makes it easier for drivers to monitor oncoming traffic and make safe merging decisions (Gattis et al. 2009).

A synthesis of current literature and policies and practices related to channelized right turns was prepared as part of National Cooperative Highway Research Project (NCHRP) Web-Only Document 208 (Potts et al. 2011). Channelized right turns are defined in that document as turning roadways at intersections that provide free-flow or nearly free-flow right-turn movements. The synthesis included a discussion on right-turn geometric design issues for the following areas: warrants, design principles, island size and design, design speed, radius and superelevation for turning roadways, width of turning roadways, angle of entry to cross street, deceleration lanes, and acceleration lanes.

2.2 TRAFFIC CRASH–BASED STUDIES

In the past, right-turn lanes were thought to improve safety by removing the turning vehicle from the through lane. However, there is a growing concern that right-turn lanes may not provide the safety benefit once believed. Rear-end crashes may increase due either to collisions with vehicles stopping at the end of the turn lane or to the driver's being more focused on looking upstream in preparation for the merging task rather than scanning to the right for potential pedestrians. To investigate the safety performance of right-turn lanes, a study was conducted in Texas using nine sites located in Irving and College Station (Fitzpatrick et al. 2006a).

Fitzpatrick et al. (2006a) analyzed three years of crash data at nine intersections based on types of crashes. At the nine intersections, 16 of the total 211 crashes involved right-turning vehicles. The right-turn crashes were further divided into rear-end, angle/merge, sideswipe, and other, as shown in Figure 2.3. The two categories that were of most interest were rear-end right-turn, in which ten crashes occurred, and right-turn angle/merge, in which four crashes occurred. The study intersections included four types of right-turn treatments: right-turn lane with lane line, right-turn lane with island, shared through/right lane, and share through/right lane with island. The authors concluded that most right-turn rear-end crashes occurred at channelized right-turn lanes, while shared through/right lanes experienced the lowest number of crashes (Fitzpatrick et al. 2006a).

The results of the 2006 study in Texas by Fitzpatrick 2006 were compared with those from a similar study conducted in the late 1990s in Georgia (Dixon et al. 1999). It was found that the treatment with the highest number of crashes in the Georgia study and the second highest number of crashes in the Texas study was the right-turn lane with raised island. In both studies, the shared through/right-turn lane had the lowest number of crashes. The authors noted, however, that a larger, more comprehensive study is needed to provide definitive results on the safety effects of the various right-turn treatments because in both the Texas and Georgia studies, the number of sites were limited and information on traffic volumes in the right-turn lane was lacking (Fitzpatrick et al. 2006a).

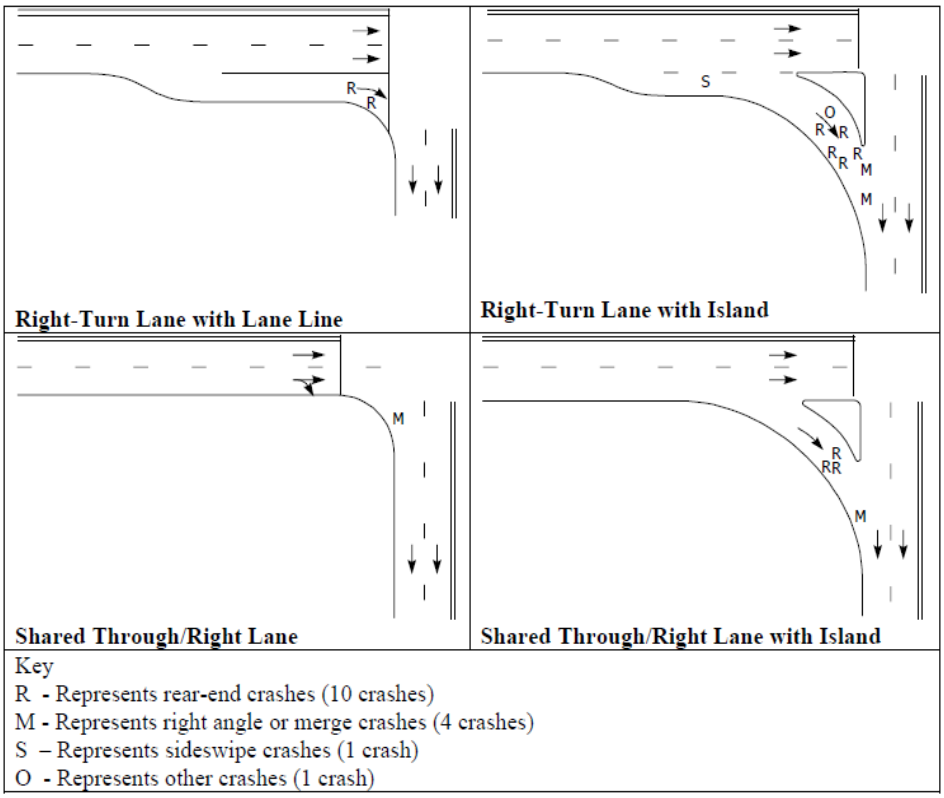
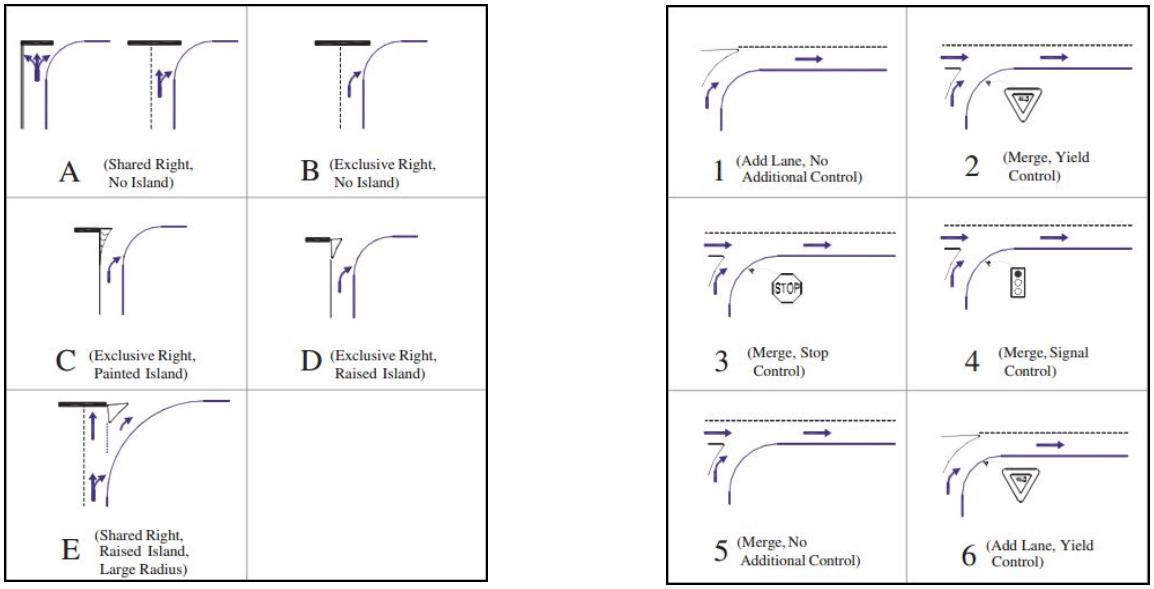


Figure 2.3 Crashes by type at various right-turn lane configuration (Source: Fitzpatrick et al. 2006a).

The study conducted in Georgia (Dixon et al. 1999) sought to observe the crash percentages of different right-turn lane configurations. Study sites were selected in Cobb County, with various entrance and exit treatments, as shown in Figure 2.4.



(a) Entrance Treatments ***(b) Exit Treatments***

Figure 2.4 Entrance and exit treatments for various right-turn lane configurations (Source: Dixon et al. 1999).

The authors noted that some of the entrance and exit treatment cases may cause confusion and misconceptions for the driver. For example, for entrance treatments B or C, during periods of high right-turn traffic, drivers might attempt to use the single right-turn lane as dual right-turn lanes involving the right-turn lane and the adjacent through lane. Entrance treatment E can often be perceived as entrance treatment D if the pavement width on the intersection approach is generous. Exit treatment 1 is often a source of confusion for drivers who anticipate a merge with oncoming through traffic and assume the existence of a yield sign. Exit treatments 2, 3, and 5 can also create confusion in terms of assigning right of way between right-turning vehicles and conflicting left- or U-turning vehicles (Dixon et al. 1999).

A preliminary crash evaluation for common right-turn scenarios in Cobb County was performed at 17 randomly selected signalized intersections. A total of 57 of the right-turn movements could be classified as one of the five following common treatment scenarios (Table 2.1):

- A5—shared right, no island, merge, no additional control
- B5—exclusive right, no island, merge, no additional control
- D1—exclusive right, raised island, additional lane, no additional control
- D2—exclusive right, raised island, merge, yield control
- E2—shared right, raised island, large turning radius, merge, yield control

Table 2.1 Right-Turn Crashes at Five Common Right-Turn Scenarios (Source: Dixon et al. 1999)

Treatment	A5 Shared Rt., Merge, No Island, No Additional Control	B5 Exclusive Rt., Merge, No Island, No Additional Control	D1 Exclusive Rt., Lane Add, Raised Island, No Additional Control	D2 Exclusive Rt., Merge, Raised Island, Yield Control	E2 Shared Rt. With Large Radius, No Island, Merge, Yield Control
Crash Type	**Percent of Right-Turn Crashes Observed**				
Right Angle	50%	31%	22%	23%	0%
Rear-end	28%	23%	64%	59%	90%
Sideswipe	17%	31%	7%	18%	0%
Other	5%	15%	7%	0%	10%
Number of Sites Evaluated	29	8	5	7	8
Number of Rt. Turn Crashes for Two-Year Period	18	13	14	22	10
Avg. Number of Rt. Turn Crashes per Site per Year	0.31	0.81	1.40	1.57	0.63

It can be seen from Table 2.1 that scenario A5 experienced fewer crashes per site than the other four right-turn treatments. Approximately 50% of the observed crashes at A5 were right-turn angle crashes, which tend to be more severe than rear-end and sideswipe crashes. Scenarios D1 and D2 both exhibited the largest number of right-turn crashes per site, with the rear-end crash being the most prevalent crash type. The authors found that shared through/right-turn lanes had the lowest number of right-turn crashes per site, while exclusive right-turn lanes with raised channelized islands had the highest number of crashes per site (Dixon et al. 1999).

A study conducted by Potts et al. (2012) sought to evaluate how the safety performance of intersection approaches with channelized right-turn lanes compared with that of intersection approaches having conventional right-turn lanes or shared through/right-turn lanes (Potts et al. 2012). As a part of Potts study, models were developed to examine various types of right-turn crashes. This study included seven years (1999 to 2005) of motor-vehicle and pedestrian crash data; volume data were obtained for 103 four-leg signalized intersections in Toronto, Ontario, and categorized into the following groups:

- Shared through/right-turn lane: 217 intersection approaches
- Conventional right-turn lane: 95 intersection approaches
- Channelized right-turn lane: 83 intersection approaches

To analyze the data, negative binomial regression modeling was used to generate seven crash prediction models. Each of the seven models focused on different types of crashes such as sideswipe, right angle, and rear-end. It was found that intersection approaches with channelized right-turn lanes had a lower estimate of total crash frequency than conventional right-turn lanes, but a higher estimate than shared through/right-turn lanes. This was found to be the case at the downstream end

of the channelized right-turn lane, where the right-turning vehicle merged with the cross street. Another statistically significant finding was that intersection approaches with channelized right-turn lanes had similar pedestrian safety performance as approaches with shared through/right-turn lanes. Intersection approaches with conventional right-turn lanes had substantially more pedestrian crashes (approximately 70% to 80% more) than approaches with channelized right-turn lanes or shared/through right-turn lanes (Potts et al. 2012).

A study conducted by Harwood et al. (2003) sought to investigate the safety effectiveness of installing left- and right-turn lanes for at-grade intersections. Three types of sites were identified and selected for the study: improved or treatment sites, comparison sites, and reference sites. The database included data for 580 intersections of the three site types located in eight different states that participated in the study: Illinois, Iowa, Louisiana, Minnesota, Nebraska, North Carolina, Oregon, and Virginia. There were 280 improved or treatment sites where left- or right-turn lanes were added.

Traffic volume data for the intersections were collected for both the major and minor road legs by counts or estimates from highway agency records for as many years as possible for each intersection. Traffic crash records were collected from each participating highway agency for periods before and after the lanes were installed. A total of 26,056 crashes that occurred at the 580 study intersections were analyzed and evaluated (Harwood et al. 2003).

The authors found that the addition of right-turn lanes was effective in improving safety at signalized and unsignalized intersections in both rural and urban areas. The authors also concluded that the installation of a single right-turn lane on a major road approach would be expected to reduce total intersection crashes at rural unsignalized intersections by 14% and reduce crashes at urban signalized intersections by 4%. Right-turn lane installations reduced crashes on individual approaches to four-leg intersections by 27% at rural unsignalized intersections and by 18% at urban signalized intersections (Harwood et al. 2003).

Sayed et al. (2013) conducted a safety evaluation of “smart channels” (modified right-turn channels that reduce the right-turn angle to approximately 70°) at three right-turn approaches in British Columbia by observing traffic conflicts at the improved locations. The three test approaches were all cross streets to a major thoroughfare. The authors state that the goal of the smart channels is to “reduce the pedestrian crossing distance which can lead to shorter distance exposure, shorter signal cycles, and reduced potential for pedestrians to be in conflict with vehicles” (Sayed et al. 2013) as well as to improve “vehicle to vehicle interactions since the new approach angle affords drivers a better view of the traffic stream they are to merge with” (Sayed et al. 2013). The authors collected before- and after-modification traffic conflict data at the three test sites and at one control site, and concluded that an overall 67% reduction in traffic conflicts was observed, significant at 90% level of confidence (LOC) (Sayed et al. 2013).

2.3 TRAFFIC OPERATIONS–BASED STUDIES

A study conducted in Texas (Fitzpatrick et al. 2006b) sought to derive equations for the free-flow speeds of vehicles in an exclusive right-turn lane with a raised channelized island and compare the accuracy with three equations used to predict the speeds of vehicles in horizontal curves. Seven

intersections in College Station and twelve intersections in Irving were selected for this study. These intersections were located in urban/suburban areas, were all signalized, and had sufficient traffic volumes of at least 30 free-flow vehicles per approach during the four-hour collection period. Twelve of the sites had raised channelized islands, while the other seven sites had exclusive right-turn lanes with pavement markings only. The corner radii at each site ranged from 27 to 86 feet.

To collect data on driver behavior and vehicles speeds, video cameras were positioned to view vehicles entering and exiting each right-turn lane and to provide additional information on driver behavior traveling through the turn. Speed data were collected during off-peak periods to obtain free-flow speeds for 100 vehicles or for a maximum time span of four hours. Free-flow vehicle data were acceptable only if the vehicle maintained a minimum five-second headway and three-second separation between the subject and following vehicle. Seven codes were developed to describe the driver behavior through the right-turn (Fitzpatrick et al. 2006b):

- Clear = driver is operating under free-flow conditions with no obstacles
- SL-Look = driver slowed to look for traffic
- SL-Traffic = driver slowed as a result of traffic
- ST-Look = driver stopped to look
- ST-Traffic = driver stopped for oncoming traffic
- ST-Other = driver stopped because of other traffic waiting to turn
- ST-Ped = driver stopped for pedestrian crossing

Only vehicles observed using the “Clear” code were counted in data to meet the free-flow requirements. The data were then analyzed to determine the 85th percentile speeds on the approach and within the right-turn. These values ranged from 13.1 to 20.5 mph within the turn and 17.4 to 28.5 mph on the approach. The data analysis revealed that vehicles decelerate on the approach to the right-turn and within the first half of the turn and then accelerate thereafter. The amount of reduction in deceleration in the turn did not appear to be a function of the corner radius. The authors then developed equations to predict the speeds of vehicles at the beginning and middle of the right turn, which were a function of right-turn lane channelization, size of corner radius, and right-turn lane length and width (Fitzpatrick et al. 2006b).

From that study, the authors concluded that equations for horizontal curves should not be used to predict speeds in right turns because they significantly over- or underestimate the true speed. The study also concluded that the approach speed was always observed greater than the middle point, which is an important aspect that should be used to design where the pedestrian crossing should be located to ensure safer crossing conditions for pedestrians (Fitzpatrick et al 2006b).

A study was conducted to investigate the effects of the geometrics of right-turn lanes on the turning performance of drivers and also the likelihood a driver will stop before making a right-turn on red (RTOR) with respect to age and gender (Tarawneh et al. 1995). Data were collected on the

performance of 100 subjects (with nearly equal gender distribution) within three age groups (25 to 45, 65 to 74, and 75+) at four intersections of different right-turn lane channelization and skew:

- A channelized right-turn lane at a 90° intersection with an acceleration lane on the cross street
- A channelized right-turn lane at a 90° intersection without an acceleration lane on the cross street
- A channelized right-turn lane at a 65° intersection without an acceleration lane on the cross street
- A 90° intersection where drivers turn right from a shared through and right-turn lane around a curb with a 40-foot radius

All subjects who volunteered for the study were allowed to perform the experiment with their own vehicle. Each driver drove around each of the four circuits a total of four times. A member of the research team rode with the subject to observe his or her behavior and rate the driver's performance based on certain measures of effectiveness. Additionally, two video cameras were used to collect data. The first camera view covered the vehicles approaching the intersection to make the right turn, paths of the vehicles after completing the right turn, and conflicting traffic approaching on the cross street. The second camera view was used to determine the speed at which the right turn was made (Tarawneh et al. 1995).

The authors concluded that right-turn channelization affects the speed at which drivers make right turns and the likelihood that they will stop before making a RTOR. Drivers, particularly ages 25 to 45, turn right at speeds three to five mph higher on intersection approaches with channelized right-turn lanes. Drivers are much less likely to stop before making a RTOR on approaches with channelization. Intersections with unfavorable skew angles affect driver behavior differently. Drivers were less likely to attempt to make a RTOR at skewed intersections where the viewing angle to conflicting traffic from the left on the cross street is greater than 90°. Skewed intersections also led to more drivers using their side mirrors to make a RTOR than at non-skewed intersections (Tarawneh et al. 1995).

A follow-up study by Tarawneh and McCoy (1996) sought to examine the effects of intersection geometrics on driver performance at different intersections. The study involved examining the effects of right-turn curb radii and observing left- and right-turn maneuvers of 200 test subjects at 11 signalized intersections. The 200 subjects were nearly equal numbers of male and female drivers in three age groups: 25 to 45, 65 to 75, and 75+. The study sites for right-turn lanes included four intersections with different degrees of right-turn lane channelization and skew. Three of the intersections were 90° intersections and the other one was a 65° skewed intersection. The study sites for examining effect of curb radii included three intersections with curb radii lengths of 15, 25, and 40 feet. The posted speed limit on all streets was 35 mph, and all intersections were controlled by traffic signals.

The measures of effectiveness used to evaluate the effects of different curb radii on driver performance were entrance distance, center distance, exit distance, and free-flow speed. All

distances were radial distances measured from the front right wheel of the vehicle to the edge of the pavement at certain points along the curb of the curve. Entrance, center, and exit distance were measured to the edge of the pavement at the beginning, center, and end of the curb curve, respectively (Tarawneh and McCoy 1996).

The authors concluded that drivers 75 years and older are not as likely as younger drivers to attempt to make a RTOR, use side mirrors when making a RTOR, and make a RTOR with stopping. It can also be noted that older drivers make right turns at slower speeds than younger drivers. Older drivers are not as sensitive as younger drivers to changes in right-turn geometry. At a channelized right-turn lane, younger drivers are more likely to increase side mirror use, make a RTOR without stopping, and increase their turning speed. The analysis to examine effect of right-turn curb radii revealed that the right-turn paths of both younger and older drivers are more in line with the pavement edge of the corner when the curb radius is larger. It can also be noted that larger curb radii increased the turning speeds of all drivers. Although drivers 75 years and older turn at slower speeds than younger drivers, there is no difference in their turning paths (Tarawneh and McCoy 1996).

2.4 PEDESTRIAN AND OLDER-DRIVER CONCERNS

Safety of older drivers continues to be a concern among transportation professionals. Some older drivers have reduced visual acuity, reduced field of vision, and loss of reflexes and limb mobility, including neck flexibility. In fact, one of the most commonly reported problems for older drivers is that they have trouble turning their heads at skewed angles to view intersecting traffic (Garcia and Belda-Esplugues 2007; Campbell et al. 2012). It is critical for drivers to have good neck flexibility to ensure proper visibility of sight triangles and safety at skewed intersections, channelized right-turn lanes, and merging roads, as illustrated in Figure 2.5.

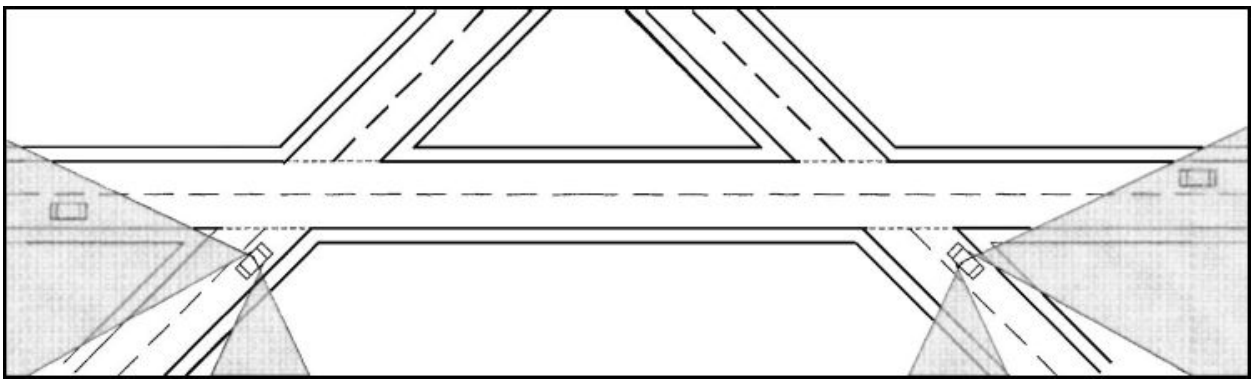


Figure 2.5 Sight triangles at skewed intersections (Source: Garcia and Belda-Esplugues 2007).

Older drivers have difficulty at skewed intersections that require an exaggerated degree of head rotation to check for oncoming vehicles before merging. When the intersection angle between two roads ranges from 40° to 55°, an older driver is physically not flexible enough to turn and look for traffic, and the outside mirror provides no assistance either. Thus, there is a need for improved lateral visibility at skewed intersections. Reducing the angle of intersection makes detection and recognition of oncoming vehicles increasingly more difficult. Intersections that meet at 90° have been

found to be the most effective for driver safety (Garcia and Belda-Esplugues 2007). According to the Institute of Traffic Engineers' *Traffic Engineering Handbook*, intersection roads should always meet at 90° when possible and not less than 75° (ITE 2009).

Figure 2.6 depicts the conflictive zones and associated visual angles. The authors, Garcia and Belda-Esplugues, recommend a maximum admissible deviation of 20° in skewed intersections. They also concluded that the most effective angles of rearview mirrors for lane changes are 17° for the left mirror and 16° for the right mirror. Lastly, when designing merging areas, the angle should be less than 7° to ensure safe merging. Approximately two-thirds of possible conflicting angles correspond to potentially dangerous situations. These critical angles should be avoided when designing intersections to promote a safe driving environment (Garcia and Belda-Esplugues 2007).

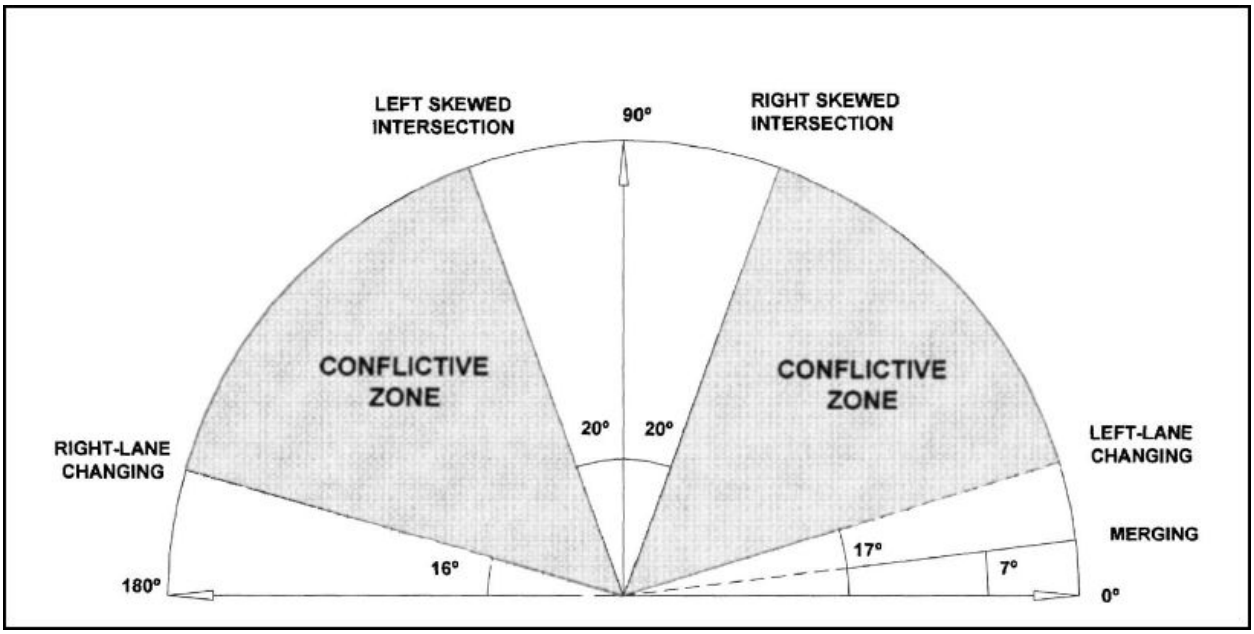


Figure 2.6 Conflictive zones and associated visual angles (Source: Garcia and Belda-Esplugues 2007).

A study conducted in the San Francisco Bay area sought to identify how specific roadway intersection characteristics (such as number of lanes, roadway crossing width, and traffic volume) are associated with pedestrian crashes. The authors were also trying to determine which intersection design characteristics are associated with greater numbers of pedestrian injuries and fatalities (Schneider et al. 2010). There are many factors that can increase or decrease the amount of pedestrian crashes, such as crosswalk distance, traffic signal phases and pedestrian wait times, right-turn channelization, right-turn-on-red movements, speed limits, sidewalk availability, and median islands.

The experiment focused on 81 of the 7,466 intersections along arterial and collector roadways in Alameda County in the San Francisco Bay Metropolitan region. Intersections were located in urban and suburban areas and had a variety of different designs and variables, as well as varying levels of pedestrian activity. Pedestrian crash data were obtained from the California Highway Patrol Statewide Integrated Traffic Records System database for the ten-year period between 1998 and

2007. Data were considered only if the crash occurred at or within 50 feet of the intersection. Pedestrian volumes were collected manually and by automated sensors for each intersection in 2008 and 2009. These data were used to estimate the total number of pedestrians crossing each intersection during a 10-year period (Schneider et al. 2010).

To analyze the data, a negative binomial regression model was used to represent the crash count data. This method was used to generate an equation with the following factors that affect the number of pedestrian crashes at each intersection: pedestrian volume, vehicle volume, mainline median, cross-street median, right-turn-only lanes, non-residential driveways, commercial properties, and proportion of population under the age of 18 (Schneider et al. 2010).

The authors concluded that more pedestrian crashes occurred at intersections with right-turn-only lanes, more nearby non-residential driveway crossings, more nearby commercial properties, and a larger percentage of children under the age of 18 living near the intersection. Medians were associated with fewer pedestrian crashes at intersections. These findings should be considered when designing an intersection to provide safer conditions for pedestrians (Schneider et al. 2010).

Research was conducted to develop a field-calibrated, statistically reliable pedestrian level of service (LOS) model for signalized intersections (Petritsch et al. 2004). To obtain data, participants were placed on actual metropolitan area roadways and through signalized intersections under typical traffic conditions. A large event was created to generate a significant number of pedestrians on the roadway course that led participants through various signalized intersections. Approximately 100 people participated in the event, and each individual was given a score card to carry during his or her walk through the course. The score card allowed each person to give each of the signalized intersections a letter grade from A to F, with A representing the best condition and F being the worst. Participants were given specific instructions to obey all traffic laws while crossing the streets. Any violations would make their data invalid. Video cameras were mounted near each intersection to study pedestrian behavior as they activated the pedestrian push buttons. Cameras were also used to count vehicle turning movements. Tube counters were used to collect real-time traffic volume data for 15-minute intervals (Petritsch et al. 2004).

The data were analyzed to determine whether the pedestrian LOS model for roadway segments can adequately predict how well intersections serve pedestrians. To test this, the existing pedestrian LOS model for roadway segments, per the *Highway Capacity Manual*, was used to calculate the parallel and perpendicular segment LOS for each intersection. The calculated pedestrian LOS values were then compared with the intersection LOS scores received from the study participants. The results of the t-test showed there were significant differences in the calculated LOS values for parallel approaches vs. the actual participants scores (p-value = 1.06×10^{-8}) and in the calculated LOS for perpendicular approaches vs. the actual scores (p-value = 1.48×10^{-24}). The results suggest that a specific pedestrian LOS model for signalized intersections needs to be developed (Petritsch et al. 2004).

The authors also sought to mathematically express the geometric, operational, and traffic characteristics that affect pedestrian perceptions of quality of service, or level of accommodation, through intersections. The following variables were considered: perceived conflicts, perceived

exposure, delay, right-turn-on-red volumes, permissive left-turn volume, traffic volumes on the street being crossed, number of lanes being crossed, pedestrian delay, and presence of a right-turn channelized island (Petritsch et al. 2004).

The authors concluded that the perceived conflicts perceived by pedestrians mostly involved right-turn-on-red movements and permissive left turns. These two movements created the most sense of danger to pedestrians crossing the street. It was also found that the presence of channelized right-turn islands has a detrimental effect on pedestrian perceptions of LOS when there is a low volume of perpendicular approach vehicles. As the perpendicular approach volume increases, the negative affect is reduced. Eventually, as the volume becomes high enough, pedestrians view the addition of channelized right-turn islands as improving their accommodation. The use of a channelized right-turn island should be considered when designing an intersection where the perpendicular approach volume is significantly high (Petritsch et al. 2004).

Research was conducted in Indiana to evaluate pedestrian crossing outcomes at a skewed intersection (Clark et al. 2005). Three outcomes were defined for pedestrian crossings: non-conflicting, compromised, and failed. A crossing is considered a non-conflicting crossing if the pedestrian leaves the curb during the walk phase and arrives at the far-side curb by the end of the pedestrian crossing phase. A crossing is considered comprised if the pedestrian reaches the far-side curb by the end of the pedestrian clearance interval but is delayed by turning vehicles or takes evasive action in response to a turning vehicle. A failed crossing is observed if a pedestrian did not reach the far-side curb by the end of the pedestrian clearance interval or if there was a pedestrian crash.

The study intersection was located in West Lafayette, Indiana, on a university campus and was fully actuated for traffic and had actuated pedestrian signals at each crosswalk. All pedestrian-phase intervals are in accordance with the guidelines in the *Manual on Uniform Traffic Control Devices* (FHWA 2009). The posted speed limit on both streets was 30 mph. The surrounding area generates a lot of pedestrian traffic throughout the day. Video data were collected on weekdays in April 2005 on each of the four pedestrian crossings during four separate nine-hour periods from 8 a.m. to 5 p.m. A signal indication data file was used to identify when a pedestrian activated the pedestrian phase. A trained observer watched the video and categorized the characteristics of the pedestrian crossing. The observer then classified each pedestrian crossing as non-conflicting, compromised, or failed (Clark et al. 2005).

Once the field data were analyzed, it was found that the likelihood of a pedestrian successfully and safely completing a crossing maneuver varies, depending on the geometry and the volume of conflicting turns into the pedestrian crosswalk. The greatest incidence of failed and compromised pedestrian crossings was observed at the crosswalk with a higher volume of right turns, turning from an exclusive right-turn lane that subtends an obtuse angle with a large turning radius. The authors concluded that exclusive right-turn lanes could lead to more pedestrian conflicts and crashes. This conclusion should be considered when designing an intersection in a highly populated pedestrian area such as a school campus (Clark et al. 2005).

CHAPTER 3: STUDY PURPOSE AND METHODOLOGY

The Illinois Department of Transportation (IDOT) commissioned a two-part research study in the fall of 2011 to (1) evaluate the safety impacts at ten intersection approaches in the Peoria, Illinois, area that were reconfigured to improve the right-turn approach angle, and to (2) conduct a crash causation analysis of right-turn crashes. The effectiveness evaluation of the ten site-specific approaches involved both a field observational study of driver behavior and a traffic crash-based evaluation at the study intersections. The crash causation analyses were used to identify the geometric variables that influence right-turn crashes in order to offer recommendations on what types of sites (based on their characteristics) would make good candidates for installation of the modified right-turn lane design in Illinois.

3.1 SITE-SPECIFIC EVALUATION OF TEST SITES

Overall, the changes made to the study approaches included sharpening the flat approach angle typical in the traditional design, reducing the radius, adjusting the stop bar position, and modifying the corner island to improve safety by increasing the line of sight of approaching through traffic. The actual changes made to each intersection varied depending on the initial design as well as site-specific conditions. At some locations, the corner island was removed; at other locations, pavement markings were provided to delineate the path for passenger vehicles. Figure 3.1 shows a generic sketch of the traditional and modified right-turn lane designs.

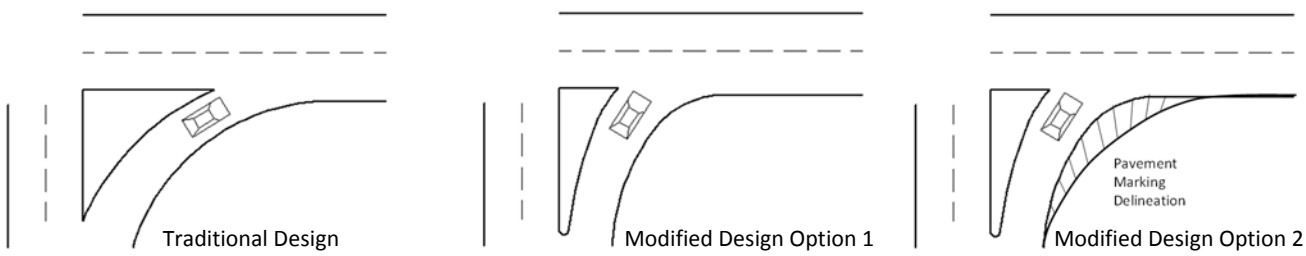


Figure 3.1 Generic right-turn lane configuration for traditional and modified design.

The traditional design has a flatter approach angle combined with a larger radius, which may increase the speed of right-turning vehicles. The flatter approach angle also requires a large degree of head rotation by the driver to screen for oncoming traffic from the left. The modified design has several expected advantages compared with the traditional: better vehicle position at the approach, improved sight distances, fewer roll-and-go stop types, and a smaller degree of head rotation by drivers. The reduced head rotation would make it easier for drivers to monitor oncoming traffic and make safe merging decisions.

To fulfill the research objectives, comprehensive driver behavior-based and crash-based evaluations were conducted to analyze the safety effects of the modified design. For all of the test sites listed in Table 3.1, field observations of driver behavior were conducted. However, not all of the sites could be included in the crash-based analysis because of a lack of availability of crash data. The test sites are

listed in Table 3.1, along with their inclusion in the driver behavior analysis and/or crash-based analysis.

Table 3.1 Test Sites and Their Inclusion in Driver Behavior and Crash-Based Analyses

No.	Test Site	City	Driver Behavior Analysis	Crash-Based Analysis
1	Prospect and Knoxville (IL-40)	Peoria	✓	✓
2	Northmoor and Knoxville (IL-40)	Peoria	✓	✓
3	Airport and IL-116	Peoria	✓	✓
4	Wesley and IL-29	Pekin	✓	✓
5	Douglas and IL-116	Metamora	✓	✓
6	Adams NB Ramp and War Memorial (US-150)	Peoria	✓	✓
7	Adams SB Ramp and War Memorial (US-150)	Peoria	✓	✓
8	I-155 SB Ramp and Broadway	Pekin/Morton	✓	
9	McClugage and IL-8	Washington	✓	
10	Sterling and Farmington (IL-8)	Peoria	✓	

In addition to the ten test sites listed in Table 3.1, the traffic crash data at one additional site located outside of the Peoria area was monitored—the intersection of Cheney and IL-29 in Taylorville. The Cheney and IL-29 intersection was excluded from the evaluation because the changes made at this intersection were substantially different from the Peoria design and did not include the critical geometric design features of the modified right-turn lane design.

3.1.1 Driver Behavior Analysis

The driver behavior analysis sought to quantify differences in head-turn extensions, stop/yield behavior of motorists, and stop location at the ten test sites, compared with ten paired control sites. The control sites had the traditional right-turn lane design, and the test sites were reconstructed to the modified design. Comparisons in driver behavior were then made between the ten paired test and control sites and were tested for statistical significance using the Z-test for proportions at 95% LOC. Additionally, at three sites, field observations were conducted before and after the approaches were reconstructed. This allowed driver behavior comparisons to be made between the results of the test and control observations and the before- and after observations.

3.1.2 Crash-Based Analysis

The crash-based evaluation included an extensive analysis of three years of before-modification data and three years of after-modification data at seven test sites. Three of the ten sites were excluded from the crash-based evaluation presented here because limited after-modification data were available. Three of the sites were reconstructed in 2013 and 2014, and only 19 to 20 months of after-data were available, which is not sufficient for inclusion in a safety effectiveness evaluation (AASHTO 2010).

The four crash categories analyzed to determine the effectiveness of the modified right-turn lane design were total intersection crashes, total intersection injury (K, A, B, C) crashes, crashes at the subject approach, and right-turn crashes at the subject approach. In the case of injury crashes, traffic crash severity is defined as follows: K represents a fatal crash, A represents a crash with an incapacitating injury, B represents a crash with a non-incapacitating injury, and C represents a crash with a possible injury (FHWA 2010). When no injuries result from a crash, the label PDO is given, representing a property-damage-only crash. The crash is represented by the highest severity observed in the incident.

Two methods were used to evaluate the crash experience at the test approaches: the naïve before and after, and the empirical Bayes (EB). These two methods seek to determine the effectiveness, or percentage reduction in crashes, that can be attributed to the countermeasure or improvement. In both methods, the actual after-modification crash frequency is compared with an expected value. The expected value represents the crashes that would have occurred in the after-modification period had the safety improvements not been made at the test site. This expected value will never be known with 100% certainty because the conditions at the test site changed as a result of the improvements. The difference in the two evaluation methods lies in the determination of the expected value of crashes. The percent reductions were determined for each of the two methods, and crash modification factors (CMFs) were calculated based on the EB results. The observed crash reductions were tested for statistical significance at 95% LOC.

3.1.2.1 Naïve Before and After

In the naïve before and after method, as shown in Figure 3.2, the expected crash frequency in the after-modification period, had the improvements not been made, is assumed to be the before crash frequency. Because the only major changes made to the intersection were related to the modified right-turn lane, it can be assumed that any significant change observed in crash frequencies would be a result of the modified design. Although this method fails to account for fluctuations common in crash frequencies over time, it still provides useful insight into the impact of the treatment.

