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Safety analysis of interchange functional areas

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Safety analysis of interchange functional areas

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DEDICATION

I would like to dedicate this research effort to my brother, Tolulope Abatan for encouraging me to enroll into a graduate program, and to my friend, Adedotun Akintayo for being a true and reliable friend.

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NOMENCLATURE

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
ADA	Americans with Disabilities Act
Adv. speed	Advisory speed
AIC	Akaike Information Criterion
ANOVA	Analysis of Variance
CMF	Crash Modification Factor
DOT	Department of Transportation
FHWA	Federal Highway Administration
GIMS	Geographic Information Management System
GIS	Geographic Information System
GLM	Generalized Linear Model
HSM	Highway Safety Manual
IIA	Interchange Influence Area
ISAT	Interchange Safety Analysis Tool
ISATe	Enhanced Interchange Safety Analysis Tool
JCHRP	Joint Cooperative Highway Research Program
KABCO	Fatal, Incapacitating, Minor, Possible Injury, Property Damage only crashes
LOS	Level of Service
NHTSA	National Highway Traffic Safety Administration
QA/QC	Quality Assurance/ Quality Control
SPF	Safety Performance Function
Std. Dev	Standard Deviation
Std. Error	Standard Error

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DISCLAIMER

The findings and conclusion of this study are those of the author and do not necessarily represent the views of the Iowa Department of Transportation or Iowa State University.

ABSTRACT

Limited access facilities, such as freeways and expressways, are generally designed to the highest standards among public roads. Consequently, these facilities demonstrate crash, injury, and fatality rates that are significantly lower than other road facility types. However, these rates are generally elevated in the immediate vicinity of interchanges due to increases in traffic conflicts precipitated by weaving, merging, and diverging traffic. Given the extensive costs involved in interchange construction, it is important to discern the expected operational and safety impacts of various design alternatives. To this end, the objective of this study was to analyze the safety performance within the functional areas of interchanges. The study involves the integration of traffic crash, volume, and roadway geometric information using data from 2010 to 2014 from the state of Iowa in order to assess the relationships between these factors and frequency of crashes within the interchange functional area. Separate analyses were conducted for the freeway mainline and ramp connections.

Safety performance functions (SPFs) were estimated for the interchange mainline and ramps using negative binomial regression models, and random effects models were estimated to account correlation in crash counts at the same location over time. The results from this study suggest that speed limit and interchange configuration have a significant impact on crash rates. Lower ramp advisory speeds (10 mph to 35 mph) were associated with fewer crashes on-ramps. Off-ramps were also associated with elevated crash risk in comparison to on-ramps and freeway-to-freeway ramps.

Comparison SPF models were also developed using Iowa-specific data to relate the outcomes of these simple SPFs with Florida-specific SPFs and the national default *SafetyAnalyst* SPFs with varying results.

CHAPTER 1: INTRODUCTION

1.1 Background

Interchanges are critical to long-term access management policies. Effectively designed interchanges allow for traffic and land use control that optimize both traffic safety and operations. There are an estimated 17,800 interchanges on the U.S Interstate Highway System and about 6,900 on other access-controlled highways. Less than 35 percent of these interchanges are also located in more rural areas (Bonneson, 2012). As the U.S freeway system ages and becomes more congested, many existing systems will need rehabilitation, re-design, improvement, or even reconstruction (Torbic et al., 2009).

Many interchanges provide freeway access to other high-volume roads with more limited access control. Therefore, interchanges present potential points of conflict on such roads. There is evidence that crash risk is significantly heightened on freeways in the interchange functional area as compared to areas upstream or downstream of this area (Kiattikomol, 2005). Opportunities for conflicts and, consequently, crashes are usually also more prominent in closer proximity to interchanges. Consequently, the development of crash prediction models specific to interchanges is expected to provide better predictive capabilities as compared to broader, aggregate-level models that analyze freeways without regard to the nuances of functional area. A better or more reasonable approach may therefore involve analyzing interchange functional areas as a separate explicit facility type (Kobelo, 2013).

To this end, the Interchange Safety Analysis Tool (ISAT) was designed by the Federal Highway Administration (FHWA) to carry out safety assessments of freeway-to-freeway and freeway-to-arterial interchanges. ISAT includes analysis modules for mainline freeway segments, interchange ramps, and crossroad (non-freeway) segments. ISAT makes use of safety performance functions (SPFs), developed for *SafetyAnalyst* based upon data from California, Minnesota, Ohio, and Washington. Therefore, to provide directly applicable results for other states, calibration coefficients based on state-specific data are recommended (Torbic et al., 2007).

An attractive alternative to using calibrated SPFs from ISAT to carry out safety analyses for interchange areas is the development of jurisdiction-specific SPFs for these facility types. Jurisdiction-specific SPFs also provide the opportunity to examine additional characteristics (depending on the availability of data) that may not be possible with a calibrated SPF. The other alternative to analyze the functional area of interchanges would be operational knowledge of ISAT to calibrate the application for a particular jurisdiction. The first edition of the Highway Safety Manual (HSM) does not provide any SPFs specific to interchanges, but only provides crash modification factors (CMFs) for various treatments applied at interchanges and interchange ramp terminals (AASHTO, 2010).

The characteristics of crashes at interchange areas were examined by Torbic et al. (2009). From this study, it was observed that interchange-related fatal crashes constituted 22 percent of fatal crashes on or related to the freeway system. Thus, interchange related crashes represented a substantial percentage of fatal crashes occurring on freeways. The results from this study estimate that each interchange experiences, on average, 0.05 fatal crashes per year and about 12.5 total crashes per year when considering all crash severity levels.

As engineers develop new and unconventional designs, like the folded interchange, and aging infrastructure on state-maintained roads is upgraded to resolve present or potential issues, it will beneficial to be able to quantify the relative advantages and disadvantages of various interchange configurations. Important impacts within the interchange functional area include economic, environmental, and right-of-way impacts, as well as safety benefits or disbenefits (Riniker, 2009).

1.2 Research Objectives

Given that highway interchanges are a complex part of the interstate system, which have received only limited coverage in statewide safety analyses, the objective of this study is to carry out a series of statistical analyses focused on interchange functional areas to determine the relationship between various operational and geometric roadway factors on crashes within the functional area. Various roadway databases, a crash database, and other resources maintained by the Iowa DOT were leveraged as a part of this study. Initially, a list of about 340 interchanges in Iowa provided a starting point, which was subsequently expanded to include a total of 423 interchanges in this statewide crash analysis.

Empirical crash analyses of the interchange influence areas for various interchange configurations were conducted to contrast their safety performance. A series of SPFs, in the form of generalized linear regression models, were estimated to determine those factors associated with crashes on interchanges based on a physical classifications of interchange segments. Different interchange configurations were investigated to determine whether any specific configurations are associated with better, or worse, safety performance. The resulting SPFs can be applied to assess the expected safety performance on proposed or existing interchanges as a form of guidance for the state DOT or as part of an Interchange Justification Report (IJR).

1.3 Thesis Structure

This thesis is organized into six major chapters, which detail the background of the research problem of interest, provide context with respect to previous research in this area, outline the study methods used to carry out the data analyses, and present key research findings from this work prior to presenting final conclusions and recommendations. A brief description of these chapters follows:

- Chapter 2: Literature Review – This chapter is structured into five sections to summarize the extant literature regarding safety at the interchange influence area. The first section provides a summary of prior research, including reports to state DOTs and federal agencies related to crash risk and comparisons of crash rates. The second section provides an overview of various interchange configurations existing in the U.S. highlighting advantages and drawbacks that engineers, planners and other decision makers may consider when considering different interchange configurations. The third section explores SPFs as related to the interchange influence area and *SafetyAnalyst* software developed by the FHWA. The fourth section outlines salient factors to consider in determining the extents of the interchange influence area as prescribed by other studies.
- Chapter 3: Data Description – This chapter provides a brief description of the various roadway information and traffic crash datasets utilized in this study, in addition to

detailing the processes employed to collect additional data, as well as quality assurance/quality control procedures (QA/QC) conducted to ensure data consistency.

- Chapter 4: Methodology – The statistical methods used for the purpose of this study are described in this chapter. General formulation of the statistical methods is provided, including a justification of the appropriateness of these methods for the nature of the data that are analyzed.

- Chapter 5: Results and Discussion – This chapter begins by using an empirical method to analyze interchange safety for the different interchange configurations existing in the study area. The results of the statistical regression linear models developed for the interchange influence area are presented as well as an explanation of variables that are significant in the models developed. These results are accompanied by a discussion as to the practical implications of the findings, as well as a discussion of potential drawbacks and limitations that exist in the presented results.

- Chapter 6: Conclusion – This chapter presents the conclusions of this research study and a summary discussion of key research findings. It also details how these findings could apply to real-world problems, State DOTs and other state agencies, as well as outlines potential directions for future research.

CHAPTER 2: LITERATURE REIVIEW

2.1 General Overview

The AASHTO *Policy on Geometric Design of Highways and Streets* (also known as the "Green Book") describes an interchange as a system of interconnecting roadways, in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways or highways on different levels (AASHTO, 2011). Safety on interchanges and within the interchange functional area has long been an area of concern for civil engineering designers and researchers due to elevated crash rates in the immediate vicinity of interchanges compared to the rest of the freeway due to the higher amount of conflict introduced by merging, diverging, and weaving traffic. Mulinazzi (1973) related uniformity (consistency) of interchange design types along specific corridors, as well as design simplicity, to safety performance. Safety was also directly related to good operational characteristics on interchanges (Mulinazzi, 1973).

More recent studies have focused on specific improvements and interventions to current interchange designs to promote operational efficiency and safety at the interchange and around the interchange areas (Bonneson et al., 2003). States like Oregon have created access management policies for interchange and interchange areas and provided guidelines for interchange design and construction. Specific management strategies, such as traffic signals on the cross-street, ramp metering, access control, land use control, required weaving distances, recommended length of acceleration lanes, and interchange spacing recommendations were included in the access management policy to promote safety and efficiency (Layton, 2012).

Various state DOTs have conducted roadway safety assessments involving an analysis of crash data for interchanges proposed for redesign and reconstruction. These studies often serve as partial justification for such projects in Interchange Justification Reports (IJRs). Green et al. (2010) compared crash rates for interchange corridors with the average rates of adjacent corridors, as well as the statewide and national averages, showing higher comparative crash rates on the interchange corridors. The interchange corridors were generally different than the adjacent corridors and provided additional complexities for drivers in terms of additional conflict points, as well as potential confusion and more difficult decision-making, which resulted in higher overall crash rates (Green et al., 2010).

Fewer studies have considered interchange crashes on a statewide level. Kiattikomol et al. (2008) estimated crash prediction models that could be used for interchange segments on urban freeways in North Carolina and Tennessee. The study focuses on freeway mainline segments and excluded the cross-streets and interchange ramp segments. These models can be used for cost-benefit analyses of freeway networks as a part of long-term transportation planning. The authors recommend the models are suitable for analyses within these states or others with similar conditions (Kiattikomol, 2008).

2.2 Overview of interchange configurations in the U.S.

Various interchange configurations have emerged in the U.S over the years. The diamond interchange has been the most constructed with several improvements to its basic design over the years. This section provides a detail of different interchange configurations and their operational and safety-related performance.

2.2.1 Diamond interchanges

The diamond interchange configuration (shown in Figure 1) is the most frequently used type of interchange facility in the U.S. Nationwide, 45 percent of all urban interchanges have a diamond configuration. (Song et al., 2012). A nationwide survey of state engineers revealed diamond interchanges to be the simplest and most common type of interchange, in addition to being low-cost and easy to implement (Song et al., 2012). Compared with other interchange designs, the diamond interchange configuration entails the use of minimal right of way (Wang J, 2007).

Figure 1: Diamond interchange on I-35, Iowa

Diamond interchanges offer several other advantages. One benefit is that all traffic can leave the major roads at relatively high speeds. Driver expectations are also met because all the entrances and exits are on the right side (Thompson, 2003). A major disadvantage of the diamond interchange is the limited capacity of the ramp terminal. If the through and turning volumes are too high, the ramp terminal may not be able to handle the capacity leading to delays (AASHTO, 2011).Diamond interchanges are generally used at locations having a low crossroad AADT where the traffic is not expected to increase significantly in the future (Garber et al., 1999).

2.2.2 Full cloverleaf interchanges

When two access controlled highways intersect, a full cloverleaf interchange (shown in Figure 2) is the minimum interchange design that provides connectivity for all movements between the highways (MassDOT, 2006). The full cloverleaf interchange configuration has loop ramps in all four quadrants. The benefit of this configuration is that they allow free flow of traffic in all directions, with no traffic control (Holzman et al., 1993). However, full cloverleaf interchanges are typically plagued by congestion problems and conflicts in its weaving sections (Kaisar et al., 2015). Weaving is an undesirable traffic issue where vehicles must cross paths with each other while attempting to merge or diverge. Due to this, many of the existing interchanges that undergo reconstruction are cloverleaf interchanges (Garber et al., 1999). A statewide study carried out in Virginia also showed that the full cloverleaf configuration had a larger percentage of fixed object collisions (37%) than any other interchange type, primarily as a result of the many run-off-road type accidents on loop ramps and in weaving areas.

Figure 2: I-35/US-20 full cloverleaf interchange

2.2.3 Partial cloverleaf interchanges

Figure 3 shows a partial cloverleaf (parclo) interchange with one loop ramp. The partial cloverleaf configuration is similar to the full cloverleaf interchange design except that the loop ramps are only present in three or fewer quadrants of the interchange. It generally has a smaller footprint compared to a full cloverleaf configuration. Parclo interchanges represent about 16 percent of all interchanges in the U.S, which is twice the number of the full cloverleaf configurations (Garber et al., 1999). Partial cloverleaf interchanges also have more frequent wrong-way driving (WWD) incidents and crashes than other interchange configurations. A study by the Michigan Department of Transportation (MDOT) showed that, despite the fact that partial cloverleaf interchanges comprised only 20 percent of all interchange configurations in the state, 60 percent of all WWD incidents were associated with parclo interchanges. The use of appropriate signage was recommended to reduce WWD incidents at parclos (Zhou et al., 2014).

Figure 3: Partial cloverleaf (Parclo) on US-20

2.2.4 Directional interchanges

Directional interchange configurations (shown in Figure 4 and Figure 5) are the most effective design for connecting intersecting freeways (WSDOT, 2015). This involves the construction of a direct connector for high capacity movements within the interchange. The directional pattern has the advantage of increased speed of operations, elimination of weaving and generally higher capacity than other interchange designs that can handle intersecting freeways like full cloverleaves. Directional interchanges are not usually justified, however, due to their extremely high construction costs, right-of-way requirements and often involve the construction of multiple bridge structures (Garber et al., 1999). Directional interchanges can be full directional interchanges which use direct connections for all major left-turn movements or semi-directional configuration (shown in Figure 5) which combines the use of both loop ramps and direct connectors. Depending on the individual projects, full and semi-directional interchanges may not always be feasible or practicable due to nearby development and land use characteristics (Riniker, 2009).

Figure 4: All Directional Interchange I-480/1-29 (left) and Three leg Directional Interchange I-380/US 20 (right)

Figure 5:Semi-directional Interchange IA-58/US-218

2.2.5 Trumpet interchanges

Figure 6 shows a trumpet interchange at the location in Cedar Rapids, Iowa where US-151 terminates at US-30. The trumpet interchange, as the name implies, has the resemblance of a trumpet and is used exclusively when three intersecting legs are present and one highway terminates at another highway (Garber, 1999). In this three-leg interchange, three of the turning movements are accommodated with directional ramps and semi-directional ramps and one of the movements is accommodated with a loop ramp. The "bell" of the trumpet is oriented such that it accommodates the priority turning movements at the interchange. An advantage of implementing a trumpet interchange configuration is the low cost of construction as it can be constructed with only one bridge structure. This interchange configuration is also fully free-flowing without a need for traffic signals. However, it can be prone to accidents if the loop is not very big and disorienting to drivers navigating in the direction that utilizes the loop.

Figure 6: US 30/US151 Trumpet interchange

2.2.6 Diverging diamond interchanges

Some interchanges are designed with specific safety or operational interests in mind. They could be upgrade of popular interchanges like the diamond interchange. Figure 7 shows a diverging diamond interchange on I-80 in Iowa. The diverging diamond interchange (DDI), also called the double crossover diamond interchange (DCD), eliminates the left-turn phase at two intersections within the interchanges, which reduces traffic conflicts and the number of signal phases (Hughes et al., 2009). Since the implementation of the first DDI in the United States in 2009, the DDI has continuously gained a rising popularity with over 30 installations (Edara et al., 2015). The operational benefits of the DDI interchange and lower costs of converting a diamond interchange to a DDI have contributed to its rising popularity. In a study conducted by Claros et al. (2016) to evaluate the safety of DDIs in Missouri, DDIs offered significant safety benefits over the conventional diamond interchange. Using naïve, empirical Bayes, and cross-sectional analyses, they were able to arrive at a total crash frequency reduction of 40.8 percent to 47.9 percent. Among the different crash severity levels, the highest reduction were in fatal crashes which were 59.3 percent to 63.2 percent lower depending on the safety evaluation method used (Edara et al., 2015). Due to the complexity and unconventional nature of the DDI interchange in the U.S, there is a potential for wrong-way crashes. During human factors studies using a driving simulator, the U.S Federal Highway Administration (FHWA) was able to show that wrong-way maneuvers in DDIs were not statistically different from those in conventional diamond interchanges (Inman, 2007). Claros et al. (2015) were able to provide empirical evidence of this using real-world crash data. They showed that only 4.8 percent of all fatal and injury crashes occurring on DDIs were wrong-way crashes (Edara et al., 2015).

Figure 7: Diverging Diamond Interchange I-80/US-20 Grand Prairie Parkway, Iowa

2.2.7 Single point urban interchange

The single point urban interchange (shown in Figure 8) is usually common in urban areas with large traffic volumes. The single point urban interchange (SPUI) has been called other names including the single signal interchange, urban interchange, single-point diamond, compressed diamond and urban grade separated interchange. The SPUI converges all movements into one grade separated signalized area. It can be an overpass SPUI in which the freeway is above the crossroad, or an underpass SPUI in which the freeway is under the crossroad. The topography of the area is the major determinant of an underpass SPUI versus overpass SPUI, with the overpass SPUIs being favored in hilly areas. Some research favors SPUIs in handling capacity compared to diamonds while others favor a diamond configuration. However, there are not enough SPUIs under operation from which definitive conclusions can be drawn (Qureshi et al., 2004). The main advantage of using SPUIs is that they allow the freeway's through movement to be separated from signal phasing without a significant increase in right of way required like the diamond. They also allow for a simpler phase sequence to be used since they have only one signalized area compared to a signalized diamond interchange. However, SPUI construction cost is much higher than for a diamond interchange. It typically costs about $1-$2 million dollars more than the diamond interchanges (Qureshi et al., 2004). Several other site characteristics such as the presence of frontage roads, accommodating pedestrians and meeting ADA requirements may significantly reduce performance or increase the cost of SPUIs.

Figure 8: Single Point Urban Interchange on I-35/Mills Civic Parkway

Figure 9 shows allowed movement for the crossroads, mainline and ramps of different interchange configurations. Some movements at the ramps could be non-direct such as cloverleaf loops, fully directional such as the all-directional four leg ramps where each major movement requires the construction of a bridge, or semi-direct such as the semi-directional interchange. The interchange configuration, especially the allowed direction of movement at the ramps, may add more complexity or improve safety and operational performance of that particular interchange configuration.

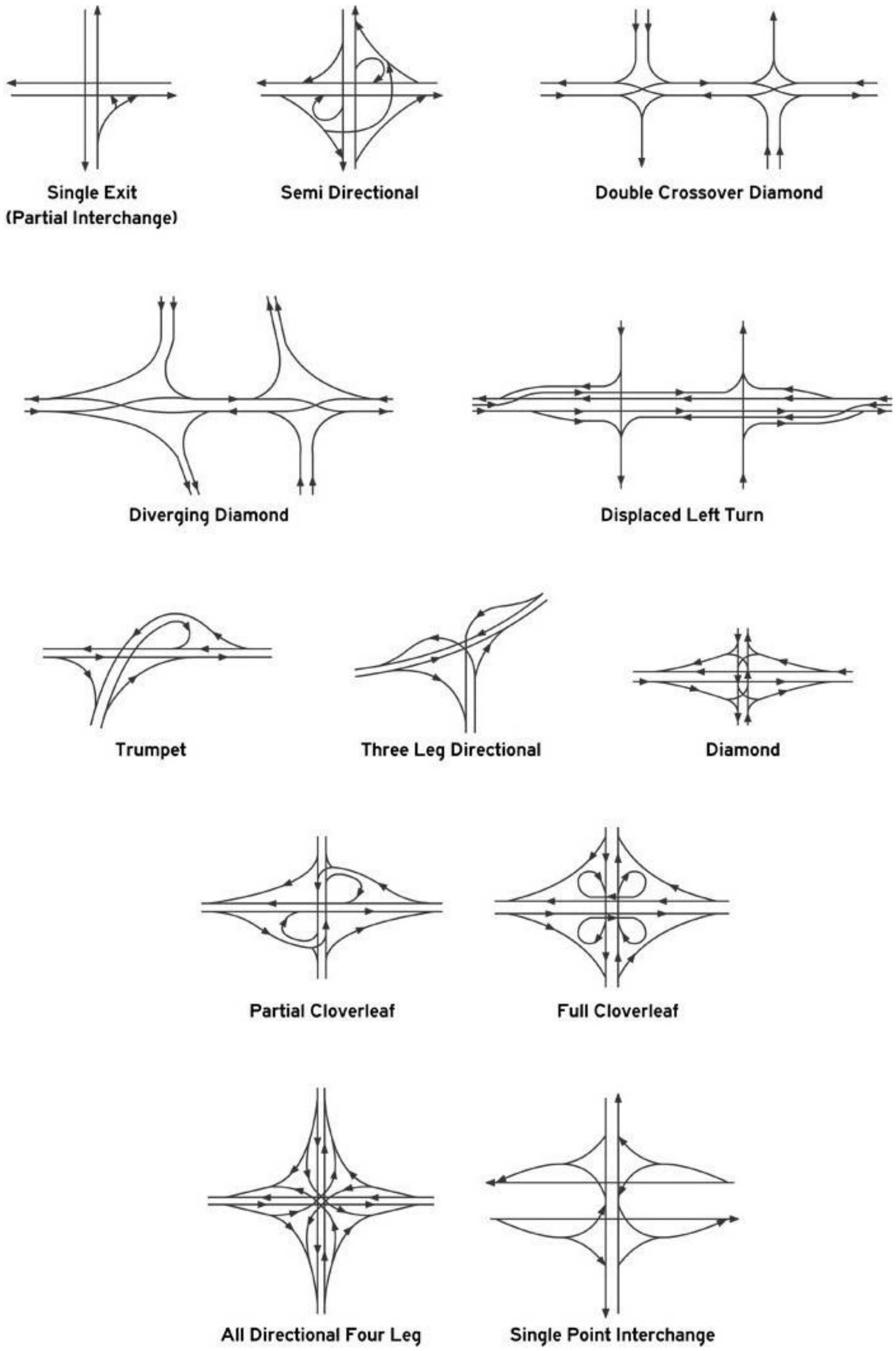


Figure 9: Directional Movements for typical interchanges (Lefler et al., 2010)

2.3 Safety Performance Function of the Interchange Influence Area (IIA)

Safety performance functions (SPFs) are mathematical equations derived from crash statistics that can be used for estimating crashes at a roadway segment or traffic corridor by relating a series of highway geometric and operational characteristics such as AADT, number of lanes, and speed limit to the number of crashes experienced. Interchange rehabilitation and reconstruction projects can especially benefit from the development of accurate interchange segment SPFs. As highlighted by the Highway Safety Manual (HSM), one of the major applications of SPFs is in determining the safety impacts of roadway design changes at a project level. Part C of the HSM provides methods for evaluation of proposed freeway and ramp design conditions against existing conditions.

Poisson and negative binomial (NB) models are two common models used in SPF development. A known limitation of applying the Poisson distribution in SPF development is that the frequency of crash data on roadway segments often has a variance that exceeds the mean. This condition, called "overdispersion" may result in biased and inconsistent parameter estimates if a Poisson regression model is used in SPF development (Lord & Mannering, 2010). The NB model accounts for overdispersion in crash data and has been widely used in transportation safety research. SPFs used for estimating crashes on freeway and ramp segments can be simple AADT-only SPFs or fully specified models that include both roadway geometric and operational characteristics. A problem that can arise from fully specified SPFs is autocorrelation of the independent variables which led the current situation where simple SPFs were used in both the HSM and default-mode *SafetyAnalyst* (Lu et al., 2014).

SafetyAnalyst uses SPFs for 16 subtypes of ramps. For the freeway within the interchange area and between interchanges, it has the capability to handle between 4 lanes and up to 8 or more lanes for urban freeway segments and between 4 lanes and 6 or more lanes for rural freeway segments. The interchange ramp categories in *SafetyAnalyst* are diamond, parclo and free-flow loops and ramps. It is an example of an end-product FHWA tool developed from SPFs that can be used for network level screening and economic or safety appraisal. It is recommended that SPFs in *SafetyAnalyst* should be calibrated for local conditions through user-defined inputs (Krammes, 2009). Since the development of safety analysis systems and tools like the HSM and *SafetyAnalyst*, several attempts have been made to examine how state or jurisdiction SPFs match up with default models, with varying results obtained.

In a study by Bornheimer et al. (2012), locally-developed SPFs for rural two-lane highways in Kansas were compared with the default SPFs in the HSM that were calibrated for the state of Kansas, with the authors finding the results to be similar. However, in a study conducted in Florida by Lu et al. (2014) to compare locally-developed SPFs versus calculating calibration factors for the *SafetyAnalyst* SPFs, the Florida-specific SPFs were found to provide better fit than the *SafetyAnalyst* default SPFs calibrated to Florida data. This study developed local, simple SPFs for Florida, for both urban and rural basic freeway segments and the interchange influence areas. Freeway segments within interchange influence areas resulted in higher crash frequencies compared to basic freeway segments. This is likely due to the complex conflict points due to weaving (merging and diverging) at the interchange influence areas. The results from this study therefore suggest that agencies could benefit from developing their own local or state-specific SPFs, similar to the examples in Figure 10, Figure 11 and Figure 12 for both basic freeway segments and the interchange areas (Lu et al., 2014).