3.1.2.2 Empirical Bayes (EB)

The random nature of crashes makes it impossible to truly predict the expected number of crashes in an after-modification period, had the improvements not been made. The EB method is commonly accepted as a more precise estimation of the expected crashes than any other method because of its ability to account for regression-to-the-mean bias.

Regression-to-the-mean effects are typically observed at sites with high crash frequencies and are defined as “the tendency of the response variable to fluctuate about the true mean value” (FHWA 1980). Thus, the decrease in the crash frequency during the after-modification period cannot be completely attributed to the improvements made at the site unless proper care has been taken to guard against regression-to-the-mean effects. The regression-to-the-mean phenomenon is illustrated in Figure 3.3.

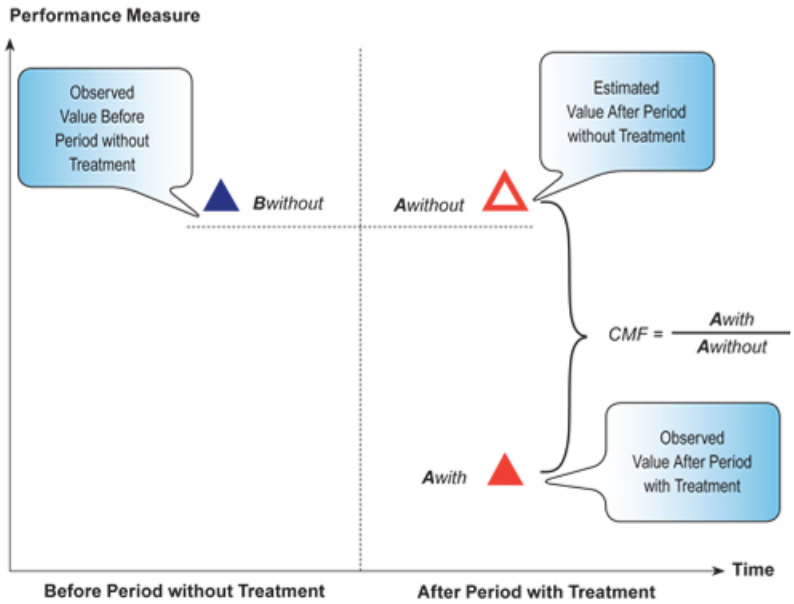


Figure 3.2 Naïve before and after method (Source: FHWA 2010).

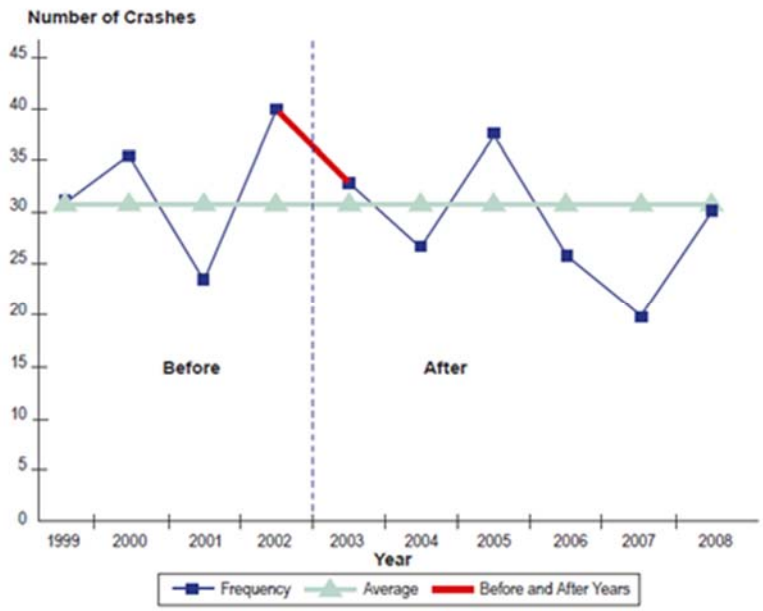


Figure 3.3 Regression-to-the-mean phenomenon (Source: FHWA 2010).

The EB method takes into account both the crash experience of the test sites and a crash prediction model, called a safety performance function (SPF), derived from the crash experience of numerous comparison sites (Figure 3.4). Once developed, this model is then weighted against the observed crash experience of the test site to more accurately predict the expected crashes.

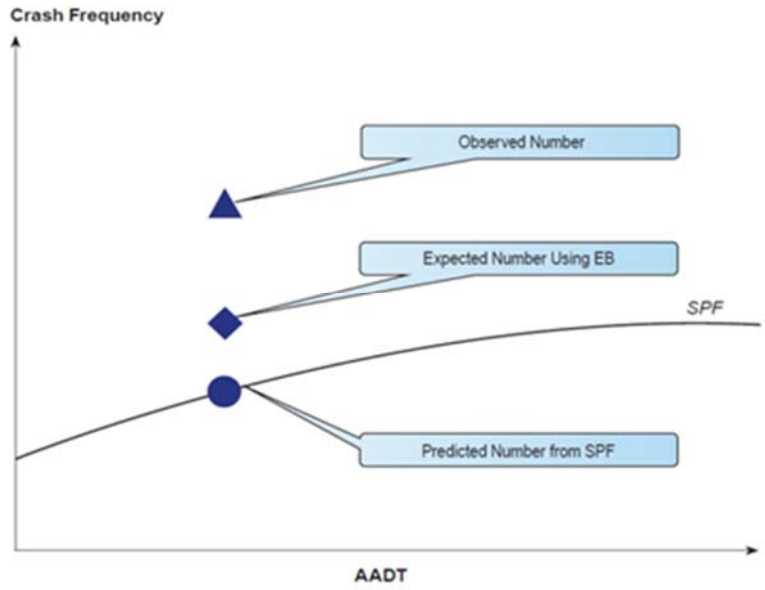


Figure 3.4 Empirical Bayes method (Source: FHWA 2010).

As a part of this research, four SPF models were developed to predict crashes for the four crash categories: total intersection crashes, total intersection injury (K, A, B, C) crashes, crashes at the subject approach, and right-turn crashes at the subject approach.

The crash history, traffic volumes, and operational features of a group of 116 comparison approaches located throughout the State of Illinois were compiled and analyzed using the IBM SPSS statistical analysis software. Assuming an underlying Poisson/negative binomial distribution, which is a common assumption in modeling traffic crashes per the *Highway Safety Manual* (AASHTO 2010), SPF models were then developed to predict crashes using variables that were found to have a statistically significant influence on crashes observed at the comparison sites.

3.2 CRASH CAUSATION STUDY

A crash causation analysis was conducted in Illinois that focused on right-turn crashes at 116 intersection approaches throughout the state. The purpose of this study is to identify and quantify the geometric design variables that correlate with significantly more right-turn crashes and injuries. This research is expected to provide guidance on what types of right-turn locations would proactively benefit from the modified right-turn lane design.

Using the Illinois statewide computerized crash database, which is geographic information system (GIS) based, queries were made to identify intersection approaches that experienced a pattern of right-turn crashes. A threshold of five right-turn crashes per approach in any one of four analysis years (2009, 2010, 2011, and 2012) was selected as the minimum crash experience for inclusion in the crash causation analysis. This number was chosen because a set of five or more crashes of a specific type is generally considered in pattern analyses. It should be noted that five crashes are typically not considered a high crash experience, yet having five crashes of a certain type (right-turn) provides a reasonable basis for correlation with geometric variables. The statewide computerized GIS

crash data query resulted in a list of 116 unique approaches with 3,174 crashes occurring over four years.

Detailed information on the crash, traffic, and geometric characteristics of the 116 study approaches was collected for the following variables: location information, intersection control (signalized, stop-controlled), laneage, intersection angle, right-turn angle, head-turn angle, annual average daily traffic (AADT) volumes for right-turn approach, intersecting through-approach and total intersection, speed limit, right-turn channelization, average annual intersection an approach crashes for total, injury, and right-turn crash types.

Statistical analyses were conducted using the Wilcoxon rank sum test, also called the Mann-Whitney test, and the Kruskal-Wallis test to determine whether there was a significant difference in crashes among a series of the following comparison groups:

- Right-turn approach location (acute quadrant vs. 90° intersection angle vs. obtuse quadrant)
- Intersection angle (intersection angle ≥ 75° vs. intersection angle < 75°)
- Right-turn angle (right-turn angle ≥ 45° vs. right-turn angle < 45°)
- Head-turn angle (head-turn angle ≤ 140° vs. head-turn angle > 140°)
- Length of right-turn radius (radius ≤ 100 feet vs. radius > 100 feet)
- Channelized right-turn approach (channelized vs. unchannelized)

Correlations between right-turn crashes and the geometric characteristics and traffic volume data were investigated through regression analyses using a negative binomial underlying distribution. Statistically significant variables resulting from the regression analysis, with right-turn crashes as the independent variable, would provide support for their relationship and influence on crashes.

In the Wilcoxon rank sum test, Kruskal-Wallis test, and the regression analyses, a 95% LOC was used to determine statistical significance.

CHAPTER 4: SITE DESCRIPTION OF TEST SITES

Ten test intersection approaches in the Peoria area were reconstructed to include the modified right-turn lane design. One additional test site, located outside of the Peoria area was also investigated; this site is located in Taylorville, Illinois. For each of the test sites, various geometric, operational, and volume characteristics were compiled from IDOT resources, and other sources. The characteristics include: intersection traffic control, intersection angle, right-turn angle, head-turn angle, right-turn radius, laneage, right-turn channelization and lane characteristics, design hourly volumes (DHV), AADT, speed limit, and other road characteristics.

Figure 4.1 depicts the angles described above for the intersection angle, right-turn angle, and head-turn angle for a sample intersection. Aerial images were imported into AutoCAD, and the dimensions of the intersection angles were measured, extracted, and recorded. The right-turn radius dimensions were extracted from the engineering design drawings, when available, or were estimated from the aerial images imported into AutoCAD. All daily traffic volume data were obtained from IDOT's traffic count website. The IDOT staff provided hourly volumes from design plans or other sources.

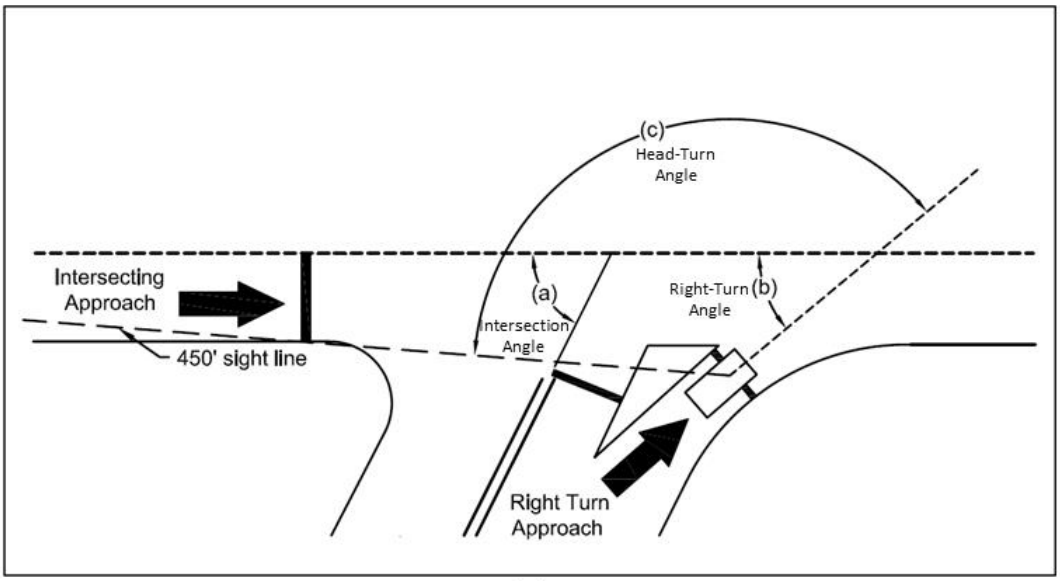


Figure 4.1 Illustration of various geometric features and measured angles.

As shown in Figure 4.1, the intersection angle (a) is the measured acute angle between the centerline of the street containing the intersecting approach and the centerline of the street containing the right-turn approach. Similarly, the right-turn angle (b) is the measured angle between the centerline of the street containing the intersecting approach and the forward direction of a right-turning vehicle at the subject approach. Finally, the head-turn angle (c) is the measured angle between the forward direction of a right-turning vehicle at the subject approach and a 450-foot line of sight to the center of the closest travel lane in the intersecting approach. This line of sight is deemed sufficient for a right-turning vehicle to judge whether it is safe to turn or not.

The main characteristics affected by the modified right-turn lane design are shown in Table 4.1 for each test approach, with the values for the before- and after-modification conditions for right-turn angle, head-turn angle, and right-turn radius. It should be noted that pedestrian volumes at all the test intersections were very low. Details of the geometric, operational, and traffic volume characteristics for the tests sites are included in Appendix A.

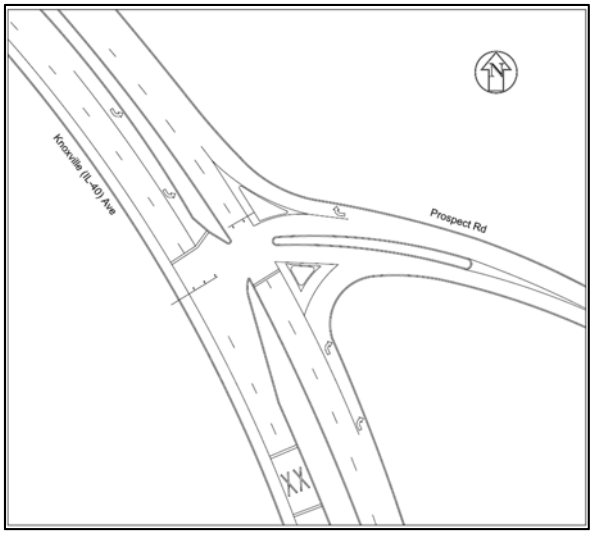
Table 4.1 Characteristics of the Test Sites

Test Site No.	City	Intersection Name	Subject Right-Turn Approach	Installation Date	Inter-section Angle (a)	Right-Turn Angle (b)		Head-Turn Angle (c)		Right-Turn Radius (ft)	
						Before	After	Before	After	Before	After
1	Peoria	Prospect & Knoxville (IL-40)	WB	2006	52	22	50	141	115	240	25
2	Peoria	Northmoor & Knoxville (IL 40)	EB	2010	90	42	60	133	115	85	78
3	Bellevue	Airport & IL-116	NB	2009–2010	59	30	44	142	128	296	75
4	North Pekin	Wesley & IL-29	SB	2010	59	30	49	142	127	302	100
5	Metamora	Douglas & IL-116	NB	2010	62	34	56	148	131	206	65
6	Peoria	Adams Ramp (NB) & War Memorial	NB	2011	84	25	41	153	138	70	70
7	Peoria	Adams Ramp (SB) & War Memorial	SB	2011	81	33	49	143	131	70	70
8	Pekin/ Morton	I-155 Ramp (SB) & Broadway	SB	2013	67	37	45	137	131	132	170
9	Washington	McClugage & IL-8	SB	2013	80	37	66	136	111	75	59
10	Peoria	Sterling & Farmington (IL-8)	SB	2014	72	38	81	139	96	92	52
11	Taylorville	IL-29 & Cheney	EB	2011	67	41	41	148	148	178	178

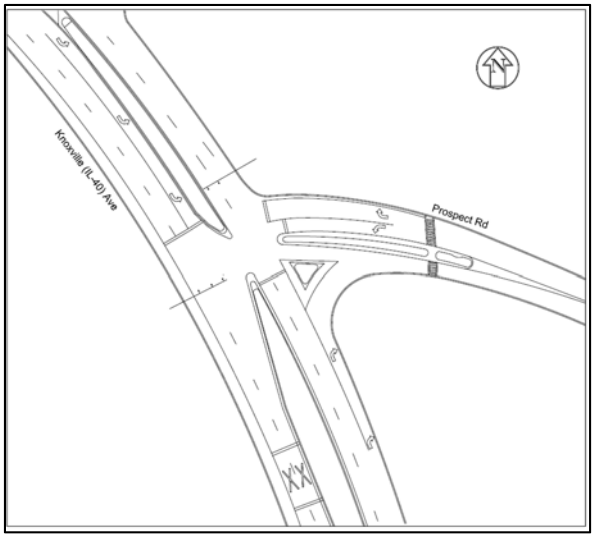
The following sections provide a summary of each of the test sites.

4.1 PROSPECT AND KNOXVILLE (IL-40)

Prospect Road is a minor arterial with a two-lane cross-section and a speed limit of 30 mph. Knoxville Avenue (IL-40) is a four-lane principal arterial with a speed limit of 45 mph. The intersection of Prospect Road and Knoxville Avenue is signalized and surrounded by a mix of residential, commercial, recreational, and institutional land uses. The intersection is skewed substantially (52°) mainly as a result of sharp horizontal curves on both of the roads at the intersection. In 2006, the westbound right-turn approach was reconstructed to the modified design. The countermeasures included removing the channelized right-turn island, reducing the right-turning radius, and relocating the stop bar and traffic signal mast arm. The modified design has a more favorable approach angle for right-turning vehicles despite the intersection skew due to roadway curvature. The before- and after-modification conditions for the intersection of Prospect Road and Knoxville Avenue are displayed in Figure 4.2.



Before Condition



After Condition

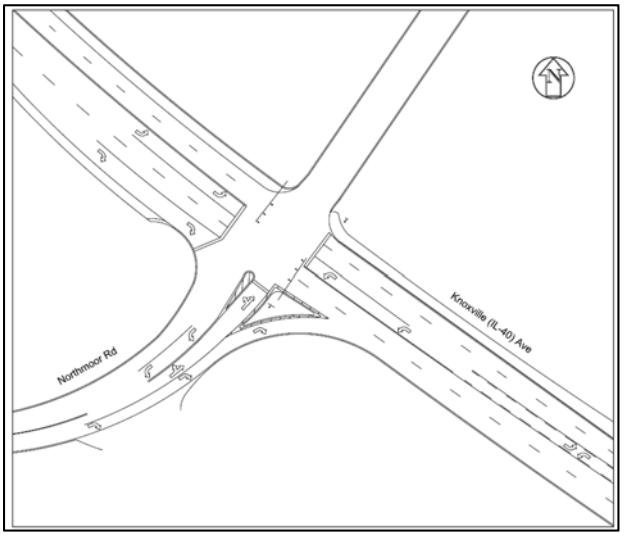
After Condition (Aerial)

Figure 4.2 Prospect Road and Knoxville Avenue (IL-40) before and after conditions.

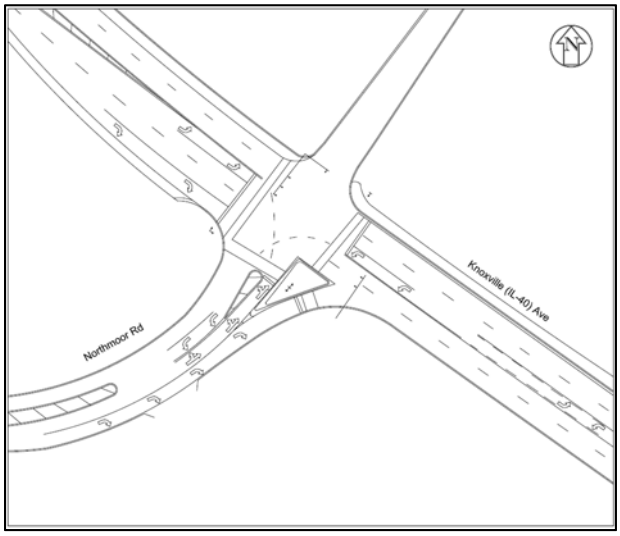
4.2 NORTHMOOR AND KNOXVILLE (IL-40)

Northmoor Road is a minor arterial with a three-lane cross-section and a speed limit of 35 mph. Knoxville Avenue (IL-40) is a four-lane principal arterial with a speed limit of 45 mph. The intersection of Northmoor Road and Knoxville Avenue is signalized and surrounded by a mix of residential, commercial, recreational, and institutional land uses. In 2010, the eastbound right-turn approach was reconstructed to the modified design. The countermeasures included adjusting the size and shape of the channelized right-turn island, reducing the right-turn radius, and relocating the stop bar. The modified design has a more favorable line of sight for right-turning vehicles because the size of the island has been reduced and the stop bar has been relocated further upstream. These

countermeasures place the driver at an approach angle that is more perpendicular to Knoxville Avenue, reducing the amount of head-turn movement required to scan for vehicles from the left. The before- and after-modification conditions for the intersection of Northmoor Road and Knoxville Avenue are depicted in Figure 4.3.



Before Condition



After Condition

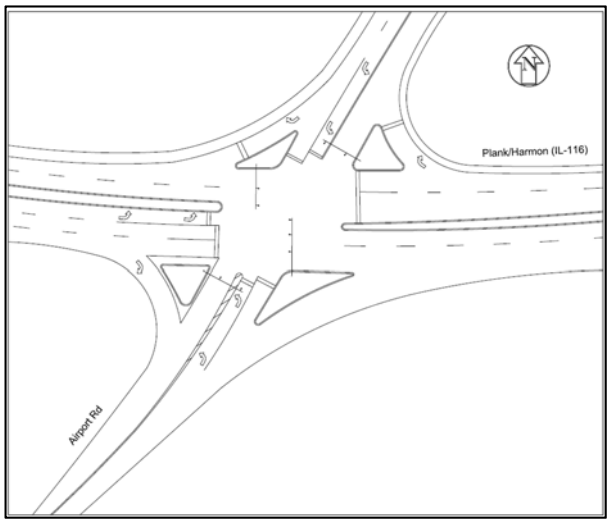
After Condition (Aerial)

Figure 4.3 Northmoor Road and Knoxville Avenue (IL-40) before and after conditions.

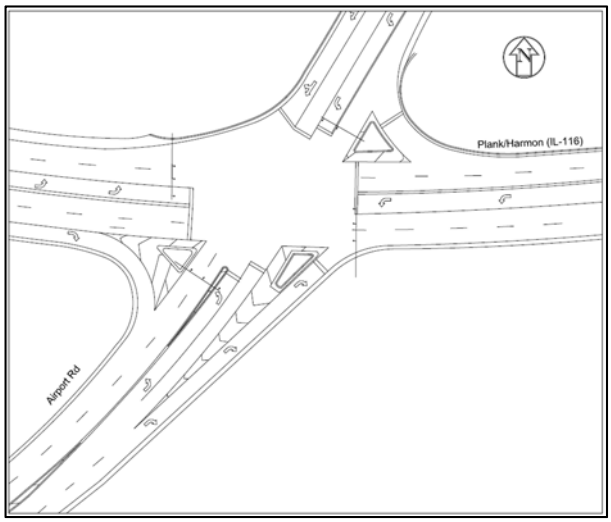
4.3 AIRPORT ROAD AND IL-116

Airport Road is a minor arterial with a four-lane cross-section and a speed limit of 45 mph. IL-116 is a principal arterial with posted speed limit of 45 mph that widens from a two-lane cross-section to a four-lane cross-section 700 feet west of the intersection with Airport Road. The intersection of Airport Road

and IL-116 is signalized and surrounded by residential and industrial land uses, as well as wooded areas. In 2009, the northbound right-turn approach was reconstructed and included the following countermeasures: resizing the right-turn island and striping pavement markings around it, substantially reducing the right-turn radius, and relocating the stop bar. Drivers now approach the right-turn more perpendicular to IL-116 and have an improved line of sight to the left. Adjustments to the right-turn islands were also made at the other three approaches. However, they do not contain the critical elements of the modified right-turn lane design and thus were not considered for use as test approaches. The before- and after-modification conditions for the intersection of Airport Road and IL-116 are illustrated in Figure 4.4.



Before Condition



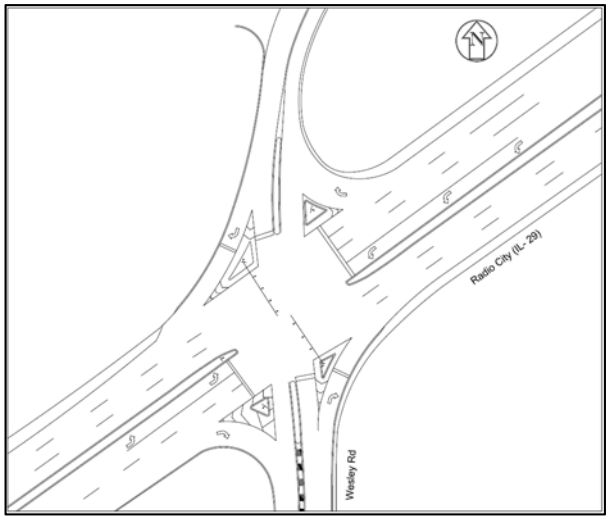
After Condition

After Condition (Aerial)

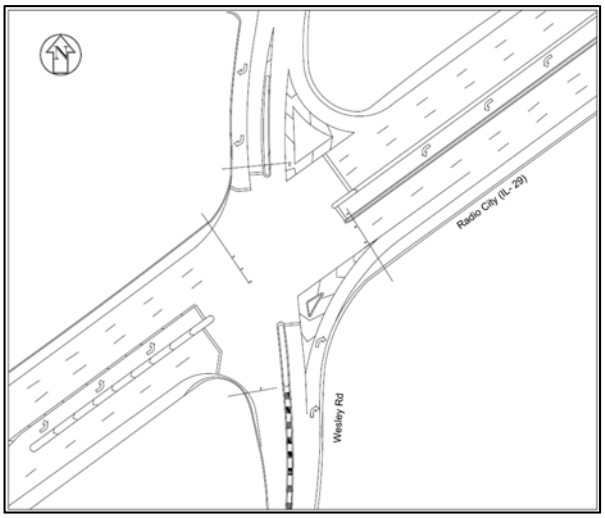
Figure 4.4 Airport Road and IL-116 before and after conditions.

4.4 WESLEY AND IL-29

Wesley Road is a local road with a two-lane cross-section and a speed limit of 25 mph. IL-29 is a principal arterial with a five-lane cross-section and a speed limit of 45 mph. The intersection of Wesley Road and IL-29 is signalized and surrounded by residential, commercial, and agricultural land uses. As Wesley Road approaches the intersection, there is a moderate horizontal curve, which when combined with the large corner island, placed drivers at a flat approach angle. In 2010, the southbound right-turn approach was reconstructed to include the following countermeasures: removing the right-turn island, substantially reducing the right-turn radius, and relocating the stop bar. Adjustments to the right-turn islands were also made at the other three approaches. However, they do not contain the critical elements of the modified right-turn lane design and thus were not considered for use as test approaches. The before- and after- conditions at Wesley Road and IL-29 are shown in Figure 4.5.



Before Condition



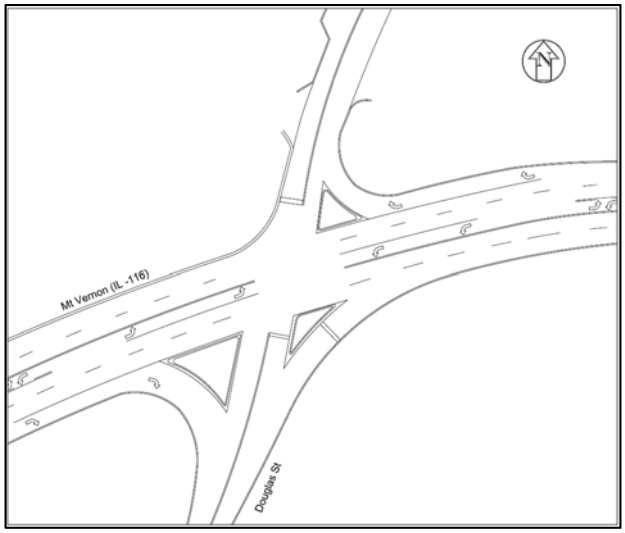
After Condition

After Condition (Aerial)

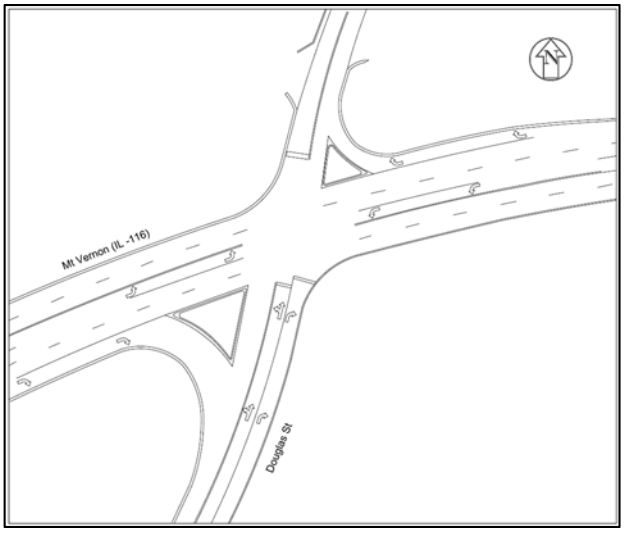
Figure 4.5 Wesley Road and IL-29 before and after conditions.

4.5 DOUGLAS AND IL-116

Douglas Street is a minor arterial with a two-lane cross-section and a speed limit of 45 mph. IL-116 is also a minor arterial and has a four-lane cross-section with posted speed limit of 40 mph. The intersection is two-way stop controlled with stop signs located on Douglas Street. The intersection is surrounded by residential, commercial, and agricultural land uses. In 2010, the northbound right-turn approach was reconstructed to the modified design. The right-turn approach countermeasures included removing the right-turn island, reducing the right-turn radius, and relocating the stop bar. The before- and after-modification conditions at the Douglas Street and IL-116 intersection are depicted in Figure 4.6.



Before Condition



After Condition

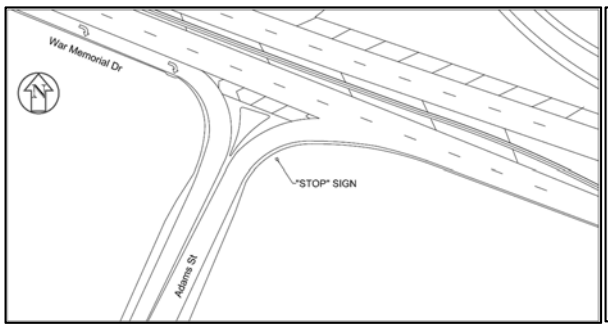
After Condition (Aerial)

Figure 4.6 Douglas Street and IL-116 before and after conditions.

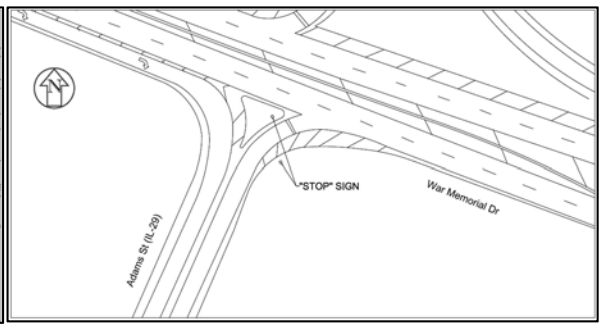
4.6 ADAMS NB AND SB RAMPS AND WAR MEMORIAL (US-150)

The ramps entering onto War Memorial (US-150) from Adams are the focus of the next two test sites. The interchange between Adams and War Memorial is complex because of the space constraints caused by being located next to the Illinois River, as well as high traffic volumes on these roads. The interchange is surrounded by residential, recreational, public works, and commercial land uses. Both Adams and War Memorial are principal arterials with four-lane cross-sections. The entrance ramps from Adams to War Memorial are two-way stop-controlled and have a suggested speed of 25 mph. War Memorial has a speed limit of 45 mph. In 2011, both the northbound and southbound entrance ramps were reconstructed with the modified right-turn design.

At the northbound approach, the countermeasures included reshaping the raised median, relocating the stop bar, striping additional pavement markings, and placing an additional stop sign located on the left side of the raised median. The additional pavement markings were placed around the radius to help delineate a better approach for passenger cars, while also allowing large trucks to complete the turn without striking the curb. The before- and after-modification conditions of the NB Adams ramp and War Memorial intersection are shown in Figure 4.7



Before Condition

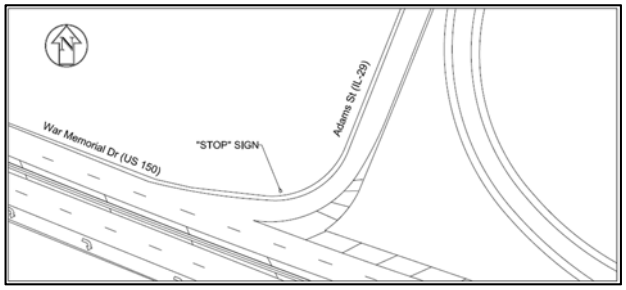


After Condition

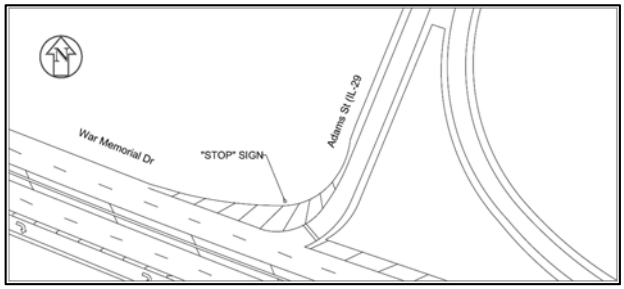
After Condition (Aerial)

Figure 4.7 Adams NB ramp and War Memorial (US-150) before and after conditions.

At the southbound approach, the countermeasures included relocating the stop bar and striping additional pavement markings, similar to the northbound approach. The before- and after-modification conditions of the SB Adams ramp and War Memorial intersection are displayed in Figure 4.8.



Before Condition



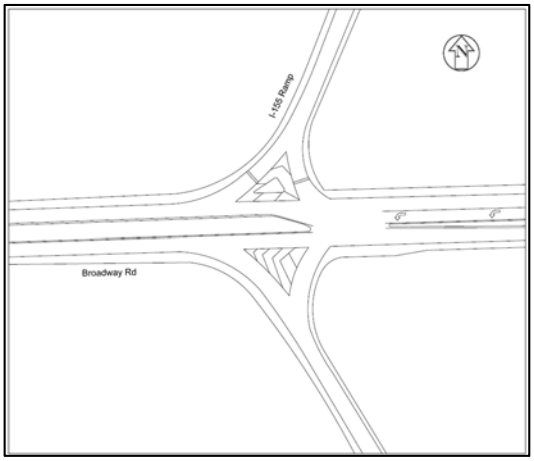
After Condition

After Condition (Aerial)

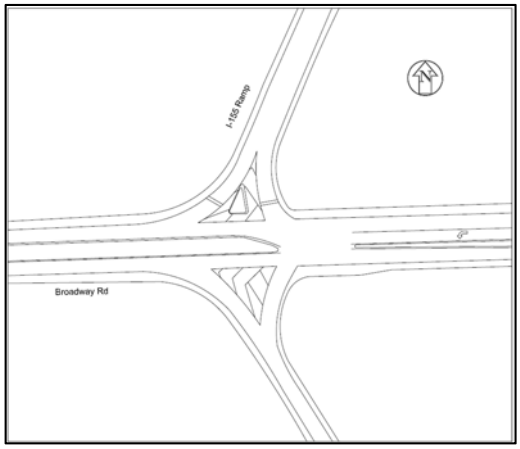
Figure 4.8 Adams SB ramp and War Memorial (US-150) before and after conditions.

4.7 I-155 SB RAMP AND BROADWAY

The southbound ramp from interstate I-155 onto Broadway is two-way stop-controlled and consists of one lane. Broadway is a major collector with a two-lane cross-section. The speed on the ramp is 45 mph, while the speed limit on Broadway is 55 mph. The intersection is surrounded by residential, recreational, public works, and commercial land uses. In 2013, the southbound right-turn approach was reconstructed to include the following countermeasures: widening the ramp to include an exclusive right-turn lane, reshaping the raised median and pavement markings, reducing the right-turn radius, and relocating the stop bar. The before- and after-modification conditions at the I-155 SB ramp and Broadway are shown in Figure 4.9.



Before Condition



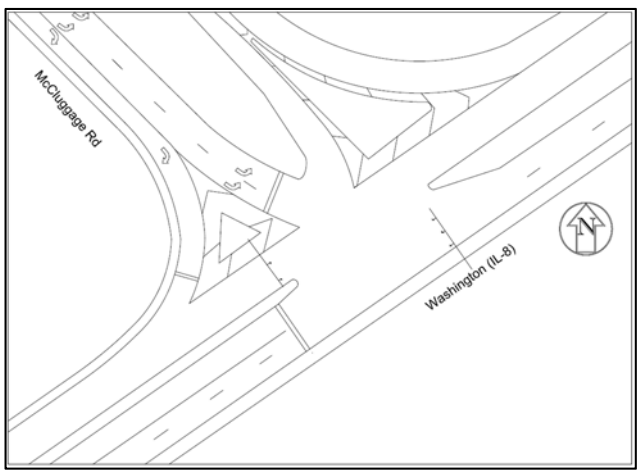
After Condition

After Condition (Aerial)

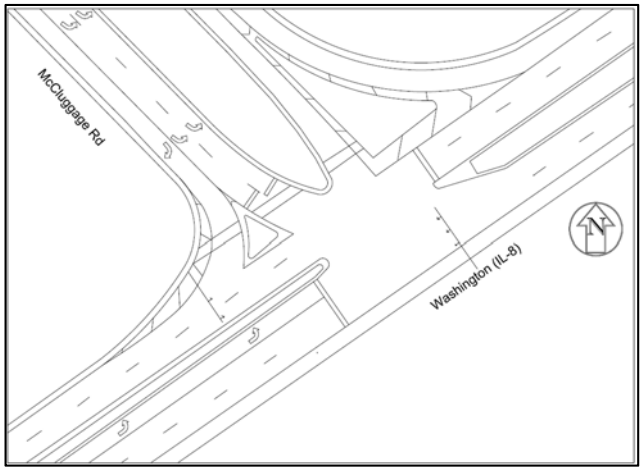
Figure 4.9 I-155 SB ramp and Broadway before and after conditions.

4.8 MCCLUGAGE AND IL-8

McClugage Road is a minor arterial with a four-lane cross-section and a speed limit of 50 mph. IL-8 is a principal arterial with a four-lane cross-section and a speed limit of 45 mph. The intersection is surrounded by agricultural, commercial, and residential land uses. In 2013, the southbound right-turn approach was reconstructed to the modified design. The countermeasures included reducing the right-turn radius, restriping the pavement markings around the radius, relocating the stop bar, adding a pedestrian crosswalk, and reshaping the corner island to place drivers at a better approach angle. The before- and after-modification conditions at the McClugage Road and IL-8 intersection are displayed in Figure 4.10.