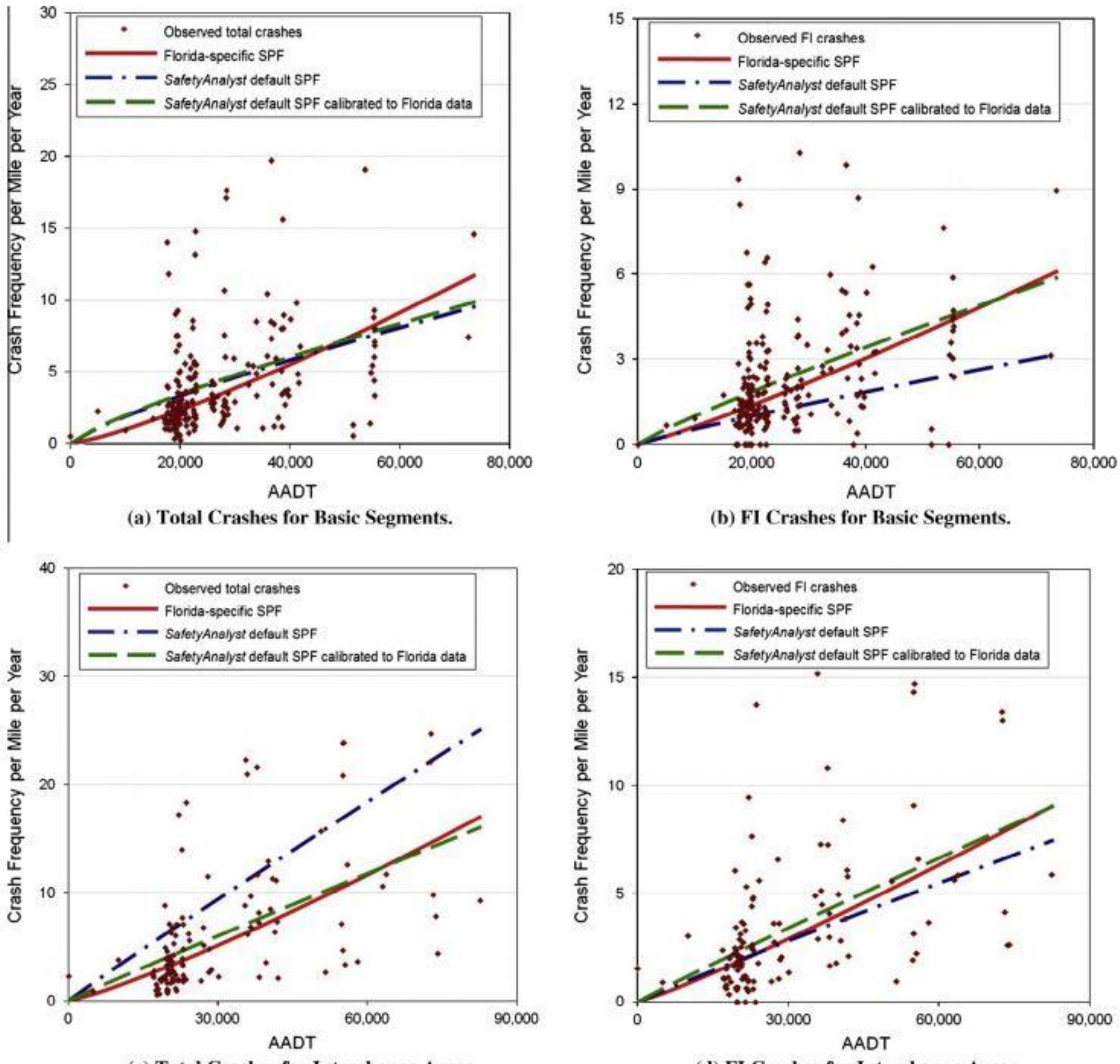
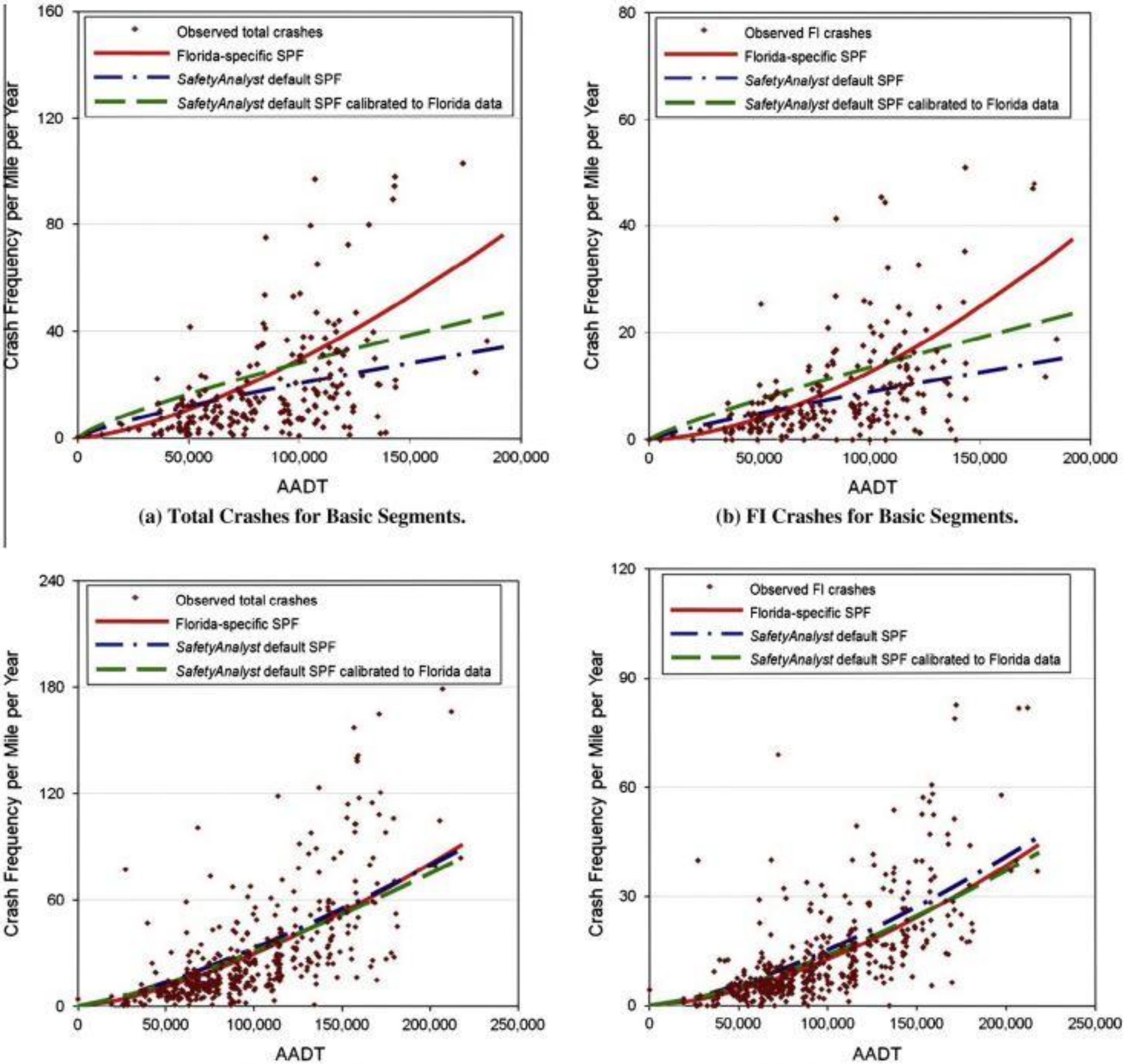


Figure 10: Predicted crashes vs AADT for urban 4-lane freeways (Lu et al., 2014)



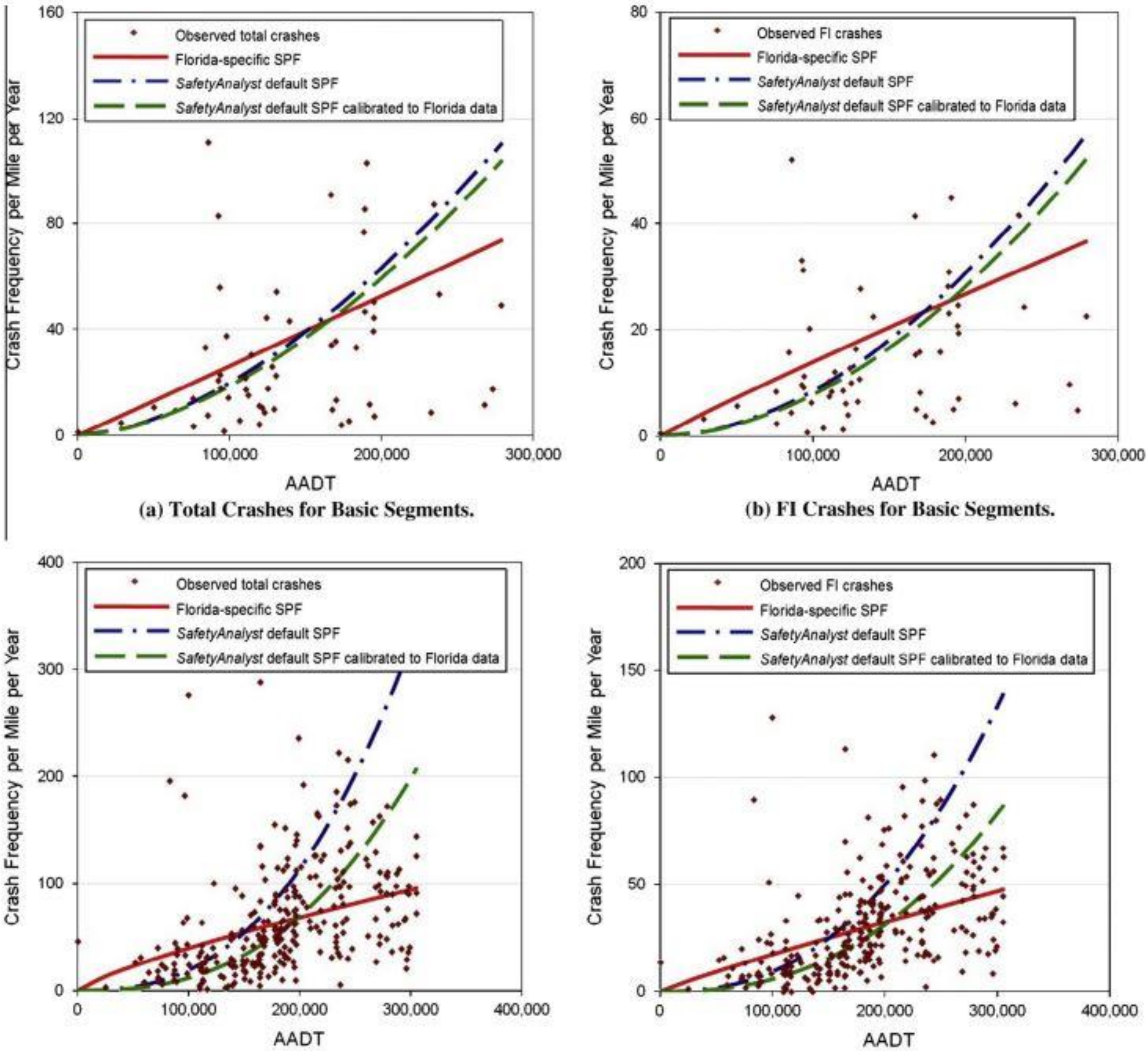
(a) Total Crashes for Basic Segments.

(b) FI Crashes for Basic Segments.

(c) Total Crashes for Interchange Areas.

(d) FI Crashes for Interchange Areas.

Figure 11: Predicted crashes vs. AADT for urban 6-lane freeways (Lu et al., 2014)



(a) Total Crashes for Basic Segments.

(b) FI Crashes for Basic Segments.

(c) Total Crashes for Interchange Areas.

(d) FI Crashes for Interchange Areas.

Figure 12: Predicted crashes vs. AADT for urban (8+)-lane freeways (Lu et al., 2014)

It is becoming evident through past and on-going research that developing calibrated SPFs or locally-developed SPFs could provide better-fitting models than SPFs developed using data from other states or jurisdictions. The Interchange Safety Analysis Tool (ISAT) is based upon data from four states in the U.S and SPFs from the tool are only valid for application to states and time periods for which the models were developed. The SPFs were developed using available data from FHWA Highway Safety Information System (HSIS) for California (1997-2001), Minnesota (1995-1999), Ohio (1997-1999), and Washington (1993-1996) (Torbic et al., 2009). ISAT has an input for calibration coefficients that allow calculations performed within ISAT to reflect better local safety experience.

The FHWA recommends that the calibration coefficients be updated annually, with more recent data from current research works. When crash data is available, ISAT allows the user to input the data and the results are combined with those from the default SPFs using an empirical Bayes (EB) methodology. The EB method improves reliability of estimated average crash frequency by pooling the estimate from the predictive crash model and site-specific crash data. The EB Method proportions the expected crash frequency from site-specific crash data and the predictive model based on the level of certainty that can be allocated to each of these (Bonneson et al., 2012). The interchange influence area and freeways within the interchange are classified differently from areas outside the interchange influence area in ISAT. This is a result of weaving, lane changing, and acceleration/deceleration that is expected to take place upstream and downstream between interchange ramps. Though ISAT uses a complicated process, it is an effective tool that can be used to predict crash frequencies for design alternatives for freeways and ramps within the interchange influence area (Torbic et al., 2007).

2.4 Interchange Influence Area

According to the ISAT user manual, the Interchange Influence Areas (IIA) for mainline freeway segments are defined to extend 0.3 miles upstream from the gore point of the first ramp of a particular interchange to approximately 0.3 miles downstream from the last ramp of a given interchange. Conversely, all freeway segments beyond this limit are assumed not to be under the influence of the interchange conditions and safety effects of weaving are expected to have been completely dissipated (Torbic et al., 2007). Many states currently do not explicitly define the IIA in their roadway inventory database. The extent of influence area of the interchange to freeway within also seems to be logical rather than empirical. In a study conducted by the by Gan et al. (2012) to develop state-specific SPFs for deployment of *SafetyAnalyst* across the state of Florida, 0.3 miles was considered as the IIA cut-off point. In this study, the IIA freeway segments were separated from other freeway segments by creating a 0.3 mile buffer for each ramp of the interchange (Gan et al., 2012). Figure 13 shows the extent of the interchange influence area from the *SafetyAnalyst* User Manual for freeway segments.

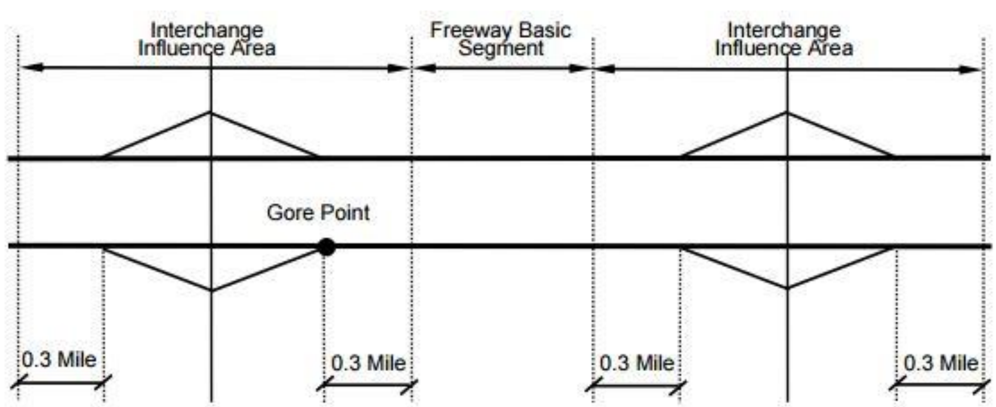


Figure 13: Interchange Influence Area (Gan et al., 2012)

In a study by McCartt et al. (2004) in Northern Virginia, the interchange influence area had more crashes despite less mileage occupancy, especially on ramps for entering and exiting freeways. The Fatality and Analysis Reporting System (FARS) and General Estimates System (GES) data indicate that in 2001, 83 percent of crashes within the interchange influence area occurred at the entrance or exit ramps (McCartt et al., 2004). Figure 14 shows crashes that are interchange-related crashes according to American National Standards Institute (ANSI).Crashes occurring 100 feet to the merge or diverge point of the interchange are classified as interchange related crashes (ANSI, 2007).

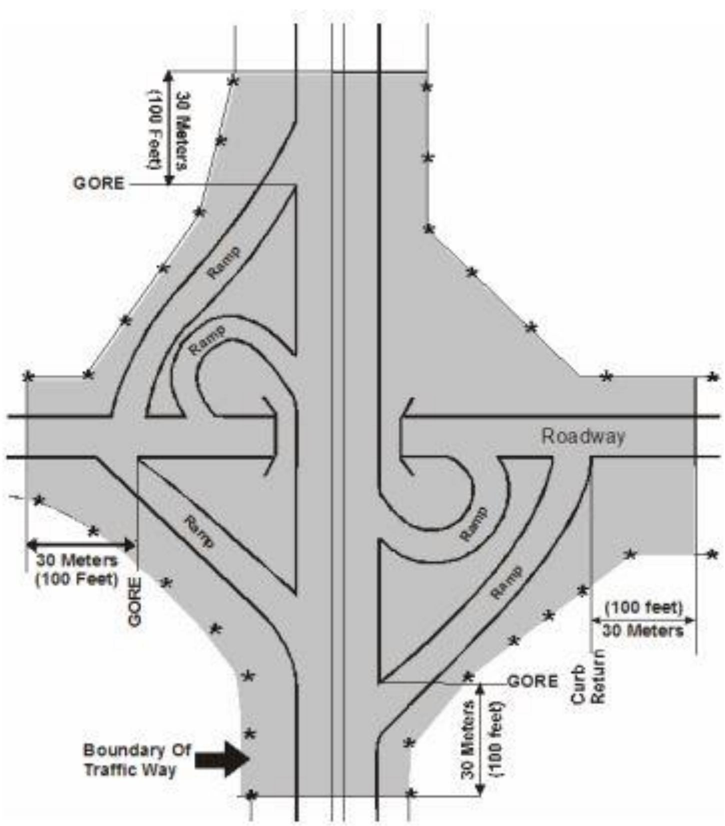


Figure 14: Interchange related crashes (Accidents which occur within the shaded area are interchange related crashes) – ANSI D16.1-2007

In a study by Moon and Hummer (2009) in North Carolina to develop safety prediction models for influence areas of freeway ramps, crashes that occurred in speed-change lanes on the freeway up to 1500 feet from the gore point were classified as freeway ramp-related crashes. The results of this study showed that entrances or exits located on the left side had as much as 150% more crashes than those on the right-side (Moon & Hummer, 2009). However, in a similar study by Zhao and Zhou (2009) to evaluate the safety of left-side ramps on freeways, the segment of freeway mainline of concern was 1000 feet from the beginning of the gore area. Their conclusion was that left-side exits are 180 percent more dangerous than right-side exits (Zhao & Zhou, 2009).

CHAPTER 3: DATA

3.1 Data Description

For the purpose of this study, roadway information and crash data from the state of Iowa were used. The Iowa Department of Transportation (DOT) maintains a series of databases as a part of its Geographic Information Management System (GIMS), which were utilized for the purposes of this study. The GIMS database contains statewide information detailing roadway geometric features and traffic operational characteristics, which are georeferenced and hosted on an Iowa DOT public server. In this study, the GIMS data were extracted for the five-year analysis period from 2010 to 2014. The project also utilized the Iowa DOT intersection database, which contains information regarding all roadway intersections on state-maintained roads in Iowa. This comprehensive database also contains information on ramp terminal control. Information about ramp advisory speeds was obtained from the Iowa DOT sign inventory. The Iowa DOT also maintains a crash database that is updated annually to include all crashes reported on Iowa roads. This data is based upon crash report forms collected by law enforcement investigating officers. Information not available or limited in any of the Iowa DOT databases such as interchange configuration and facility type were collected manually. The following sub-sections explain, in detail, the information available in each of the aforementioned databases.

3.1.1 GIMS Road Info

The *GIMS Road Info* is hosted publicly on an Iowa DOT public server as a GIS shapefile. Each road segment is coded to the shapefile at approximately the centerline of the roadway in a series of polylines. Each GIMS segment is also coded as an *MSLINK* attribute in the GIMS shapefile attribute table. The *MSLINK* represents a unique identification number for each GIMS road segment. Roadways are segmented based upon changes in roadway cross-sectional characteristics along each corridor. Segment breaks occur wherever heterogeneity is present across adjacent segments. For instance, a change in the number of lanes along a freeway would prompt the terminus of a GIMS segment. Another instance could be a change in speed limit along the freeway. As these roadway information or characteristics can change from year to year, the GIMS files are updated yearly to reflect up-to-date roadway information. The attribute table for the shapefile contains several attributes that explain roadway characteristics. Figure 15 shows a GIMS shapefile overlay on an ESRI World Imagery layer in ArcGIS.

Figure 15: Iowa GIMS Shapefile overlay on aerial imagery - interchange I-80/1A-224

3.1.2 GIMS Traffic

The *GIMS traffic* shapefile contains useful AADT information for various categories of vehicles on roads in Iowa. The segmentation process is based on the observed AADT for vehicle categories ranging from motorcycles, buses, two axle vehicles and other larger vehicles. The shapefile also contains the total AADT that includes all vehicle subtypes. As AADT can change from year to year, the *GIMS traffic* files are updated annually to reflect up-to-date roadway information.

3.1.3 GIMS Direct lane

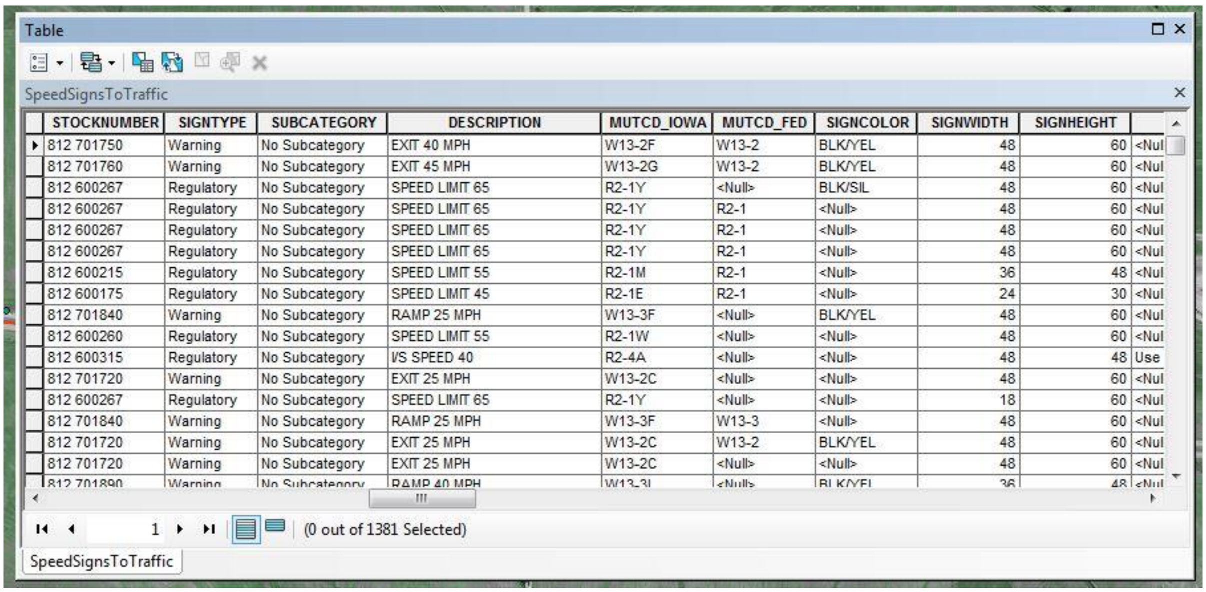
The GIMS *Direct lane* GIS shapefile contains additional detailed roadway information for each travel direction. The database contains useful information such as length and width of each road segment, shoulder width and type, width of rumble strips, and speed limit for each road segment on roads in Iowa.

3.1.4 Intersection database

The intersection database contains the location of intersections on the Iowa DOT maintained roads in both rural and urban areas. Each intersection point is geocoded into the database to include the number of legs at the intersection as well as the type of traffic control at each intersection.

3.1.5 Iowa DOT Sign Inventory

The Iowa DOT sign inventory contains a database of all Manual for Uniform Traffic Control Devices (MUTCD) approved signs located on Iowa DOT maintained roads. The MUTCD is an evolving document that contains a database of FHWA-approved traffic signs and signals. This database is coded as a point shapefile for each inventoried sign. The information contained in the sign inventory ranges from sign type, sign height, sign description and even more detail sign information such as sign retro-reflectivity. Figure 16 shows the attribute table of the Iowa DOT sign inventory.



STOCKNUMBER	SIGNTYPE	SUBCATEGORY	DESCRIPTION	MUTCD_IOWA	MUTCD_FED	SIGNCOLOR	SIGNWIDTH	SIGNHEIGHT	
812 701750	Warning	No Subcategory	EXIT 40 MPH	W13-2F	W13-2	BLK/YEL	48	60	<Nul
812 701760	Warning	No Subcategory	EXIT 45 MPH	W13-2G	W13-2	BLK/YEL	48	60	<Nul
812 600267	Regulatory	No Subcategory	SPEED LIMIT 65	R2-1Y	<Null>	BLK/SIL	48	60	<Nul
812 600267	Regulatory	No Subcategory	SPEED LIMIT 65	R2-1Y	R2-1	<Null>	48	60	<Nul
812 600267	Regulatory	No Subcategory	SPEED LIMIT 65	R2-1Y	R2-1	<Null>	48	60	<Nul
812 600267	Regulatory	No Subcategory	SPEED LIMIT 65	R2-1Y	R2-1	<Null>	48	60	<Nul
812 600215	Regulatory	No Subcategory	SPEED LIMIT 55	R2-1M	R2-1	<Null>	36	48	<Nul
812 600175	Regulatory	No Subcategory	SPEED LIMIT 45	R2-1E	R2-1	<Null>	24	30	<Nul
812 701840	Warning	No Subcategory	RAMP 25 MPH	W13-3F	<Null>	BLK/YEL	48	60	<Nul
812 600260	Regulatory	No Subcategory	SPEED LIMIT 55	R2-1W	<Null>	<Null>	48	60	<Nul
812 600315	Regulatory	No Subcategory	I/S SPEED 40	R2-4A	<Null>	<Null>	48	48	Use
812 701720	Warning	No Subcategory	EXIT 25 MPH	W13-2C	<Null>	<Null>	48	60	<Nul
812 600267	Regulatory	No Subcategory	SPEED LIMIT 65	R2-1Y	<Null>	<Null>	18	60	<Nul
812 701840	Warning	No Subcategory	RAMP 25 MPH	W13-3F	W13-3	<Null>	48	60	<Nul
812 701720	Warning	No Subcategory	EXIT 25 MPH	W13-2C	W13-2	BLK/YEL	48	60	<Nul
812 701720	Warning	No Subcategory	EXIT 25 MPH	W13-2C	<Null>	<Null>	48	60	<Nul
812 701890	Warning	No Subcategory	RAMP 40 MPH	W13-3I	<Null>	BLK/YEL	36	48	<Nul

Figure 16: Attribute table, Iowa DOT sign inventory

3.1.6 Iowa DOT Crash Database

The Iowa DOT crash database contains details of the characteristics of all crashes occurring on Iowa roads. These data are aggregated into person-level, vehicle-level and crash-level files. The person-level file contains information about each vehicle occupant who was involved in a crash. These data include demographic characteristics, such as age and gender, as well as the degree of injury sustained by each occupant. The vehicle-level file contains information about each vehicle that was involved in a crash. This includes data regarding the type of vehicle, details of any damage sustained as a result of the crash, and other pertinent characteristics. The crash-level file contains information regarding the universal characteristics associated with each crash, such as the crash type, location, and severity level. Crash severity is categorized on the KABCO scale, which classify each crash based upon the worst injury suffered by any of the vehicle occupants involved in the crash. These include crashes that result in fatal (K), incapacitating (A), non-incapacitating (B), possible (C), and no (O) injuries. According to the Iowa DOT Motor Vehicle Division's Office of Driver Services, a fatal crash indicates that the crash resulted in death within 30 days of the crash. An incapacitating injury indicates that the crash led to an acute injury that prevented the victims from walking, driving or carrying out any normal activity that they were capable of before the crash occurred. Non-incapacitating injury indicates the crash led to a minor injury such as an abrasion, bruises and/or minor lacerations that is evident at the crash scene. Possible injury indicates that the crash led to a reported injury, but injuries or wounds are not evident. This includes momentary loss of memory, and claim of injury, limping, complain of pain, nausea or dizziness. A property damage only crash indicates that the crash led to the damage of public or private property such as car(s), building, barriers, and/or fences. Unknown indicates that the crash victim(s) were not at the crash scene at the time of reporting (Iowa Department of Transportation Motor Vehicle Divisions's Office of Driver Services, 2015). Each crash in the database is identified by a unique attribute, the *crash key*. The crash database is updated yearly by the Iowa DOT using information from the investigating officer's crash report form.

3.2 Data collection and database preparation

The data collection procedure and database preparation processes included a series of tasks carried out to compile and prepare the analysis dataset. For this study, 423 interchanges in the state of Iowa were considered. The tasks involved as a part of this process are briefly summarized here:

- Identification of the physical location of all interchanges in the state of Iowa.
- Collection of GIMS data for each interchange in the study area in ArcGIS through a series of ArcGIS filters and manual review.
- Classification of each segment based on facility and interchange configuration.
- Truncation of crossroad GIMS segments at ramp terminals and freeway mainlines along GIMS segment at distances approximately 0.25 miles from each ramp terminal (See Figure 17).
- Crashes occurring from 2010 to 2014 within the vicinity of each interchange were associated with the appropriate GIMS segment in the interchange influence area by a spatial join in ArcGIS (Figure 18). Since crashes in the crash database have not been explicitly coded as occurring on a particular facility type, a spatial selection tool in ArcGIS was used to attach crashes to either the mainline, ramp, or crossroad, depending upon which of these features the crash was located nearest to.