Before Condition



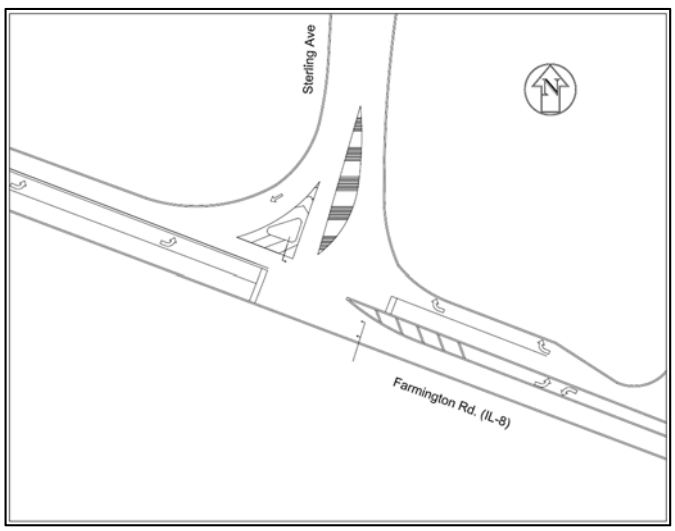
After Condition

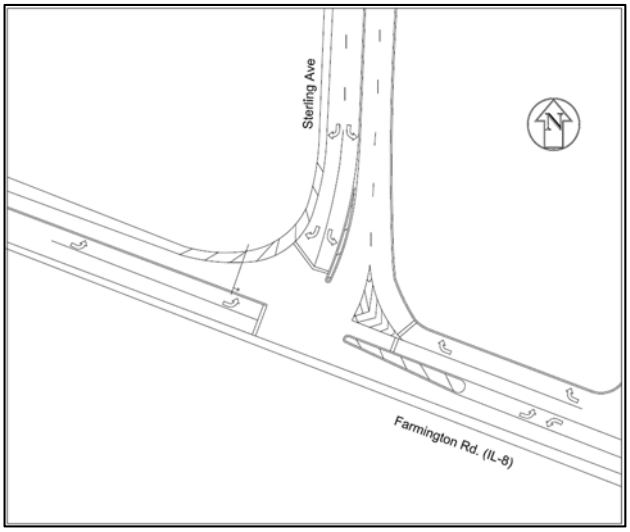
After Condition (Aerial)

Figure 4.10 McClugage Road and IL-8 before and after conditions.

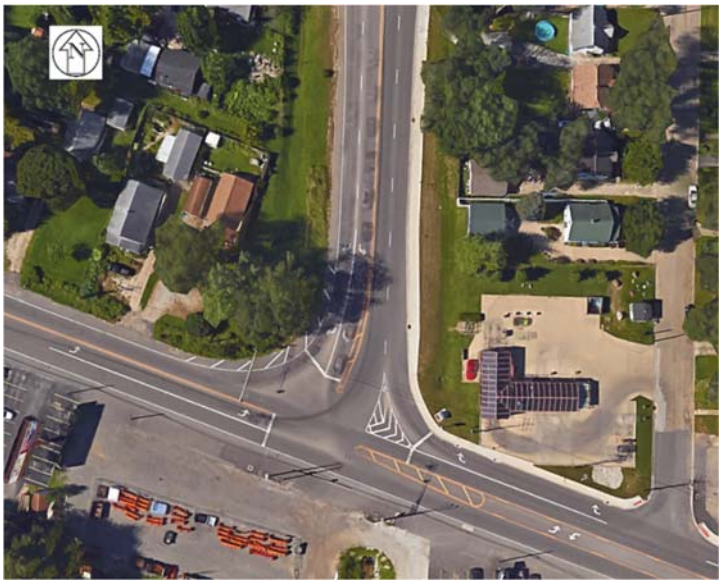
4.9 STERLING AND FARMINGTON (IL-8)

Sterling Avenue is a minor arterial with a four-lane cross-section and a speed limit of 45 mph. Farmington Road (IL-8) is a minor arterial with a three-lane cross-section and a speed limit of 40 mph. The intersection is surrounded by residential and commercial land uses. In 2014, the intersection of Sterling Avenue and Farmington Road was reconstructed. Sterling Avenue was widened from two lanes to four lanes. As part of the reconstruction, the southbound right-turn approach was treated with the modified right-turn lane design. The countermeasures included removing the right-turn island and pavement markings, reducing the right-turn radius, relocating the stop bar, and adding pavement markings around the radius to delineate the path for passenger vehicles while allowing large trucks to safely complete the turn. A right-turn island was also striped at the westbound approach to delineate the corner areas because there are two receiving lanes on Sterling Avenue. The before- and after-modification conditions at the Sterling Avenue and Farmington Road (IL-8) intersection are shown in Figure 4.11.





Before Condition ***After Condition***



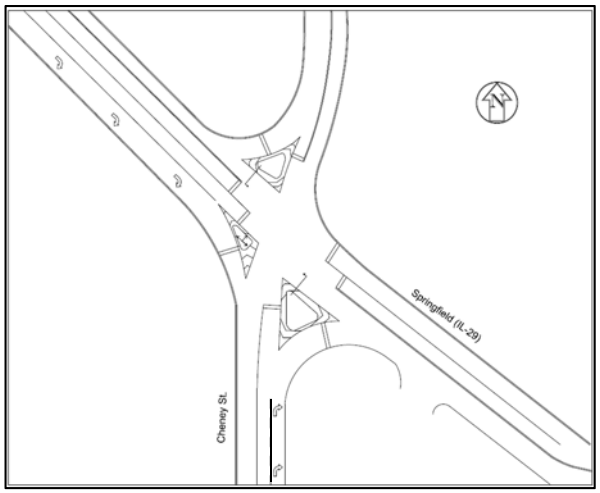
After Condition (Aerial)

Figure 4.11 Sterling Avenue and Farmington Road (IL-8) before and after conditions.

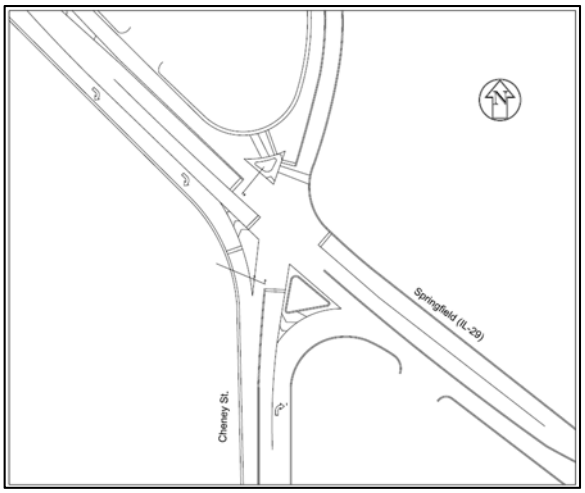
4.10 IL-29 AND CHENEY

IL-29 is a principal arterial with a three-lane cross-section and a speed limit of 30 mph. Cheney Street is a minor arterial with a two-lane cross-section and a speed limit of 40 mph. The intersection is surrounded by commercial, agricultural, and institutional land uses and is located in Taylorville (outside the Peoria area). In 2011, the eastbound right-turn approach was reconstructed; the countermeasures included removing the raised corner island, restriping the pavement markings, relocating the stop bar, and relocating the traffic signals. Geometric changes to the right-turn radius and approach angle were not made. Because the changes at this intersection did not include the critical geometric elements of the modified right-turn lane design, it was excluded from the

evaluation. The before- and after-modification conditions of the IL-29 and Cheney Street intersection are displayed in Figure 4.12.



Before Condition



After Condition

After Condition (Aerial)

Figure 4.12 IL-29 and Cheney Street before and after conditions.

Enlarged versions of the images depicted in Figures 4.2 through 4.12 are included in Appendix B.

CHAPTER 5: FIELD INVESTIGATION OF DRIVER BEHAVIOR

Test and control sites were used in the field investigation of driver behavior. The control sites were selected based on their similarity to the respective test site's pre-construction (before) characteristics. Test and control sites were paired based on their similarities in intersection control, intersection angle, right-turn angle, head-turn angle, right-turn radius (measured as a single curve as opposed to a three-centered curve), annual average daily traffic (AADT), presence of exclusive right-turn lanes, downstream characteristics of the right-turn movement, and other characteristics. The research team used aerial images to extract the intersection angle, right-turn angle, head-turn angle, and radius. All traffic volume data were gathered from IDOT's traffic count website. Table 5.1 presents the characteristics at the ten paired test and control sites. The angles described in Table 5.1 were previously illustrated in Figure 4.1.

Table 5.1 Characteristics of Test and Control Sites

	Intersection	Subject Right-Turn Approach	Traffic Control at Subject Right Turn	Inter-section Angle[a] (a)	Right-Turn Angle[a] (b)	Head-Turn Angle[a] (c)	Radius[a]	Inter-section AADT[b]
Test	Prospect & Knoxville	WB	Signalized	52	22	141	240	29,217
Control	Allentown & Court	WB	Signalized	76	40	126	136	24,525
Test	Northmoor & Knoxville	EB	Signalized	90	42	133	85	28,483
Control	N Frye & Knoxville	EB	Signalized	75	32	143	199	25,242
Test	Airport & IL 116	NB	Signalized	59	30	142	296	15,650
Control	War Memorial & Willow Knolls	WB	Signalized	68	23	154	240	29,825
Test	Wesley & IL 29	SB	Signalized	59	30	142	302	29,992
Control	Veterans Parkway & Morrissey	WB	Signalized	63	33	137	207	37,363
Test	Douglas & IL 116	NB	Stop	62	34	148	206	15,558
Control	Park Ave & Court	EB	Stop	65	45	132	45	23,550
Test	Adams & War Memorial	NB	Stop	84	25	153	70	19,254
Control	I-74 & Washington	NB	Stop	87	39	131	83	13,200
Test	Adams & War Memorial	SB	Stop	81	33	143	70	19,004
Control	I-74 & Washington	NB	Stop	87	39	131	83	13,200
Test	I 155 & Broadway	SB	Stop	67	37	137	132	5,675
Control	I 155 & Broadway	WB	Stop	69	38	139	143	3,300
Test	McClugage & IL 8	SB	Signalized	80	37	136	75	22,250
Control	War Memorial & Adams	EB	Signalized	89	39	139	124	23,717
Test	Sterling & Farmington (IL-8)	SB	Signalized	72	38	139	92	41,300
Control	War Memorial & Adams	EB	Signalized	89	39	139	124	23,717

[a]Represents the before condition value at the test approaches. Angles are defined in Figure 4.1.
[b]Average of 2008 to 2012 AADT data.

5.1 DATA COLLECTION

To observe and quantify changes in driver behavior at the approaches with the modified right-turn lane design, field data were collected at each of the ten test sites and ten paired control sites. The field data consist of four main variables: driver head-turn behavior, lateral placement in the turn lane, stop type, and stop location.

Field data were collected between May 2012 and July 2015. To have a comprehensive understanding of driver behavior, eight hours of data were collected at each site, resulting in 160 hours of observations (80 hours at the test sites plus 80 hours at the control sites.) An extended PM peak period (3:00 p.m. to 7:00 p.m.) on typical weekdays (Tuesday, Wednesday, or Thursday) was chosen for the observation period, and the data were collected on two days at each site. A total of nearly 19,000 vehicles were observed at the test and control sites combined.

To ensure a comprehensive set of data, a video recorder and two observers with voice recorders were used to collect driver behavior data. The observers took a verbal account of driver behavior with the voice recorders, and the video was used as a reference. Every right-turning vehicle that approached the subject right-turn when a stop was required was observed. One observer recorded the driver head-turn behavior prior to completing the turn, as well as the lateral placement of the vehicle in the turn lane. The other observer recorded the type of stop and the location of the stop relative to the stop bar. The variables collected are defined in Table 5.2.

Table 5.2 Driver Behavior Variables Collected When Required to Stop

Type of Head Turn	Description
No Look	The driver does not observe upstream traffic within the data collector's vantage point
Normal	The driver simply turns his or her head to observe upstream traffic
Exaggerated	The driver turns his or her shoulders or body, leans far forward, or makes other extreme movements to observe upstream traffic
Lateral Placement	**Description**
Inside/Curbside	The vehicle travels in the part of the lane closest to the curb
Middle	The vehicle travels in the middle of the lane
Outside	The vehicle travels in the part of the lane closest to the right-turn island
Stop Type	**Description**
Complete or Near Complete	The vehicle either comes to a complete stop, or stops all forward momentum but the tires don't completely stop
Rolling	The vehicle slows down substantially but does not stop its forward momentum
Roll-and-Go	The vehicle fails to slow down or reduce speed
Stop Location	**Description**
On or Before the Stop Bar	The vehicle stops either with its front tires before the stop bar or with the stop bar between its front and back tires
Past the Stop Bar	The vehicle stops with its back tires beyond the stop bar

Figures 5.1 through 5.4 depict the percentage of the total observations for each driver behavior variable for head-turn behavior, lateral vehicle placement, stop type, and stop placement, respectively, for the combined observations at the ten test sites and ten paired control sites.

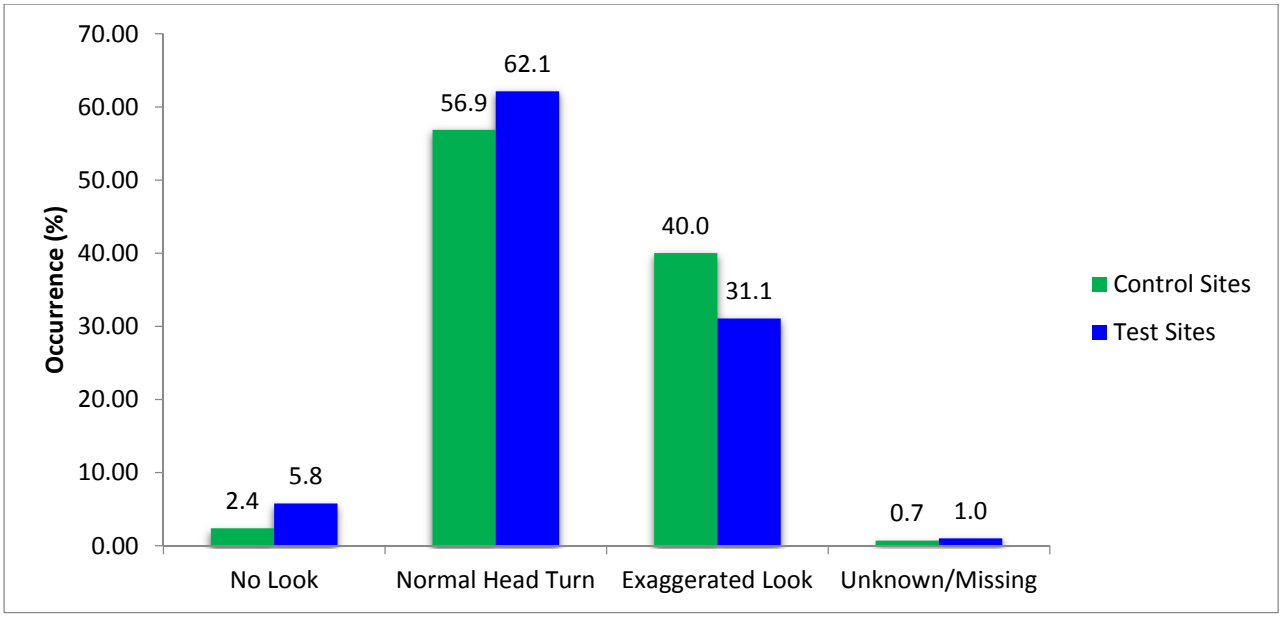


Figure 5.1 Head-turn behavior at test and control sites.

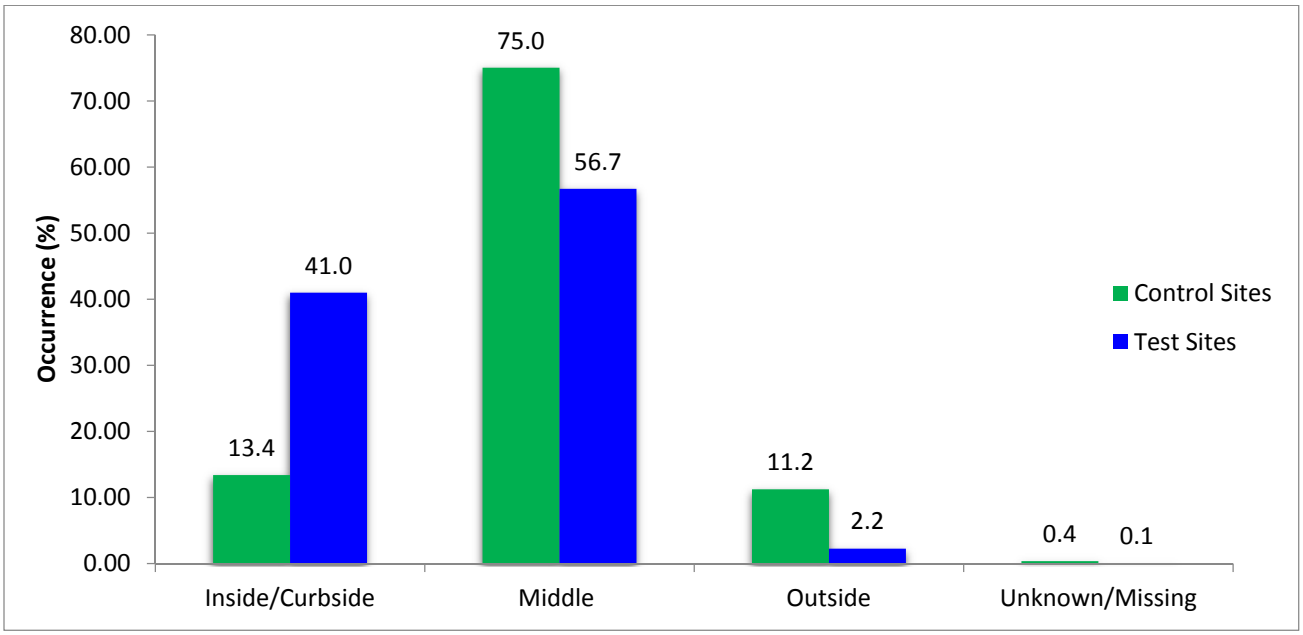


Figure 5.2 Lateral vehicle placement at test and control sites.

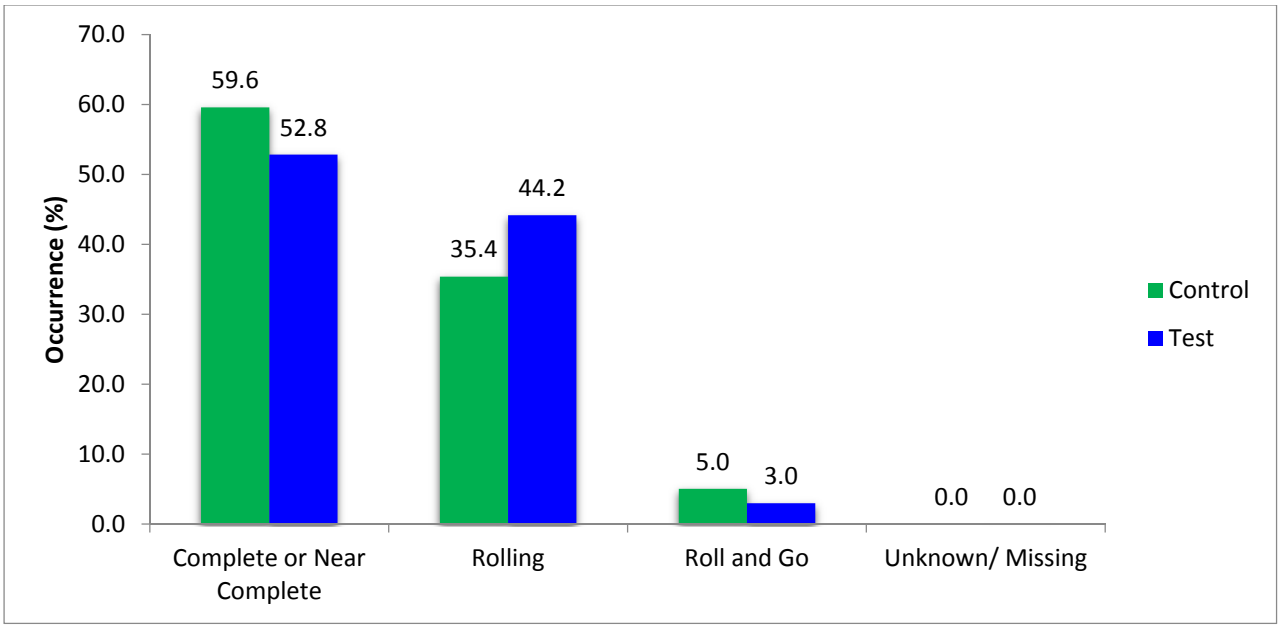


Figure 5.3. Stop type at test and control sites.

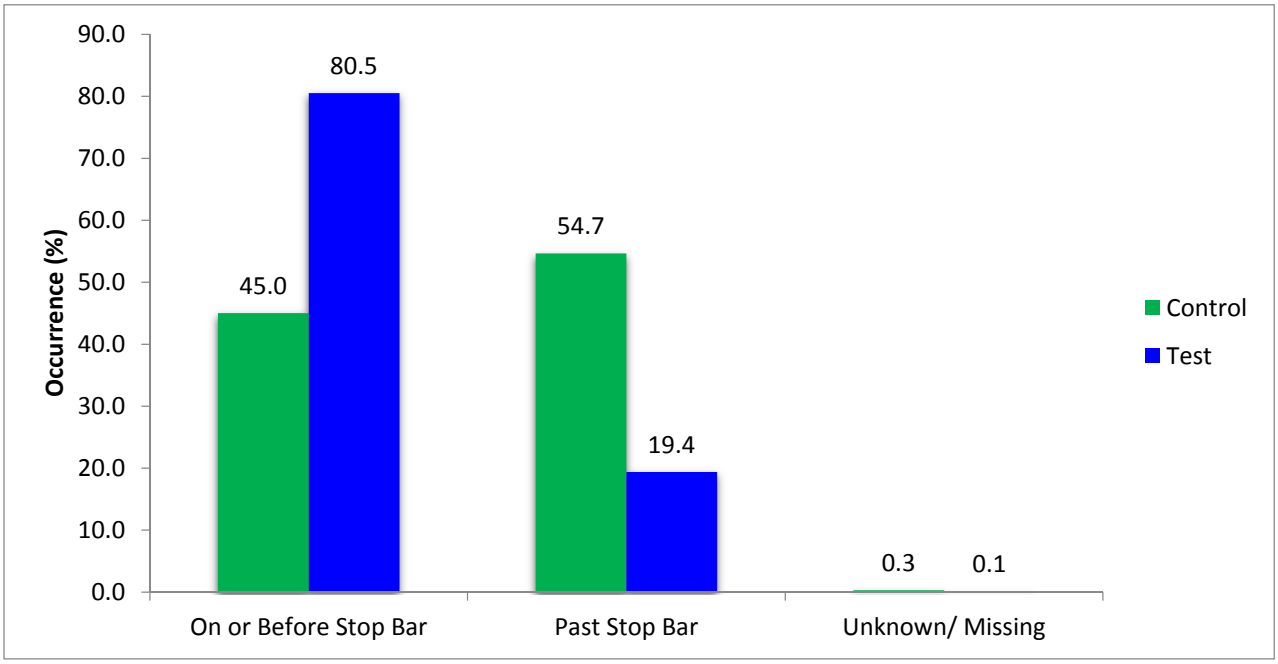


Figure 5.4. Stop placement relative to stop bar for test and control sites.

After comparing the test and control sites on an aggregated basis, it was found that normal head turns occurred more at the test sites (62.1%) than the control sites (56.9%), and exaggerated head turns occurred less at the test sites (31.1%) than at the control sites (40.0%). Additionally, drivers used the inside/curbside of the right-turn lane more at the test sites (41.0%) than the control sites (13.4%).

Complete stops or near-complete stops were observed frequently at both the test and control sites, representing over 50% of the observations. A higher percentage of roll-and-go stops were observed at the control sites compared with the test sites. When a complete or near-complete stop was used, drivers stopped on or before the stop bar more frequently at the test sites (80.5%) compared with the control sites (45.0%).

Prior to collecting any data or conducting any statistical analyses, four main hypotheses were developed to describe differences in driver behavior in response to the modified right-turn lane design, compared with the traditional design, when stopped in the right-turn lane. Each of the hypotheses state that fewer occurrences of the critical/undesirable variables would be observed at the test sites compared with the control sites. The four critical variables are exaggerated head turns, inside/curbside lateral placement (hugging the curb), roll-and-go stop type, and the stop location past the stop bar. The four main test (alternative) hypotheses (H_a) are listed below. The null hypothesis (H_o) for each case would be that there is no difference in the driver behavior variables between the test and control sites.

1. The proportion of exaggerated head turns at the test sites would be less than the proportion at the control sites. This was expected because of the improvement in the driver's line of sight. {H_o: $P_t = P_c$; H_a: $P_t < P_c$}
2. The proportion of vehicles with inside/curbside lateral placement at the test sites would be less than the proportion at the control sites. This was expected because of the shortened radii and the modification of the channelized island. {H_o: $P_t = P_c$; H_a: $P_t < P_c$}
3. The proportion of vehicles "rolling and going" at the test sites (when a stop was required) would be less than the proportion at the control sites. This was expected because it was assumed that drivers would prefer to travel in the middle of the lane with the geometric improvements, even when turning right. {H_o: $P_t = P_c$; H_a: $P_t < P_c$}
4. The proportion of vehicles who stopped past the stop bar at the test sites would be less than the proportion at the control sites. This was expected because the improved approach angle and improved sight lines meant that drivers would not have to travel past the stop bar to see oncoming traffic. {H_o: $P_t = P_c$; H_a: $P_t < P_c$}

The field observations of driver behavior for the critical variables were analyzed for the ten test sites and ten control sites, and the results were then compared on an aggregated basis.

In addition, before- and after-modification comparisons were made for three sites where it was possible to collect before- and after-modification data, based on the installation dates of the modified design, at the intersections of: McClugage and IL-8, Sterling and Farmington (IL-8), and I-155 SB ramp and Broadway.

5.2 ANALYSIS AND RESULTS

Statistical analyses were conducted to determine whether significant differences existed in the critical variables as a result of the modified right-turn lane design. The Z-test for proportions (Equation 5.1) was used to determine the significance of the observed differences at a 95% LOC,

significance level (α) of 0.05, and a one-tailed test, because a direction in the critical variables was hypothesized.

$$z_{statistic} = \frac{P_c - P_t}{\sqrt{p\ (1-p)\left(\frac{1}{N_c} + \frac{1}{N_t}\right)}} \tag{5.1}$$

where

P_c = Proportion of driver behavior variable observed at control site
P_t = Proportion of driver behavior variable observed at test site
N_c = Number of observations at control site
N_t = Number of observations at test site
p = Estimate of overall proportion = $(N_cP_c + N_tP_t) / (N_c + N_t)$

The Z-statistic values were calculated and the corresponding p-values were obtained. A p-value is defined as the lowest level of significance at which the calculated value of the test statistic is significant. Thus, for a one-tailed test, if the p-value is less than 0.05, then the null hypothesis is rejected and the finding is significant. The results of the statistical analysis for the test and control comparisons for the critical variables are shown in Table 5.3. For the before- and after-modification analysis, the aggregated before- and after-modification observations were used in place of the control and test variables in Equation 5.1, the comparisons are shown in Table 5.4.

Table 5.3 Comparison of Critical Driver Behavior Variables at Test and Control Sites

	No. Observations		Proportion				
Description	**Control Sites**	**Test Sites**	**Control Sites**	**Test Sites**	**Z-statistic**	**p-value**	**Significant at 95% LOC?**
Head-Turn Behavior							
Exaggerated Head Turns	2,342	3,688	0.400	0.311	11.82	< 0.00001	Yes
Total Observed	5,852	11,869					
Lateral Placement							
Inside/ Curbside	783	4,864	0.134	0.410	–37.08	< 0.00001	Yes
Total Observed	5,852	11,869					
Stop Type							
Roll-and-Go	299	413	0.050	0.033	5.52	< 0.00001	Yes
Total Observed	5,948	12,358					
Stop Placement							
Past Stop Bar	1,937	1,310	0.547	0.196	36.2	< 0.00001	Yes
Total Observed	3,544	6,676					

Table 5.4 Before and After Comparison of Critical Driver Behavior Variables at Test Sites

	No. Observations		Proportion				
Description	**Before**	**After**	**Before**	**After**	**Z-statistic**	**p-value**	**Significant at 95% LOC?**
Head-Turn Behavior							
Exaggerated Head Turns	741	642	0.395	0.240	11.18	< 0.00001	Yes
Total Observed	1,876	2,674					
Lateral Placement							
Inside/ Curbside	505	758	0.247	0.283	–2.82	0.00241	Yes
Total Observed	2,046	2,674					
Stop Type							
Roll-and-Go	75	96	0.051	0.032	3.13	0.00088	Yes
Total Observed	1,477	3,020					
Stop Location							
Past Stop Bar	333	660	0.440	0.510	–3.07	0.00108	Yes
Total Observed	757	1,294					

The comparison between test and control right-turn approach locations (Table 5.3) revealed that significantly fewer drivers used exaggerated head turns at the test sites (31.1%) than at the control sites (40.0%). This supports the assumption that the modified right-turn lane design provides a better line of site for drivers turning right, reducing the need to use an excessive head turn to check for upstream traffic. The comparison also showed that more drivers traveled through the inside part of the lane at the test sites (41.0%) than at the control sites (13.4%). This result does not support the stated hypothesis that the modified right-turn lane design would encourage drivers to position their vehicles in the middle of the lane. It was found that significantly lower number of drivers used roll-and-go stops at the test sites (3.3%) than the control sites (5.0%). At the test locations, a significantly lower number of vehicles stopped past the stop bar (19.6%) compared with the control sites (54.7%), which indicates that the modified right-turn lane design allows vehicles to stop at the proper location, with adequate sight distance. Because drivers at the modified right-turn lane design use fewer exaggerated head turns, roll-and-go stops, and stops past the stop bar, they will travel at slower speeds, have improved sight lines, and approach the right turn at a better angle.

As shown in Table 5.4, the before- and after-modification results were similar to the results of the test and control site comparisons. Although the before and after comparisons were drawn from a smaller sample of test sites, they do provide further support for the noted results.

CHAPTER 6: TRAFFIC CRASH–BASED SAFETY EVALUATION

Traffic crash analyses were performed for each of the seven test intersections and approaches based on three years of before-modification data and three years of after-modification data for the following crash types: total intersection crashes, intersection injury crashes, subject approach crashes, and right-turn crashes at the subject approach. The analysis of traffic crashes and the effectiveness evaluations were based on the naïve before and after method and the EB method. Crash modification factors were determined based on the EB method results. Before- and after-modification crash comparisons were made for older and younger age groups to understand how the modified right-turn lane design may impact human factors issues.

For the intersections where fewer than three years of after-modification crash data were available, or were located outside of the Peoria area, the before- and after-modification crash comparisons are presented in the report, but were not used as the basis for the conclusions. These intersections are I-155 SB ramp and Broadway, McClugage and IL-8, Sterling and Farmington, and Cheney and IL-29.

6.1 DATA COLLECTION

Three years of before-modification data were obtained and analyzed for each of the test approaches. Three years of after-modification data are typically desired for crash-based evaluations, and seven of the sites have three years of after-modification data available. Three of the test sites were excluded from this analysis because only 19 to 20 months of after-modification crash data, at most, were available. The fourth intersection was excluded from the analysis because it is located outside of the Peoria area, and its modified design varied from the Peoria designs. The test sites included in the crash-based evaluation are as follows:

1. Prospect and Knoxville (IL-40)
2. Northmoor and Knoxville (IL-40)
3. Airport and IL-116
4. Wesley and IL-29
5. Douglas and IL-116
6. Adams Northbound ramp and War Memorial (US-150)
7. Adams Southbound ramp and War Memorial (US-150)

Crash data were obtained from IDOT, typically in batches ranging from 6 months to 1 year of data, depending on availability. Once the crash database files were received, the research team extracted the data for each intersection, downloaded the police traffic crash report forms, and then filed them for ease of use and organization. Because the police traffic crash reports were used in this evaluation study, the research team extracted data from the report forms one crash at a time.

All crash types were determined by reviewing the diagrams/narratives prepared by the police officers on the form and/or from the direction of movements of the involved drivers as noted on the crash report forms. The crash diagrams and narratives included on the crash report provided an accurate

assessment of the type of crash that actually occurred, regardless of the crash type coded on the form, which helped minimize potential coding errors. For each test approach, crashes were collected within a 200-foot radius, and information was summarized for location details, crash details, and driver characteristics. Additionally, crashes that involved a vehicle making a right-turn maneuver from the subject approach were specifically noted. This set of right-turn crashes was of primary interest in this research because the treatment has the largest expected impact on this targeted crash type.

Data extracted from the crash report forms included the following:

- Date, time, and day of week of crash
- Weather conditions
- Road surface conditions
- Crash type
- Crash severity (K, A, B, C, PDO)
- Approach of intersection that the crash occurred at
- Driver(s) age and gender
- Primary contributory cause of crash

The traffic crash data were aggregated into four main categories for analyses:

- Total intersection crashes
- Intersection injury (K, A, B, C) crashes
- Subject approach crashes
- Right-turn crashes at the subject approach

These categories were chosen to gain a comprehensive understanding of the safety effects of the modified design. Right-turn crashes at the subject approach are considered to be the targeted crash types for the modified design. Once the traffic crash data had been extensively analyzed and summarized, crash comparisons were made to determine whether the modified design had statistically significant effects on reducing crashes.

Because two of the test sites are located at the same intersection (Adams and War Memorial), the total intersection and intersection injury crashes are aggregated for six test intersections, while the subject approach crashes and right-turn crashes at the subject approach are aggregated for seven test approaches.

A total of 435 traffic crashes occurred over a six-year period at the seven test intersections. Table 6.1 presents the overall crash frequencies for the before- and after-modification periods. Details of the crash frequencies by year, by crash type, and by intersection/approach can be found in Appendix C.

Table 6.1 Aggregated Crash Frequencies Before and After Treatment at Seven Test Sites

Crash Type	3-Year Crash Data Totals		Average Annual Crashes per Year	
	Before	After	Before	After
Total Intersection Crashes (6 Test Intersections*)	274	161	91.33	53.67
Intersection Injury (K, A, B, C) Crashes (6 Test Intersection*)	64	39	21.33	13.00
Subject Approach Crashes (7 Test Approaches)	172	65	57.33	21.67
Right-Turn Crashes at Subject Approach (7 Test Approaches)	161	45	53.67	15.00

*War Memorial and Adams intersection was counted only once, even though it contains two test approaches.

Figure 6.2 provides a graphical display of the annual average before- and after-modification crash comparisons.

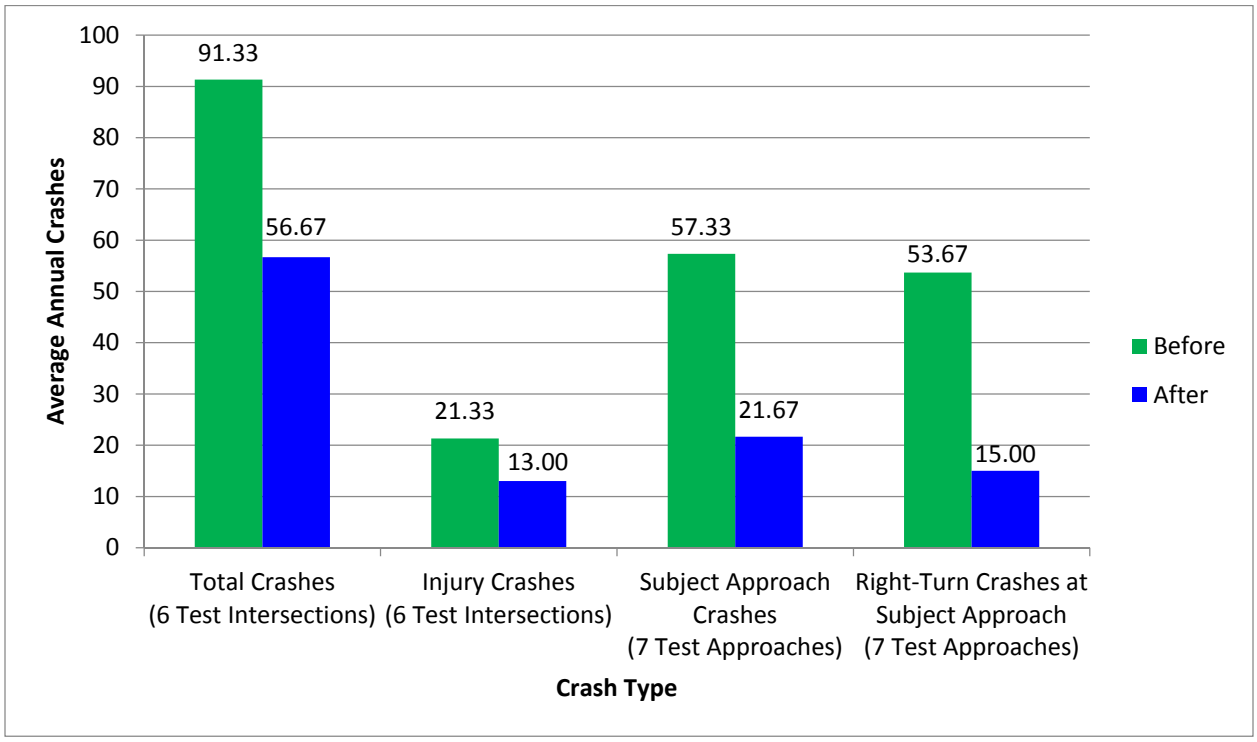


Figure 6.2 Comparison of before- and after-modification crashes at test sites.

The trends in Figure 6.2 indicate that crashes have reduced after the modified right-turn lane design was installed. It should be noted that the modified design was not installed at all approaches of the test intersections. Thus, the total intersection crash reductions are not as great as the subject

approach reductions. It can also be observed that the predominant crash type at the subject approaches were right-turn crashes, as 54 of the 57 crashes (before-modification) were right-turn related. The greatest reductions in crashes occurred for the targeted crash type, right-turn crashes at the subject approach.

6.2 ANALYSIS AND RESULTS

Two methods were used to evaluate the crash experience at seven test right-turn approaches: the naïve before and after, and the empirical Bayes method. These two methods seek to determine the effectiveness, or percent reduction in crashes, that can be attributed to the countermeasure or improvement. In this research, the improvement is the modified right-turn lane design described earlier. In both methods, the actual after-modification crash frequency is compared with an expected value. The expected value represents the crashes that would have occurred in the after-modification period had the safety improvements not been made at the test site. This expected value will never be known with 100% certainty because the conditions at the test site changed as a result of the improvements. The difference in the two evaluation methods lies in the determination of the expected value of crashes.

6.2.1 Naïve Before and After (B&A)

The naïve before and after method involves comparing the crash frequency of the traditional right-turn island design (before-modification period) to the crash frequency with the modified right-turn lane design (after-modification period.) The before-modification period crashes are considered to be the "expected" value, based on the assumption that the crashes would have remained the same over time, had the modified design not been implemented. The result of this comparison is a theoretical difference in crash frequency that can be attributed to the modified design, if the finding is found to be statistically significant at 95% LOC. The naïve before and after analysis was performed for each of the four crash categories analyzed as a part of this study.

The observed crash reductions were tested for statistical significance using the Poisson test at 95% LOC. Because traffic crash data are discrete and assumed to occur randomly, the Poisson test was used to test the significance of changes in crash frequencies. A one-tailed test was used at a 95% LOC and significance level (α) of 0.05 because it was hypothesized that traffic crash frequencies would reduce as a result of the implementation of the modified right-turn lane design. The null (H_o) and alternative (H_a) hypotheses used in the statistical analysis of the average/mean (μ) crash frequencies are as follows:

H_o: $\mu_{\text{after crashes}} = \mu_{\text{before crashes}}$

H_a: $\mu_{\text{after crashes}} < \mu_{\text{before crashes}}$

The naïve before and after method was the only method used to compare crashes by age group. The necessary age-specific SPFs for the EB method are not available in the literature nor could be developed as a part of this research because of small sample sizes and the small subset of crashes that they represent. Crashes involving older drivers and younger drivers were analyzed to see the effects of the modified design on these age groups. Crashes which involved a driver of age 65 or older

were considered older-driver crashes. Crashes involving drivers 16 to 21 years of age were considered younger-driver crashes. It is to be noted that the fault of the crash was not assigned to the drivers in this analysis. For example, just because an older driver was involved in the crash, it does not mean that the older driver was at fault, and caused the crash. The older/younger-driver comparisons were performed for each of the four crash categories studied.