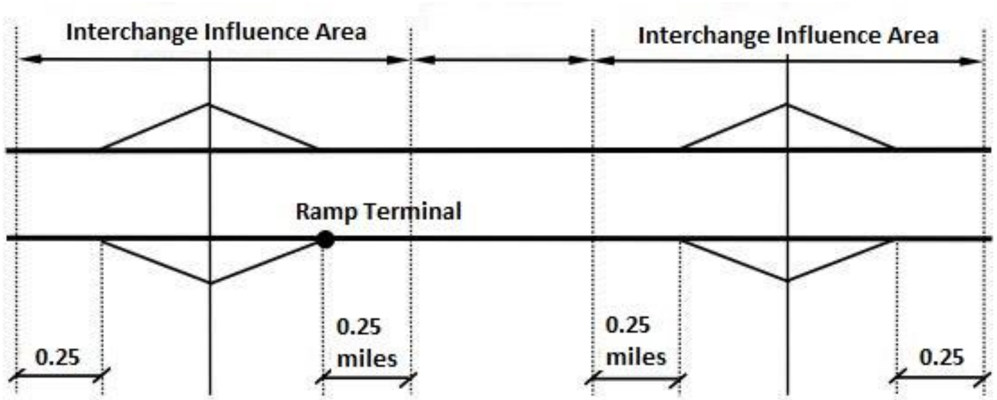


Figure 17: Interchange Influence Area

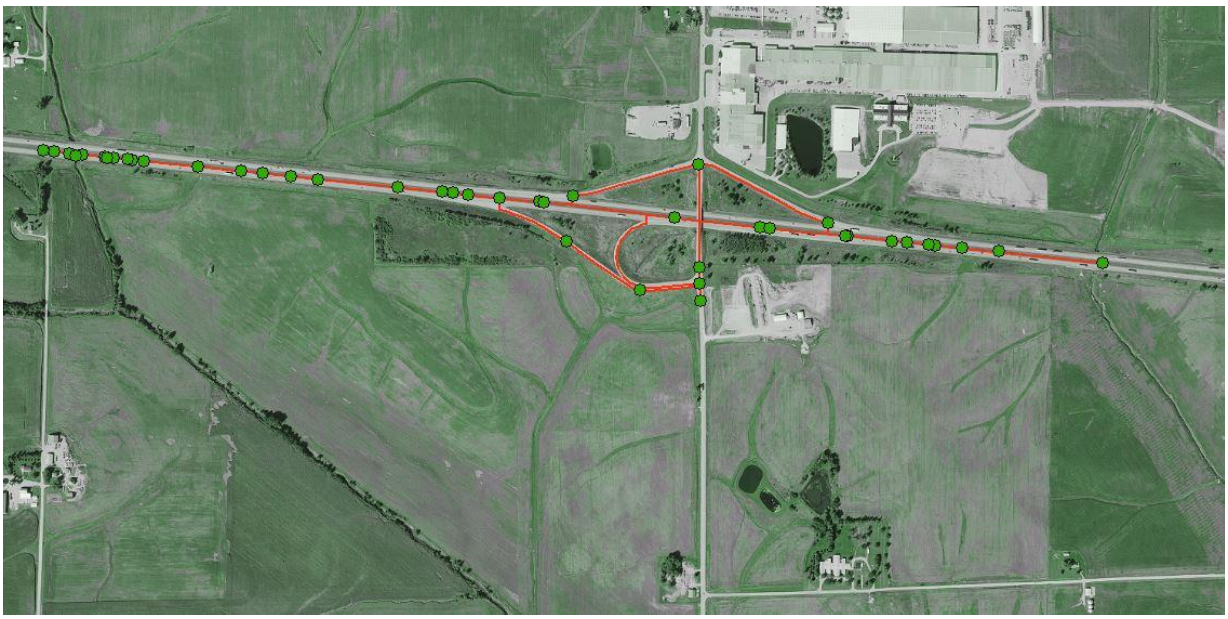


Figure 18: Geocoded crashes on interchange I-80/M Ave (2010-2014)

While it would be ideal to terminate all freeway mainline segments at 0.25 miles from the ramp terminal for consistency, there were cases were this could not be done due to the segmentation of GIMS, which are often arbitrary in length. Also, in situations where two interchanges were close in the direction of the mainline (as in Figure 19), the extent of the mainline segment attached to each interchange was split (approximately) equally between the two interchanges. This was also subject to the segmentation of the GIMS segments.

Figure 19: Freeway-to-Freeway interchanges close to each other

The most common interchange configuration in Iowa is the diamond interchange with 278 diamond interchanges included as part of this study. A distribution of interchanges by configuration in Iowa is shown in Table 1. Since there were more diamond interchanges in the state of Iowa than any other interchange configuration, the highest number of crashes – 11,550 crashes – was associated with this interchange configuration in the analysis database. Out of the 22,699 crashes occurring at vicinity interchanges between 2010 to 2014, about 51 percent were on diamond interchanges, 26 percent on partial cloverleaf interchanges, 13 percent on unconventional/hybrid interchanges and about 4 percent of the total crashes occurred on full cloverleaf interchanges.

Table 1: Interchange configuration frequency and crash rate distribution in the study area

Interchange configuration	Number of interchanges	Percent of total	Crashes, 2010-2014	Percent of total
Diamond	278	65.7	11,550	50.9
Partial cloverleaf	96	22.7	5,893	26.0
Unconventional/hybrid	23	5.4	2,879	12.7
Trumpet	10	2.4	536	2.4
Full cloverleaf	8	1.9	857	3.8
Semi-directional	3	0.7	516	2.3
Three-leg directional	3	0.7	205	0.9
All directional	1	0.2	116	0.5
Single Point	1	0.2	147	0.6
Total	423	100	22,699	100

3.3 Data QA/QC

The facility type – ramps, loop ramps, mainline and crossroad – is distinguished in GIMS in the *function* field of the GIMS attribute table. The field designates between mainline and non-mainline segments and other roadway uses. Table 2 shows part of the key for the *function* field in GIMS. The mainline code (0-14) contained both mainline and crossroad facility types. Mainline facility types were separated from crossroad facility types through a manual data review of each GIMS segment. (50) was coded for special case ramps which were mostly directional ramps, (51-54) were coded for diagonal ramps and (55-58) were coded for loop ramps in GIMS. For each interchange, segmented mainline segments were combined into one data entry by summing or taking an average of their attributes. Similarly, this was done for crossroads and ramps.

Since GIMS does not specifically differentiate between off ramps, on ramps and freeway-to-freeway ramps, information relating to these were obtained from an Iowa DOT database indicating driver behavior (acceleration and deceleration) on the roadway segment. Drivers are expected to accelerate on an on-ramp, decelerate at an off-ramp and decelerate shortly for a period and accelerate on a freeway-to-freeway ramp.

Table 2: GIMS function field

Mainline	Non-mainline	Description
0	--	NORMAL SECTION
--	50	SPECIAL CASE
1	51	NE RAMP CURVE
2	52	SE RAMP CURVE
3	53	SW RAMP CURVE
4	54	NW RAMP CURVE
5	55	NE LOOP
6	56	SE LOOP
7	57	SW LOOP
8	58	NW LOOP
9	59	1ST INNERLEG
10	60	2ND INNERLEG
11	61	3RD INNERLEG
12	62	4TH INNERLEG
13	63	5TH INNERLEG
14	64	6TH INNERLEG

Another issue associated with GIMS was that a fair amount of road segments were incorrectly coded. The speed limits for some road segments were also incorrectly coded. Also, some mainline segments with grass medians were coded as “no median” in the GIMS database. In general, the variables that were used in the final SPFs went through extensive QA/QC with aerial imagery, as this was essential for reliable results.

The presence of slip ramps at interchanges (see Figure 20) presented a major challenge because the slip ramps were inconsistently coded at the facility type function level. The joining process described above would provide erroneous results if it was applied to ramp and loop segments of such interchanges. For this reason, such segments belonging to such interchanges were remove from the analysis dataset.

Figure 20: Interchange I-380 at Coldstream Ave NE with slip ramp

Interchanges identified as having undergone major re-construction or rehabilitation during the analysis period were removed from the dataset. For example, the interchange in Figure 21 was reconstructed into a partial cloverleaf interchange from a diamond interchange between 2013 and 2014 through the addition of loop ramps in the NE and SW quadrants of the interchange. There is also a noticeable median at the crossroad of the interchange. Due to potential issues that may arise from including such interchanges, they were excluded from the analysis dataset.

Figure 21:1-29/US-34 Interchange in 2010 and 2014

3.4 Joining Segments

The process of joining GIMS segments was such that each facility type was joined to corresponding facility types. Segmented facility types at each interchange were joined to form a continuous homogenous segment. For instance, if the SW Ramp curve of a particular interchange was segmented, each GIMS segment that was part of the SW Ramp of that particular interchange was combined into one analysis segment by summarizing attributes. Crashes occurring on each segment were summarized, by summation for the analysis segment. Table 3 shows the summary function for variables in the dataset occurring across GIMS segments. For example, since it is not unusual for the number of lanes to vary along a particular interchange mainline, an average summary function was used for the number of lanes at that interchange mainline.

Table 3: Summarizing function used for GIMS and crash attributes

Attribute type	Summary function
Level of service	Mode
Median type	Mode
Median width	Average
Number of Lanes	Average
AADT	Average
Crashes Total	Sum
Crashes K	Sum
Crashes A	Sum
Crashes B	Sum
Crashes C	Sum
Crashes PDO	Sum
Divided	Average
Segment length	Sum
Surface width	Average
Surface type	Mode
Shoulder type	Mode
Shoulder width	Average
Rumble strip presence	Mode
Speed limit	Average
SLOPE	Average
KIPSANNUAL	Average
Ramp Advisory speed	Average

CHAPTER 4: STATISTICAL METHODOLOGY

Statistical investigation was carried out in SPSS and R Studio to determine the variables associated with the frequency of crashes occurring in the vicinity of interchanges. Areas of concern examined as a part of this investigation included the freeway mainlines, the cross-streets, and the ramp areas, with particular interest on the signal control at the ramp terminals. Since interchange configurations have different geometric designs that may influence crashes, the extent of this influence was also investigated.

First, an exploratory analysis of the dataset was conducted to determine general trends, discern sample sizes within various categories of interest, and to provide descriptive statistics of the datasets used. Safety performance functions (SPFs) were developed to relate the frequency of crashes within the interchange functional areas to a series of observable roadway characteristics. For traffic safety data, SPFs generally include annual average daily traffic (AADT) and length in the case of segment SPFs. These models may also consider various other site characteristics, such as shoulder width, lane width, traffic control and others. The Highway Safety Manual (HSM) contains a general formulation that is used to predict crashes on roadway segments using a negative binomial linear regression model.

$$N_{spf} = \exp(a + b \times ln(AADT) + ln(L)) \quad \text{(AASHTO, 2010)}$$

where:

N_{spf} = crash counts

AADT = annual average daily traffic on roadway in vehicles/day

L= length of roadway segment in miles

a,b = regression coefficients.

Crashes are examples of count data and are typically modeled using a Poisson or negative binomial regression model. These belong to a category of models known as generalized linear models, or GLMs. The regression coefficients of GLMs and the standard error are estimated by maximizing the likelihood or log-likelihood function of the observed parameters. Although the Poisson distribution provides a reasonable alternative to model crash data, the major drawback in using the model is that the Poisson distribution assumes the variance is equal to the mean. Crashes, by their nature of being rare and random occurrences (some segments may have high frequency of crashes and others very low frequency of crashes), are better modelled using a negative binomial framework that allows the variance to be greater than the mean. A negative binomial model regression model was therefore used in this study to analyze the overdispersed crash data used in the study.

The natural log of the AADT was used as a predictor variable in the model and the length of the segment (mainline or ramp) was used as an offset variable, implying that crashes are explicitly assumed to increase proportionately with length. This implies that doubling the length of the segment would also mean doubling the predicted crashes. Several other predictor variables besides AADT can also be included in an SPF to better predict crashes. The coefficient of the predictor variable represents the impact of that variable on the predicted number of crashes. Predictor variables with positive coefficients would be associated with an increase in the number of predicted crashes. Similarly, predictor variables with negative coefficients would be associated with a decrease in the number of predicted crashes.

The random effects model may be viewed as a generalization of the regression analysis models. When the number of data clusters is small and the number of observations per cluster is large, the cluster-specific coefficients can be treated as fixed and ordinary regression analysis with a dummy variable applied, as in the ANOVA model. Such a model is termed a naïve-pooled model. However, when the number of data clusters is large but the number of observations per cluster is small, a random effects model would be more adequate when the cluster-specific coefficients are random. Since classical statistics assumes independent and identically distributed (IID) observations, and the observation period for the analysis segments in this study is 5 years (2010-2014), it is expected that some observations may not be truly independent and developing a random effects model for the dataset may provide better results. Since each analysis segment in the dataset was coded with a unique name for the five-year study period, the approach used was to add a random effect for the unique name. This allows R Studio to resolve this non-independence by assuming a different "baseline" for each analysis segment. These individual differences can be modelled by assuming different random intercepts for each analysis segment.

Though this analysis could result in a more complicated computation, it ensures that a dataset that has observations of same category but on the other hand individually differ is taken into consideration. The Likelihood test for the best model fit for the dataset can then be performed to determine which model best fits the analysis dataset. The model with the greater log-likelihood and the log-likelihood closest to zero would provide a better description of the coefficients in the model.

CHAPTER 5: RESULTS & DISCUSSION

This chapter presents the results of a series of safety analyses focused on discerning those factors affecting the number of crashes occurring along the interchange influence area. Separate analyses focus on the performance of the interstate mainline, as well as the ramp connections.

First, an exploratory crash analysis was conducted on the different interchange configurations to identify general trends at the interchange-configuration level. The diamond interchange, which is the most frequent interchange configuration in Iowa, also has the lowest total crashes per interchange configuration per year from the empirical study. Between the years of 2010 and 2014, an average of 8.3 crashes were observed to have occurred per diamond interchanges per year and at the immediate vicinity of each diamond interchange located in Iowa. Partial cloverleaf interchanges have a higher average number of total crashes occurring with 12.3 crashes occurring on average on each parclo in the state per year. Interchanges classified as semi-directional interchanges were observed to have the highest total crashes per interchange configuration per year in the study area. Table 4 shows a summary table of the total crashes per interchange per configuration year for existing interchange configurations in Iowa between 2010 and 2014. The SPUI interchange in Iowa has the second highest total crashes per interchange configuration per year. However, it should be noted that the first SPUI in Iowa was opened to traffic in 2003 and is relatively new in the state. Also, the result for this SPUI is based only one interchange. It should also be noted that there are very few directional interchanges in Iowa as well and the crash rate results shown for these interchange configuration (shown in table 4) are based on a very small sample of those interchange configurations existing in Iowa.

Table 4: Crashes per interchange configuration from exploratory study

Interchange configuration	Number of Interchanges	Average Annual Crashes	Crashes per interchange per year
Diamond	278	2,304	8.3
Partial cloverleaf	96	1,179	12.3
Unconventional/hybrid	23	576	25.0
Trumpet	10	107	10.7
Full Cloverleaf	8	171	21.4
Semi-directional	3	103	34.3
Three-leg directional	3	42	14.0
All directional	1	23	23.0
Single-point Urban (SPUI)	1	29	29.0

A similar interchange exploratory analysis was carried out at the interchange influence areas for select interchanges in Missouri. This research, however, considered only non-terminal related crashes. The results from study show that diamond interchanges experienced on the average 5.5 total crashes per year and parclos experienced 6.0 total crashes per year (Zhang, 2016). Full cloverleaf interchanges were also observed to experience 43.5 annual crashes per interchange (Zhang, 2016). This is markedly higher than the results observed in this study. Since both states may have differing characteristics such as AADT and crash reporting procedures, it is expected that crash rates per interchange configuration may also vary. However, the very high crash rates for full cloverleaf interchanges in both states suggests that full cloverleaf interchanges and other interchange with similar high crash rates in this study may be associated with higher crash risks.

Subsequent to this exploratory analysis, a series of negative binomial models were estimated for both the mainline and ramp facilities based on the statewide interchange data.

5.1 Interchange mainline analysis

Summary statistics for the interchange mainline is shown in Table 5. There are a total of 2,015 analysis segments at the interchange mainlines investigated in this study. The mean AADT on the mainline is 21,609 vehicles per day. Most of the interchange mainlines are in the 60 mph and 65 mph category followed by mainline speed limit 70 mph. Diamond and parclo interchanges contribute to about 90 percent of all the analysis mainline segments.

Table 5: Summary statistics for variables at the interchange mainline

Parameter	Minimum	Maximum	Mean	Std. Deviation	Number of mainline segments
Total Crashes	0.00	54.00	6.01	6.96	2015
AADT	1735.71	121812.50	21609.39	19977.67	2015
mainline length	0.23	5.07	1.29	0.55	2015
35 mph speed limit	0.00	1.00	<0.01	0.05	5
45 mph speed limit	0.00	1.00	0.01	0.11	25
50 mph speed limit	0.00	1.00	<0.01	0.05	5
Sub 55 mph speed limit	0.00	1.00	0.02	0.13	35
55 mph speed limit	0.00	1.00	0.20	0.40	410
60 mph speed limit	0.00	1.00	0.03	0.17	65
65 mph speed limit	0.00	1.00	0.43	0.50	875
60/65 mph speed limit	0.00	1.00	0.47	0.50	940
70 mph speed limit	0.00	1.00	0.32	0.47	640
Diamond	0.00	1.00	0.69	0.46	1395
Parclo	0.00	1.00	0.24	0.43	480
Full Cloverleaf	0.00	1.00	0.02	0.15	45
Trumpet	0.00	1.00	0.03	0.16	50
Any Directional	0.00	1.00	0.02	0.15	45
Single Point	0.00	1.00	<0.01	0.05	5
Urban area mainline	0.00	1.00	0.40	0.49	810
4 lane mainline	0.00	1.00	0.89	0.31	1805
6 lane mainline	0.00	1.00	0.08	0.27	165
8 lane mainline	0.00	1.00	0.02	0.13	35
LOSA	0.00	1.00	0.50	0.50	1005
LOSB	0.00	1.00	0.48	0.49	975
LOSC	0.00	1.00	<0.01	<0.01	5
LOSD	0.00	1.00	<0.01	<0.01	15

About 40 percent of the interchange mainlines are in urban area boundaries as designated by the FHWA. 50 percent of the interchange mainlines in Iowa are operating under LOS A while nearly 49 percent of the interchange mainlines are operating under LOS B with only 1 percent operating under either LOS C or LOS D

Using these data, a series of negative binomial models were estimated in R Studio. The analysis segment length was used as an offset variable in this model (i.e., it was assumed that crashes increase proportionally to length). Annual average daily traffic (AADT) was included in log-form and, as such, the parameter estimate for this variable indicates the elasticity of crashes with respect to volume. The other variables in each model are binary indicators, which distinguish differences among various speed limit categories and interchange configurations.

Classical statistics assumes that observations are independent and identically distributed (IID). Within the context of this study, a five-year analysis period was used and, as such, it is expected that crash counts would be correlated within the same locations (i.e., segments and ramps) over this period. Consequently, a random effect framework is utilized to account for this correlation. To this end, a site-specific random effect term is added that is unique to each analysis segment or ramp in the database. The resulting model is shown in Table 6. The random effects framework shows significant improvement as compared to a simpler, pooled model that does not consider this within-site correlation. Examining model goodness-of-fit, the log-likelihood improves from -4810.49 to -4629.52 and the Akaike Information Criterion (AIC) improves from 9643 to 9283.

Table 6: Negative Binomial Models for interchange mainlines (Random Effects)

Parameter	Estimate	Std. Error	Z-value	P-value
(Intercept)	-7.91	0.29	-27.77	<0.01
log (AADT)	0.96	0.03	31.85	<0.01
Speed limit 70 mph (1 if yes; 0 otherwise)	-0.36	0.07	-5.09	<0.01
Speed limit 60/65 mph (1 if yes; 0 otherwise)	-0.11	0.07	-1.75	0.08
Speed limit <55 mph (1 if yes; 0 otherwise)	0.44	0.19	2.34	0.02
Parclo (1 if yes; 0 otherwise)	0.09	0.07	1.64	0.10
Cloverleaf (1 if yes; 0 otherwise)	0.32	0.14	2.26	0.02
Trumpet (1 if yes; 0 otherwise)	0.23	0.14	1.64	0.10
Any Directional (1 if yes; 0 otherwise)	0.26	0.16	1.63	0.10
SPUI (1 if yes; 0 otherwise)	0.27	0.42	0.64	0.52

When examining the results from this model, as expected, crashes are found to increase roughly proportionally with respect to exposure (i.e., AADT). A one-percent increase in volume is associated with a nearly elastic 0.96-percent increase in crashes along the interchange mainline.

An observation of the speed limit parameter variables in the interchange mainline result shows a decrease in crashes as the speed limit on the mainline increases. Interchanges with mainline speed limits of 70 mph were associated with a 30 percent decrease in crashes. The interchange mainlines with speed limits of 60 or 65 mph were associated with an 11 percent decrease in crashes. Interchange mainlines that have speed limits below 55 mph were associated with a 55 percent increase in crashes. A study carried out on interchanges in Florida showed comparable results with an increase in speed limit which was used as a continuous variable in this study resulting in a decrease in predicted crashes (Kobelo, 2013). Figure 22 shows expected crashes on interchange mainlines by AADT for the various interchange mainline speed limits. SPF graphs were developed for the various speed limits at the mainlines in Iowa. For interchange variables where no actual traffic volumes were available to develop SPFs, the available results were extrapolated for those AADT ranges as shown by the dashed portion of the SPF graphs shown in Figure 22. As there is no clear relationship between variables for the extrapolated results, the expected crashes obtained through the means may only be interpreted with caution.

It may appear counterintuitive that an increase in speed limit may result in fewer crashes but this may be indicative of the more rural nature of the mainline locations that have higher speed limits. As the corridor is developed and the interchange location is more urbanized, it is assumed that the speed limit of such interchange mainlines would be reduced.

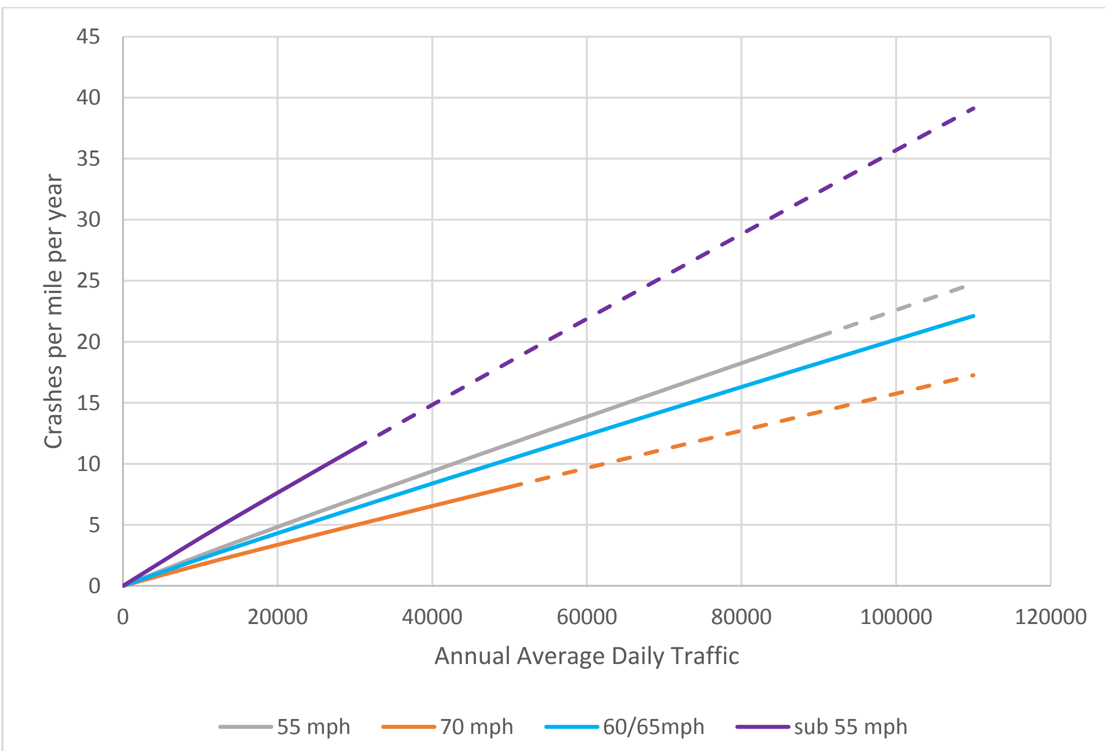


Figure 22: Crashes per mile vs Annual Average Daily Traffic by speed limit

Differences among various interchange configurations in terms of crashes on the mainline were also investigated. The diamond interchange was used as a baseline for the various interchange configuration variables in the model. The results show that Partial cloverleaf interchange mainlines were associated with about 10 percent more crashes while full cloverleaf interchange mainlines were associated with about 38 percent more crashes on average. The mainlines of directional interchanges were associated with 30 percent more crashes but resulted in lesser crash risk compared to full cloverleaf interchanges. Figure 23 shows expected crashes on the interchange mainline by AADT for different interchange configurations. For the parclo and full cloverleaf interchanges, the results were extrapolated when no actual traffic volumes were available as shown by dashed portion of the SPF graphs.

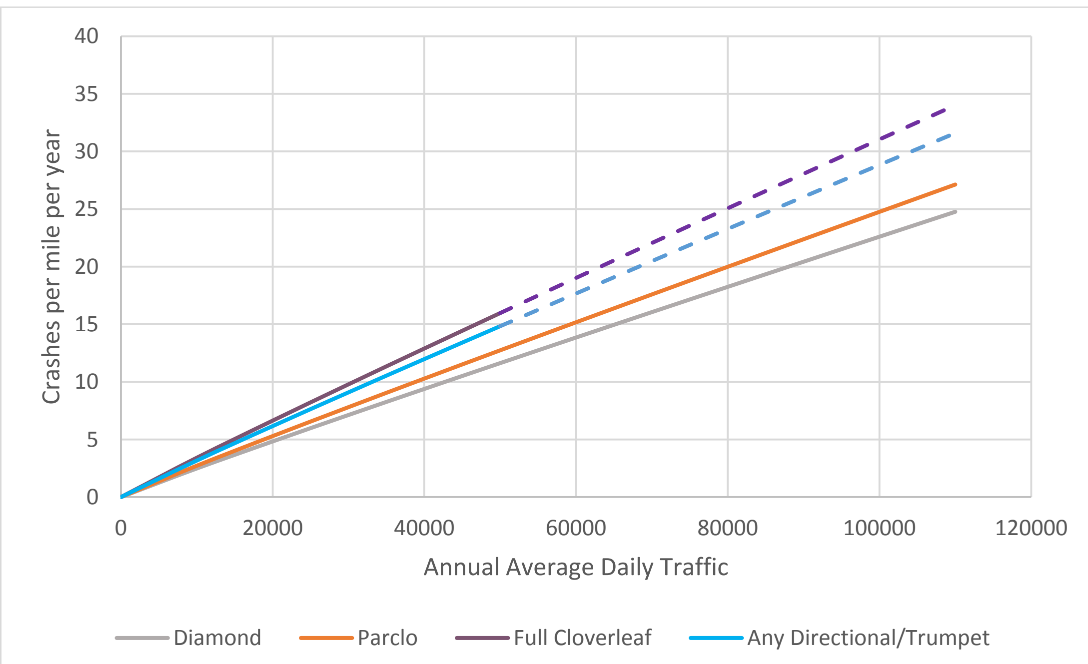


Figure 23: Crashes per mile vs Annual Average Daily Traffic by interchange configuration

5.2 Interchange ramp analysis

Table 7 shows summary statistics for the interchange ramps. This dataset has 8125 ramp analysis segments, representing ramps at interchanges in Iowa that were investigated. About 86 percent of the ramps have no advisory speed with about 7 percent of ramps having advisory speeds of between 10 mph and 35 mph and about 7 percent having advisory speeds between 40 mph and 55 mph. Most of the ramps are 1-lane ramps representing about 96 percent of ramps in the analysis dataset. Diagonal ramps represent 87 percent and Loop ramps 13 percent of the interchange ramps dataset. On-ramps represent 48 percent, off-ramps 47 percent and freeway-to-freeway ramps 5 percent of the entire ramps dataset. 93 percent of interchange ramps are operating at either LOS A or LOS B.