The aggregated before and after results for the seven test sites are shown in Table 6.2. Table 6.3 and Table 6.4 depict the evaluation results of the older-driver (age 65+ years) and younger-driver (age 16 to 21 years) analyses, respectively. It should be noted that older-driver crashes represent 18% to 19% of the before- and after-modification crashes, while the younger-driver crashes represent 23% to 24% of the crashes. The observed percent reductions in crash frequency were tested for statistical significance using the Poisson test at 95% LOC, and the p-values were also obtained. For a one-tailed test, if the p-value is less than 0.05, then the null hypothesis is rejected and the finding is significant.

Table 6.2 Naïve Before and After Evaluation of Seven Test Sites

	Annual Average Crash Frequency			
		Naïve B&A		
Crash Type	**Observed/ After**	**Expected/Before**	**Reduction**	**Significant?* (p-value)**
Total Intersection Crashes (6 Test Intersections)	53.67	91.33	41.2%	Yes (< 0.01)
Intersection Injury (K, A, B, C) Crashes (6 Test Intersections)	13.00	21.33	39.1%	Yes (0.04)
Subject Approach Crashes (7 Test Approaches)	21.67	57.33	62.2%	Yes (< 0.01)
Right-turn Crashes at Subject Approach (7 Test Approaches)	15.00	53.67	72.0%	Yes (< 0.01)

*Based on Poisson test of crash frequencies at 95% LOC and significance level $\alpha = 0.05$.

Table 6.3 Naïve Before and After Evaluation of Seven Test Sites for Older-Driver Crashes

	Annual Average Crash Frequency			
		Naïve B&A		
Crash Type	**Observed/ After**	**Expected/Before**	**Reduction**	**Significant?* (p-value)**
Total Intersection Crashes (6 Test Intersections)	16.67	16.33	–2.1%	No (> 0.20)
Intersection Injury (K, A, B, C) Crashes (6 Test Intersection)	4.67	2.00	–133.5%	Too low frequency to test
Subject Approach Crashes (7 Test Approaches)	3.33	5.67	41.3%	No (0.13)
Right-Turn Crashes at Subject Approach (7 Test Approaches)	1.67	5.67	70.6%	Yes (0.05)

*Based on Poisson test of crash frequencies at 95% LOC and significance level $\alpha = 0.05$.

Table 6.4. Naïve Before and After Evaluation of Seven Test Sites for Younger-Driver Crashes

	Annual Average Crash Frequency			
		Naïve B&A		
Crash Type	**Observed/ After**	**Expected/Before**	**Reduction**	**Significant?* (p-value)**
Total Intersection Crashes (6 Test Intersections)	12.67	22.33	43.3%	Yes (0.02)
Intersection Injury (K, A, B, C) Crashes (6 Test Intersection)	4.00	8.00	50.0%	No (0.07)
Subject Approach Crashes (7 Test Approaches)	4.67	12.67	63.1%	Yes (0.02)
Right-turn Crashes at Subject Approach (7 Test Approaches)	3.67	11.00	66.6%	Yes (0.01)

*Based on Poisson test of crash frequencies at 95% LOC and significance level $\alpha = 0.05$.

When comparing the naïve before-modification and after-modification crash frequencies of the four crash types studied, it was found that total intersection crashes were reduced by 41%, intersection injury crashes were reduced by 39%, subject approach crashes were reduced by 62%, and right-turn crashes at the subject approach were reduced by 72%. Each of these crash reductions are statistically significant at 95% LOC using the Poisson test. It should be noted that the largest reductions were observed at the approach level, with the largest impact on right-turn crashes.

The naïve before and after comparison of crashes involving older drivers revealed that there was statistically no difference in before and after crashes for total intersection and subject approach crashes. The intersection injury crashes could not be tested for statistical significance due to the value of the expected crash frequency being too low ($\leq$ 4 crashes). However, older driver right-turn crashes at the subject approach were significantly reduced by 70%. The impact of the modified design on right-turn crashes at the subject approach is similar for older drivers (70%) and all drivers (72%).

The naïve before and after comparison for younger drivers revealed decreases in crashes for all four of the crash categories; however, only three of these reductions were statistically significant: intersection crashes reduced by 43%, approach crashes reduced by 63%, and right-turn crashes at the subject approach reduced by 66%. The impact of the modified design on right-turn crashes at the subject approach is similar for younger drivers, older drivers, and drivers of all ages (66% to 72%).

The before- and after-modification crash comparisons for the intersections where fewer than three years of after-modification data were available, or that were located outside of the Peoria area, are presented in Appendix C. These intersections are the I-155 SB ramp and Broadway, McClugage and IL-8, Sterling and Farmington (IL-8), and Cheney and IL-29. The first three test sites were excluded from the crash analysis because only 20 months or fewer of after-data are currently available. The fourth intersection was excluded from the analysis because the redesign for this intersection did not include

the critical geometric design elements of the modified right-turn lane design, even though a full three years of after-data were available at this site.

6.2.2 Empirical Bayes (EB)

The random nature of crashes makes it impossible to truly predict the expected number of crashes in an after-modification period, had the improvements not been made. The EB method is commonly accepted as a more precise estimation of the expected crashes than any other method because of its ability to account for regression-to-the-mean bias. The EB method takes into account both the crash experience of the test sites, and a crash prediction model, called a safety performance function (SPF), derived from the crash experience of numerous comparison sites. Once developed, this model is then weighted against the observed crash experience of the test site to more accurately predict the expected crashes. In this research, the procedure for the EB method, as documented in the *Highway Safety Manual* (AASHTO 2010) was used to conduct the analysis.

In the empirical Bayes method conducted in this research, SPFs were developed by the research team to predict crash frequencies. The development of SPFs involved locating and obtaining characteristics of 116 comparison approaches at intersection approaches in Illinois. The comparison approaches, much like the test sites in the before-modification period, had the traditional right-turn island design. Four years of crash data were obtained for the comparison sites from 2009 through 2012.

The geometric characteristics, crash history, and traffic volumes of the 116 comparison approaches were compiled and analyzed using the IBM SPSS statistical analysis software. Details of the traffic volume, crash data, and other characteristics for the comparison sites can be found in Appendix D. Assuming an underlying Poisson/negative binomial distribution, which is a common assumption in modeling traffic crashes per the *Highway Safety Manual* (AASHTO 2010), SPF models were then developed to predict crashes using variables that were found to have a statistically significant influence on crashes. As a part of this research, four SPF models were developed to predict total intersection crashes, intersection injury crashes, subject approach crashes, and right-turn crashes at the subject approach. Equations 6.1, 6.2, 6.3, and 6.4 show the form of the SPFs developed in this research, and Tables 6.5 shows the coefficients, standard error, and overdispersion factors for the SPFs developed.

$$\mu_{Total} = e^{\alpha} \times e^{\beta_1 * RTappADT} \times e^{\beta_2 * INTRSappADT} \tag{6.1}$$

$$\mu_{Injury} = e^{\alpha} \times e^{\beta_1 * RTappADT} \times e^{\beta_2 * INTRSappADT} \tag{6.2}$$

$$\mu_{SubjectApp} = e^{\alpha} \times e^{\beta_1 * RTappADT} \times e^{\beta_2 * RTappSpeedLimit} \tag{6.3}$$

$$\mu_{RT} = e^{\beta_1 * HeadTurnAngle} \times e^{\beta_2 * RT\ Radius} \tag{6.4}$$

where

μ_{Total} = Average annual expected total intersection crashes
μ_{Injury} = Average annual expected intersection injury (K, A, B, C) crashes
$\mu_{SubjectApp}$ = Average annual expected crashes at the subject approach
μ_{RT} = Average annual expected right-turn crashes at the subject approach

RTappAADT = AADT of the right-turn approach
INTRSappAADT = AADT of the intersecting through-approach
RTappSpeedLimit = Posted speed limit (mph) at the right-turn approach
HeadTurnAngle = Measured head-turn angle for right-turning vehicle at approach
RT Radius = Measured radius of right-turn lane (feet)
α = Regression model intercept
β_1, β_2 = Regression coefficients

Table 6.5 Variable Coefficients for Safety Performance Functions Developed

	Regression Coefficients			
	Intercept (α) (St. Error)	**Coefficient (β_1) (St. Error)**	**Coefficient (β_2) (St. Error)**	**Overdispersion Parameter, k (St. Error)**
Total Intersection Crashes	2.210 (0.0991)	5.249×10^{-5} (6.084×10^{-6})	4.020×10^{-5} (5.8085×10^{-6})	0.109 (0.0204)
Intersection Injury (K,A,B,C) Crashes	0.848 (0.1125)	4.570×10^{-5} (6.1203×10^{-6})	3.315×10^{-5} (6.2823×10^{-6})	0.029 (0.0242)
Subject Approach Crashes	1.253 (0.2834)	2.807×10^{-5} (6.8050×10^{-6})	0.018 (0.0069)	0.079 (0.0269)
Right-Turn Crashes at Subject Approach	NA	0.012 (0.010)	0.003 (0.012)	0.145 (0.0364)

NA = Not Available

The standard error of the coefficients helps measure the quality of an SPF and represents the ability of an SPF to predict crashes accurately. A small standard error indicates that the SPF predicts crashes accurately. As shown in Table 6.5, the standard errors are very small with values ranging from 0.000006 to 0.2834.

The overdispersion parameters were derived from the negative binomial modeling process. Traffic crashes are typically assumed to follow a Poisson distribution, where the mean and variance are equal. If the mean and variance of the crashes are not equal, the negative binomial model should be used to account for this overdispersion. Using the predicted crashes per year, the overdispersion factor, the weight factor, the expected number of after crashes without treatment can be predicted using the EB method, according to the specific steps listed below, per the *Highway Safety Manual* (AASHTO 2010).

Step 1—Calculate the predicted average crash frequency, $N_{predicted,\,B}$ for each site *i* using a SPF

Step 2—Calculate $N_{expected,\,B}$

$$N_{expected;\,B} = w_{i,B} N_{predicted,B} + (1 - w_{i,B}) N_{observed,B}$$

where the weight, $w_{i,B}$, for each site *i*, is determined as

$$w_{i,B} = \frac{1}{1 + k \times N_{predicted,B}}$$

and

$N_{expected}$ = Expected average crash frequency at site *i*

$N_{observed}$ = Observed crash frequency at site *i*

$N_{predicted}$ = Predicted crash frequency at site *i* from a safety performance function

k = Overdispersion parameter for the applicable SPF

A, B = After period, Before period, respectively

Step 3—Calculate the predicted average crash frequency, $N_{predicted,A}$ for each site *i* using a SPF

Step 4—Calculate r_i, an adjustment factor to account for differences between before- and after-modification periods in duration and traffic volume at each site *i*

$$r_i = \frac{\sum N_{predicted,A}}{\sum N_{predicted,B}}$$

Step 5—Calculate $N_{expected,A}$, the expected average crash frequency for each site *i* in the after-modification period without treatment

$$N_{expected,A} = N_{expected,B} \times r_i$$

Step 6—Calculate the crash modification factor (CMF), θ′, for all sites combined

$$\theta' = \frac{\sum_{All\ sites} N_{observed,A}}{\sum_{All\ sites} N_{expected,A}}$$

Step 7—Calculate the unbiased estimate of the CMF, θ

$$\theta = \frac{\theta'}{1 + \frac{Var\left(\sum_{All\ sites} N_{expected,A}\right)}{\left(\sum_{All\ sites} N_{expected,A}\right)^2}}$$

where

$$Var\left(\sum_{All\ sites} N_{expected,A}\right) = \sum_{All\ sites} \left[(r_i)^2 \times N_{expected,B} \times \left(1 - w_{i,B}\right)\right]$$

Step 8—Calculate the unbiased safety effectiveness, or unbiased percent reduction for all sites combined

$$Safety\ Effectiveness = Percent\ Reduction = 100 \times (1 - \theta)$$

Step 9—Calculate the variance of the unbiased crash modification factor, Var(θ)

$$Var(\theta) = \frac{(\theta')^2 \left[\frac{1}{\sum_{All\ sites} N_{observed,A}} + \frac{Var\left(\sum_{All\ sites} N_{expected,A}\right)}{\left(\sum_{All\ sites} N_{expected,A}\right)^2} \right]}{\left[1 + \frac{Var\left(\sum_{All\ sites} N_{expected,A}\right)}{\left(\sum_{All\ sites} N_{expected,A}\right)^2} \right]}$$

Step 10—Calculate the standard error of the unbiased crash modification factor, SE(θ), as the square root of its variance

$$SE(\theta) = \sqrt{Var(\theta)}$$

Step 11 – Assess the statistical significance of the estimated safety effectiveness by making comparisons with the following measure and criteria:

If $\left| \frac{Safety\ Effectiveness}{SE(Safety\ Effectiveness)} \right|$ > 2.0, conclude that the treatment effect is significant at the (approximate) 95% confidence level.

where SE(Safety Effectiveness) = 100 x SE(θ)

The effectiveness evaluation results and the crash reduction factors (CRF) based on the empirical Bayes methods are shown in Table 6.6. The observed crash reductions were tested for statistical significance using the Poisson test (Table 6.6) and according to Step 11 above (Table 6.7) at 95% LOC. Because traffic crash data are discrete and assumed to occur randomly, the Poisson test was used to test the significance of changes in crash frequencies. A one-tailed test was used at a 95% LOC and significance level (α) of 0.05 because it was hypothesized that traffic crash frequencies would reduce as a result of the implementation of the modified right-turn lane design. The null (H_o) and alternative (H_a) hypotheses used in the statistical analysis of the average/mean (μ) crash frequencies are as follows:

H_o: $\mu_{\text{after crashes}} = \mu_{\text{before crashes}}$

H_a: $\mu_{\text{after crashes}} < \mu_{\text{before crashes}}$

The Poisson charts of significance were used to determine the significance and corresponding p-values. For a one-tailed test, if the p-value is less than 0.05, then the null hypothesis is rejected and the finding is significant.

Table 6.6 Empirical Bayes Evaluation of Seven Test Sites

	Aggregated Annual Average Crash Frequency						Unbiased Safety Effectiveness*/ Crash Reduction Factor	Significant?** (p-value)
	Before-Modification Period			After-Modification Period				
	Observed	Predicted	Expected	Observed	Predicted	Expected		
Total Intersection Crashes (6 Test Intersections)	91.32	97.72	94.58	53.67	99.41	95.47	44.2%	Yes (< 0.01)
Intersection Injury (K, A, B, C) Crashes (6 Test Intersection)	21.34	22.72	22.60	13.00	23.05	22.93	43.6%	Yes (0.02)
Subject Approach Crashes (7 Test Approaches)	57.33	45.64	48.95	21.66	48.24	52.43	59.0%	Yes (< 0.01)
Right-Turn Crashes at Subject Approach (7 Test Approaches)	53.67	70.03	57.60	15.00	39.58	36.74	59.6%	Yes (< 0.01)

*Unbiased safety effectiveness = Unbiased percent reduction = Crash reduction factor.
**Based on Poisson test of crash frequencies at 95% LOC and significance level $\alpha = 0.05$.

As shown in Table 6.6, the EB method resulted in significant reductions in all four of the crash categories at the seven test approaches. Significant reductions in right-turn crashes at the subject approaches, which is the targeted crash type, of 59.6% was observed, as well as a significant reduction in crashes at the subject approaches (59.0%). The modified right-turn lane design implemented in the Peoria area provides significant safety improvements at approaches where installed.

Significant crash reductions were also observed for total intersection crashes (44.2%), and intersection injury crashes (43.6%). However, the intersection results should be viewed with caution because other improvements at two of the test sites were made, in addition to the subject right-turn approaches.

6.2.3 Crash Modification Factors

The expected countermeasure effectiveness is commonly expressed as a crash modification factor (CMF). A CMF is a multiplicative factor used to compute the expected number of crashes after implementing a given countermeasure at a specific site. The crash reduction factors (CRF) presented in Table 6.6, as the unbiased safety effectiveness, provide the percentage crash reduction that might be expected after implementing the modified right-turn lane design countermeasure.

Using the procedures outlined in the *Highway Safety Manual* (AASHTO 2010), CMFs were determined, per the empirical Bayes results for the four crash categories. The procedure and equations used to calculate the unbiased index of effectiveness (θ)—which is the CMF—as well as the variance and standard error, were presented in Section 6.2.2. The confidence interval on the CMF was also calculated. The 95% confidence interval is $\theta \pm Z_{\alpha/2} \times$ standard error, where $Z_{\alpha/2}$ is the two-tailed Z-statistic = 1.96. Table 6.7 provides the unbiased safety effectiveness (percent reduction), the unbiased CMFs, and the variance and standard error of the CMFs.

Table 6.7 Crash Modification Factors for Seven Test Sites

	Unbiased Safety Effectiveness	CMF (θ)	Variance of θ	Standard Error (SE) of θ	Standard Error of Safety Effectiveness	Ratio of Safety Effectiveness/ SE of (Safety Effectiveness)* and Significance?
Total Intersection Crashes (6 Test Intersections)	44.2%	0.558	0.0080	0.0895	8.95	4.93, Significant
Intersection Injury (K, A, B, C) Crashes (6 Test Intersection)	43.6%	0.564	0.0260	0.1614	16.14	2.70, Significant
Subject Approach Crashes (7 Test Approaches)	59.0%	0.410	0.0090	0.0949	9.49	6.21, Significant
Right-Turn Crashes at Subject Approach (7 Test Approaches)	59.6%	0.404	0.0127	0.1127	11.27	5.28, Significant

* If ratio > 2.0, conclude that the treatment effect is significant at the (approximate) 95% confidence level, according to Step 11.

In summary, the resulting crash modification factors along with their confidence intervals for the modified right-turn lane design are as follows:

- Total intersection crashes CMF = 0.558
    - 95% Confidence Interval = $0.558 \pm 1.96 \times 0.0895$ = 0.383 to 0.734
- Intersection Injury (K, A, B, C) Crashes CMF = 0.564
    - 95% Confidence Interval = $0.564 \pm 1.96 \times 0.1614$ = 0.248 to 0.881

The above two intersection CMFs should be viewed with caution because at two of the test sites, improvements were made to the other approaches, in addition to the subject right-turn approach.

- **Subject approach crashes CMF = 0.410**
    - **95% Confidence Interval = $0.410 \pm 1.96 \times 0.0.0949$ = 0.224 to 0.596**
- **Right-turn crashes at the subject approach CMF = 0.404**
    - **95% Confidence Interval = $0.404 \pm 1.96 \times 0.1127$ = 0.183 to 0.625**

These CMF can be used to estimate the expected safety benefits of installing the modified right-turn lane design countermeasure at locations throughout Illinois, as a part of benefit/cost analyses.

CHAPTER 7: ECONOMIC ANALYSIS

An analysis was conducted to determine the economic effectiveness of the modified right-turn lane design in the Peoria area using the equivalent uniform annual benefit/cost (EUAB and EUAC) method. Economic costs and benefits were calculated and annualized to determine the benefit/cost ratio of the modified design. The following assumptions were made:

- Economic life = 15 years
- Discount rate of 3% per IDOT's approved rates (IDOT 2015; Holland 2012)
- Additional annual maintenance cost required for the modified design = $0
- Salvage value = $0

The initial construction costs varied at each site because the extent of reconstruction to implement the modified right-turn lane improvement varied. At some test sites, in addition to the geometric redesign of the right-turn lanes, other line items were included in the cost, such as the relocation of traffic signals, pavement resurfacing, and median reconstruction/removal. Table 7.1 shows the initial cost to implement the modified right-turn lane design at each test site, as well as the date of construction. To evaluate economic benefits and costs accurately, all costs were converted to 2010 dollars. It is also important to note that the modified right-turn lane design does not add any new maintenance costs to the test sites because the modified design is not expected to incur any additional annual costs compared with the traditional design. Thus, the only cost component used in this economic analysis was the initial cost.

Table 7.1 Cost of Modified Right-Turn Lane Design and Geometric Improvements

Intersection Name	Initial Cost of Improvement	Year of Installation	Initial Cost in 2010 dollars
Prospect & Knoxville (IL-40)	$122,000	2006	$137,372
Northmoor & Knoxville (IL 40)	$105,000	2010	$105,000
Airport & IL-116	$112,000	2010	$112,000
Wesley & IL-29	$141,800	2010	$141,800
Douglas & IL-116	$109,270	2010	$109,270
Adams Ramp (NB & SB) & War Memorial	$81,360	2011	$78,992
		Total	**$684,434**

The present worth of costs in 2010 dollars for the modified right-turn lane design is calculated by converting the initial cost from the year of construction to the year 2010 using a discount rate of 3%. The present worth of costs for all seven test sites in 2010 dollars was found to be $684,434. To annualize this present value, converting it to an EUAC value, it is multiplied by the capital recovery factor (A/P) for i = 3% and n = 15 years, which is 0.0838. The resulting EUAC value is

EUAC = $57,356 per year

The benefits of the modified design are seen through their safety improvement and calculated by multiplying the reduction in crashes of a given severity (Fatal; Injury A, B, and C; and PDO) by the applicable crash costs. Crash costs include the monetary losses related to medical care, emergency services, property damage, lost productivity, etc. to society as a whole. The crash costs published in the *Highway Safety Manual* (AASHTO 2010) represent the crash costs in 2001 dollars. They were updated to 2010 dollars, as presented in Table 7.2. The procedure for updating the crash costs to current-year costs is outlined in Appendix A4 of the *Highway Safety Manual* and involves a two-step process using data from the U.S Bureau of Labor Statistics. In general, "the annual adjustment of crash costs uses federal economic indexes (such as the Consumer Price Index and Employment Cost Index) to account for the economic changes between the documented past year and the year of interest" (AASHTO 2010).

Comparing the "crash savings" between the annual average of three years before-modification data with the average annual three years after-modification data resulted in an annual average crash savings by severity (K, A, B, C, PDO) attributable to the modified design. Table 7.3 displays the average crash frequencies by crash severity in the before- and after-modification periods at seven test intersections.

Table 7.2 Societal Crash Costs by Crash Severity

Crash Severity Type	Comprehensive Societal Crash Cost (2001 dollars)	Comprehensive Societal Crash Costs (2010 dollars)
Fatal (K)	$4,008,900	$5,127,900
Injury A	$216,000	$273,200
Injury B	$79,000	$99,800
Injury C	$44,900	$56,400
PDO	$7,400	$9,200

Table 7.3 Annual Crash Frequency by Severity for the Seven Test Sites

	Average Annual Crashes	
	Before-Modification Period	After-Modification Period
Fatal Crashes (K)	0.0	0.0
Injury A Crashes	2.7	2.3
Injury B Crashes	7.7	6.7
Injury C Crashes	11.0	5.3
PDO Crashes	70.0	39.3

The annualized monetary benefits resulting from crash savings can be calculated by finding the difference in average annual crashes from the before- to after-modification periods for each severity class and then by multiplying this crash savings by the respective crash cost. The resulting value is the EUAB.

EUAB = (0.0 – 0.0) × $5,127,900 + (2.7 – 2.3) × $273,200 + (7.7 – 6.7) × $99,800 + (11.0 – 5.3) × $56,400 + (70.0 – 39.3) × $9,200

EUAB = $792,600 per year

Table 7.4 displays the equivalent uniform annual benefits and equivalent uniform annual costs of the modified design, as well as the resulting benefit/cost ratio.

Table 7.4 Resulting Annual Benefits and Costs of the Modified Design

Equivalent Uniform Annual Benefits	**$792,600**
Equivalent Uniform Annual Costs	**$57,356**
B/C Ratio	**13.8**

As shown in Table 7.4 the overall benefit to cost ratio of the modified design at the seven test sites is 13.8 to 1.0, which indicates that the accrued benefits in dollar value is exceeds the annualized cost of the modified design improvements over a period of 15 years by a factor of nearly 14. It should be noted that the economic analysis was performed on an intersection basis. It is expected that benefit to cost ratios calculated on an approach basis may even be higher because the crash reductions on an approach basis were greater.

CHAPTER 8: CRASH CAUSATION STUDY

A crash causation analysis was conducted in Illinois that focused on right-turn crashes at 116 intersection approaches throughout the state. The purpose of this study was to identify and quantify the geometric design variables that correlate with significantly more right-turn crashes and injuries. This research is expected to provide guidance on what types of right-turn locations would proactively benefit from a modified right-turn lane design approach.

Using the Illinois statewide computerized crash database, which is geographic information system (GIS) based, queries were made to identify intersection approaches that experienced a pattern of right-turn crashes. When compiling the database of intersection approaches to study, the initial determining factor was to be sites with frequencies exceeding the average right-turn crashes for the State of Illinois. It was found that the statewide average number of right-turn crashes was only 0.09 crashes per year per intersection. This was considered too low of a threshold for site selection because any site that experienced a crash during the analysis years would be considered. Therefore, a threshold of five right-turn crashes per approach, in any one of the four analysis years was selected as the minimum crash experience for inclusion in the analysis. This number was chosen because five crashes of a specific type is generally considered in pattern analyses of crashes; for example, the *Manual on Uniform Traffic Control Devices* (MUTCD) cites five correctable crashes as the threshold for justifying a traffic signal based on crash experience, per Warrant 7 (FHWA 2009). It should be noted that five crashes is typically not considered to be a high crash experience, yet having five crashes of a certain type (right-turn) provides a reasonable basis for correlation with geometric variables.

Figure 8.1 provides an example of the output from the statewide computerized GIS crash database to illustrate the methodology for the crash queries. The first step in identifying sites was to find all intersection crashes within the state (Figure 8.1a). Once this was determined, the location of the crashes was determined using the ROUTE, MILE, XCOORD, and YCOORD fields (Figure 8.1b). After the output was organized in this manner, all sites not meeting the threshold value of five crashes were not considered, while sites with crashes greater than or equal to the threshold were more closely examined for right-turn crashes, based on the recorded vehicle maneuvers (Figure 8.1c). Finally, the specific approach of the intersection involving the right-turn crash was identified as the direction of the vehicle turning right (Figure 8.1d); in the case of both vehicles turning right the direction was taken to be that of vehicle one if the directions differed. Through this process, approaches meeting the threshold value of at least five right-turn crashes in a year were selected.

The statewide computerized GIS crash data query resulted in a list of 68 unique approaches based on 2009 and 2010 crash data. The database was expanded to include another 58 unique approaches based on 2011 and 2012 crash data. These approaches had five or more right-turn crashes in at least one of the study years. The complete database includes all 116 unique approaches, along with 2009, 2010, 2011, and 2012 crash data for each approach.

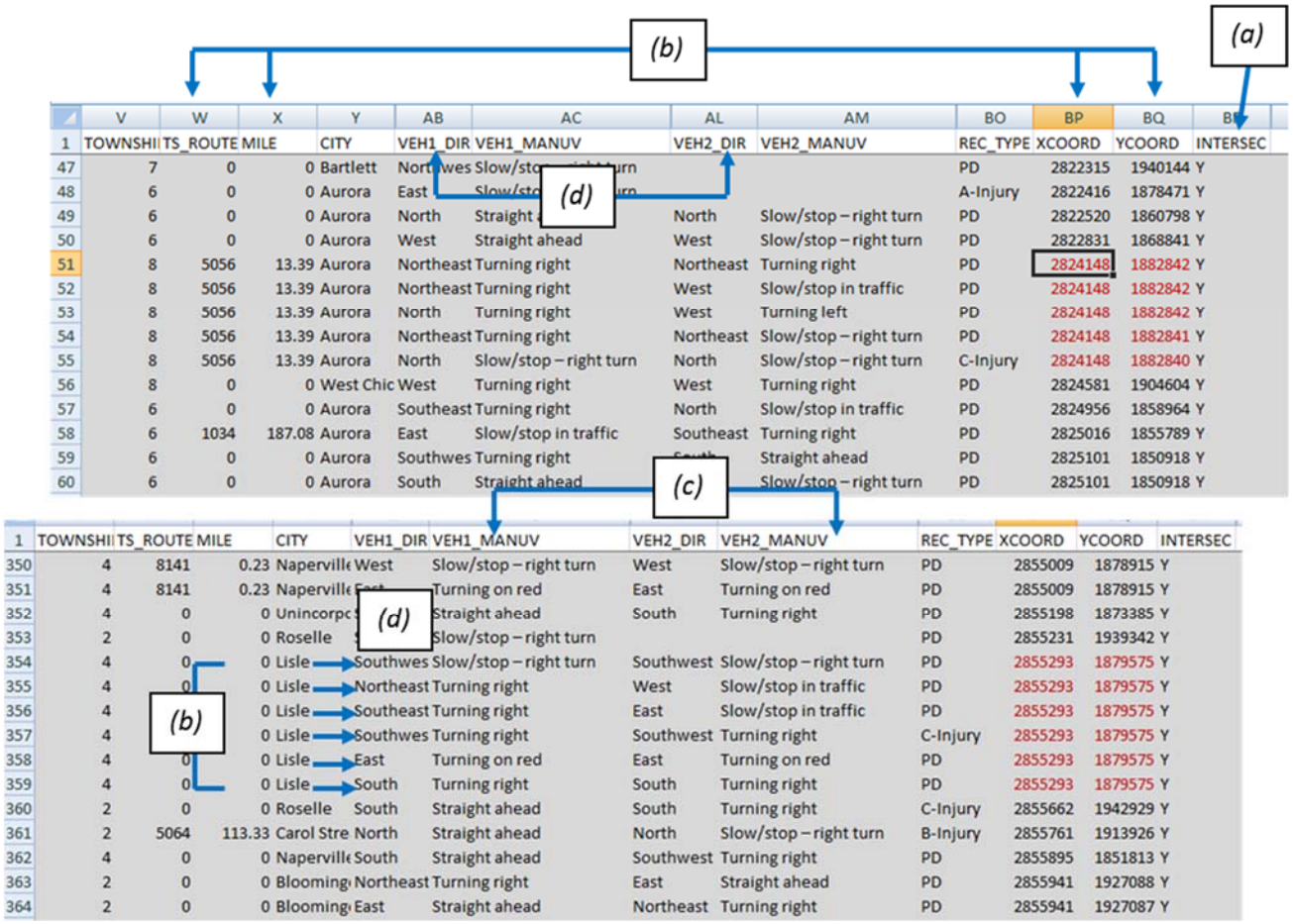


Figure 8.1 Example of computerized data output used in crash queries.

8.1 DATA COLLECTION

To gain an understanding of the correlation between traffic volume, geometric characteristics, and crash characteristics, an extensive database of the characteristics of the 116 study approaches was created.

8.1.1 Crash Data

After the 116 study approaches were identified, the results of the computerized crash data queries for right-turn crashes for the study approaches were verified by carefully reviewing the information contained in the traffic crash report forms. The traffic crash report forms filled out by police officers were downloaded from the IDOT crash report form image website and were analyzed to confirm crash frequencies and injuries from the computerized crash database output from GIS. Crash type, injury level, location information (intersection and approach), and crash frequencies were confirmed and updated for all intersections. Each narrative was carefully analyzed for all crashes at these intersections, and additional information was gathered for right-turn crashes, including vehicle maneuver prior to the crash for each vehicle, driver action for both drivers, and a description of the crash as reported in the narrative. This additional information helped in further categorizing the right-

turn crashes. Categories of right-turn crashes that were considered were right-turn rear-end, right-turn angle, right-turn sideswipe, and right-turn other.

The right-turn crash causation study is based on 3,174 right-turn crashes at 116 study approaches over a four-year period (2009 to 2012). Table 8.1 provides a summary of the aggregated crash data for all four years at all 116 sites, categorized by crash type.

Table 8.1 Summary by Crash Type and Severity for the 116 Study Approaches

Crash Type		2009	2010	2011	2012	Total
Right-turn rear-end						
	No.	734	770	789	691	**2,984**
	%	93.4%	93.6%	94.4%	94.8%	**94.0%**
Right-turn sideswipe						
	No.	34	34	32	25	**125**
	%	4.3%	4.1%	3.8%	3.4%	**3.9%**
Right-turn angle						
	No.	4	8	4	2	**18**
	%	0.5%	1.0%	0.5%	0.3%	**0.6%**
Right-turn other						
	No.	14	11	11	11	**47**
	%	1.8%	1.3%	1.3%	1.5%	**1.5%**
Total right-turn crashes						
		786	**823**	**836**	**729**	**3,174**
Right-turn injury crashes						
	No.	**161**	**166**	**132**	**110**	**569**
	%	**20.5%**	**20.2%**	**15.8%**	**15.1%**	**17.9%**

Most of the right-turn crashes that occurred at the study approaches were right-turn rear-end collisions (94.0%), the remaining crash types (right-turn sideswipe, right-turn angle, and right-turn other) accounted for only 6.0% of the right-turn crashes. Right-turn injury crashes represented nearly 18% of all right-turn crashes studied.

8.1.2 Geometric Data

Detailed information on the traffic and geometric characteristics of the 116 study approaches were collected for the following variables:

- Location information (county, city, street names, study right-turn approach)
- Operation of right-turn approach (one-way or two-way)
- Intersection control (signalized, stop-controlled)
- Laneage of subject road
- Intersection angle (measured by acute angle)

- Right-turn angle
- Head-turn angle
- AADT volumes for right-turn approach, intersecting through-approach, and total intersection
- Speed limit
- Right-turn channelization (presence or absence)
- Exclusive right-turn lanes

Nearly all characteristic information was gathered from online aerial map images and street views. The AADT volumes were obtained from IDOT's interactive AADT maps available online. Once collected, the data were entered into a database for analysis. The full database of traffic and geometric characteristics of the 116 study approaches can be found in Appendix D. Please note that these 116 approaches are the same 116 approaches used as the comparison sites discussed in Section 6.2.2.

Various studies and design guides were used to determine the critical geometric characteristics that may correlate with right-turn crashes, as well as their desirable limits or values where possible (Garcia and Belda-Esplugues 2007; ITE 2010), and are listed below:

- Location of the study right-turn approach—in the acute, right, or obtuse angle (Figure 8.2)
- Intersection angle > 75°—measured by the acute angle (Garcia and Belda-Esplugues 2007)
- Right-turn angle > 45°
- Head-turn angle of right-turning driver ≤ 140° (ITE 2010)
- Length of right-turn radius (≤ 100 feet assumed to be a small radius)
- Right-turn channelized islands (present or not)

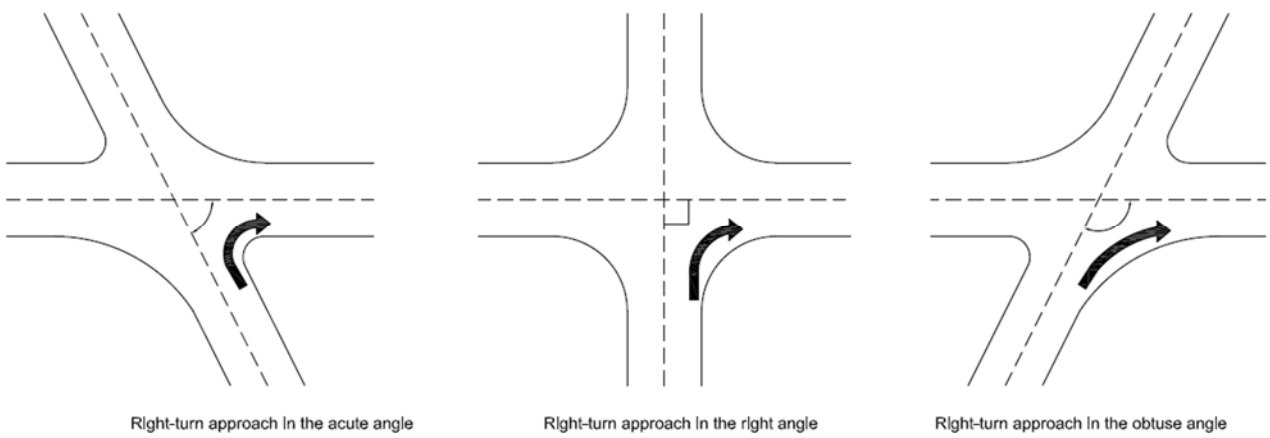


Figure 8.2 Right-turn approach locations within an intersection.

These characteristics were of special interest in the crash causation analysis, to examine whether they influence the safety performance of the approaches. It should also be noted that the right-turn angle was split into two groups at 45° based on the current practice of using the right-turn angle as half of the intersection angle; anything greater than this is considered desirable because it provides the right-turn vehicle with better range of vision of the intersection street both ways. Table 8.2 shows a breakdown of the approaches based on various geometric characteristics.