Table 7: Summary statistics for variables at the interchange ramps

Parameter	Minimum	Maximum	Mean	Std. Deviation	Number of ramp segments
Total Crashes	0.000	20.00	0.57	1.31	8125
AADT	5.00	27450.00	1806.49	2316.97	8125
Ramp length	0.04	1.09	0.28	0.09	8125
No ramp advisory speed	0.00	1.00	0.86	0.35	6960
10 mph ramp advisory speed	0.00	1.00	<0.01	0.03	5
20 mph ramp advisory speed	0.00	1.00	0.01	0.01	75
25 mph ramp advisory speed	0.00	1.00	0.04	0.19	295
30 mph ramp advisory speed	0.00	1.00	0.02	0.15	190
35 mph ramp advisory speed	0.00	1.00	0.01	0.07	45
10-35 mph ramp advisory speed	0.00	1.00	0.08	0.26	610
40 mph ramp advisory speed	0.00	1.00	<0.01	0.07	35
45 mph ramp advisory speed	0.00	1.00	0.06	0.23	450
50 mph ramp advisory speed	0.00	1.00	0.09	0.09	65
55 mph ramp advisory speed	0.00	1.00	<0.01	0.03	5
40-55 mph ramp advisory speed	0.00	1.00	0.07	0.25	555
1 lane ramp	0.00	1.00	0.96	0.20	7835
2 lane ramp	0.00	1.00	0.03	0.17	205
3 lane ramp	0.00	1.00	<0.01	0.04	10
Diagonal Ramp indicator	0.00	1.00	0.87	0.34	7010

Table 7 continued

Loop ramp indicator	0.00	1.00	0.13	0.34	1030
Freeway to Freeway ramp Ind.	0.00	1.00	0.05	0.10	410
On-ramp indicator	0.00	1.00	0.48	0.50	3890
Off-ramp indicator	0.00	1.00	0.47	0.50	3825
Signalized on-ramp indicator	0.00	1.00	0.06	0.24	480
Signalized off-ramp indicator	0.00	1.00	0.08	0.28	680
LOS A	0.00	1.00	0.51	0.50	4130
LOS B	0.00	1.00	0.42	0.49	3385
LOS C	0.00	1.00	0.01	0.12	115
LOS D	0.00	1.00	<0.01	0.09	65

As with the mainline segments, a random effects negative binomial model was also estimated for the interchange ramps. The results from this model are presented in Table 8. In comparison to a simpler, pooled model, the effects of each factor are generally similar. However, it is important to note that several factors (e.g., ramp advisory speeds 40 mph to 55 mph) are not statistically significant in the random effects model at a 95 percent confidence level. The random effects model again shows significant improvements in log-likelihood (from -7058 to -6573) and AIC (from 14135 to 13166).

The natural log of the ramp length was used as an offset variable in this model. The AADT is included in the model as a continuous variable while other variables are discretized by using a binary indicator to establish the presence or absence. The base category for the two ramp advisory speed groups were ramps that did not have advisory speed signs. The base category for on-ramp and freeway to freeway ramps were interchange off-ramps.

Advisory ramp speeds from 55 mph to 40 mph are associated with 20 percent more crashes when compared with ramps with no advisory speeds while ramp advisory speeds from 35mph to 10 mph, which is the lowest ramp advisory speed category, are associated with 27 percent fewer crashes on average (Figure 24). The procedure used to set advisory speeds on ramps has recommended by the MUTCD is based on the 85th percentile speed of free-flowing traffic, the Ball-Bank indicator test from the ramp or speed otherwise determined by an engineering study because of unusual circumstances (MUTCD, 2003). This may differ from speed setting on the interchange mainline which most often is directly related to the corridors rural or urban setting and functional class. Many of the ramps with very low advisory speeds may also often have MUTCD requirements for additional traffic safety signs such as chevrons or curve signs, which may be providing additional safety benefits and feedback to drivers on such ramps.

Table 8: Negative Binomial Models for interchange ramps (Random Effects)

Parameter	Estimate	Std. Error	Z-value	P-value
(Intercept)	-5.02	0.23	-21.97	<0.01
log (AADT)	0.71	0.03	-21.97	<0.01
Adv. Speed 55mph to 40 mph (1 if yes; 0 otherwise)	0.18	0.13	1.44	0.15
Adv. Speed 35mph to 10 mph (1 if yes; 0 otherwise	-0.32	0.13	-2.48	0.01
Freeway to Freeway ramp indicator (1 if yes; 0 otherwise)	-0.12	0.14	-0.83	0.40
On ramp indicator (1 if yes; 0 otherwise)	-0.12	0.08	-1.57	0.12
Signalized on ramp (1 if yes; 0 otherwise)	0.42	0.13	3.22	<0.01
Signalized off ramp (1 if yes; 0 otherwise)	0.76	0.11	6.61	<0.01

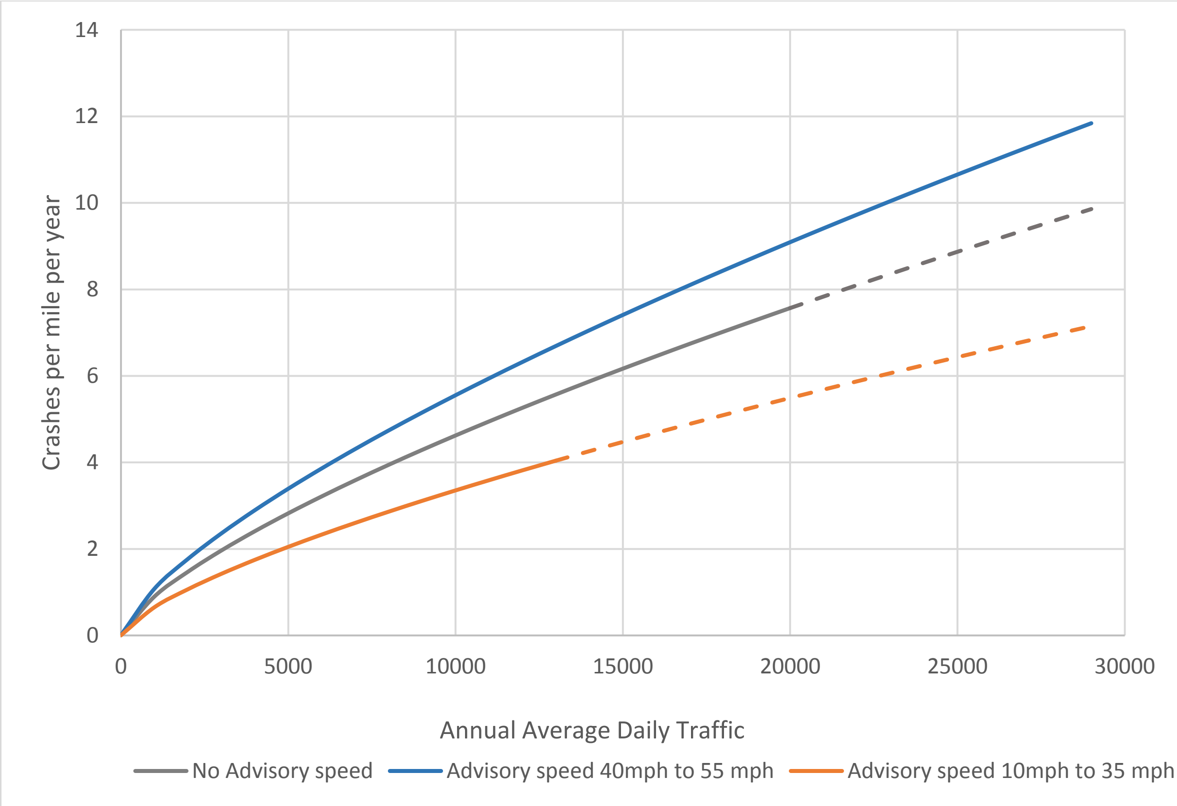


Figure 24: Crashes per mile vs Annual Average Daily Traffic by ramp advisory speeds

On-ramps and freeway-to-freeway ramps at the influence area are associated with fewer crashes in comparison to off-ramps as shown by their negative coefficients. The results show that both on-ramps and freeway-to-freeway ramps were associated with an 11 percent decrease in crashes. In a study conducted by Bared et al. (1999) to relate ramp deceleration lane length to crash risk, the models developed also suggested that off-ramps suffered more crashes compared to on-ramps. Figure 25 shows expected total crash frequencies by AADT for off-ramps, on-ramps and freeway-to-freeway ramps.

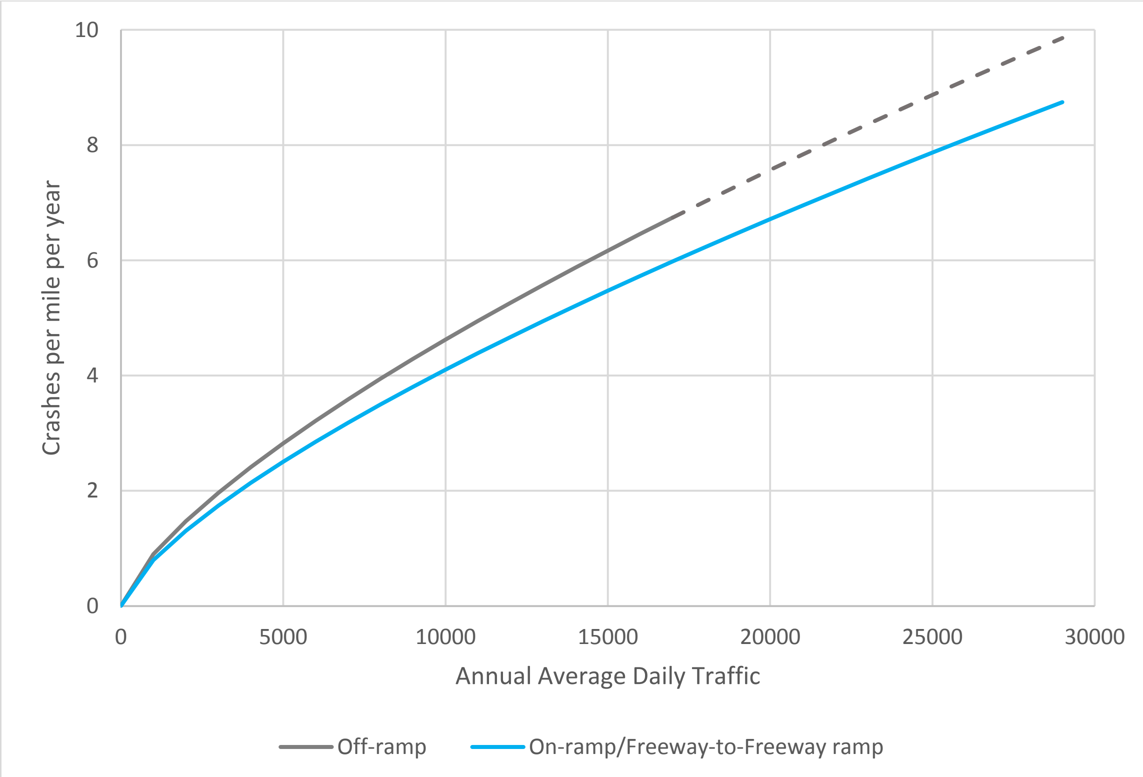


Figure 25: Crashes per mile vs Annual Average Daily Traffic for off-ramps, on-ramps and freeway-to-freeway ramps

If the beginning of an on-ramp had a traffic signal, it was coded as a signalized on-ramp while if the end of an off-ramp had a traffic signal, it was coded as a signalized off-ramp. Signalized on-ramps and off-ramps generally experienced more crashes. Signalized on-ramps were associated with a 52 percent increase in crashes when compared to unsignalized on-ramps while signalized off-ramps were associated with 114 percent increase in crashes when compared to unsignalized off-ramps. Figure 26 shows expected total crash frequencies by AADT for Signalized off-ramps, Signalized on-ramps and unsignalized ramp terminals.

Figure 26: Crashes per mile versus Annual Average Daily Traffic by ramp terminal control

The ramps were also analyzed with respect to their general interchange configuration (e.g., diamond, full cloverleaf). The diamond interchange served as the baseline configuration in this model. Table 9 shows the results for the random effects model. In general, ramps that are a part of directional interchanges, trumpet interchanges and full cloverleaf interchanges are associated with a lesser crash risk in comparison to the ramps of partial cloverleaf and SPUIs. However, most of these differences are not statistically significant when considering within-site correlation. Ramps from these interchange configurations configuration show similar trends as in the naïve pooled model but are not significant at a 95 percent confidence interval. The trumpet interchanges, however, are significant at a 95 percent confidence level in both models. The random effects model developed for the ramps has a log-likelihood of -6603.74

and AIC of 13225 compared to the naïve-pooled model with a log-likelihood of -7126.87 and AIC of 14269 resulting in a better fitting model. An SPF graph is shown in figure 27 and points out differences in safety associated with ramps from the various interchange configurations.