Table 8.2 Critical Geometric Characteristics of the 116 Right-Turn Study Approaches

Critical Geometric Characteristic	Category	Number of Approaches	%
Right-Turn Approach Location	Acute (< 85°)	16	13.8%
	Right (85°–95°)	65	56.0%
	Obtuse (> 95°)	35	30.2%
Intersection Angle	≥ 75° Desirable	85	73.3%
	< 75°	31	26.7%
Right-Turn Angle	≥ 45° Desirable	54	46.6%
	< 45°	62	53.4%
Head-Turn Angle	≤ 140° Desirable	90	77.6%
	> 140 °	26	22.4%
Right-Turn Radius	≤ 100 ft—smaller (desirable)	58	50.0%
	> 100 ft—larger	58	50.0%
Right-Turn Channelization	Channelized	99	85.3%
	Unchannelized	17	14.7%

8.2 CRASH CAUSATION DATA ANALYSIS AND RESULTS

The crash data were analyzed on a rate-related basis using two definitions of crash rate, including the traditional crash rate equation (8.1) and a slight variation of this equation (8.2).

$$\text{RT CR 1} = \frac{\text{C}*1{,}000{,}000}{\text{V}\times 365} \tag{8.1}$$

where

RT CR 1 = Right-turn crashes per million entering vehicles (MEV)
C = Annual average right-turn crashes
V = Intersection AADT

The second crash rate calculation used a form similar to Equation 8.1, but instead of V being the total intersection AADT, it consisted of the sum of two of the intersection approach AADTs—the right-turn approach AADT and the intersecting through-approach AADT. The unit of this second equation is "right-turn crashes per million entering approach and intersecting vehicles." Right-turn crash rates (RT CR 1 and RT CR 2) and right-turn injury crash rates (RT I-CR 1 and RT I-CR 2) were calculated using

the traditional crash rate equation (8.1) and the alternate equation (8.2) for the 116 study approaches, as shown in Table 8.3.

$$\text{RT CR 2} = \frac{C*1{,}000{,}000}{V' \times 365} \tag{8.2}$$

where

RT CR 2 = Right-turn crashes per million entering approach and intersecting vehicles (MEV – A + I)

C = Annual average right-turn crashes

V′ = Right-turn approach AADT and intersecting through-approach AADT

Table 8.3 Summary of Right-Turn Crash and Injury Rates for 116 Study Approaches

Approach No.	RT CR 1	RT CR 2	RT I-CR 1	RT I-CR 2	Approach No	RT CR 1	RT CR 2	RT I-CR 1	RT I-CR 2
1	0.47	0.96	0.11	0.22	**59**	0.47	0.97	0.07	0.16
2	0.36	0.77	0.03	0.07	**60**	0.32	0.63	0.05	0.10
3	0.33	0.66	0.06	0.11	**61**	0.23	0.43	0.04	0.08
4	0.40	0.90	0.04	0.09	**62**	1.92	4.20	0.17	0.38
5	0.15	0.26	0.01	0.02	**63**	0.41	0.82	0.05	0.11
6	0.99	1.81	0.36	0.66	**64**	0.46	0.85	0.06	0.11
7	0.37	0.68	0.19	0.35	**65**	0.63	1.06	0.03	0.06
8	0.21	0.50	0.07	0.17	**66**	0.25	0.50	0.03	0.06
9	0.17	0.32	0.01	0.01	**67**	0.58	1.18	0.06	0.12
10	0.30	0.62	0.02	0.04	**68**	0.48	0.99	0.15	0.30
11	0.83	1.38	0.08	0.13	**69**	0.27	0.55	0.07	0.14
12	0.13	0.26	0.04	0.07	**70**	0.67	1.33	0.04	0.09
13	0.99	2.00	0.30	0.60	**71**	0.95	1.53	0.22	0.35
14	0.31	0.63	0.06	0.13	**72**	1.04	1.87	0.24	0.43
15	0.39	0.76	0.21	0.41	**73**	0.14	0.27	0.02	0.04
16	0.78	1.55	0.19	0.37	**74**	0.20	0.39	0.02	0.04
17	0.63	1.39	0.12	0.26	**75**	0.19	0.37	0.04	0.09
18	0.51	1.00	0.17	0.33	**76**	0.45	0.91	0.08	0.17
19	0.37	0.67	0.06	0.12	**77**	1.38	2.59	0.27	0.50
20	0.55	0.94	0.08	0.15	**78**	1.22	2.59	0.22	0.47
21	1.08	1.87	0.19	0.34	**79**	1.16	2.15	0.17	0.32
22	0.66	1.57	0.07	0.16	**80**	0.26	0.50	0.03	0.06
23	0.48	0.74	0.09	0.14	**81**	0.24	0.48	0.05	0.09
24	0.47	0.76	0.00	0.00	**82**	0.35	0.62	0.05	0.09
25	0.46	0.90	0.05	0.10	**83**	1.60	2.86	0.43	0.77
26	1.16	1.84	0.17	0.27	**84**	0.33	0.62	0.03	0.05
27	0.28	0.52	0.04	0.07	**85**	1.55	2.64	0.29	0.50

28	1.14	2.07	0.17	0.31	**86**	0.14	0.25	0.02	0.04
29	0.84	1.55	0.18	0.34	**87**	0.33	0.56	0.04	0.06
30	0.53	0.97	0.12	0.22	**88**	1.72	3.95	0.26	0.60
31	1.67	2.83	0.32	0.54	**89**	0.92	1.75	0.20	0.38
32	0.41	0.83	0.11	0.22	**90**	0.48	0.90	0.16	0.30
33	0.53	1.23	0.13	0.29	**91**	0.28	0.54	0.05	0.09
34	0.35	0.63	0.04	0.08	**92**	0.20	0.36	0.04	0.08
35	1.58	2.50	0.29	0.46	**93**	0.50	0.93	0.07	0.13
36	0.74	1.19	0.19	0.31	**94**	0.26	0.47	0.03	0.06
37	0.64	1.41	0.04	0.10	**95**	1.20	2.24	0.17	0.32
38	0.81	1.54	0.10	0.19	**96**	0.96	1.73	0.25	0.45
39	1.64	3.03	0.18	0.33	**97**	0.71	1.23	0.06	0.11
40	0.30	0.59	0.01	0.02	**98**	0.26	0.51	0.09	0.17
41	0.35	0.69	0.05	0.10	**99**	0.24	0.50	0.05	0.11
42	1.63	3.10	0.14	0.26	**100**	0.37	0.73	0.07	0.13
43	0.35	0.69	0.09	0.17	**101**	0.52	1.10	0.10	0.22
44	0.60	1.20	0.22	0.44	**102**	0.67	1.30	0.06	0.12
45	0.41	0.79	0.03	0.05	**103**	0.75	1.60	0.19	0.40
46	0.13	0.30	0.03	0.07	**104**	0.21	0.43	0.00	0.00
47	0.10	0.20	0.04	0.09	**105**	0.57	1.12	0.14	0.28
48	0.17	0.34	0.01	0.02	**106**	0.63	1.20	0.12	0.23
49	0.19	0.38	0.04	0.08	**107**	0.52	0.94	0.04	0.07
50	0.24	0.56	0.03	0.08	**108**	0.30	0.45	0.05	0.08
51	0.16	0.28	0.06	0.11	**109**	0.57	1.32	0.05	0.13
52	0.11	0.23	0.01	0.01	**110**	0.60	1.08	0.14	0.26
53	0.68	1.01	0.08	0.11	**111**	0.33	0.67	0.02	0.05
54	0.59	1.21	0.11	0.23	**112**	0.72	1.28	0.14	0.26
55	0.25	0.50	0.09	0.19	**113**	0.67	1.44	0.10	0.21
56	0.33	0.61	0.06	0.11	**114**	0.29	0.53	0.02	0.04
57	0.22	0.44	0.03	0.06	**115**	0.26	0.49	0.02	0.04
58	0.25	0.42	0.07	0.12	**116**	0.35	0.74	0.02	0.05

RT CR 1: Right-turn crash rate per MEV (total intersection AADT).
RT CR 2: Right-turn crash rate per MEV – I + A (right-turn and intersecting through approach AADTs).
RT I-CR 1: Right-turn injury crash rate per MEV (total intersection AADT).
RT I-CR 2: Right-turn injury crash rate per MEV – I + A (right-turn and intersecting through approach AADTs).

Once the crash and injury rates were calculated for the 116 approaches, statistical analyses were performed to determine whether there was a significant difference in crashes among a series of two or three comparison groups for the following variables:

- Right-turn approach location (acute quadrant vs. 90° intersection angle vs. obtuse quadrant)
- Intersection angle (intersection angle ≥ 75° vs. intersection angle < 75°)
- Right-turn angle (right-turn angle ≥ 45° vs. right-turn angle < 45°)

- Head-turn angle (head-turn angle ≤ 140° vs. head-turn angle > 140°)
- Length of right-turn radius (radius ≤ 100 feet vs. radius > 100 feet)
- Channelized right-turn approach (channelized vs. unchannelized)

Statistical analyses were conducted using the Wilcoxon rank sum and Kruskal-Wallis tests at 95% LOC and significance level, α, = 0.05. Specifically, one-tail tests were used because a direction in the measures of effectiveness (right-turn crashes) was hypothesized based on past research.

The Wilcoxon rank sum test, also called the Mann-Whitney test, is a non-parametric test for two independent datasets where the populations are not normal. This test is described in statistical textbooks (Gilbert 1987; Hollander and Wolfe 1999; Conover 1999) and is judged to be appropriate for analysis of the datasets presented in this research. The null and alternative hypotheses are as follows:

H_o: The populations from which the two datasets have been drawn have the same mean

H_a: One population has a mean smaller or larger than the other population mean

The hypothesis test consists of computing a sample statistic calculated from a ranking of all the data compared with a critical test statistic. The two sets are combined and a rank (1, 2, …, $n_1+n_2 = m$) is assigned to each data value in ascending order. The sum of the ranks assigned to the n_1 measurements for X_1 to be W_{rs}. If $n_1 \leq 10$, and $n_2 \leq 10$ is defined, then reference tables (Gilbert 1987; Hollander and Wolfe 1999) are available to compare W_{rs} with a critical value. For large sample sets, the sample statistic below may be used.

$$Z_{rs} = \frac{W_{rs} - n_1\frac{(m+1)}{2}}{\sqrt{n_1 n_2 (m+\frac{1}{12})}} \quad n_1 n_2 \geq 10 \tag{8.2}$$

where

Z_{rs} = calculated test statistic
W_{rs} = Wilcoxon rank sum
n_1 = sample size of group 1
n_2= sample size of group 2
$m = n_1 + n_2$

For a α-level one-tailed test, the null hypothesis, H_o, is rejected and H_a accepted if $|Z_{rs}| \geq |Z_{1-\alpha}|$ where $Z_{1-\alpha}$ is the value for the random variable corresponding to 1-α according to the cumulative standard normal distribution. In other words, the null hypothesis is rejected when the calculated test statistic exceeds the critical value of the test statistic of 1.645. The Z_{rs}-values were calculated and the corresponding critical Z_{rs} value obtained.

The Kruskal-Wallis test is an extension of the Wilcoxon rank sum test for more than two independent datasets (Gilbert 1987). This test was used to compare the dataset based on the three right-turn approach locations (acute, 90°, or obtuse). The hypothesis test is as follows:

H_o: The populations from which the *j* number of datasets have been drawn have the same mean

H_a: At least one population has a mean smaller or larger than at least one other population

The hypothesis test consists of computing a sample statistic calculated from a ranking of all the data compared with a critical test statistic. The three sets are combined and a rank (1,2, …, $n_1+n_2 = m$) is assigned to each data value in ascending order. The sum of the ranks assigned to the data in the *j*th set is defined as R_j. The Kruskal-Wallis statistic (Kw) is computed using Equation 8.3. For populations with ties (multiple data points with the same value), a modified Kruskal-Wallis statistic (Kw') is computed using Equation 8.4 (Gilbert 1987).

$$K_w = \left[\frac{12}{m(m+1)}\sum_{j=1}^{k}\frac{R_j^2}{n_j}\right] - 3(m+1) \tag{8.3}$$

$$K_w' = \frac{K_w}{1-\frac{1}{m(m^2-1)}\sum_{j=1}^{g} t_j\left(t_j^2-1\right)} \tag{8.4}$$

where

K_w = calculated test statistic
K_w' = Wilcoxon rank sum
n_j = sample size of group j
$m = n_1 + n_2 + \ldots + n_j$
R_j = sum of ranks in group j
g = number of tied data groups
t_j = number of ties in data group j

For a α-level one-tailed test, the null hypothesis, H_o, is rejected and H_a accepted if $K_w' \geq \chi^2_{1-\alpha,\ k-1}$ where $\chi^2_{1-\alpha,\ k-1}$ is the 1-α quantile of the chi-square distribution with k-1 degrees of freedom. For 95% LOC, α = 0.05, and degrees of freedom = 3, the critical value for χ^2 is 5.991. In other words, the null hypothesis is rejected when the calculated test statistic exceeds the critical value of the test statistic of 5.991. The results of the statistical analyses are summarized in Table 8.4. The Wilcoxon rank sum test was used for intersection angle, right-turn angle, head-turn angle, right-turn radius, and right-turn channelization comparisons, whereas, the Kruskal-Wallis test was used for right-turn location comparisons.

Table 8.4 Statistical Analysis for Right-Turn Crash Rates and Right-Turn Injury Crash Rates

(a) Right-Turn Crash Rates

Analysis Scenario	n	Mean Crash Rate CR 1	Calculated Statistic	Significant?* (p-value)	Mean Crash Rate CR 2	Calculated Statistic	Significant?* (p-value)
RT Location Acute (< 85°)	16	0.532	2.767	No (0.429)	1.046	2.955	No (0.399)
RT Location Right (85°–95°)	65	0.545			1.036		
RT Location Obtuse (> 95°)	35	0.631			1.192		
INT ≥ 75°	85	0.545	–0.951	No (0.171)	1.047	–1.089	No (0.138)
INT < 75°	31	0.637			1.187		
RT ≥ 45°	54	0.483	**–2.297**	**Yes (0.011)**	0.906	**–2.480**	**Yes (0.007)**
RT < 45°	62	0.644			1.240		
HT ≤ 140°	90	0.537	**–1.940**	**Yes (0.026)**	1.018	**–1.920**	**Yes (0.027)**
HT > 140°	26	0.680			1.314		
RT Radius ≤ 100 ft	58	0.547	–0.536	No (0.296)	1.027	–0.420	No (0.337)
RT Radius > 100 ft	58	0.591			1.142		
Channelized RT	99	0.589	0.925	No (0.177)	1.125	1.058	No (0.145)
Unchannelized RT	17	0.452			0.846		

(b) Right-Turn Injury Crash Rates

Analysis Scenario	n	Mean Crash Rate I-CR 1	Calculated Statistic	Significant?* (p-value)	Mean Crash Rate I-CR 2	Calculated Statistic	Significant?* (p-value)
RT Location Acute (< 85°)	16	0.083	4.275	No (0.233)	0.162	4.321	No (0.229)
RT Location Right (85°–95°)	65	0.095			0.180		
RT Location Obtuse (> 95°)	35	0.120			0.227		
INT ≥ 75°	85	0.092	**–2.037**	**Yes (0.021)**	0.177	**–2.015**	**Yes (0.022)**
INT < 75°	31	0.124			0.230		
RT ≥ 45°	54	0.074	**–3.113**	**Yes (0.001)**	0.139	**–3.285**	**Yes (0.001)**
RT < 45°	62	0.124			0.237		
HT ≤ 140°	90	0.092	**–2.582**	**Yes (0.005)**	0.173	**–2.612**	**Yes (0.005)**
HT > 140°	26	0.133			0.256		
RT Radius ≤ 100 ft	58	0.102	0.375	No (0.354)	0.192	0.400	No (0.345)
RT Radius > 100 ft	58	0.100			0.191		
Channelized RT	99	0.105	0.937	No (0.174)	0.200	1.081	No (0.140)
Unchannelized RT	17	0.076			0.142		

*Denotes statistically significant difference at ≥ 95% LOC using the Wilcoxon rank sum test or the Kruskal-Wallis test.

When comparing these site characteristics with respect to the average right-turn crash rate (CR 1 and CR 2), the head-turn angle and the right-turn angle were found to be significant, which can be seen in Table 8.4a. The right-turn approaches with smaller right-turn angles (< 45°) had statistically higher crash rates than those approaches with larger right-turn angles. In addition, the right-turn approaches with excessive head-turn angles required by drivers (> 140°) had statistically higher crash rates than those approaches with smaller head-turn angles.

Further, when comparing the site characteristics with respect to the average right-turn injury crash rate (I-CR 1 and I-CR 2), three of the four geometric characteristics were found to be significant, as shown in Table 8.4b. It is seen that the right-turn injury rates were significantly higher for all intersections with the following characteristics: acute intersection skew angle less than 75°, right-turn angles less than 45°, and head-turn angles greater than 140°.

8.4 REGRESSION MODELING

Additional analyses were conducted to further investigate the relationships, and strength of the relationships, between crashes and traffic volume and/or geometric parameters at the 116 study sites. These analyses included (1) conducting a regression analysis using a negative binomial underlying distribution and (2) performing a Monte Carlo simulation to quantify the relative strength of the parameters within the developed SPF regression equations.

8.4.1 Relationship Between Right-Turn Crashes and Geometric Variables

Correlations between right-turn crashes and the geometric characteristics and traffic volume data were investigated through regression analyses using a negative binomial underlying distribution. The IBM SPSS software was used to determine which variables were statistically significant in the regression analyses. Initially, a regression model with all available variables—right-turn approach location, intersection angle, right-turn angle, head-turn angle, right-turn radius, channelization, speed limit, right-turn-approach ADT, intersecting through-ADT, and total intersection ADT—was developed. However, all the variable coefficients were not significant. Thus, the least statistically significant variable was removed and the regression model was developed again. This process was repeated until only statistically significant ($p < 0.05$) variables and coefficients resulted. To ensure reliability in the regression model, the removal process was replicated in different orders several times. After testing each variable, if none were found to be significant, the whole process was repeated using regression models without an intercept. It was then found that head-turn angle and right-turn radius had a statistically significant impact on right-turn crashes, as shown earlier in Equation 6.4 in Section 6.2 of this report.

8.4.2 Relative Impact of Parameters in SPF Regression Equations

The four SPF models, developed in this study as presented in Section 6.2, represent an important advancement with respect to predicting the number of crashes, under known intersection-specific conditions. There is value in analyzing the relative contribution of each of the regression coefficients (for each SPF model) on the number of cashes. Thus, a Monte Carlo simulation was used to analyze the relative contribution/strength of the parameters used to predict crashes per the four SPF regression equations developed.

Condition numbers (CN) can be deployed to achieve this objective. The condition number analysis method is essentially a sensitivity analysis technique to evaluate the impact of the various regression coefficients on the dependent variable. The condition number analysis begins with the regression equation or the SPF model. For each regression coefficient in a particular regression equation the condition number is developed using the general formula: $CN\beta_x = \beta_x * f'(x)/f(x)$, where $f'(x)$ is the partial derivative.

As an example, the SPF model given by Equation 6.1 presented earlier is selected.

$$\mu_{Total} = e^{\alpha} \times e^{\beta_1 * RTappADT} \times e^{\beta_2 * INTRSappADT}$$

Equation 6.1 has two regression coefficients or parameters (β_1, and β_2) directly associated with two independent variables (RTappADT and INTRSappADT), which are used to predict the average annual expected total intersection crashes. The third parameter, α, yields a value of 1.000 for the condition number.

$$CN_{\alpha} = \left(\frac{\alpha}{\mu_{TOTAL}}\right)\left(\frac{\partial \mu_{TOTAL}}{\partial \alpha}\right) = 1.000$$

No further analysis is required for α because this value (i.e., 1.000) remains a constant for all values of the regression coefficient. Additionally, it is perhaps more important to evaluate the impact of the coefficients β_1 and β_2 because they are linked directly to variables that impact the number of crashes. Condition numbers for the other two parameters are defined as

$$CN_{\beta 1} = \left(\frac{\beta_1}{\mu_{TOTAL}}\right)\left(\frac{\partial \mu_{TOTAL}}{\partial \beta_1}\right) = (\beta_1)(x_1)$$

$$CN_{\beta 2} = \left(\frac{\beta_2}{\mu_{TOTAL}}\right)\left(\frac{\partial \mu_{TOTAL}}{\partial \beta_2}\right) = (\beta_2)(x_2)$$

A Monte Carlo simulation was then used to analyze the relative contribution of the two parameters on the total number of crashes. To conduct the condition number analysis, the average values of the variables RTappAADT and INTRSappAADT were estimated using available data from the 116 comparison approaches in this study. The averages for the other variables were also estimated for other condition number analyses for equations 6.1, 6.3, and 6.4. The average values used in the condition number analysis were RT radius (feet) = 106.7; head-turn angle (degrees) = 131.6; speed (RT App) = 40.5; RT App ADT = 7680.0; Int APP ADT = 12740.0.

The range of each parameter (e.g., β_1 and β_2 for Equation 6.1) was allowed to vary between 1×10^{-5} and 9×10^{-5}. It is noted here that the user can restrict or expand this range, based on the actual value of coefficients. The Monte Carlo simulation was performed 1000 times to estimate the *relative impact and contribution* of each regression coefficient (e.g., β_1 and β_2 for Equation 6.1) on the total number of crashes. Once the condition numbers were obtained from the Monte Carlo simulation, the average condition number for each parameter was calculated and expressed as a percentage. For Equation 6.1, the Monte Carlo simulation indicated that the range of condition number values for

1000 trials for b_1 was 7.7×10^{-5} to 6.9×10^{-4} (average = 3.9×10^{-4}) and the condition number for β_2 ranged from 4.1×10^{-5} to 3.6×10^{-6} (average = 2.1×10^{-6}). This suggested that while both β_1 and β_2 are the most important parameters for this equation, the impact of β_1 is at least one order of magnitude higher compared with β_2. Similarly, the condition number analysis for the other equations was conducted using the Monte Carlo–based approach described above. These results are summarized in Table 8.5.

Table 8.5. Relative Impact of Parameters in the SPF Regression Equations

	Condition Numbers (CN)			Relative Impact
	Maximum	**Minimum**	**Average**	
Equation for Predicting Total Intersection Crashes $\mu_{Total} = e^{\alpha} \times e^{\beta_1 * RTappADT} \times e^{\beta_2 * INTRSappADT}$ (6.1)				
β_1 Right-Turn-Approach ADT	6.9×10^{-4}	7.7×10^{-5}	3.9×10^{-4}	**99%**
β_2 Intersecting Through-Approach ADT	4.1×10^{-4}	3.6×10^{-5}	2.1×10^{-4}	**1%**
Equation for Predicting Total Intersection Injury Crashes $\mu_{Injury} = e^{\alpha} \times e^{\beta_1 * RTappADT} \times e^{\beta_2 * INTRSappADT}$ (6.2)				
β_1 Right-Turn-Approach ADT	1.2×10^{-5}	1.3×10^{-6}	6.6×10^{-6}	**55%**
β_2 Intersecting Through-Approach ADT	9.6×10^{-6}	1.1×10^{-5}	5.3×10^{-6}	**45%**
Equation for Predicting Subject Approach Crashes $\mu_{SubjectApp} = e^{\alpha} \times e^{\beta_1 * RTappADT} \times e^{\beta_2 * RTappSpeedLimit}$ (6.3)				
β_1 Right-Turn-Approach ADT	6.9×10^{-4}	7.7×10^{-5}	3.7×10^{-4}	**37%**
β_2 Speed Limit of Right-Turn Approach	1.2×10^{-3}	1.3×10^{-5}	6.3×10^{-4}	**63%**
Equation for Predicting Right-Turn Crashes at Subject Approach $\mu_{RT} = e^{\beta_1 * HeadTurnAngle} \times e^{\beta_2 * RT\ Radius}$ (6.4)				
β_1 Head-Turn Angle at Right-Turn Approach	6.9×10^{-4}	7.8×10^{-5}	3.8×10^{-4}	**37%**
β_2 Right-Turn Radius	1.2×10^{-3}	1.3×10^{-5}	6.4×10^{-4}	**63%**

Based on the results, one may conclude that for Equation 6.1, right-turn approach ADT has more than an order of magnitude larger impact on total intersection crashes compared with intersecting through-approach ADT. For Equation 6.2, the impact of right-turn approach ADT is higher than the impact of intersecting through-approach ADT (55% to 45%) on total intersection injury (K, A, B, C) crashes. For Equation 6.3, the right-turn approach ADT coefficient is lower than the impact of speed limit at the right-turn approach (37% to 63%) on crashes at the subject approach. For Equation 6.4, head-turn angle at the right-turn approach is also lower than the impact of right-turn radius (37% to 63%) on right-turn crashes at the subject approach.

It is emphasized here that the above analyses show the *relative impact* of the different regression coefficients (e.g., α, β_1, β_2) on the dependent variable (e.g., number of crashes). This method is not intended as a replacement to developing the appropriate regression equation(s) using standard sensitivity methods such as a Bayesian analysis using the negative binomial, Poisson, or other applicable distributions In fact, these methods are necessary to develop the regression equations, which can then further be analyzed using the condition number analysis presented here. It is noted that additional data and accompanying analyses are needed to verify these initial findings.

CHAPTER 9: CONCLUSIONS AND RECOMMENDATIONS

From 2006 to 2014, IDOT implemented modifications to ten right-turn approaches in the Peoria area. The changes were intended to improve the line of sight of right-turning passenger vehicles by reducing the skew of the approach angle, while allowing semi-tractor trailer trucks to make right turns without encroachment. Bradley University researchers were retained to conduct a two-part study in which they (1) performed site-specific effectiveness evaluations of the modified sites and (2) conducted an area-wide crash causation analysis of right-turn crashes. In the site-specific evaluation, driver behavior analyses and traffic crash-based safety analyses were performed to assess the safety performance of the modified right-turn lane design. The results of the crash causation analysis were used to identify the geometric variables that influence right-turn crashes in order to offer recommendations on what types of sites (based on their characteristics) would make good candidates for installation of the modified right-turn lane design in Illinois.

9.1 DRIVER BEHAVIOR ANALYSIS

Field observations of driver behavior at right-turn lanes at ten test sites and ten paired control sites were collected to assess the modified right-turn lane design. The control sites were selected based on their similarity to the respective test site's pre-construction (before modification) characteristics in terms of intersection control, subject road laneage, intersection angle, right-turn angle, head-turn angle, right-turn radius, AADT, and other characteristics.

To observe and quantify changes in driver behavior at the approaches with the modified right-turn lane design, 160 hours of field data were collected at the test and control sites, and nearly 19,000 vehicles were observed. The field data consist of four main variables: driver head-turn behavior, lateral placement in the turn lane, stop type, and stop location.

The test and control comparisons revealed that drivers traveling through the modified right-turn lane design (test sites) demonstrated safer driving behaviors compared with drivers at the control sites with the traditional design. Specifically, compared with the control sites (traditional design), drivers at the test sites with the modified design

- Used fewer exaggerated head turns → improved line of sight when checking for upstream traffic to the left
- Used fewer roll-and-go stops → minimized opportunity for traffic conflicts
- Stopped on or before the stop bar more frequently → drivers are able to stop at the proper location with adequate sight distance

Because drivers at the modified right-turn lane design use fewer exaggerated head turns, roll-and-go stops, and stops past the stop bar, they will travel at slower speeds, have an improved line of sight, and approach the right turn with less skew.

Before- and after-modification comparisons of driver behavior from the field investigations were available at only three test sites because of the dates of construction. However, the results were

similar to the test and control comparisons for three of the four critical variables and provide support for the comparative test and control results.

9.2 SAFETY EVALUATION AND CRASH MODIFICATION FACTORS

The traffic crash–based evaluation of seven test approaches with three years of before- and three years of after-modification data provides insight into the safety impact of the modified right-turn lane design. The results of the empirical Bayes (EB) method indicated statistically significant reductions in total intersection crashes (44.2%), intersection injury crashes (43.6%), subject approach crashes (59.0%), and right-turn crashes at the subject approaches (59.6%) at a 95% level of confidence.

Additional analyses were conducted to determine the impact of the modified design on two subsets of drivers, older (age 65+) and younger (age 16 to 21 years). The findings from the older- and younger-driver analyses were based on the naïve before and after method because the needed safety performance functions (SPFs) for the EB method are not available for those age categories. The following are the statistically significant reductions observed:

- Older-driver crash analysis revealed a 70% significant reduction in right-turn crashes at the subject approach.
- Younger-driver crash analysis revealed significant reductions of 43% for intersection crashes, 63% for approach crashes, and 66% for right-turn-related crashes at the subject approach.

Using the procedures outlined in the *Highway Safety Manual* (AASHTO 2010), crash modification factors (CMFs) were developed for each crash type, based on the empirical Bayes results. The resulting CMFs, along with their confidence intervals (CI), are as follows:

- **Subject approach crashes CMF = 0.410** with 95% CI = 0.224 to 0.596
- **Right-turn crashes at the subject approach CMF = 0.404** with 95% CI = 0.183 to 0.625

Overall, the safety evaluation provides evidence that modifying right-turn lanes on state routes to improve the approach angle of right-turn vehicles improves safety by reducing the frequency of right-turn crashes at the approaches, as studied in the Peoria area.

An analysis was conducted to determine the economic effectiveness of the modified right-turn lane design at seven test sites in the Peoria area by comparing the equivalent uniform annual benefits and costs (EUAB and EUAC). Economic costs and benefits (in 2010 dollars) of the modified design were calculated and annualized to determine the benefit to cost ratio. The resulting benefit to cost ratio was 13.8 to 1.0, which indicates that the accrued benefits of traffic crash reductions in dollar value exceeds the annualized cost of the modified right-turn lane design over a period of 15 years by a factor of nearly 14.

9.3 CRASH CAUSATION ANALYSIS

In addition to the effectiveness evaluation of the test sites in Peoria, an area-wide investigation of right-turn crash causation was conducted. In total, 116 right-turn approaches with 3,174 right-turn crashes over a 4-year period were included in this analysis. Crash rates for right-turn crashes and

injury crashes were calculated using the traditional definition of crash rate (crashes per million entering vehicles) and an alternate one (crashes per million entering right-turn approach and intersecting approach vehicles). This resulted in four crash rates, with two rates for right-turn crashes and two for right-turn injury crashes for each of the 116 study approaches.

Once the crash and injury rates were calculated, statistical analyses were performed to determine whether there was a significant difference in crashes among a series of two or three comparison groups for the following variables at 95% LOC:

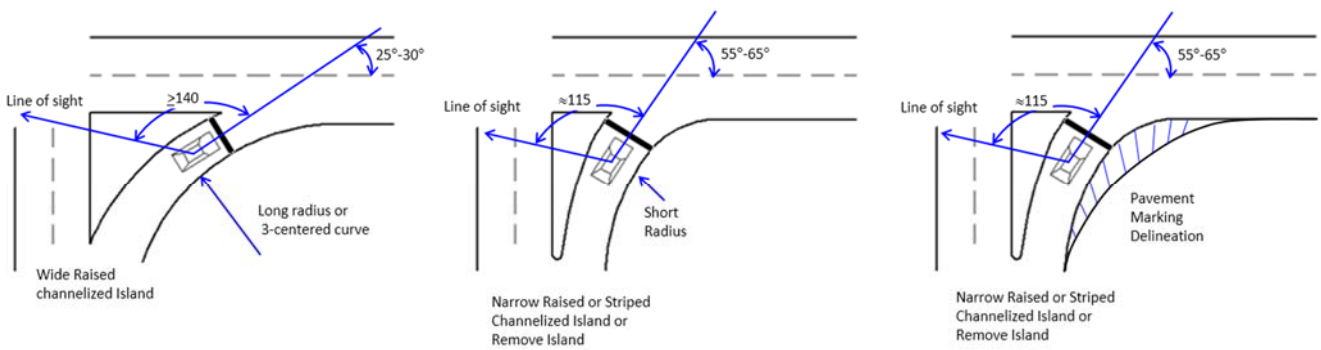
- Right-turn approach location (acute quadrant vs. 90° intersection angle vs. obtuse quadrant)
- Intersection angle (intersection angle ≥ 75° vs. intersection angle < 75°)
- Right-turn angle (right-turn angle ≥ 45° vs. right-turn angle < 45°)
- Head-turn angle (head-turn angle ≤ 140° vs. head-turn angle > 140°)
- Length of right-turn radius (radius ≤ 100 feet vs. radius > 100 feet)
- Channelized right-turn approach (channelized vs. unchannelized)

Based on the analysis of right-turn crashes, two characteristics were found to be significant from these comparisons. Approaches with right-turn angles less than 45° and head-turn angles greater than 140° were associated with significantly higher crash rates. Based on the analysis of right-turn injury crashes, three of the geometric variables were found to be significant. Right-turn injury crash rates were significantly higher for approaches with intersection angles less than 75°, right-turn angles less than 45°, and head-turn angles greater than 140°.

Correlations between right-turn crashes and the geometric characteristics and traffic volume data were investigated through regression analyses using a negative binomial underlying distribution. It was found that head-turn angle and right-turn radius had a statistically significant impact on right-turn crashes. The results of a Monte Carlo simulation conducted to quantify the relative strength of the parameters within the developed SPF regression equations revealed that the following variables have a stronger relationship with right crashes: right-turn approach ADT, right-turn radius, and right-turn approach speed.

9.4 RECOMMENDATIONS AND DESIGN FLEXIBILITY

The overall findings of the study, including both the driver behavior and safety evaluations, are evidence that the modified right-turn lane design has significant safety benefits. The modified design included sharpening the flat approach angle typical in the traditional designs, reducing the radius, adjusting the stop bar position, and modifying the corner island to improve safety by increasing the line of sight of approaching through traffic. At some locations, the corner island was removed or redesigned to achieve the geometric improvements. At other locations, additional pavement markings were placed around the radius to help delineate a better approach for passenger cars, while also allowing large trucks to complete the turn without striking the curb. Figure 9.1 shows a generic sketch of the traditional design and two options of the modified right-turn lane design for a 90° intersection. When implementing the modified design at skewed intersections, dimensions different from those depicted may be necessary, based on site-specific geometric conditions. Please note that the modified design is not a standardized (cookie-cutter) design; site specific customization may be needed for urban and rural applications based on engineering judgement. For example, in urban areas, adjustments to the stop bar locations depicted in Figure 9.1 may be needed to accommodate pedestrian crosswalks and traffic signal mast arms, at the discretion of the designer.



Traditional Design ***Modified Design (Option 1)*** ***Modified Design (Option 2)***

Figure 9.1 Traditional and modified right-turn lane designs.

It is recommended that similar modifications to improve the right-turn approach angle/position be considered as potential improvements for intersection approaches on state routes that experience a pattern of right-turn crashes in Illinois. Flexibility in design may be needed for intersections and right-turn approaches where one or more of the following characteristics exist because higher right-turn crash rates may result:

- Channelized island present with large right-turn radius
- Intersection angle < 75°
- Right-turn angle 25° to 30°, or < 45°
- Head-turn angle > 140°

- Right-turning volume > 250 vph, or right-turn approach AADT > 3,125 vpd
- Moderate truck volumes
- Five or more right-turn crashes at an approach per year

Intersections and right-turn approaches that exhibit a combination of these characteristics may experience higher right-turn crash rates. Thus, it is recommended that locations with these values be considered for safety improvement and installation of the modified right-turn lane design to reduce right-turn crashes.

It is recommended that IDOT promote the use of the modified right-turn lane design as an alternative to the current policy, thereby improving intersection safety for Illinois motorists.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). Highway Safety Manual, 1st ed. AASHTO, Washington, D.C., 2010.

American Association of State Highway and Transportation Officials (AASHTO). A Policy on Geometric Design of Highways and Streets, 6th ed. AASHTO, Washington, D.C., 2011.

Campbell, J.L., Lichty, M.G., Brown, J.L., Richard, C.M., Graving, J.S., Graham, J., O'Laughlin, M., Torbic, D., and D. Harwood. Human Factors Guidelines for Road Systems, 2nd ed. NCHRP Report 600. Transportation Research Board of the National Academies, Washington, D.C., 2012.

Clark, Z. T., D. M. Bullock, and S. M. L. Hubbard. "Quantitative Measurement Procedures for Pedestrian Service at Signalized Intersections: A Case Study at a Skewed Intersection." Transportation Research Board 85th Annual Meeting, 2005.

Conover, W. J. Practical Nonparametric Statistics, 3rd ed., John Wiley & Sons, Inc., 1999.

Dixon, K. K., J. L. Hibbard, and H. Nyman. "Right-Turn Treatment for Signalized Intersections." Urban Street Symposium, 1999.

Federal Highway Administration (FHWA). Accident Research Manual. DOT-FH-11-9424, U.S. Department of Transportation, Washington D.C., 1980.

Federal Highway Administration (FHWA). Manual on Uniform Traffic Control Devices (MUTCD) for Streets and Highways. U.S. Department of Transportation, Washington, D.C., 2009.

Federal Highway Administration (FHWA). Highway Safety Improvement Program. FHWA-SA-09-029. U.S. Department of Transportation, Washington, D.C., 2010.

Federal Highway Administration. National Agenda for Intersection Safety. Publication FHWA-SA-02-007. FHWA, U.S. Department of Transportation, 2011.

Fitzpatrick, K., E.S. Park, and W. H. Schneider IV. "Operation and Safety of Right-Turn Lane Designs." Transportation Research Board: Journal of the Transportation Research Board, 2006a.

Fitzpatrick, K., E. S. Park, and W. H. Schneider IV. "Predicting Speeds in an Urban Right-Turn Lane." Journal of Transportation Engineering © ASCE, Vol. 132, 2006b.

Garcia, A. and E. Belda-Esplugues. "Lateral Vision Angles in Roadway Geometric Design." Journal of Transportation Engineering, Vol. 133, 2007.

Gattis, J. L., Barlow, J. M., R. W. Eck, J. S. Gluck, W. F. Hecker, Jr. and H. S. Levinson. "Multimodal Driveway Design." Transportation Research Board 89th Annual Meeting, 2009.

Gilbert, Richard O. Statistical Methods for Environmental Pollution Monitoring, John Wiley & Sons, Inc. 247-253 pp., 1987

Harwood, D. W., K. M. Bauer, L. Elefteriadou, M. S. Griffith, E. Hauer, B. P. Ingrid, E. R. Kohlman Rabbani, K.R. Richard, D. J. Torbic. "Safety Effectiveness of Intersection Left- and Right-Turn Lanes." Transportation Research Board 82th Annual Meeting, 2003.

Holland, W.G. Illinois Department of Transportation's Life-Cycle Cost Analysis for Road Construction Contracts. State of Illinois, Office of the Auditor General, Springfield, IL, 2012.

Hollander, Myles and Wolfe, Douglas A. Nonparametric Statistical Methods. 2nd ed., John Wiley & Sons, Inc., 1999.

Illinois Department of Transportation (IDOT). 2011 Illinois Crash Facts & Statistics. Illinois Department of Transportation, Division of Traffic Safety, 2013.

Illinois Department of Transportation (IDOT). Highway Safety Improvement Program: User's Manual Benefit-Cost Tool. Illinois Department of Transportation, Springfield, IL, 2015.

Institute of Transportation Engineers (ITE). Traffic Engineering Handbook, 6th ed. Institute of Transportation Engineers, Washington, D.C., 2009.

Institute of Transportation Engineers (ITE). Designing Walkable Urban Thoroughfares: A Context Sensitive Approach: An ITE Recommended Practice. Publication No. RP-036A. Institute of Transportation Engineers, 2010.

National Highway Traffic Safety Administration (NHTSA). Traffic Safety Facts 2011. Publication DOT HS 811 754. NHTSA, U.S. Department of Transportation, 2013.

Petritsch, T. A., Challa, S., H. F. Huang, B. W. Landis, and P. S. McLeod. "Level of Service Model for Signalized Intersections for Pedestrians." Transportation Research Board 84th Annual Meeting, 2004.

Potts, I. B., K. M. Bauer, J. F. Ringert, and D. J. Torbic. "Safety of Channelized Right-Turn Lanes for Motor Vehicles and Pedestrians." Transportation Research Board Annual Meeting, 2012

Potts, I. B., Harwood, D.W., Bauer, K.M., Gilmore, D.K., Hutton, J.M., Torbic, D.J., Ringert, J.F., Daleiden, A., and J.M. Barlow. NCHRP Web-Only Document 208: Design Guidance for Channelized Right-Turn Lanes. Transportation Research Board of the National Academies, Washington, D.C., 2011.

Sayed, T., Zaki, M.H., and J. Autey. A Novel Approach for Diagnosing Road Safety Issues Using Automated Computer Vision Techniques. 16th Road Safety on Four Continents Conference, Beijing, China, May 2013.

Schneider, R. J., Arnold, L. S. MPH, V. Attaset, M. C. Diogenes, J. Griswold, and D. R. Ragland. "Association between Roadway Intersection Characteristics and Pedestrian Crash Risk in Alameda County, California." Transportation Research Board 89th Annual Meeting, 2010.

Tarawneh, M.S, and P. T. McCoy. "Effect of Intersection Channelization and Skew on Driver Performance." Transportation Research Record No. 1523, Geometric and Other General Design Issues, 1996.

Tarawneh, M. S., McCoy, P. T., and T. Rifaey. "Effects of Intersection Geometrics on Driver Performance." International Symposium on Highway Geometric Design Practices, Boston Massachusetts, 1995.

APPENDIX A: DETAILS OF TEST SITE CHARACTERISTICS

Test Site No.	City	Intersection Name	Subject Right-Turn Approach	Operation of Right-Turn Approach	Intersection Control	Installation Date	Inter-section Angle	Right-Turn Angle		Head-Turn Angle		Right-Turn Radius (ft)	
								Before	After	Before	After	Before	After
1	Peoria	Prospect & Knoxville (IL-40)	WB	Two-Way	Signalized	2006	52	22	50	141	115	240	25
2	Peoria	Northmoor & Knoxville (IL 40)	EB	Two-Way	Signalized	2010	90	42	60	133	115	85	78
3	Bellevue	Airport & IL-116	NB	Two-Way	Signalized	2009–2010	59	30	44	142	128	296	75
4	North Pekin	Wesley & IL-29	SB	Two-Way	Signalized	2010	59	30	49	142	127	302	100
5	Metamora	Douglas & IL-116	NB	Two-Way	Stop	2010	62	34	56	148	131	206	65
6	Peoria	Adams Ramp (NB) & War Memorial	NB	One-Way	Stop	2011	84	25	41	153	138	70	70
7	Peoria	Adams Ramp (SB) & War Memorial	SB	One-Way	Stop	2011	81	33	49	143	131	70	70
8	Pekin/Morton	I-155 Ramp (SB) & Broadway	SB	One-Way	Stop	2013	67	37	45	137	131	132	170
9	Washington	McClugage & IL-8	SB	Two-Way	Signalized	2013	80	37	66	136	111	75	59
10	Peoria	Sterling & Farmington	SB	Two-Way	Signalized	2014	72	38	81	139	96	92	52
11	Taylorville	IL-29 & Cheney	EB	Two-Way	Signalized	2011	67	41	41	148	148	178	178

Test Site No.	City	Intersection Name	Subject Right-Turn Approach	Right-Turn Approach Laneage (Before/After)	Highway Type of Right-Turn Approach	Raised Channelized Island Present at Right-Turn Approach?		Exclusive Right-Turn Lane Present at Right-Turn Approach?	Speed Limit (mph)	
						Before	After		Right-Turn Approach	Intersecting Approach
1	Peoria	Prospect & Knoxville (IL-40)	WB	2/2	Undivided	Yes	No	Yes	30	45
2	Peoria	Northmoor & Knoxville (IL-40)	EB	3/3	Undivided	Yes	Yes	Yes	35	45
3	Bellevue	Airport & IL-116	NB	2/4	Undivided	Yes	Yes	Yes	45	45
4	North Pekin	Wesley & IL-29	SB	2/2	Undivided	Yes	No	Yes	25	45
5	Metamora	Douglas & IL-116	NB	2/2	Undivided	Yes	No	Yes	45	40
6	Peoria	Adams Ramp (NB) & War Memorial	NB	1/1	Divided	Yes	Yes	Yes	25	45
7	Peoria	Adams Ramp (SB) & War Memorial	SB	1/1	Divided	No	No	Yes	25	45
8	Pekin/Morton	I-155 Ramp (SB) & Broadway	SB	1/1	Divided	Yes	Yes	Yes	45	55
9	Washington	McClugage & IL-8	SB	4/4	Divided	Yes	Yes	Yes	50	45
10	Peoria	Sterling & Farmington	SB	3/3	Undivided	Yes	No	Yes	45	40
11	Taylorville	IL-29 & Cheney	EB	3/3	Undivided	Yes	No	Yes	30	40

Test Site No.	City	Intersection Name	Subject Right-Turn Approach	Design Hourly Volume (DHV) (vehicles per hour)		Right-Turn Approach Average Daily Traffic (ADT) (vehicles per day)		Intersecting Approach ADT (vehicles per day)		Total Intersection ADT (vehicles per day)	
				Right-Turn Approach	Intersecting Approach	Before	After	Before	After	Before	After
1	Peoria	Prospect & Knoxville (IL-40)	WB	368	1,153	5,100	4,100	11,700	10,450	31,300	27,900
2	Peoria	Northmoor & Knoxville (IL-40)	EB	297	1,499	2,700	3,000	11,225	12,800	27,550	30,200
3	Bellevue	Airport & IL-116	NB	231	201	4,850	4,400	3,400	3,850	16,100	15,650
4	North Pekin	Wesley & IL-29	SB	49	1,318	325	2,475	14,975	15,300	25,875	31,900
5	Metamora	Douglas & IL-116	NB	276	667	2,050	2,550	5,417	4,913	15,442	15,188
6	Peoria	Adams Ramp (NB) & War Memorial	NB	537	1,935	2,850	2,800	16,575	16,500	19,425	19,300
7	Peoria	Adams Ramp (SB) & War Memorial	SB	279	993	3,067	3,150	16,575	16,500	19,642	19,650
8	Pekin/Morton	I-155 Ramp (SB) & Broadway	SB	139	81	1,950	1,875	1,600	1,700	6,275	6,300
9	Washington	McClugage & IL-8	SB	91	524	6,225	6,550	9,775	10,200	22,275	23,950
10	Peoria	Sterling & Farmington	SB	482	348	7,200	NA	5,350	NA	20,250	NA
11	Taylorville	IL-29 & Cheney	EB	NA	NA	8,000	7,550	513	450	20,763	19,400

APPENDIX B: BEFORE AND AFTER CONDITIONS AT THE TEST SITES

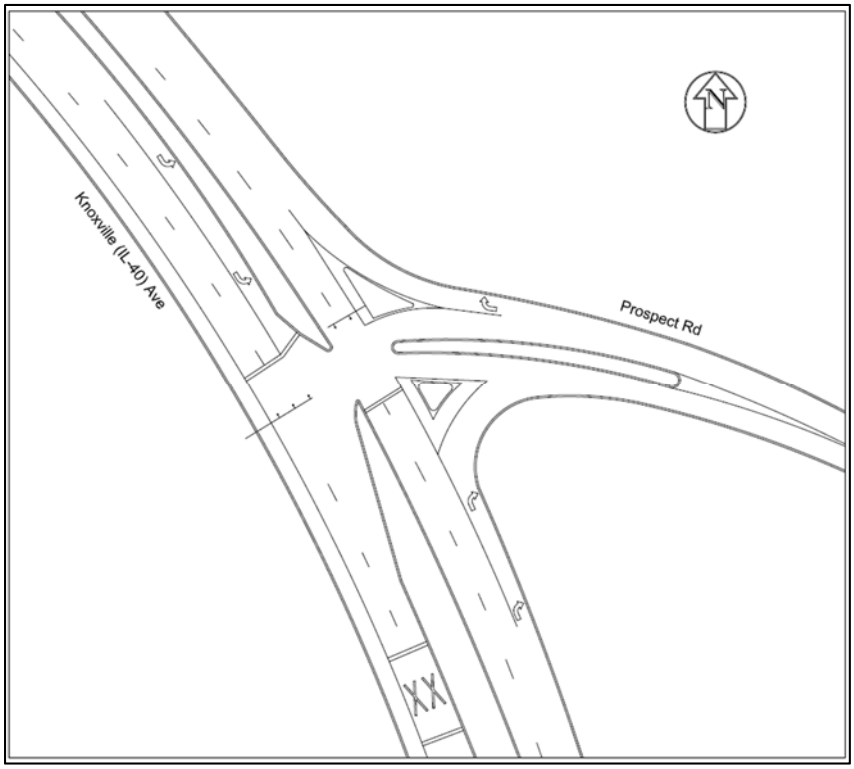


Figure B.1 Before condition of Prospect Road and Knoxville Avenue (IL-40).

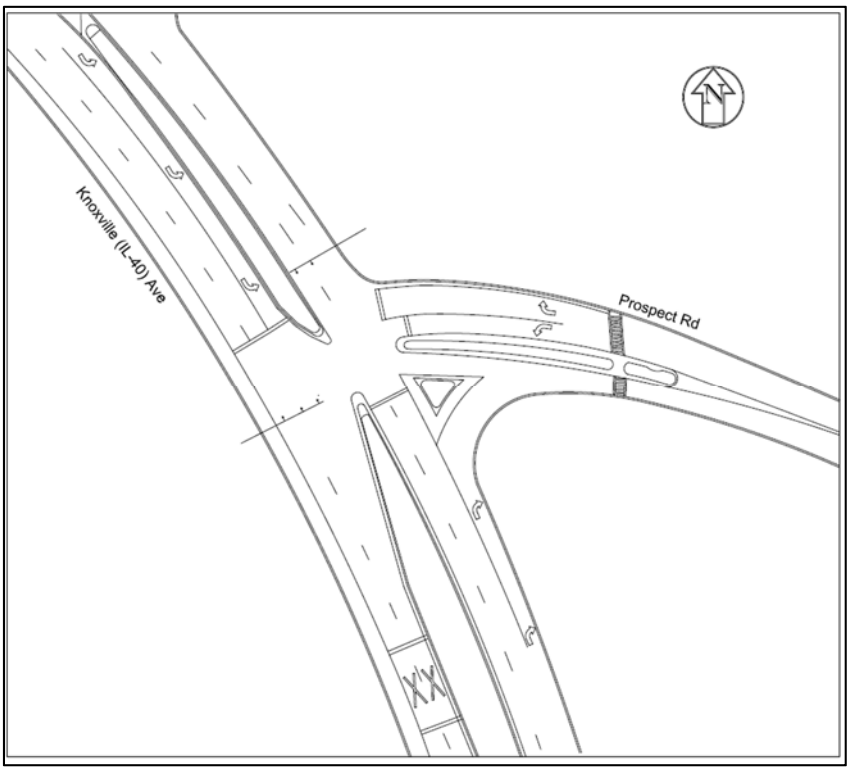


Figure B.2 After condition of Prospect Road and Knoxville Avenue (IL-40).

Figure B.3 After condition of Prospect Road and Knoxville Avenue (IL-40) (aerial).

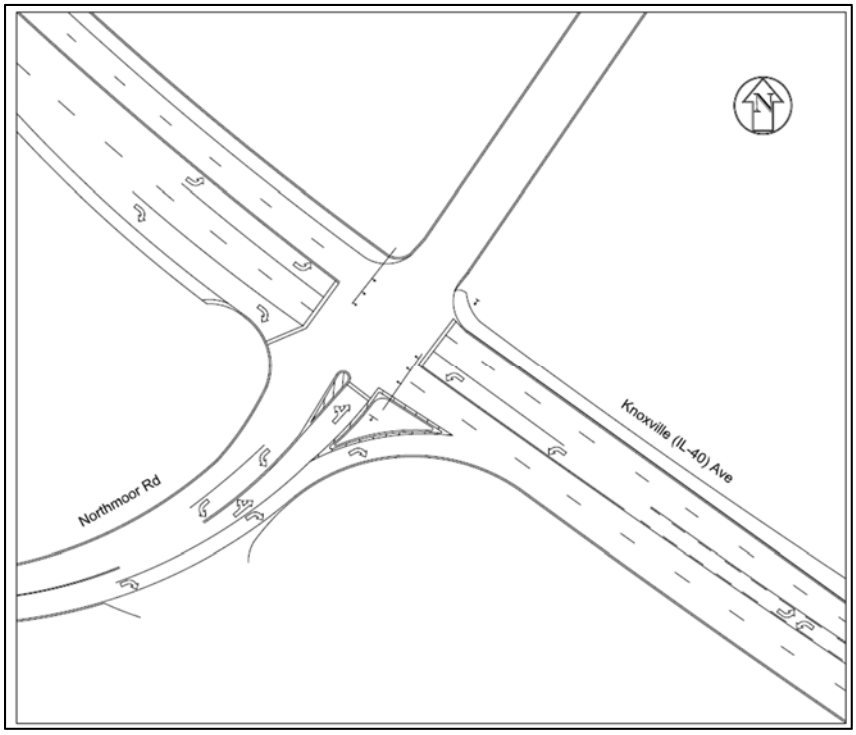


Figure B.4 Before condition of Northmoor Road and Knoxville Avenue (IL-40).

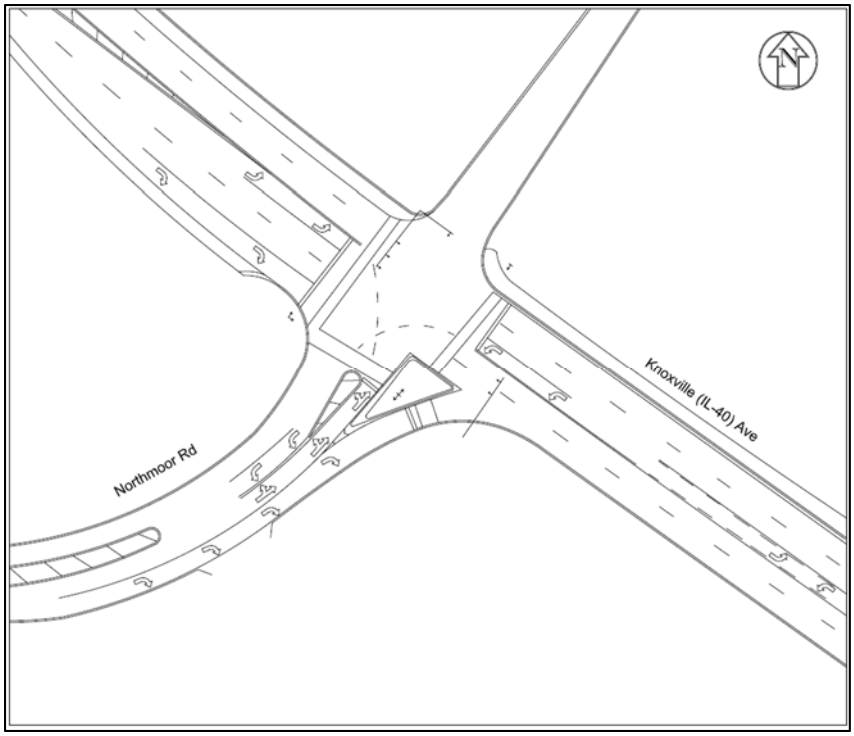


Figure B.5 After condition of Northmoor Road and Knoxville Avenue (IL-40).

Figure B.6 After condition of Northmoor Road and Knoxville Avenue (IL-40) (aerial).

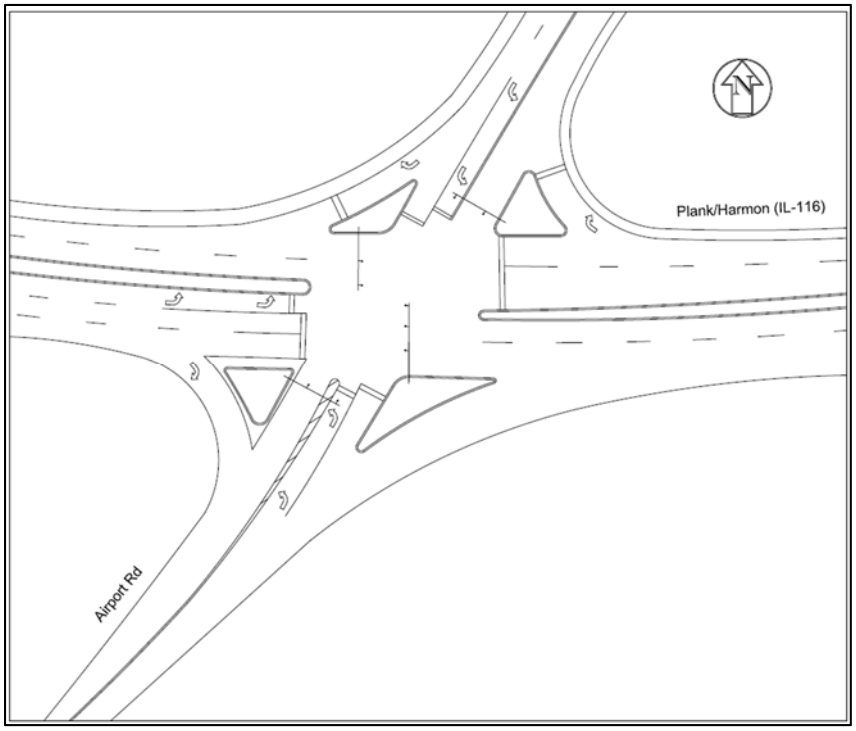


Figure B.7 Before condition of Airport Road and IL-116.

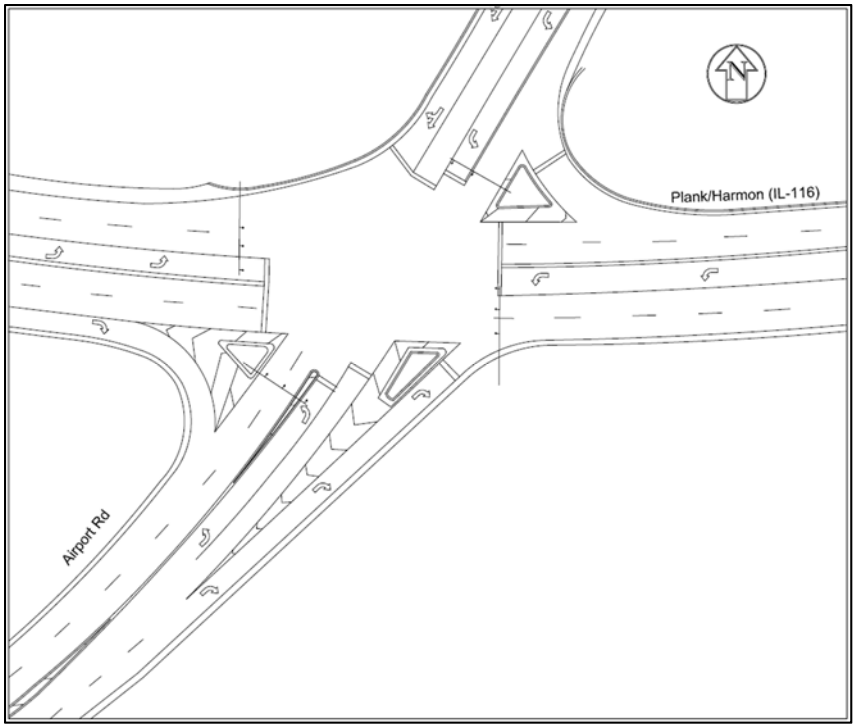


Figure B.8 After condition of Airport Road and IL-116.

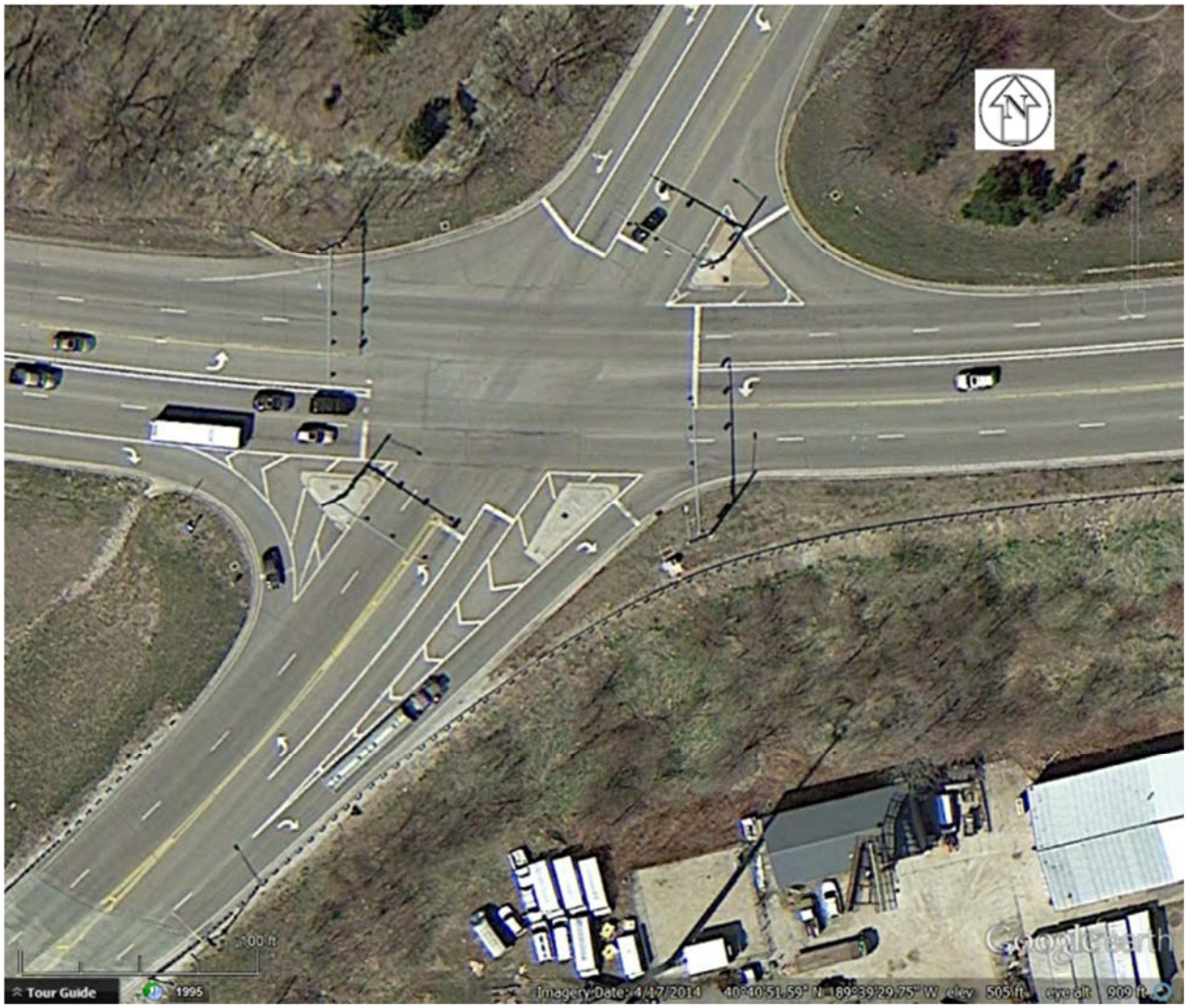


Figure B.9 After condition of Airport Road and IL-116 (aerial).

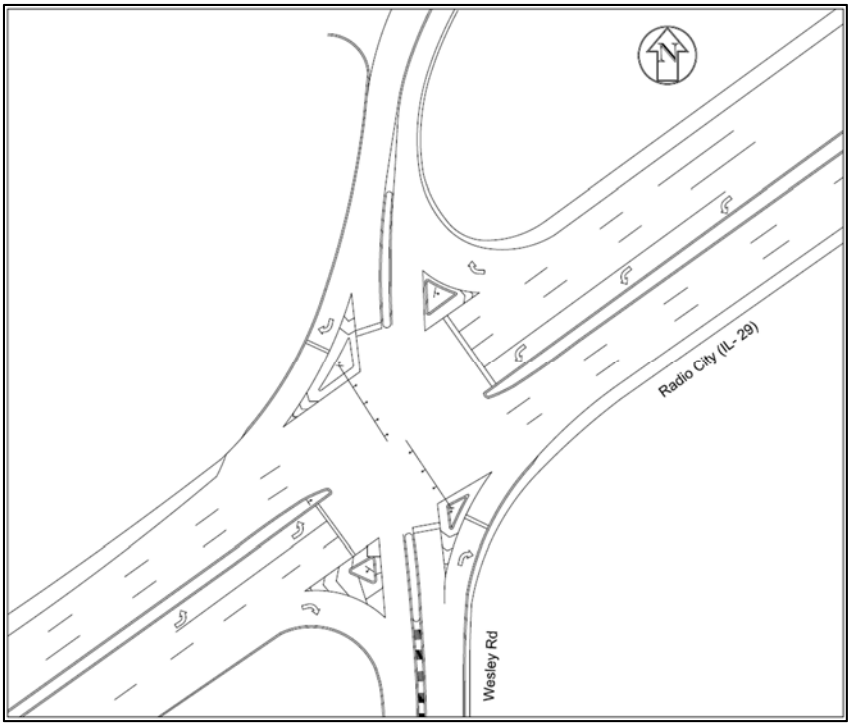


Figure B.10 Before condition of Wesley Road and IL-29.

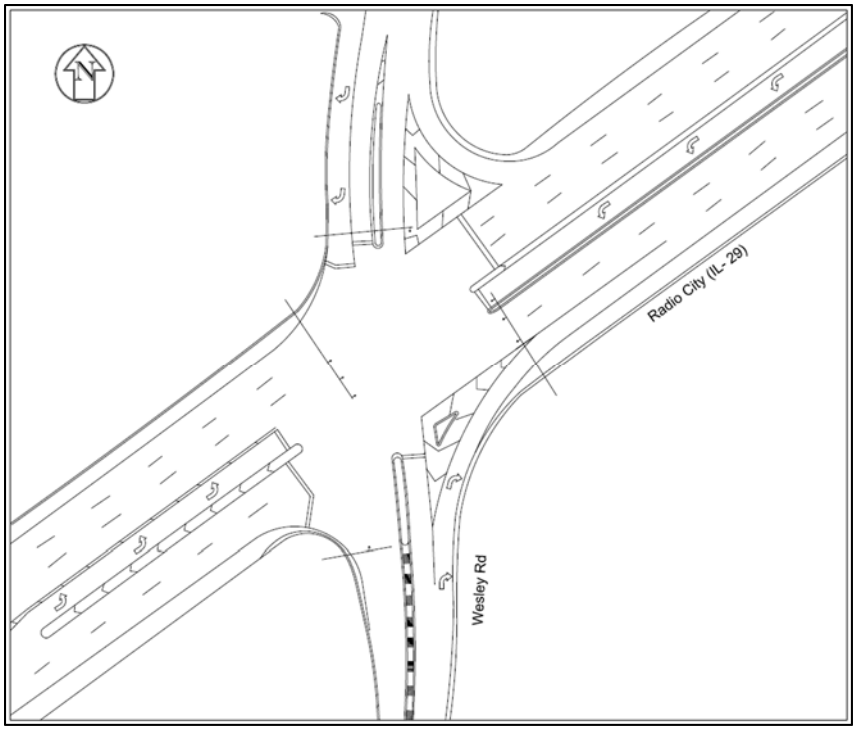


Figure B.11 After condition of Wesley Road and IL-29.

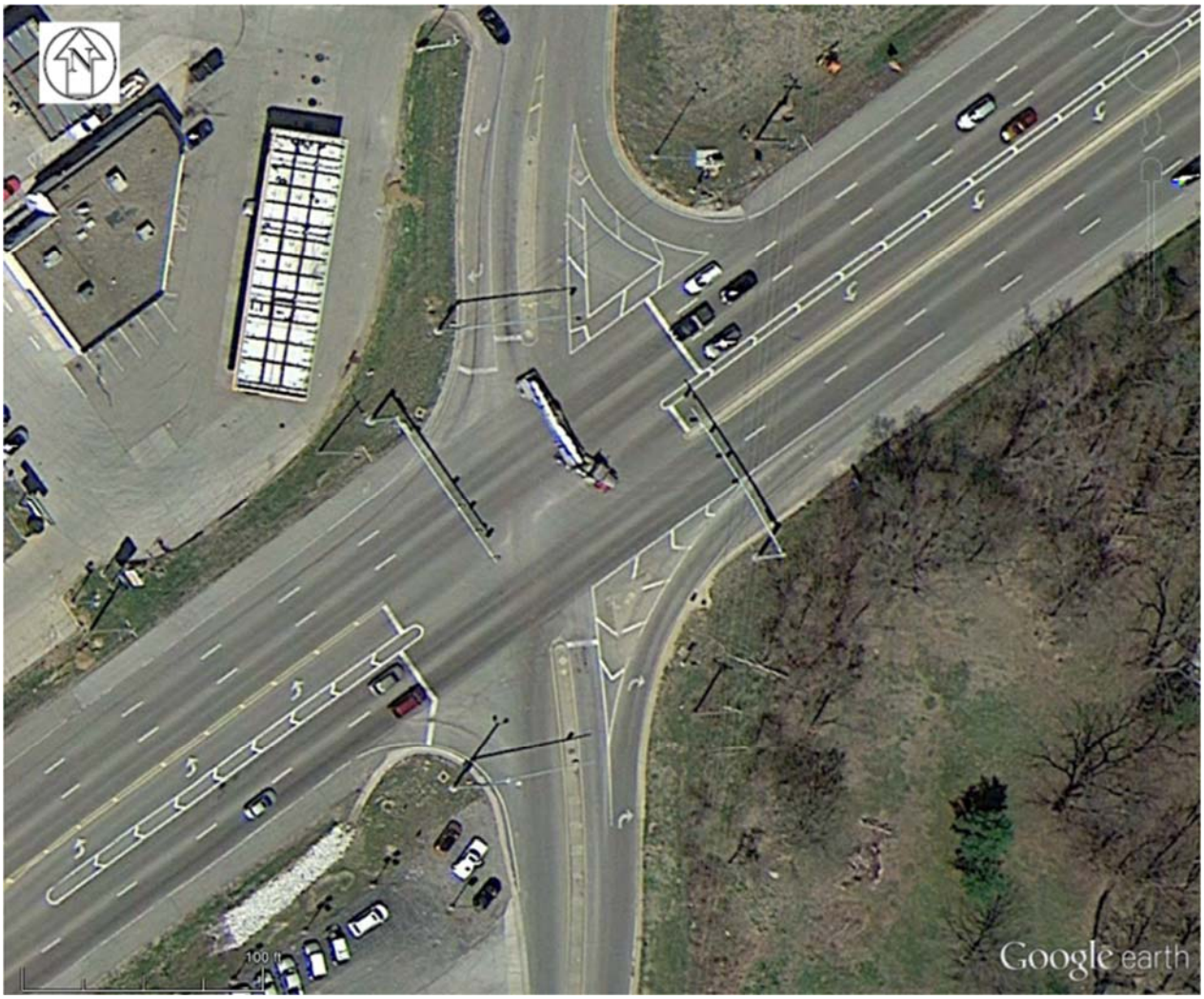


Figure B.12 After condition of Wesley Road and IL-29 (aerial).

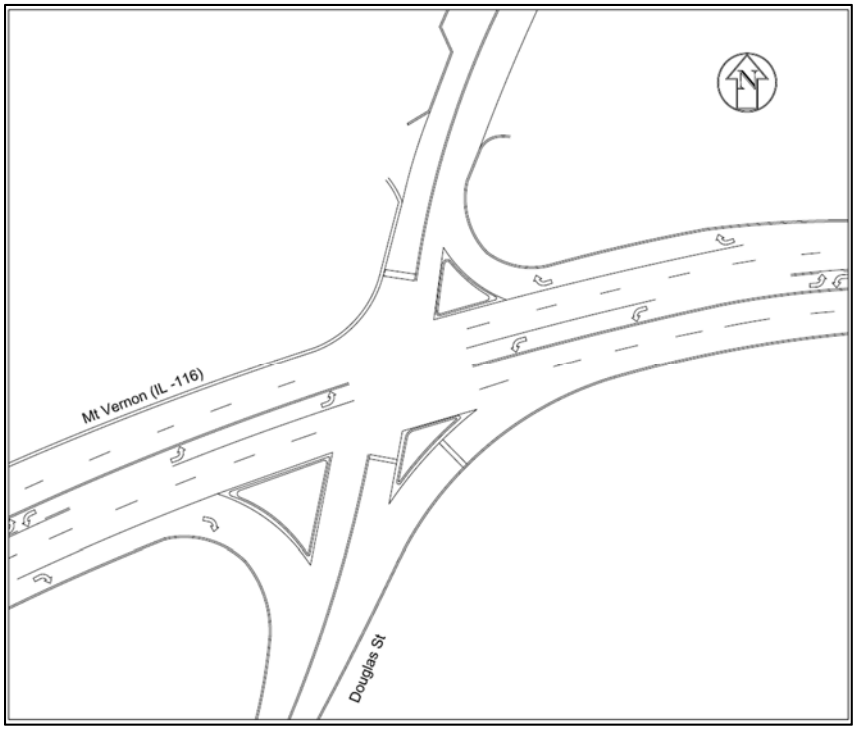


Figure B.13 Before condition of Douglas Street and IL-116.

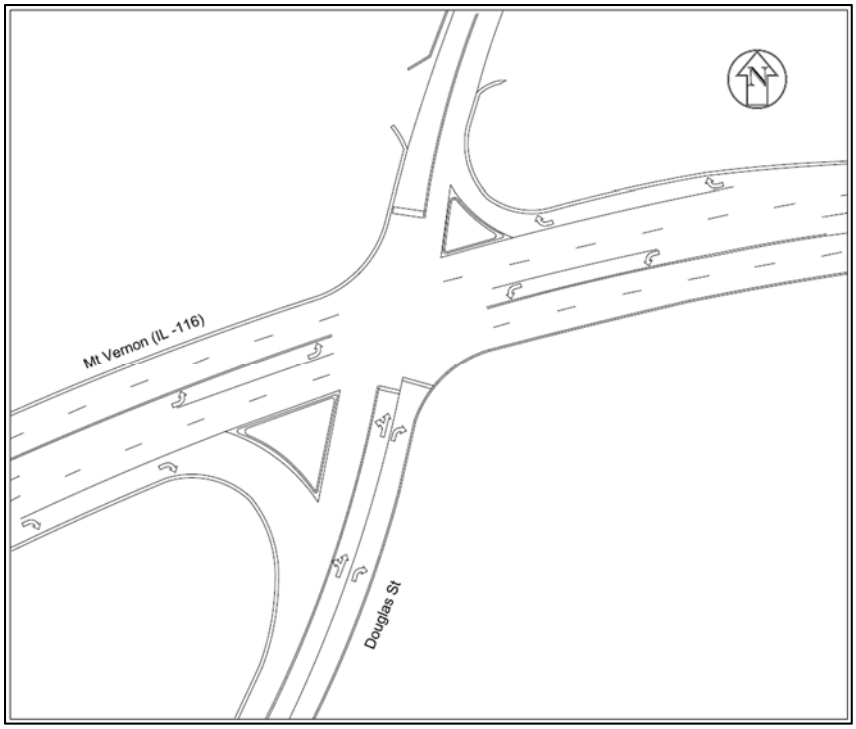


Figure B.14 After condition of Douglas Street and IL-116.

Figure B.15 After condition of Douglas Street and IL-116 (aerial).

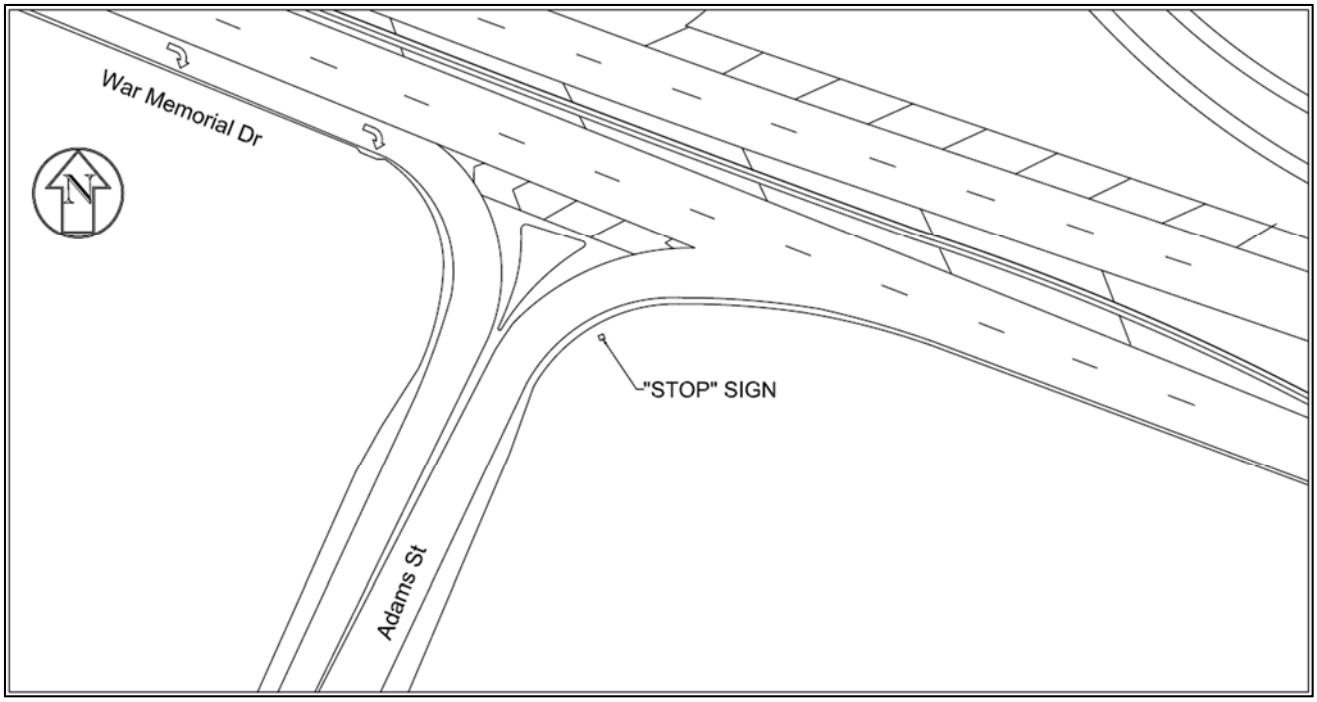


Figure B.16 Before condition of Adams NB ramp and War Memorial (US-150).

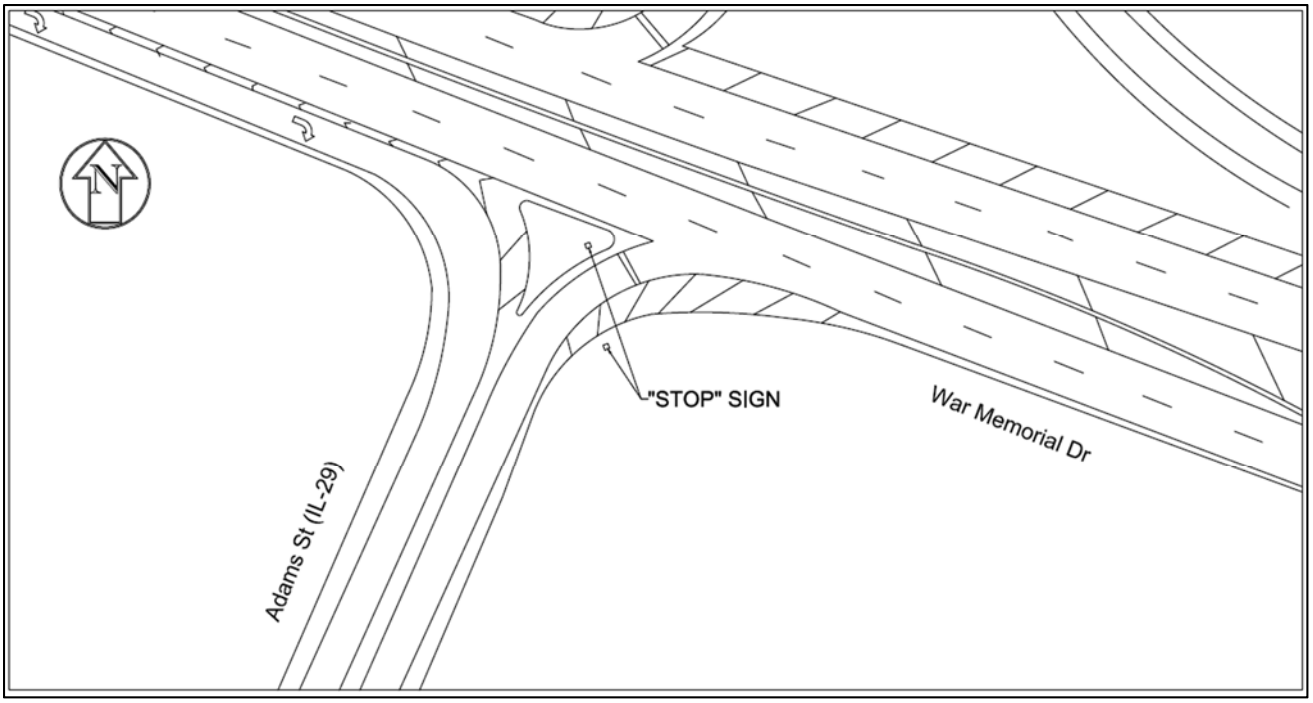


Figure B.17 After condition of Adams NB ramp and War Memorial (US-150).

Figure B.18 After condition of Adams NB ramp and War Memorial (US-150) (aerial).

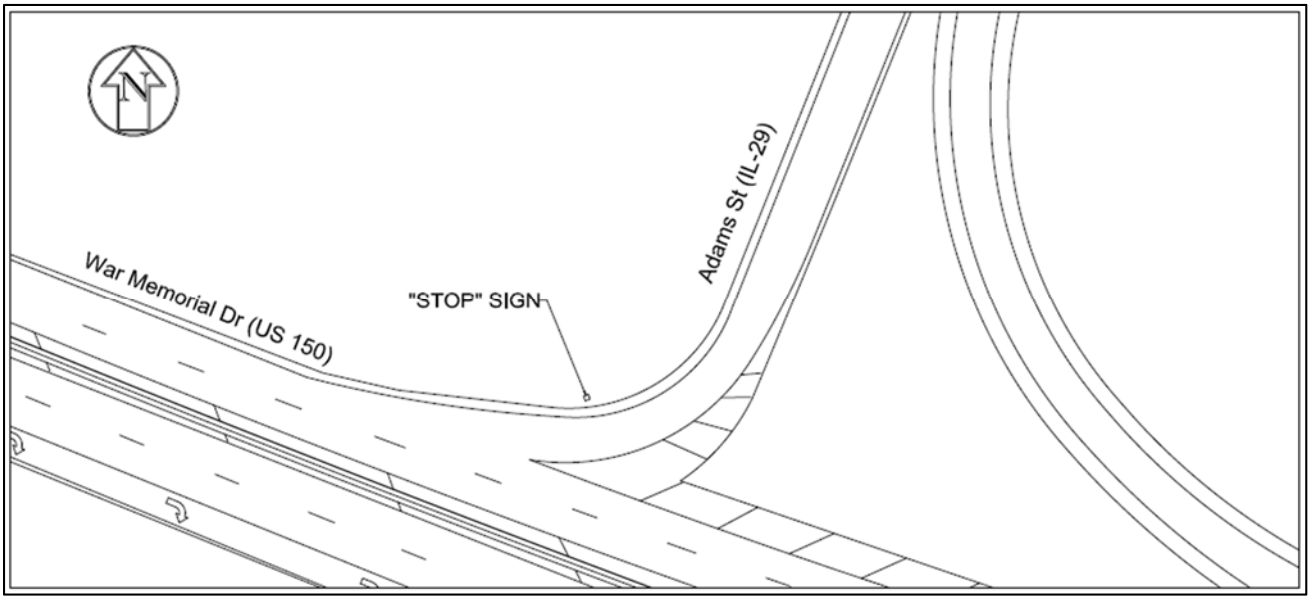


Figure B.19 Before condition of Adams SB ramp and War Memorial (US-150).

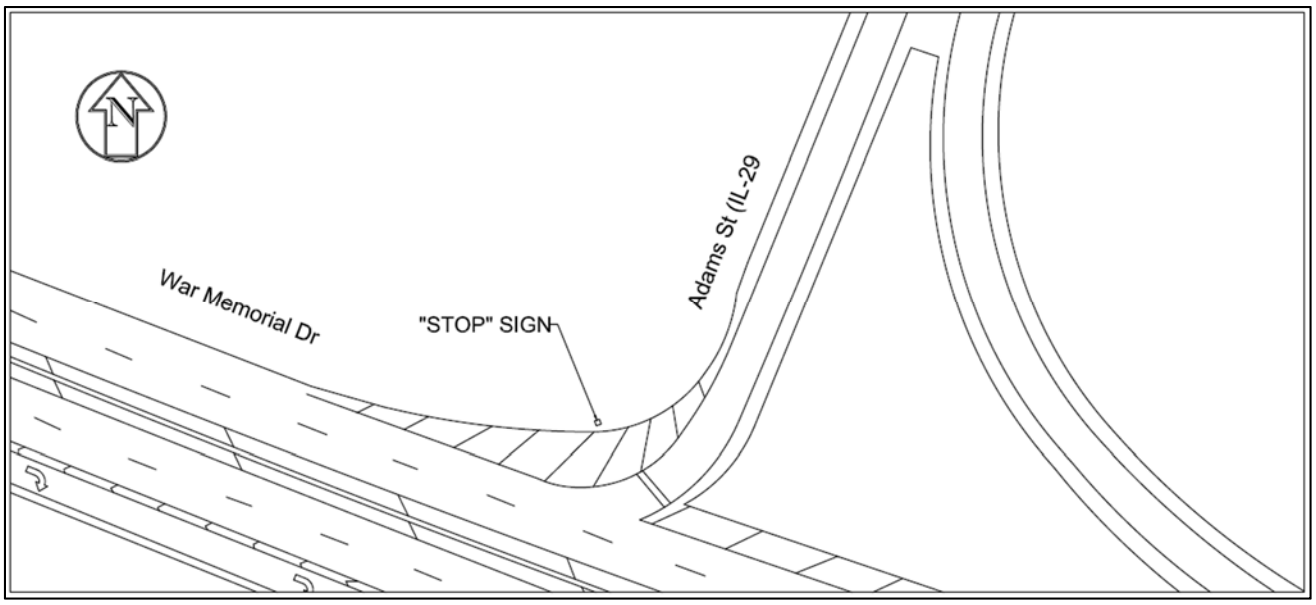


Figure B.20 After condition of Adams SB ramp and War Memorial (US-150).

Figure B.21 After condition of Adams SB ramp and War Memorial (US-150) (aerial).

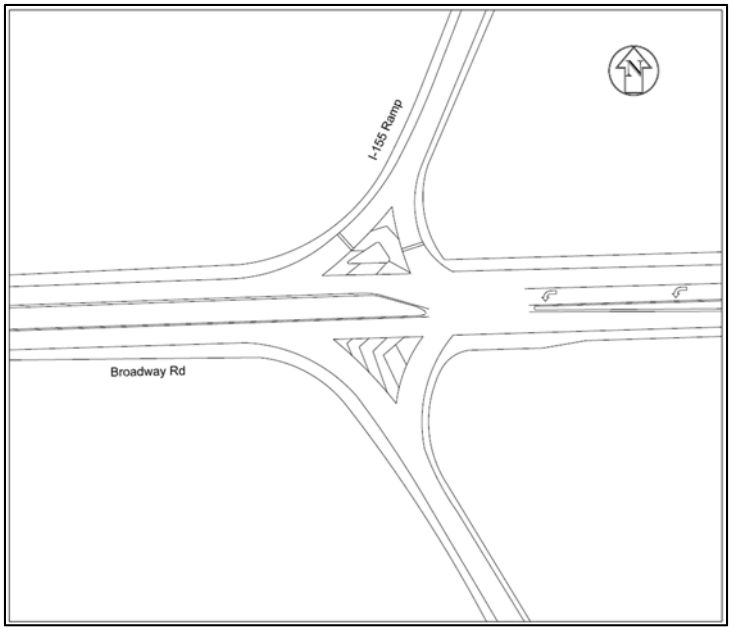


Figure B.22 Before condition of I-155 SB ramp and Broadway Road.

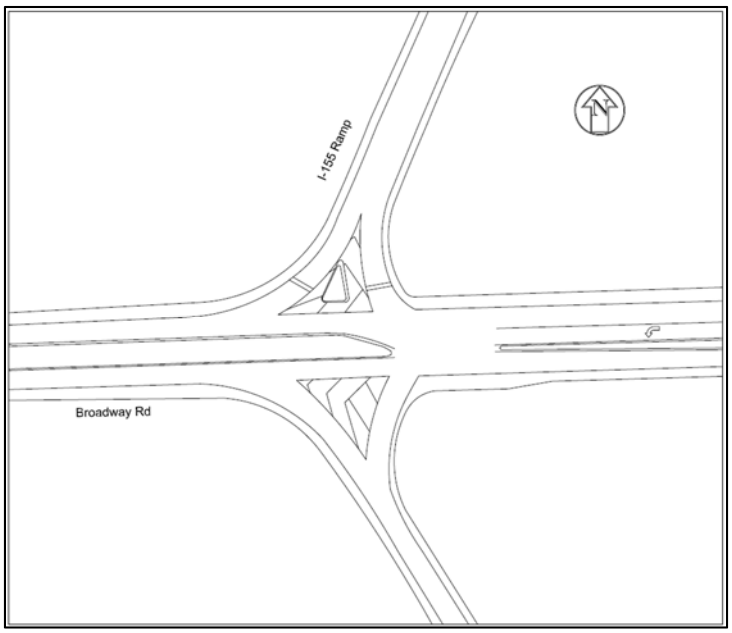


Figure B.23 After condition of I-155 SB ramp and Broadway Road.

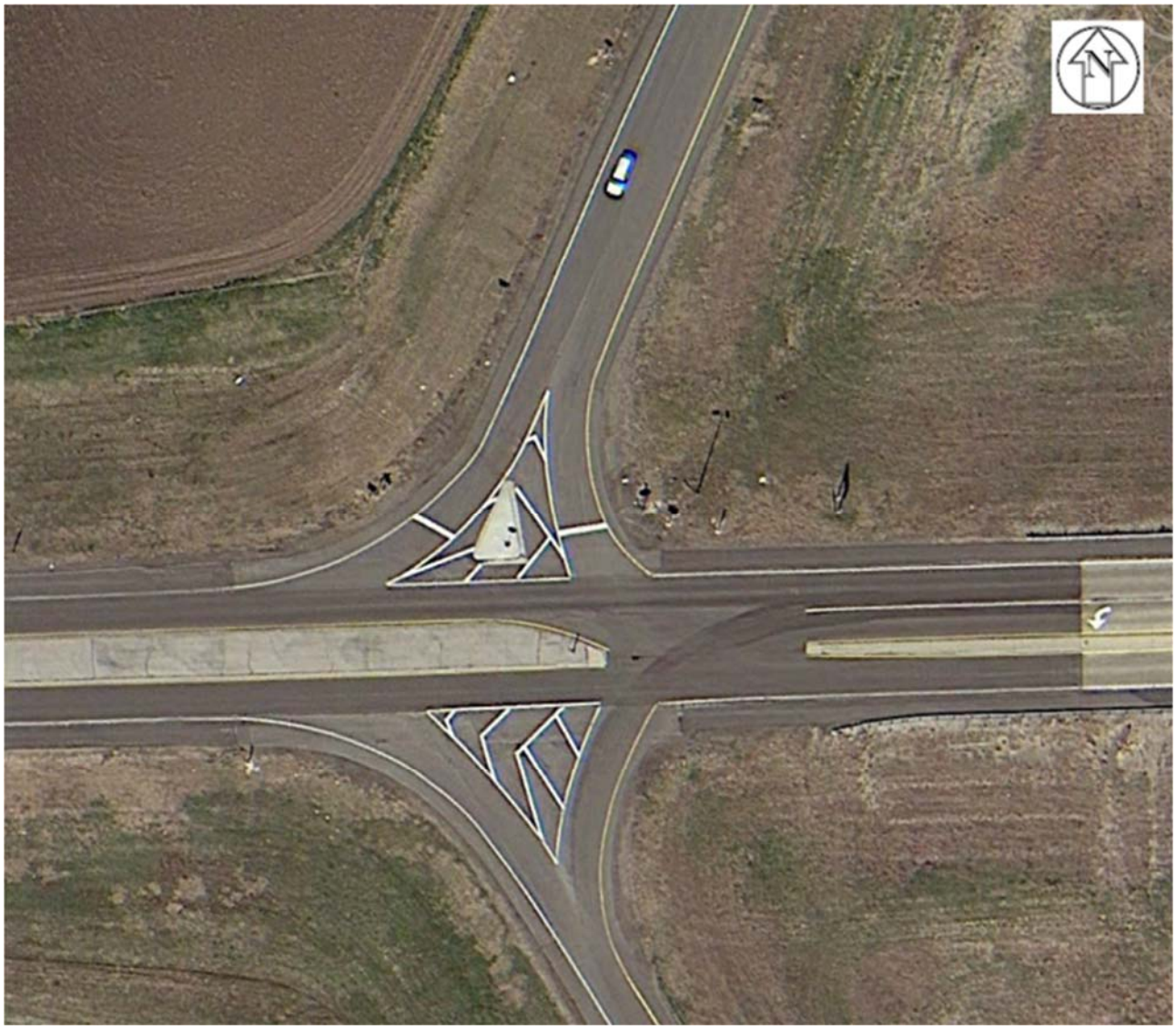


Figure B.24 After condition of I-155 SB ramp and Broadway Road (aerial).

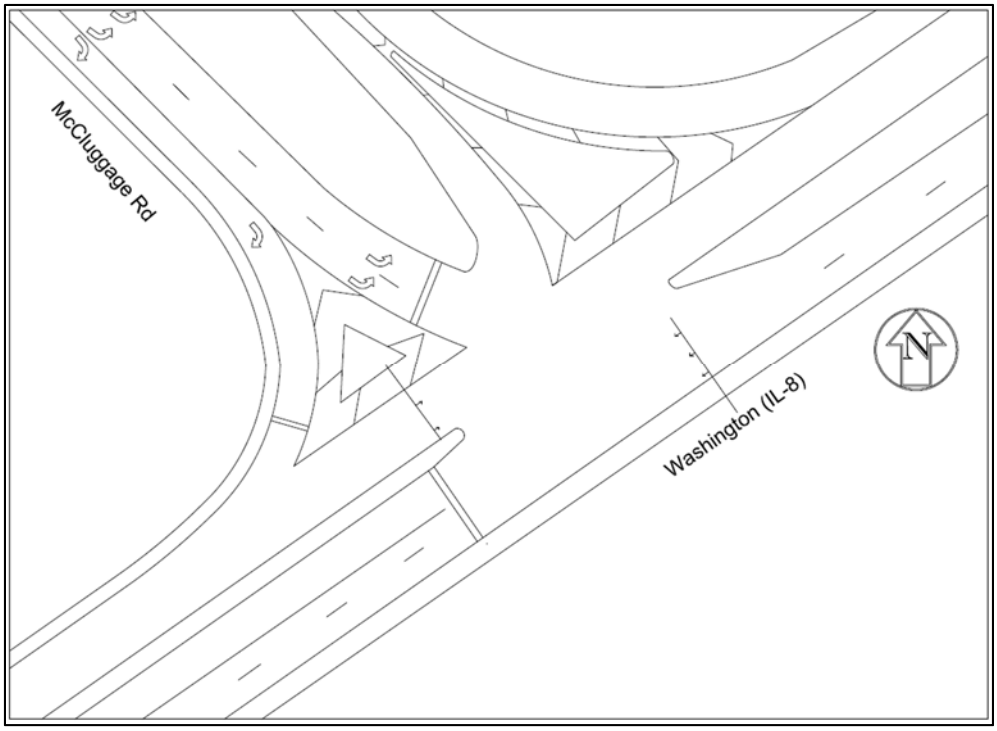


Figure B.25 Before condition of McClugage Road and IL-8.

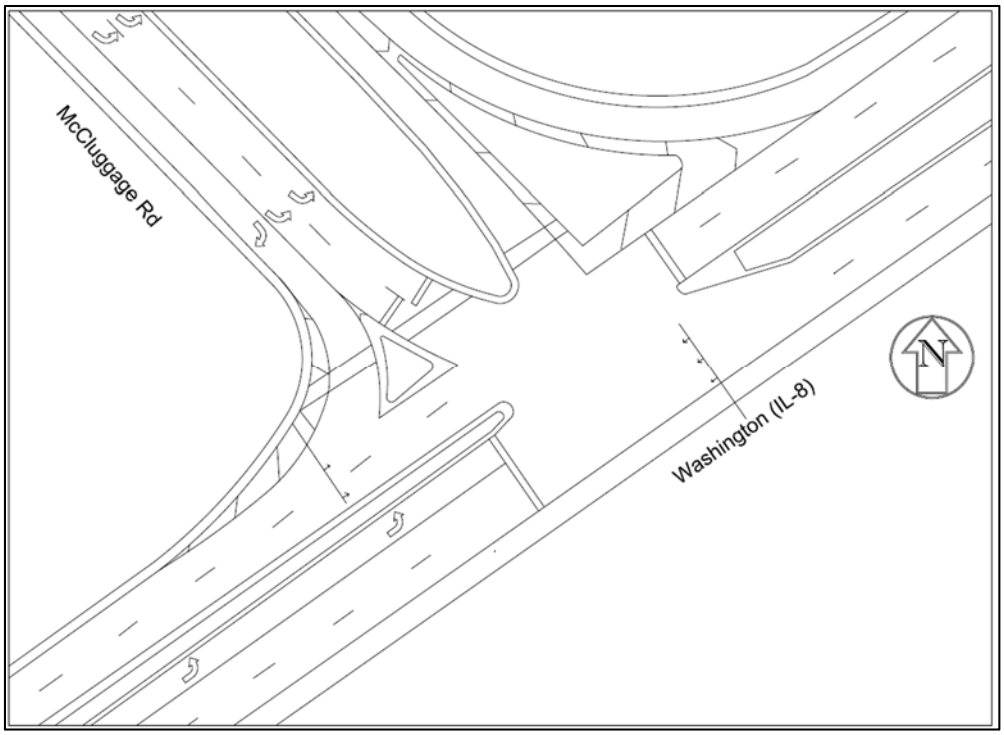


Figure B.26 After condition of McClugage Road and IL-8.

Figure B.27 After condition of McClugage Road and IL-8 (aerial).

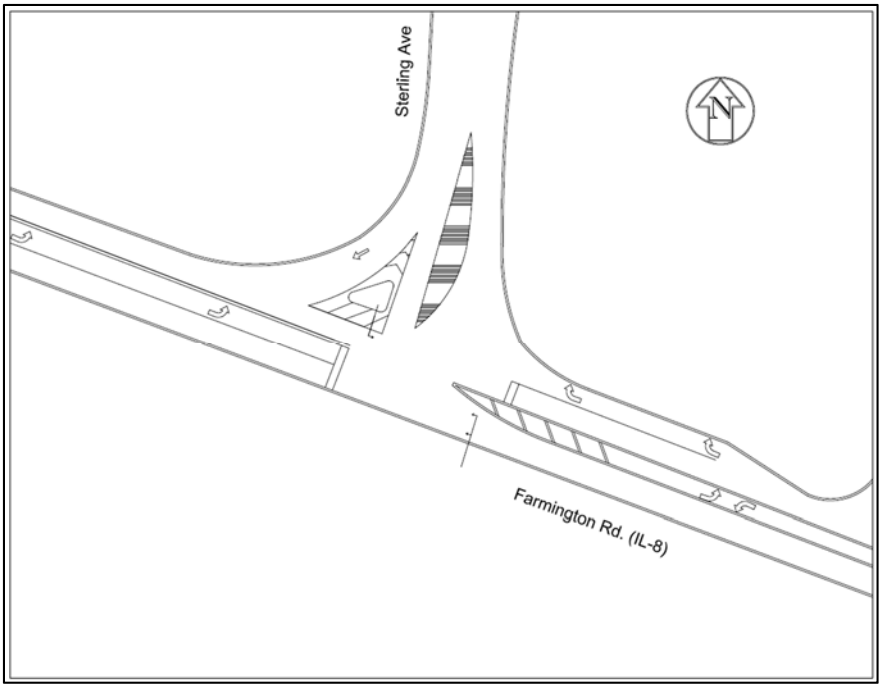


Figure B.28 Before condition of Sterling Avenue and Farmington Road (IL-8).

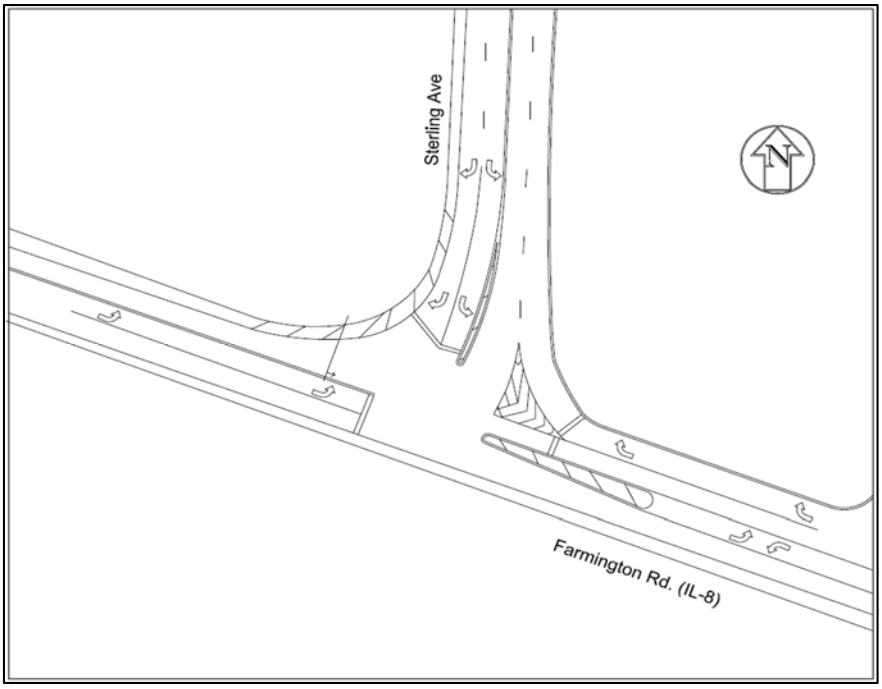


Figure B.29 After condition of Sterling Avenue and Farmington Road (IL-8).

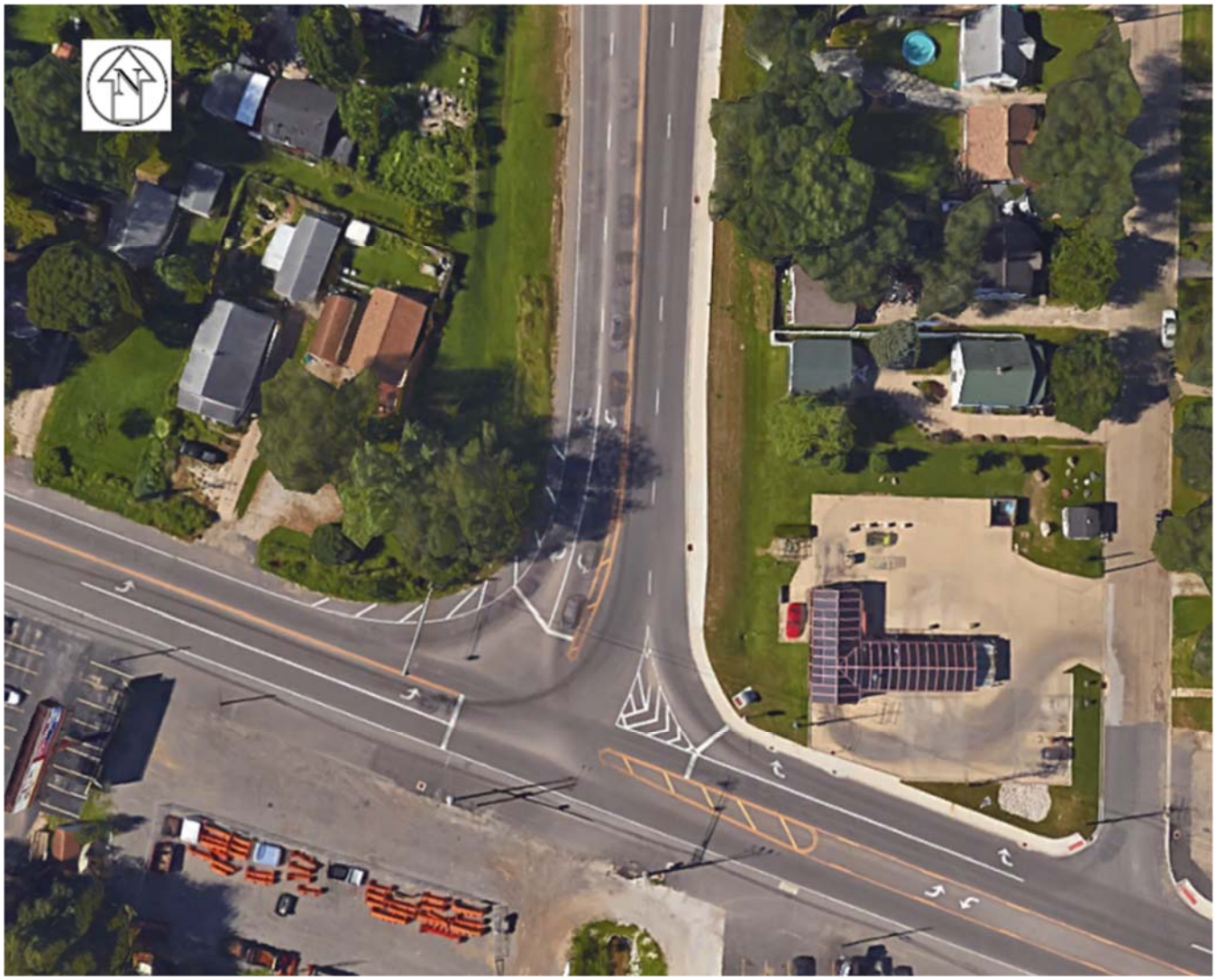


Figure B.30 After condition of Sterling Avenue and Farmington Road (IL-8) (aerial).

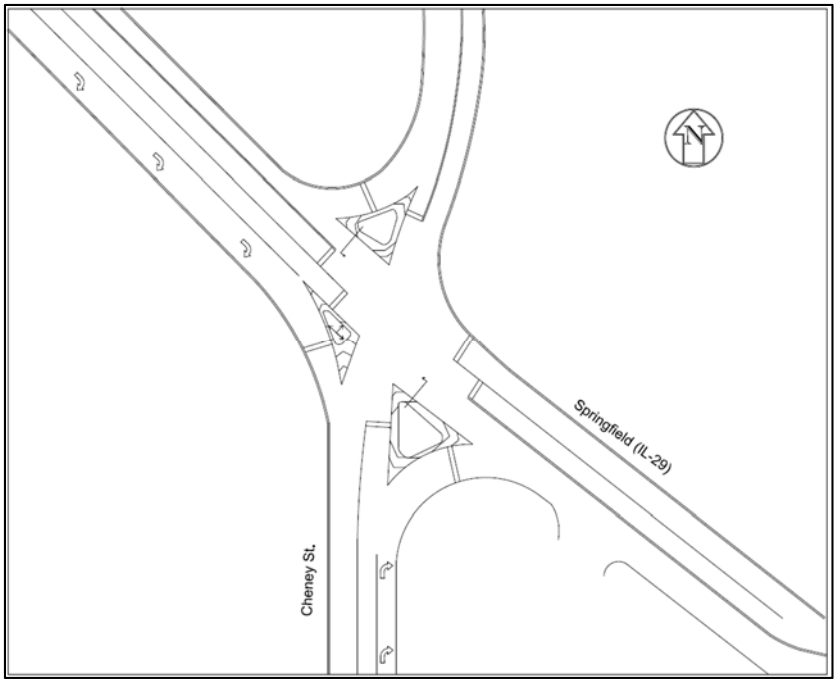


Figure B.31 Before condition of IL-29 and Cheney Street.

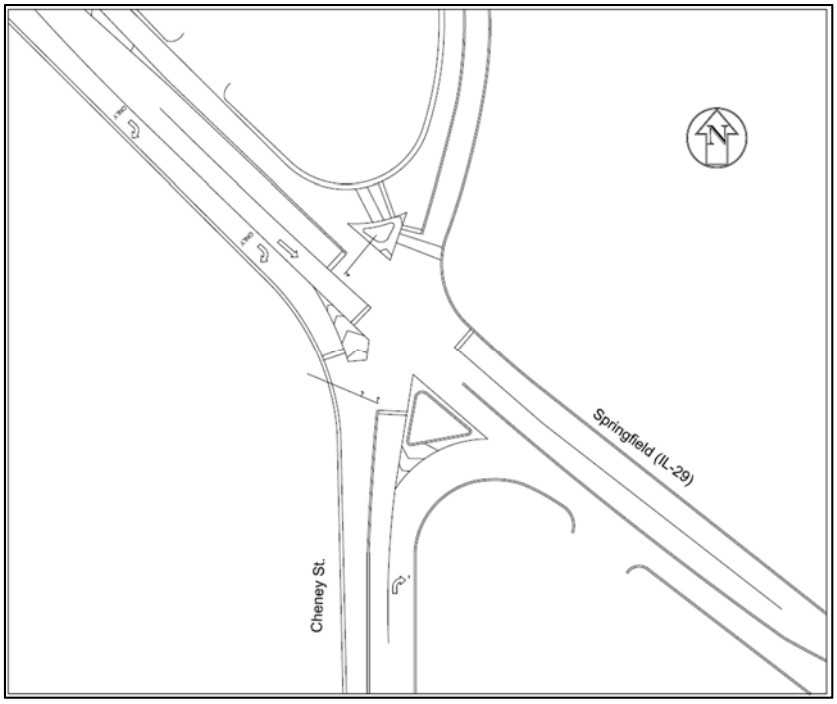


Figure B.32 After condition of IL-29 and Cheney Street.

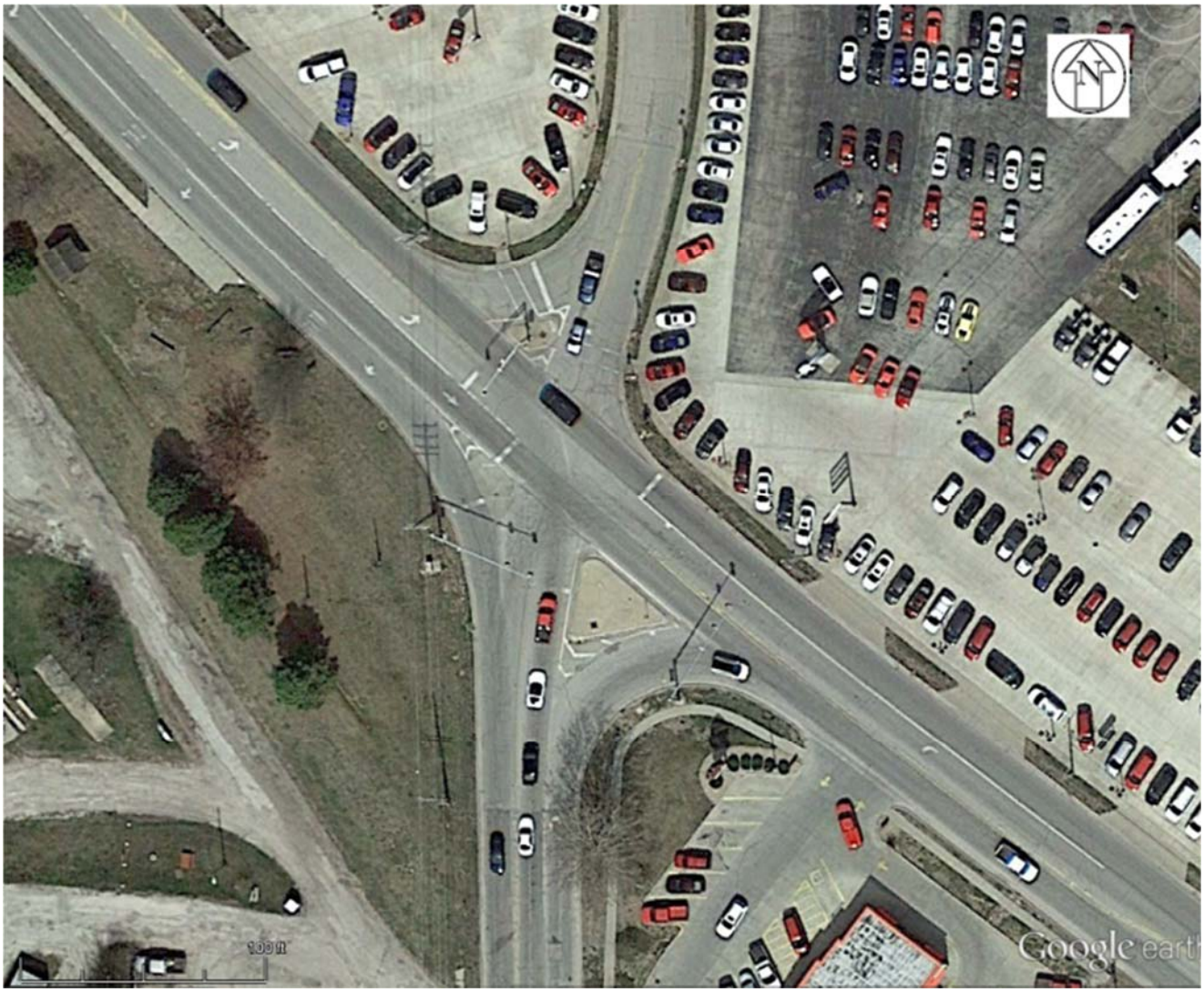


Figure B.33 After condition of IL-29 and Cheney Street (aerial).

APPENDIX C: BEFORE AND AFTER TRAFFIC CRASH DATA AT TEST SITES

Test Site No.	Intersection Name	Total Intersection Crash Frequency									Total Injury Crash Frequency (K, A, B & C)								
		"Before" Period				"After" Period				Naïve Before & After Percent Reduction	"Before" Period				"After" Period				Naïve Before & After Percent Reduction
		Yr_1	Yr_2	Yr_3	Avg.	Yr_4	Yr_5	Yr_6	Avg.		Yr_1	Yr_2	Yr_3	Avg.	Yr_4	Yr_5	Yr_6	Avg.	
1	Prospect & Knoxville (IL-40)	18	15	22	18.33	9	15	10	11.33	38.2%	3	4	4	3.67	0	1	3	1.33	63.6%
2	Northmoor & Knoxville (IL-40)	7	8	4	6.33	9	8	3	6.67	–5.3%	0	1	1	0.67	0	1	1	0.67	0.0%
3	Airport & IL-116	9	15	7	10.33	10	6	4	6.67	35.5%	4	3	5	4.00	6	3	2	3.67	8.3%
4	Wesley & IL-29	17	14	11	14.00	20	11	15	15.33	–9.5%	4	6	5	5.00	8	3	5	5.33	–6.7%
5	Douglas & IL-116	8	8	6	7.33	1	3	1	1.67	77.3%	4	1	1	2.00	1	0	0	0.33	83.3%
6	Adams Ramp (NB) & War Memorial	55	23	27	35.00	9	18	9	12.00	65.7%	4	7	7	6.00	2	2	1	1.67	72.2%
7	Adams Ramp (SB) & War Memorial	55	23	27	35.00	9	18	9	12.00	65.7%	4	7	7	6.00	2	2	1	1.67	72.2%
8	I-155 Ramp (SB) & Broadway	2	1	0	1.00	1	0	NA	0.63*	36.8%	0	0	0	0.00	0	0	NA	0.00	0.0%
9	McClugage & IL-8	6	7	6	6.33	9	3	NA	7.20*	–13.7%	2	1	2	1.67	1	0	NA	0.60	64.0%
10	Sterling & Farmington	8	20	10	12.66	NA	NA	NA	NA	NA	2	1	1	1.33	NA	NA	NA	NA	NA
11	IL-29 & Cheney	7	14	10	10.33	8	5	7	6.67	35.5%	0	4	3	2.33	0	1	1	0.67	71.4%

*Average annual crash frequency using 19 to 20 months of after-data.

Test Site No.	Intersection Name	Total Crashes at Target Approach									Right-Turn Crashes at Target Approach								
		"Before" Period				"After" Period				Naïve Before & After Percent Reduction	"Before" Period				"After" Period				Naïve Before & After Percent Reduction
		Yr_1	Yr_2	Yr_3	Avg.	Yr_4	Yr_5	Yr_6	Avg.		Yr_1	Yr_2	Yr_3	Avg.	Yr_4	Yr_5	Yr_6	Avg.	
1	Prospect & Knoxville (IL-40)	13	11	13	12.33	2	2	1	1.67	86.5%	12	11	13	12.00	2	2	1	1.67	86.1%
2	Northmoor & Knoxville (IL-40)	3	4	1	2.67	3	3	1	2.33	12.5%	2	4	1	2.33	3	2	1	2.00	14.3%
3	Airport & IL-116	0	5	2	2.33	3	2	1	2.00	14.3%	0	3	1	1.33	2	1	0	1.00	25.0%
4	Wesley & IL-29	1	3	2	2.00	1	3	10	4.67	–133.3%	1	1	0	0.67	0	0	0	0.00	100.0%
5	Douglas & IL-116	6	4	5	5.00	1	2	1	1.33	73.3%	5	4	5	4.67	0	2	0	0.67	85.7%
6	War Memorial and Adams NB	26	14	18	19.33	4	7	2	4.33	77.6%	26	14	17	19.00	4	7	2	4.33	77.2%
7	War Memorial and Adams SB	27	9	5	13.67	2	8	6	5.33	61.0%	27	9	5	13.67	2	8	6	5.33	61.0%
8	I-155 Ramp (SB) and Broadway	2	1	0	1.00	1	0	NA	0.63	36.8%	1	1	0	0.67	1	0	NA	0.63	5.3%
9	McClugage & IL-8	4	1	3	2.67	3	2	NA	3.00	–12.5%	1	0	1	0.67	0	0	NA	0.00	100.0%
10	Sterling & Farmington	6	13	8	12.33	NA	NA	NA	NA	NA	5	9	6	6.66	NA	NA	NA	NA	NA
11	IL-29 and Cheney	5	5	5	5.00	5	1	3	3.00	40.0%	1	1	1	1.00	2	1	1	1.33	-33.3%

*Average annual crash frequency using 19 to 20 months of after-data.

APPENDIX D: CHARACTERISTICS OF THE 116 COMPARISON SITES

Int. No.	IDOT District No.	County	City	Intersection Name	RT App.	Operation of RT App.	Int. Control	Sub. Road Lanes	Int. Angle	RT Angle	Head-Turn Angle	RT Radius (ft)	Divided Hwy (RT App.)?	Channelized Island at RT App.?	Exclusive RT Lane at RT App.?
1	D1	Cook (016)	Alsip	IL-50 (Cicero) & 127th	NB	Two-way	Signalized	7	90	53	122	105	Yes	Yes	Yes
2					SB	Two-way		7	90	49	125	124	Yes	Yes	Yes
3	D1	Cook (016)	Chicago	67th/Marquette & IL-50 (Cicero)	EB	Two-way	Signalized	4	90	39	138	92	No	Yes	Yes
4	D1	Cook (016)	Deer Park	Lake Cook (County Line) & US-12 (Rand)	EB	Two-way	Signalized	5	47	33	141	352	Yes	Yes	Yes
5	D1	Cook (016)	Glenview	Patriot & Willow	NB	Two-way	Signalized	5	90	63	112	72	Yes	No	Yes
6	D1	Cook (016)	Hoffman Estates	IL-72 (Higgins) & Barrington	SB	Two-way	Signalized	8	51	67	107	49	No	Yes	Yes
7	D1	Cook (016)	Matteson	Vollmer & IL-43 (Harlem)	WB	Two-way	Signalized	4	90	61	116	68	No	No	Yes
8	D1	Cook (016)	Midlothian	147th & IL-50 (Cicero)	WB	Two-way	Signalized	5	90	64	109	63	Yes	No	Yes
9	D1	Cook (016)	Northbrook	Skokie & IL-68 (Dundee)	SB	Two-way	Signalized	5	63	57	119	44	N/A	Yes	Yes
10	D1	Cook (016)	Northfield	Willow & IL-43 (Waukegan)	EB	Two-way	Signalized	5	86	35	139	96	Yes	Yes	Yes
11	D1	Cook (016)	Oak Forest	167th & IL-50 (Cicero)	WB	Two-way	Signalized	5	90	44	133	61	Yes	Yes	Yes
12	D1	Cook (016)	Palatine	IL-68 (Dundee) & US-12 (Rand)	EB	Two-way	Signalized	5	46	40	135	163	No	No	Yes
13	D1	Cook (016)	Rolling Meadows	IL-53 NB Ramp & Euclid	NB	One-way	Yield	1	81	36	141	131	No	Yes	Yes
14	D1	Cook (016)	Schaumburg	Plum Grove & IL-72 (Higgins)	SB	Two-way	Signalized	7	82	48	129	101	No	Yes	Yes
15	D1	Cook (016)	Schaumburg	Meacham & IL-58	SB	Two-way	Signalized	8	87	58	117	80	Yes	Yes	Yes
16	D1	Cook (016)	Schaumburg	West Frontage & IL-72 (Higgins)	SB	One-way	Signalized	4	81	54	121	54	No	Yes	Yes
17	D1	Cook (016)	Unincorporated	Schaumburg & Irving Park (IL-19)	SB	Two-way	Signalized	3	52	46	129	91	No	No	Yes
18	D1	Cook (016)	Unincorporated	Palatine & Algonquin (IL-62)	SB	Two-way	Signalized	3	40	36	141	192	No	No	Yes
19	D1	Cook (016)	Unincorporated	Main (IL-83) & Archer (IL-171)	WB	Two-way	Signalized	5	68	35	140	197	Yes	Yes	Yes
20	D1	Cook (016)	Unincorporated	I-90 WB ramp & IL-72 (Lee)	WB	One-way	Stop	2	59	40	138	115	No	Yes	Yes
21	D1	DuPage (022)	Addison	J F Kennedy & US-20 (Lake)	NB	Two-way	Signalized	4	85	45	130	104	Yes	Yes	Yes
22	D1	DuPage (022)	Aurora	Diehl & Eola	WB	Two-way	Signalized	6	86	45	130	89	Yes	Yes	Yes
23	D1	DuPage (022)	Downers Grove	Finley & IL-56 (Butterfield)	NB	Two-way	Signalized	5	82	59	129	80	Yes	Yes	Yes
24	D1	DuPage (022)	Itasca	IL-53 (Rohlwing) & Elgin O'Hare Expwy/Thorndale	NB	Two-way	Signalized	6	65	51	125	66	Yes	Yes	Yes
25	D1	DuPage (022)	Naperville	Naperville Wheaton & US-34 (Ogden)	SB	Two-way	Signalized	4	63	37	138	97	No	Yes	No

Int. No.	Intersection Name	RT App.	Design Hourly Volume (DHV) Vehicles per hr.		Average Daily Traffic (ADT) Vehicles per day			Speed limit (mph)		Avg. Annual 2009–2012 Intersection Crashes			Avg. Annual 2009–2012 Approach Crashes		
			RT App.	Int. Thru	RT App.	Int. Thru	Total Inter-section	RT App.	Int. Thru	Total	Injury	Right-Turn	Total	Injury	Right-Turn
1	IL-50 (Cicero) & 127th	NB	745	621	20,700	8,275	65,675	35	40	69.00	12.00	41.00	14.75	2.50	9.50
2		SB	799	1,088	22,200	14,500		35	40	69.00	12.00	41.00	14.25	2.50	3.50
3	67th/Marquette & IL-50 (Cicero)	EB	132	2,481	200	29,500	68,350	30	30	31.25	6.50	5.50	5.75	1.00	3.25
4	Lake Cook (County Line) & US-12 (Rand)	EB	338	1,811	9,400	24,150	68,200	45	35	47.00	8.50	20.50	14.50	3.50	11.75
5	Patriot & Willow	NB	36	1,414	1,000	18,850	39,700	35	45	31.00	6.00	9.75	8.50	1.50	5.00
6	IL-72 (Higgins) & Barrington	SB	459	1,040	18,900	14,000	63,300	50	45	33.75	10.00	12.75	9.75	3.25	3.25
7	Vollmer & IL-43 (Harlem)	WB	457	491	5,350	8,450	27,000	45	45	12.75	1.25	4.50	5.00	0.25	4.00
8	147th & IL-50 (Cicero)	WB	260	821	9,250	14,900	46,950	35	35	16.50	3.00	2.25	4.25	1.00	1.75
9	Skokie & IL-68 (Dundee)	SB	275	1,288	7,650	17,175	49,650	40	35	31.00	7.00	8.75	12.50	2.00	6.00
10	Willow & IL-43 (Waukegan)	EB	541	1,120	18,850	9,050	56,150	40	40	43.00	15.50	12.50	22.50	7.50	12.25
11	167th & IL-50 (Cicero)	WB	398	494	14,500	8,725	45,025	35	45	24.50	5.25	6.50	1.75	1.25	4.25
12	IL-68 (Dundee) & US-12 (Rand)	EB	416	1,045	13,700	16,350	64,000	—	35	47.75	7.25	12.50	14.00	0.25	8.50
13	IL-53 NB Ramp & Euclid	NB	63	902	1,750	12,025	25,800	55	45	13.75	2.50	13.00	13.25	2.50	13.00
14	Plum Grove & IL-72 (Higgins)	SB	400	2,525	7,600	23,875	60,950	40	45	24.00	5.25	9.25	5.75	0.50	4.50
15	Meacham & IL-58	SB	430	1,074	18,000	21,400	76,350	45	40	36.75	10.75	10.75	10.50	2.25	5.25
16	West Frontage & IL-72 (Higgins)	SB	242	1,557	4,850	23,875	57,450	45	40	35.75	9.50	16.25	17.75	3.00	9.50
17	Schaumburg & Irving Park (IL-19)	SB	139	448	3,850	5,975	15,800	40	45	10.00	2.00	5.75	6.00	1.25	5.50
18	Palatine & Algonquin (IL-62)	SB	380	460	2,800	11,500	25,800	55	45	17.00	4.00	9.75	10.00	2.25	9.75
19	Main (IL-83) & Archer (IL-171)	WB	182	627	6,025	11,425	37,150	45	45	34.75	6.25	17.00	20.00	3.50	16.50
20	I-90 WB ramp & IL-72 (Lee)	WB	522	1,449	2,900	16,250	35,400	—	—	24.00	3.25	15.00	19.50	3.00	15.00
21	J F Kennedy & US-20 (Lake)	NB	285	1,009	7,925	13,450	35,325	30	35	24.75	4.25	11.50	13.25	1.25	10.75
22	Diehl & Eola	WB	267	1,144	7,425	15,250	45,350	—	45	19.50	4.25	10.50	2.00	2.00	4.00
23	Finley & IL-56 (Butterfield)	NB	285	1,308	7,550	25,400	68,600	45	45	52.00	11.25	18.00	13.50	2.75	7.50
24	IL-53 (Rohlwing) & Elgin O'Hare Expwy/Thorndale	NB	354	2,178	6,500	39,900	90,000	40	55	35.50	5.50	8.75	6.25	0.25	5.50
25	Naperville Wheaton & US-34 (Ogden)	SB	276	1,024	7,655	13,650	39,141	40	35	19.25	7.00	5.75	8.50	3.00	5.25

Int. No.	IDOT District No.	County	City	Intersection Name	RT App.	Operation of RT App.	Int. Control	Sub. Road Lanes	Int. Angle	RT Angle	Head-Turn Angle	RT Radius (ft)	Divided Hwy (RT App.)?	Channelized Island at RT App.?	Exclusive RT Lane at RT App.?
26	D1	DuPage (022)	Oak Brook	York & 22nd	NB	Two-way	Signalized	6	90	50	124	109	Yes	Yes	Yes
27	D1	DuPage (022)	Oakbrook Terrace	IL-83 (Kingery) & 22nd	EB	Two-way	Signalized	8	89	46	130	87	Yes	Yes	Yes
28					NB	Two-way		8	89	35	142	91	Yes	Yes	Yes
29					SB	Two-way		8	89	39	138	105	Yes	Yes	Yes
30	D1	DuPage (022)	Unincorporated	Swift & IL-64 (North)	SB	Two-way	Signalized	3	76	36	140	100	No	Yes	Yes
31	D1	DuPage (022)	Warrenville	I-88 WB Ramps & Winfield	WB	One-way	Signalized	2	90	40	135	67	No	Yes	Yes
32	D1	DuPage (022)	West Chicago	IL-59 (Neltnor) & IL-64 (North)	SB	Two-way	Signalized	8	77	54	123	42	Yes	Yes	Yes
33	D1	DuPage (022)	Wheaton	Naperville & IL-56 (Butterfield)	NB	Two-way	Signalized	5	65	53	123	168	Yes	No	Yes
34	D1	DuPage (022)	Willowbrook	IL-83 (Kingery) & 75th	EB	Two-way	Signalized	5	89	46	127	105	Yes	Yes	Yes
35	D1	Kane (045)	Aurora	Galena & Orchard	WB	Two-way	Signalized	6	83	28	150	104	Yes	Yes	Yes
36	D1	Kane (045)	Bartlett	Dunham (IL-25) & Stearns (IL-29)	SB	Two-way	Signalized	6	74	58	114	110	Yes	Yes	Yes
37	D1	Kane (045)	Carpentersville	Huntley & Randall	EB	Two-way	Signalized	3	59	39	136	106	No	Yes	Yes
38	D1	Kane (045)	East Dundee	IL-72 (Higgins) & IL-25 (Dundee)	WB	Two-way	Signalized	5	62	40	134	134	Yes	No	Yes
39	D1	Kane (045)	Elgin	Big Timber & Randall	WB	Two-way	Signalized	5	60	28	146	165	Yes	Yes	Yes
40	D1	Kane (045)	Elgin	Randall to US-20 EB/Weld & Randall	EB	Two-way	Signalized	2	87	44	135	98	Yes	Yes	No
41	D1	Kane (045)	Unincorporated	I88 WB Off Ramp & Orchard	WB	Two-way	Signalized	4	90	37	141	72	Yes	Yes	Yes
42	D1	Lake (049)	Gurnee	Hunt Club & IL-132 (Grand)	NB	Two-way	Signalized	5	78	62	116	85	Yes	Yes	Yes
43	D1	Lake (049)	Gurnee	Milwaukee & Grand	NB	Two-way	Stop	2	83	50	128	59	No	Yes	No
44	D1	Lake (049)	Gurnee	IL-132 (Grand) & O'Plaine	EB	Two-way	Signalized	5	85	84	90	47	No	No	No
45	D1	Lake (049)	Lake Forest	IL-60 (Kennedy) & US-41 (Skokie)	EB	Two-way	Signalized	4	66	41	124	167	Traversable	Yes	Yes
46	D1	Lake (049)	Long Grove	Gilmer/Oakwood & IL-83	EB	Two-way	Signalized	4	64	55	120	71	No	No	Yes
47	D1	Lake (049)	North Chicago	IL-137 (Buckley) & IL-43 (Waukegan)	EB	Two-way	Signalized	8	63	56	120	151	Yes	No	Yes
48	D1	Lake (049)	Unincorporated	St. Marys & IL-137 (Buckley)	NB	Two-way	Signalized	2	61	57	108	90	No	No	Yes
49	D1	Lake (049)	Vernon Hills	Fairway/Buffalo Grove & US-45	NB	Two-way	Signalized	3	60	62	115	77	Yes	Yes	Yes
50	D1	McHenry (056)	Carpentersville	Randall & Miller	EB	Two-way	Signalized	3	90	46	131	57	No	Yes	Yes

Int. No.	Intersection Name	RT App.	Design Hourly Volume (DHV) Vehicles per hr.		Average Daily Traffic (ADT) Vehicles per day			Speed limit (mph)		Avg. Annual 2009–2012 Intersection Crashes			Avg. Annual 2009–2012 Approach Crashes		
			RT App.	Int. Thru	RT App.	Int. Thru	Total Inter-section	RT App.	Int. Thru	Total	Injury	Right-Turn	Total	Injury	Right-Turn
26	York & 22nd	NB	100	1,071	7,150	11,625	37,550	40	40	13.25	5.75	7.25	3.00	1.25	1.75
27	IL-83 (Kingery) & 22nd	EB	203	2,204	28,000	14,900	98,600	40	40	53.50	2.25	21.25	18.50	3.25	8.75
28		NB	359	1,673	35,350	20,350		40	40	53.50	2.25	21.25	13.75	5.25	5.75
29		SB	455	799	20,350	28,000		40	40	53.50	2.25	21.25	10.25	2.25	4.00
30	Swift & IL-64 (North)	SB	459	2,772	6,650	29,300	73,050	35	45	33.75	7.50	7.25	6.75	1.25	5.00
31	I-88 WB Ramps & Winfield	WB	382	521	4,900	10,650	28,350	—	45	20.25	6.50	10.50	15.75	4.75	10.25
32	IL-59 (Neltnor) & IL-64 (North)	SB	251	1,394	15,450	19,300	66,650	45	45	35.75	6.75	9.00	11.25	1.75	4.25
33	Naperville & IL-56 (Butterfield)	NB	453	562	10,525	10,150	49,550	50	45	31.00	10.50	6.50	9.00	3.25	3.75
34	IL-83 (Kingery) & 75th	EB	284	1,727	7,900	23,025	54,450	45	45	27.00	4.75	7.50	9.25	1.25	7.00
35	Galena & Orchard	WB	184	1,189	5,100	15,850	42,700	35	45	23.50	6.75	10.75	10.25	2.25	9.25
36	Dunham (IL-25) & Stearns (IL-29)	SB	308	179	6,600	7,750	26,750	45	45	23.50	5.25	4.50	7.50	0.75	3.25
37	Huntley & Randall	EB	229	1,470	6,350	19,600	52,700	50	50	42.50	13.50	10.00	8.50	1.75	6.00
38	IL-72 (Higgins) & IL-25 (Dundee)	WB	516	767	14,325	10,225	46,025	40	45	21.50	5.00	0.25	7.75	0.00	5.50
39	Big Timber & Randall	WB	297	1,755	8,250	23,400	61,600	45	45	36.25	12.00	11.50	14.75	5.75	8.75
40	Randall to US-20 EB/Weld & Randall	EB	70	1,485	1,950	19,800	43,500	—	45	23.00	1.50	0.50	7.25	0.00	4.00
41	I88 WB Off Ramp & Orchard	WB	439	874	12,200	11,650	35,500	40	50	15.25	3.75	2.00	8.75	0.25	8.75
42	Hunt Club & IL-132 (Grand)	NB	383	1,508	10,650	20,100	59,850	40	45	41.00	9.75	14.25	15.75	5.25	11.25
43	Milwaukee & Grand	NB	113	1,076	3,150	14,350	31,850	35	45	6.50	1.50	4.25	4.25	0.75	4.25
44	IL-132 (Grand) & O'Plaine	EB	519	373	14,425	4,975	33,725	40	30	16.00	4.00	1.75	5.00	1.00	1.75
45	IL-60 (Kennedy) & US-41 (Skokie)	EB	421	1,212	7,950	22,450	52,850	45	45	35.25	6.25	21.00	22.00	4.25	20.75
46	Gilmer/Oakwood & IL-83	EB	260	1,296	5,400	12,800	40,050	45	45	20.50	3.50	10.00	10.00	1.75	9.25
47	IL-137 (Buckley) & IL-43 (Waukegan)	EB	121	301	16,450	11,050	46,550	45	45	22.50	4.50	6.50	9.00	2.50	4.25
48	St. Marys & IL-137 (Buckley)	NB	261	872	7,250	11,625	32,375	40	45	14.50	2.50	8.00	9.00	1.00	6.50
49	Fairway/Buffalo Grove & US-45	NB	95	731	2,650	9,750	25,400	35	50	12.75	3.25	3.50	3.50	0.25	2.00
50	Randall & Miller	EB	593	290	2,863	19,925	44,813	40	50	29.25	6.75	5.00	6.50	1.25	4.50

Int. No.	IDOT District No.	County	City	Intersection Name	RT App.	Operation of RT App.	Int. Control	Sub. Road Lanes	Int. Angle	RT Angle	Head-Turn Angle	RT Radius (ft)	Divided Hwy (RT App.)?	Channelized Island at RT App.?	Exclusive RT Lane at RT App.?
51	D1	McHenry (056)	Woodstock	Lake & IL- 47	WB	Two-way	Signalized	3	56	20	157	166	No	Yes	Yes
52	D1	Will (099)	Bolingbrook	Boughton & Plainfield-Naperville	WB	Two-way	Signalized	3	90	52	124	69	Yes	No	Yes
53	D1	Will (099)	Channahon	I-55 SB Ramps & US-6 (Eames)	SB	One-way	Signalized	2	65	47	129	121	No	Yes	No
54	D1	Will (099)	Crest Hill	IL-7/53 (Broadway) & IL-7 (Renwick)	NB	Two-way	Signalized	5	90	37	148	89	Yes	Yes	Yes
55	D1	Will (099)	Joliet	Houbolt & US-52 (Jefferson)	NB	Two-way	Signalized	3	89	62	113	61	No	Yes	Yes
56	D1	Will (099)	Joliet	I-55 NB Ramps & US-30 (Plainfield)	NB	One-way	Signalized	1	88	57	124	94	No	No	Yes
57	D1	Will (099)	Joliet	I-80 WB Ramps & Houbolt	WB	One-way	Signalized	1	88	46	136	81	No	Yes	Yes
58	D1	Will (099)	Joliet	I-55 SB Ramps & US-30 (Plainfield)	SB	One-way	Signalized	2	89	53	125	58	No	No	Yes
59	D1	Will (099)	Joliet	I-80 EB Off Ramp to IL-53 SB & IL-53 (Chicago)	EB	One-way	Signalized	1	88	41	138	86	Yes	No	Yes
60	D1	Will (099)	Mokena	US-45 (La Grange) & 191st	SB	Two-way	Signalized	6	87	67	101	65	Yes	Yes	Yes
61					WB	Two-way		6	86	67	104	69	Yes	Yes	Yes
62	D1	Will (099)	Romeoville	I-55 SB Ramps & Weber	WB	One-way	Signalized	2	89	48	129	89	N/A	Yes	Yes
63	D1	Will (099)	Unincorporated	I-355 SB Ramp & 159th (IL-7)	SB	One-way	Signalized	2	89	43	132	121	No	Yes	Yes
64	D2	Rock Island (081)	Milan	4th St & 10th Ave	SB	Two-way	Signalized	4	89	41	133	116	No	Yes	Yes
65	D2	Rock Island (081)	Moline	23rd & IL-92 (6th)	NB	Two-way	Signalized	3	60	39	148	55	Yes	Yes	Yes
66	D2	Rock Island (081)	Moline	41st & IL-5 (John Deere)	NB	Two-way	Signalized	5	85	45	132	110	No	Yes	No
67	D2	Rock Island (081)	Moline	Ave of the Cities & 19th	WB	Two-way	Signalized	5	85	41	137	111	Yes	Yes	Yes
68	D2	Winnebago (101)	Loves Park	Harlem & Perryville	EB	Two-way	Signalized	5	89	46	129	103	No	Yes	Yes
69	D2	Winnebago (101)	Rockford	Rockton & Riverside	NB	Two-way	Signalized	3	90	48	129	104	No	Yes	Yes
70	D2	Winnebago (101)	Rockford	Mulford & Harrison	NB	Two-way	Signalized	5	82	44	133	83	Yes	Yes	Yes
71					SB	Two-way		5	82	44	133	82	Yes	Yes	Yes
72	D2	Winnebago (101)	Rockford	US-20 WB Ramp & Alpine	WB	One-way	Signalized	2	66	36	140	189	No	Yes	No
73	D2	Winnebago (101)	Rockford	US-20 BUS (State) & Perryville	SB	Two-way	Signalized	6	80	37	139	109	Yes	Yes	Yes
74	D2	Winnebago (101)	Rockton	IL-251 NB Ramp & Rockton	SB	One-way	Signalized	2	90	47	130	106	No	Yes	Yes

Int. No.	Intersection Name	RT App.	Design Hourly Volume (DHV) Vehicles per hr.		Average Daily Traffic (ADT) Vehicles per day			Speed limit (mph)		Avg. Annual 2009–2012 Intersection Crashes			Avg. Annual 2009–2012 Approach Crashes		
			RT App.	Int. Thru	RT App.	Int. Thru	Total Inter-section	RT App.	Int. Thru	Total	Injury	Right-Turn	Total	Injury	Right-Turn
51	Lake & IL- 47	WB	103	1,494	4,488	9,250	25,650	40	35	22.75	6.25	6.25	6.75	2.75	4.50
52	Boughton & Plainfield-Naperville	WB	311	720	8,625	9,600	27,825	35	45	10.50	2.50	3.50	3.50	0.75	3.00
53	I-55 SB Ramps & US-6 (Eames)	SB	526	310	2,825	7,383	18,508	35	40	8.00	0.50	3.50	4.75	0.50	3.50
54	IL-7/53 (Broadway) & IL-7 (Renwick)	NB	322	194	11,125	4,975	37,525	45	—	35.00	7.25	9.50	13.00	1.75	7.75
55	Houbolt & US-52 (Jefferson)	NB	476	1162	7,800	14,000	35,300	45	45	28.25	5.50	9.50	12.75	3.00	9.50
56	I-55 NB Ramps & US-30 (Plainfield)	NB	104	836	2,900	11,150	30,900	40	40	22.75	5.25	8.00	9.25	0.75	7.25
57	I-80 WB Ramps & Houbolt	WB	312	469	1,000	9,700	20,400	—	40	13.25	1.50	6.00	8.75	0.75	6.00
58	I-55 SB Ramps & US-30 (Plainfield)	SB	63	836	1,750	11,150	24,050	—	40	18.50	3.50	4.50	7.00	0.75	4.00
59	I-80 EB Off Ramp to IL-53 SB & IL-53 (Chicago)	EB	130	600	3,600	8,000	19,600	—	30	6.75	0.25	4.50	5.50	0.25	4.50
60	US-45 (La Grange) & 191st	SB	554	546	10,025	17,850	55,125	45	—	46.75	5.25	13.50	17.25	2.00	6.00
61		WB	587	1175	17,850	10,025		—	45				12.50	1.75	7.00
62	I-55 SB Ramps & Weber	WB	633	1282	3,000	17,550	38,100	—	45	54.75	5.25	23.50	37.25	3.25	22.75
63	I-355 SB Ramp & 159th (IL-7)	SB	119	752	3,300	10,025	24,050	40	45	9.25	3.25	5.25	12.75	1.50	5.25
64	4th St & 10th Ave	SB	169	330	4,700	4,400	16,325	-	-	11.75	3.25	6.00	6.50	1.75	5.75
65	23rd & IL-92 (6th)	NB	654	1,868	2,750	6,200	15,175	30	30	13.25	4.00	9.25	9.25	2.25	9.25
66	41st & IL-5 (John Deere)	NB	127	1,939	3,950	20,900	50,050	30	55	31.50	11.50	8.00	9.00	2.50	7.50
67	Ave of the Cities & 19th	WB	362	210	10,050	2,800	22,275	-	-	17.75	4.50	7.25	9.00	2.25	5.75
68	Harlem & Perryville	EB	325	568	6,775	9,150	30,175	45	45	28.25	6.00	19.75	19.00	2.00	18.00
69	Rockton & Riverside	NB	159	542	4,413	7,225	23,613	30	40	13.00	2.50	5.50	5.75	0.75	5.00
70	Mulford & Harrison	NB	202	739	6,975	8,025	30,450	45	45	19.25	3.75	10.25	4.00	1.00	3.00
71		SB	251	602	5,600	9,850		45	45				11.50	1.00	7.50
72	US-20 WB Ramp & Alpine	WB	291	706	1,988	12,350	29,413	—	45	7.00	0.75	3.75	7.25	0.25	3.50
73	US-20 BUS (State) & Perryville	SB	196	804	12,925	13,875	56,225	45	45	54.25	13.25	12.50	13.50	1.75	7.25
74	IL-251 NB Ramp & Rockton	SB	43	311	1,188	4,150	9,488	25	45	2.75	0.50	2.50	2.75	0.50	2.50

Int. No.	IDOT District No.	County	City	Intersection Name	RT App.	Operation of RT App.	Int. Control	Sub. Road Lanes	Int. Angle	RT Angle	Head-Turn Angle	RT Radius (ft)	Divided Hwy (RT App.)?	Channelized Island at RT App.?	Exclusive RT Lane at RT App.?
75	D2	Winnebago (101)	Roscoe	Hononegah/Elevator & 2nd (IL-251)	EB	Two-way	Signalized	5	74	37	139	164	No	Yes	Yes
76					WB	Two-way		5	75	30	146	169	No	Yes	Yes
77	D2	Winnebago (101)	Roscoe	Roscoe (Bridge) & 2nd (IL-251)	EB	Two-way	Signalized	3	90	31	145	120	Yes	Yes	No
78	D2	Winnebago (101)	Unincorporated	IL-173 (West Lane) & Perryville	EB	Two-way	Signalized	3	89	29	147	141	No	Yes	Yes
79	D2	Winnebago (101)	Winnebago	Winnebago/Elida & US-20 (State)	NB	Two-way	Signalized	3	85	50	126	111	No	Yes	Yes
80	D3	Grundy (032)	Unincorporated	Pine Bluff & IL-47 (Division)	WB	Two-way	Signalized	2	90	47	129	82	No	Yes	Yes
81	D3	Grundy (032)	Unincorporated	IL-113 (Lowery) & IL-47	WB	Two-way	Stop	3	87	42	136	83	No	Yes	Yes
82	D3	Kankakee (046)	Bourbonnais	William R Latham Sr & IL-102	SB	Two-way	Signalized	4	88	48	128	102	No	Yes	Yes
83	D3	Kendall (047)	Yorkville	IL-126 (Schoolhouse) & IL-47 (Bridge)	WB	Two-way	Signalized	2	75	35	141	164	No	Yes	No
84	D3	LaSalle (050)	Ottawa	Stevenson Dr & IL-23 (Columbus)	EB	Two-way	Signalized	4	89	60	116	87	No	Yes	Yes
85	D4	Peoria (072)	Peoria	US-150 (War Memorial) & Sterling/Glen	EB	Two-way	Signalized	6	75	39	114	220	Yes	Yes	Yes
86	D4	Peoria (072)	Peoria	Scenic & US-150 (War Memorial)	EB	Two-way	Signalized	4	49	32	143	197	Yes	Yes	Yes
87					NB	Two-way		5	56	26	152	136	Yes	Yes	Yes
88	D4	Peoria (072)	Peoria	US-24 (Adams) SB Ramps & US-150 (War Memorial)	NB	One-way	Stop Ramps	2	87	44	130	130	Yes	Yes	No
89					SB	One-way		2	85	27	151	102	N/A	Yes	Yes
90	D4	Peoria (072)	Peoria	US-150 (War Memorial) & Allen	WB	Two-way	Signalized	7	90	45	153	83	No	Yes	Yes
91	D4	Peoria (072)	West Peoria	Sterling & IL-8 (Farmington)	SB	Two-way	Signalized	2	88	49	127	85	No	Yes	Yes
92	D4	Tazewell (090)	East Peoria	River-Tractor & IL-40 (Kumpf-Washington)	SB	Two-way	Signalized	5	43	17	126	90	Yes	Traversable	Yes
93	D5	McLean (057)	Bloomington	Hershey & IL-9 (Empire)	NB	Two-way	Signalized	5	90	43	133	87	Yes	Yes	No
94	D5	McLean (057)	Bloomington	US-150 (Morrissey) & Veterans (I-55 BUS Loop)	NB	Two-way	Signalized	6	64	54	121	71	No	Yes	Yes
95	D5	McLean (057)	Bloomington	Clearwater & Veterans (I-55 BUS)	WB	Two-way	Signalized	5	90	35	142	87	Yes	Yes	Yes
96	D5	McLean (057)	Normal	College & Veterans (I-55 BUS Loop)	EB	Two-way	Signalized	6	89	41	135	79	Yes	Yes	Yes
97	D5	McLean (057)	Unincorporated	Ft. Jesse & Veterans	EB	Two-way	Signalized	6	86	38	139	61	Yes	Yes	Yes
98	D5	Monroe (067)	Columbia	Valmeyer & IL-3 (Admiral Pkwy)	EB	Two-way	Signalized	4	88	39	137	132	Yes	Yes	Yes
99	D6	Logan (054)	Lincoln	Malerich & Woodlawn (IL-10/IL-121)	NB	Two-way	Signalized	5	90	48	130	80	No	Yes	Yes

Int. No.	Intersection Name	RT App.	Design Hourly Volume (DHV) Vehicles per hr.		Average Daily Traffic (ADT) Vehicles per day			Speed limit (mph)		Avg. Annual 2009–2012 Intersection Crashes			Avg. Annual 2009–2012 Approach Crashes		
			RT App.	Int. Thru	RT App.	Int. Thru	Total Inter-section	RT App.	Int. Thru	Total	Injury	Right-Turn	Total	Injury	Right-Turn
75	Hononegah/Elevator & 2nd (IL-251)	EB	385	670	7,600	9,075	35,975	35	—	27.00	6.00	14.75	10.75	2.50	8.75
76		WB	102	621	5,850	13,450		35	—				6.75	1	3.75
77	Roscoe (Bridge) & 2nd (IL-251)	EB	192	874	4,888	13,450	33,725	40	—	16.25	2.25	4.75	5.00	0.5	3.25
78	IL-173 (West Lane) & Perryville	EB	122	112	8,550	5,300	28,275	45	45	13.25	3.25	6.25	6.75	2	5.00
79	Winnebago/Elida & US-20 (State)	NB	233	689	2,938	7,950	21,588	40	—	7.00	1.75	2.00	2.50	0.25	2.00
80	Pine Bluff & IL-47 (Division)	WB	500	570	3,350	6,900	20,750	45	45	11.50	3.75	7.50	7.75	2.25	7.50
81	IL-113 (Lowery) & IL-47	WB	182	306	2,625	3,600	11,113	55	55	7.25	2.25	6.50	6.75	2.00	6.50
82	William R Latham Sr & IL-102	SB	208	520	7,425	6,350	23,388	—	35	19.00	2.25	13.75	14.00	1.75	13.25
83	IL-126 (Schoolhouse) & IL-47 (Bridge)	WB	377	856	3,125	6,150	18,425	40	45	8.50	1.50	5.50	5.50	1.50	5.25
84	Stevenson Dr & IL-23 (Columbus)	EB	82	508	3,550	4,763	19,688	-	45	8.50	1.50	5.25	6.00	0.75	4.75
85	US-150 (War Memorial) & Sterling/Glen	EB	353	1,019	20,800	10,750	58,050	45	45	39.25	6.75	12.25	12.25	2.50	6.00
86	Scenic & US-150 (War Memorial)	EB	352	1,041	4,625	15,250	35,975	-	45	32.50	6.25	15.75	16.75	2.25	15.00
87		NB	101	1,151	2,800	15,350	33,500	25	45	21.00	4.00	17.00	10.50	2.25	10.25
88	US-24 (Adams) SB Ramps & US-150 (War Memorial)	NB	106	1,151	2,950	15,350	33,650	25	45	21.00	4.00	17.00	6.50	1.50	6.50
89		SB	702	14	19,825	4,775	44,450	50	40	44.50	6.75	12.25	19.50	3.75	4.25
90	US-150 (War Memorial) & Allen	WB	256	1,432	4,625	14,675	35,975	-	45	33.50	6.50	16.50	16.50	2.25	15.75
91	Sterling & IL-8 (Farmington)	SB	529	331	7,550	5,100	20,050	45	40	14.00	3.00	8.75	10.75	2.50	8.50
92	River-Tractor & IL-40 (Kumpf-Washington)	SB	71	75	9,550	8,250	28,175	40	—	29.50	3.75	18.00	20.75	3.50	16.25
93	Hershey & IL-9 (Empire)	NB	176	439	9,000	10,800	39,100	35	45	26.25	5.50	1.75	12.00	1.75	6.50
94	US-150 (Morrissey) & Veterans (I-55 BUS Loop)	NB	225	1,030	4,775	12,900	36,675	45	45	22.50	3.25	9.75	8.75	1.25	6.25
95	Clearwater & Veterans (I-55 BUS)	WB	235	1,157	2,750	23,700	47,850	30	45	13.00	3.00	3.50	3.50	0.75	3.50
96	College & Veterans (I-55 BUS Loop)	EB	575	1,265	9,800	17,400	53,750	30	45	30.75	6.00	7.50	10.75	1.50	6.25
97	Ft. Jesse & Veterans	EB	263	559	11,700	13,950	48,200	35	45	14.00	3.00	5.25	5.50	1.25	4.00
98	Valmeyer & IL-3 (Admiral Pkwy)	EB	502	1,805	2,700	13,500	30,375	45	30	10.50	2.00	5.75	5.75	0.75	5.50
99	Malerich & Woodlawn (IL-10/IL-121)	NB	408	423	4,750	6,350	18,450	-	45	5.00	1.00	2.50	2.50	0.25	2.25

Int. No.	IDOT District No.	County	City	Intersection Name	RT App.	Operation of RT App.	Int. Control	Sub. Road Lanes	Int. Angle	RT Angle	Head-Turn Angle	RT Radius (ft)	Divided Hwy (RT App.)?	Channelized Island at RT App.?	Exclusive RT Lane at RT App.?
100	D6	Sangamon (084)	Chatham	Plummer & Main (IL-4)	WB	Two-way	Signalized	5	89	35	142	105	No	Yes	No
101	D6	Sangamon (084)	Springfield	White Oaks & Wabash	NB	Two-way	Signalized	4	89	39	140	123	Yes	Yes	Yes
102	D6	Sangamon (084)	Springfield	IL-4 (Veterans) & Old Jacksonville	NB	Two-way	Signalized	6	66	40	136	174	Yes	Yes	Yes
103	D6	Sangamon (084)	Springfield	Wabash & IL-4 (Veterans)	EB	Two-way	Signalized	6	83	28	150	84	Yes	Yes	Yes
104	D6	Sangamon (084)	Springfield	Southwest Plaza & IL-4 (Veterans)	EB	Two-way	Signalized	5	90	38	139	103	Yes	Yes	Yes
105	D6	Sangamon (084)	Springfield	I-72 WB Ramp & 6th (I-55 BUS)	WB	One-way	Stop	1	80	33	145	113	No	Yes	Yes
106	D6	Sangamon (084)	Springfield	I-55 NB Ramp & Toronto	NB	One-way	Signalized	2	89	33	144	126	No	Yes	Yes
107	D6	Sangamon (084)	Springfield	South Grand & Dirksen (IL-29)	WB	Two-way	Signalized	5	90	40	137	120	Yes	Yes	Yes
108	D6	Sangamon (084)	Unincorporated	Sangamon & Dirksen	WB	Two-way	Signalized	5	85	49	128	89	No	Yes	Yes
109	D7	Macon (058)	Forsyth	Barnett & Bloomington (US-51)	EB	Two-way	Signalized	5	88	27	149	113	Yes	Yes	Yes
110	D8	Madison (060)	Alton	US-67 (New Clark Bridge) & Landmarks	NB	Two-way	Signalized	4	90	52	124	118	Yes	Yes	Yes
111	D8	Madison (060)	Alton	Landmarks (IL-140) & Berm (IL-143)	SB	Two-way	Signalized	4	72	35	128	165	Yes	Yes	Yes
112	D8	Madison (060)	Unincorporated	IL-255 NB Ramps & New Poag	NB	Two-way	Stop Ramps	2	90	46	130	84	Yes	Yes	Yes
113	D8	St. Clair (082)	Cahokia	IL-157 & Mississippi (IL-3)	WB	Two-way	Signalized	4	81	30	147	112	Yes	Yes	Yes
114	D8	St. Clair (082)	Lebanon	Belleville & US-50	SB	Two-way	Stop	2	90	53	124	104	No	No	No
115	D8	St. Clair (082)	O'Fallon	I-64 EB Ramp & Green Mount	EB	One-way	Signalized	2	75	33	146	99	No	Yes	Yes
116	D8	St. Clair (082)	Unincorporated	IL-158 (Air Mobility) & IL-161 (Carlyle)	SB	Two-way	Signalized	3	78	38	139	193	Yes	Yes	Yes

Int. No.	Intersection Name	RT App.	Design Hourly Volume (DHV) Vehicles per hr.		Average Daily Traffic (ADT) Vehicles per day			Speed limit (mph)		Avg. Annual 2009–2012 Intersection Crashes			Avg. Annual 2009–2012 Approach Crashes		
			RT App.	Int. Thru	RT App.	Int. Thru	Total Inter-section	RT App.	Int. Thru	Total	Injury	Right-Turn	Total	Injury	Right-Turn
100	Plummer & Main (IL-4)	WB	106	1,044	1,800	7,550	19,788	30	40	15.00	4.75	4.25	5.00	1.25	3.75
101	White Oaks & Wabash	NB	155	673	2,700	9,000	27,275	-	45	14.75	3.50	7.00	6.00	1.50	5.25
102	IL-4 (Veterans) & Old Jacksonville	NB	573	375	15,475	8,550	47,975	55	40	31.50	8.50	8.50	13.25	3.25	4.50
103	Wabash & IL-4 (Veterans)	EB	303	1,827	12,025	13,875	53,725	45	55	31.25	7.50	10.50	8.25	2.00	4.75
104	Southwest Plaza & IL-4 (Veterans)	EB	394	1,659	639	15,225	31,564	-	45	19.25	5.00	4.25	4.50	0.75	4.25
105	I-72 WB Ramp & 6th (I-55 BUS)	WB	25	776	700	10,350	21,400	30	45	5.75	0.50	5.25	5.25	0.50	5.25
106	I-55 NB Ramp & Toronto	NB	141	90	1,650	10,350	25,525	35	45	9.25	3.00	7.00	7.00	1.75	7.00
107	South Grand & Dirksen (IL-29)	WB	122	709	7,875	8,175	33,275	35	40	17.25	3.75	5.00	4.50	0.25	2.50
108	Sangamon & Dirksen	WB	391	667	10,325	8,900	38,425	45	40	41.00	11.25	10.75	10.75	1.50	5.75
109	Barnett & Bloomington (US-51)	EB	128	683	3,550	9,100	29,100	-	40	24.75	4.25	18.50	18.50	2.75	18.25
110	US-67 (New Clark Bridge) & Landmarks	NB	775	497	13,900	11,150	38,150	50	40	20.00	5.75	8.00	10.25	2.00	6.75
111	Landmarks (IL-140) & Berm (IL-143)	SB	191	544	5,300	7,250	23,750	40	40	15.00	3.75	8.25	8.25	1.75	8.00
112	IL-255 NB Ramps & New Poag	NB	593	290	2,800	4,400	11,600	50	50	2.25	0.00	2.25	2.25	0.00	2.00
113	IL-157 & Mississippi (IL-3)	WB	287	469	5,950	6,250	24,050	45	40	19.00	6.75	5.75	5.50	1.25	5.00
114	Belleville & US-50	SB	104	435	2,900	5,800	15,850	35	45	3.00	0.75	2.00	2.50	0.25	2.00
115	I-64 EB Ramp & Green Mount	EB	434	647	3,700	11,150	28,050	45	40	19.00	4.25	6.50	9.00	1.75	6.50
116	IL-158 (Air Mobility) & IL-161 (Carlyle)	SB	341	462	3,525	5,600	20,025	—	55	19.50	3.00	14.50	16.50	1.50	14.00

Illinois Department of Transportation

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