Table 9: Negative Binomial Models for interchange ramps by interchange configuration (Random Effects)

Parameter	Estimate	Std. Error	Z-value	P-value
(Intercept)	-5.73	0.21	-26.73	<0.01
log(AADT)	0.82	0.029	27.84	<0.01
Parclo (1 if yes; 0 otherwise)	0.05	0.08	0.72	0.47
Full cloverleaf (1 if yes; 0 otherwise)	-0.22	0.16	-1.41	0.16
Trumpet (1 if yes; 0 otherwise)	-0.75	0.25	-3.00	<0.01
Any Directional (1 if yes; 0 otherwise)	-0.32	0.19	-1.70	0.09
SPUI (1 if yes; 0 otherwise)	0.68	0.38	1.79	0.07

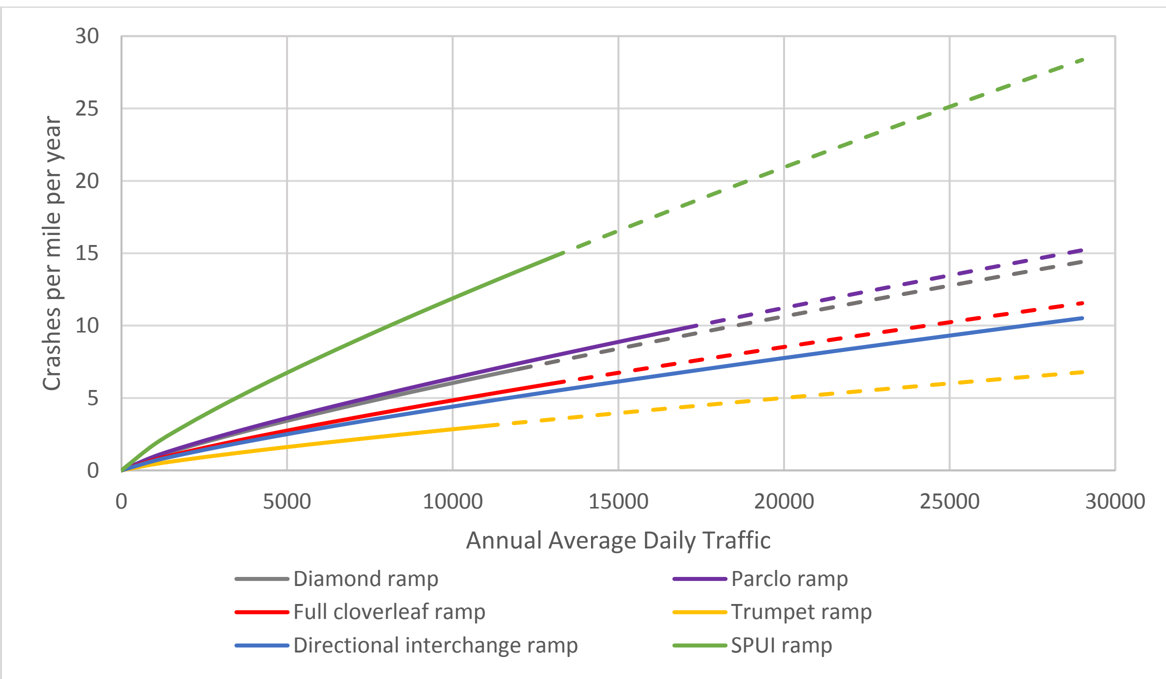


Figure 27: Crashes per mile versus Annual Average Daily Traffic by interchange configuration (ramp crashes)

5.3 Model Comparison

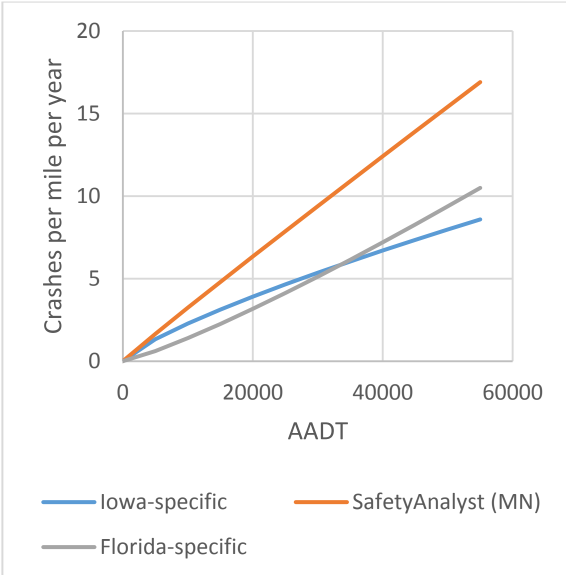
Table 10 shows simple SPFs developed for different site-subtypes at interchange influence areas. The SPFs developed for *SafetyAnalyst* for the rural mainline -4 lanes use the FHWA Highway Safety Information System (HSIS) data files from Minnesota (1995 to 1999) while for all other site subtypes compared in this study, the SPFs in *SafetyAnalyst* were developed from the state of Washington HSIS data files. In a study by Lu et al. (2014), Florida-specific SPFs were developed for different site-subtypes at the interchange influence area and compared to SPFs from *SafetyAnalyst* with varying results (Lu et al., 2014).

Table 10: Site subtype AADT only SPFs

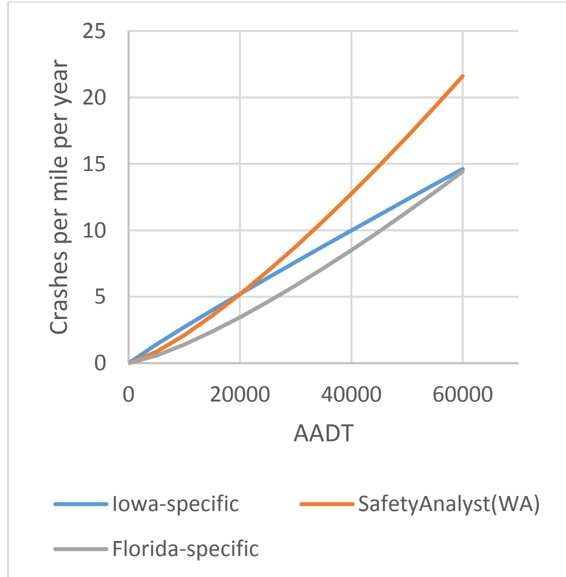
Site subtype description	Regression coefficient AADT	Regression coefficient Intercept	Std. Error	Z-value	P-value
Rural mainline-4 lanes	0.78	-6.34	0.03	24.20	<0.01
Urban mainline -4 lanes	0.94	-7.65	0.045	20.80	<0.01
Urban mainline - 6 lanes	0.55	-3.25	0.07	8.080	<0.01
Urban mainline - 8 lanes	1.13	-9.54	0.52	2.16	0.03
Rural diamond off-ramps	0.76	-5.18	0.06	12.80	<0.01
Rural diamond on-ramps	0.60	-4.13	0.07	9.20	<0.01
Urban diamond off-ramps	1.06	-6.83	0.06	17.60	<0.01
Urban diamond on-ramps	0.71	-4.44	0.07	9.95	<0.01
Rural Parclo loop off-ramps	0.39	-2.43	0.19	2.05	0.04
Rural Parclo loop on-ramps	0.72	-5.18	0.27	2.65	<0.01
Urban Parclo loop off-ramps	0.52	-2.93	0.15	3.44	<0.01
Urban Parclo loop on-ramps	0.46	-2.57	0.18	2.47	<0.01

Figure 28 compares Iowa-specific SPFs developed in this study for the interchange mainline with Florida-specific SPFs and SPFs from *SafetyAnalyst*. The results show that Iowa-specific SPFs are more similar to the Florida-specific SPFs than national default SPFs from

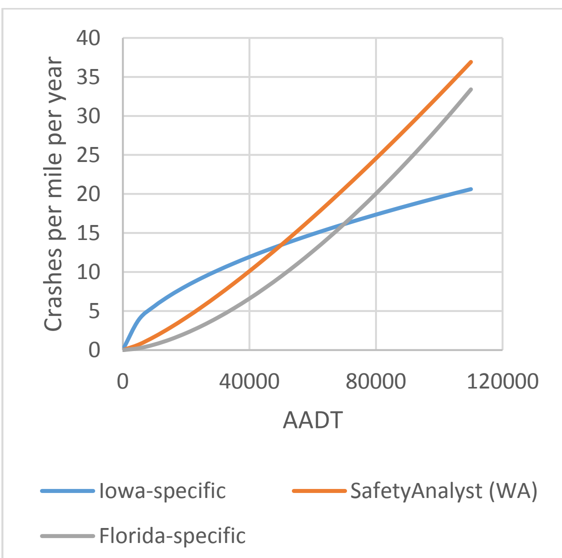
SafetyAnalyst at the interchange mainline. Figure 28(a) shows that the SPFs developed for the 4-lane rural mainline in *SafetyAnalyst* SPFs overestimate the expected crashes in comparison to both Iowa-specific and Florida-specific SPFs. Figure 28(b) shows that the SPFs developed for *SafetyAnalyst* for the 4-lane urban mainline site subtype after 20000 veh/day also overestimate the expected crashes when compared to Iowa-specific SPFs. In figure 28 (c), the *SafetyAnalyst* SPFs underestimate expected crashes until about 50000 veh/day when they begin to overestimate the expected crashes when compared to Iowa-specific SPFs. Since the SPFs developed for both Florida and *SafetyAnalyst* at the interchange mainlines were specific to only 8+ lane interchange mainlines, the graphs shown in figure 28(d) for the 8-lane urban mainlines may not completely reflect how these SPFs compare as Iowa-specific SPFs were developed for 8-lane interchange mainlines.



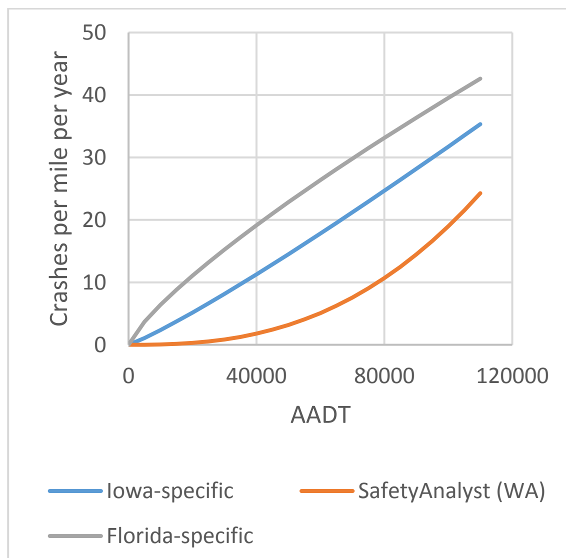
(a) Total crashes rural mainline -4 lanes



(b) Total crashes urban mainline -4 lanes



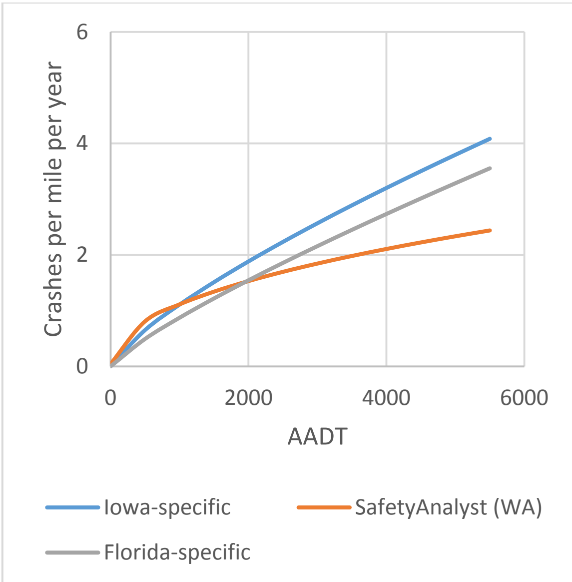
(c) Total crashes urban mainline -6 lanes



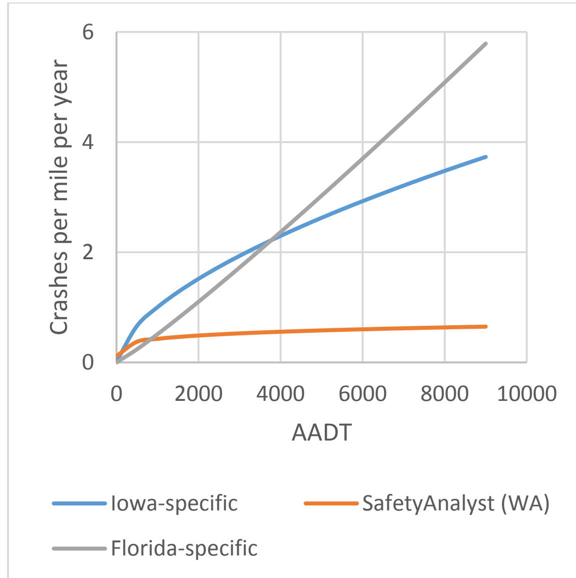
(d) Total crashes urban mainline -8 lanes
*Florida and *SafetyAnalyst* SPF models for (8+ lanes)

Figure 28: SPFs for mainlines at the Interchange Areas

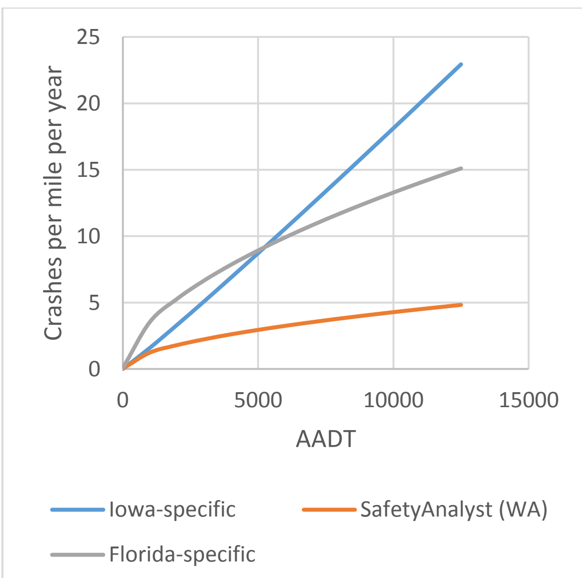
Figure 29 shows that Iowa-specific SPFs developed for diamond ramps are generally also more similar to Florida-specific SPFs than SPFs developed for *SafetyAnalyst*.



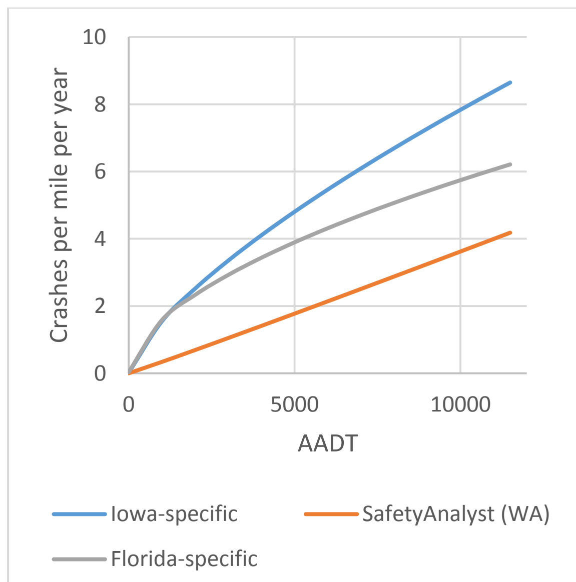
(a) Total crashes rural off-ramps



(b) Total crashes rural on-ramps



(c) Total crashes urban off-ramps

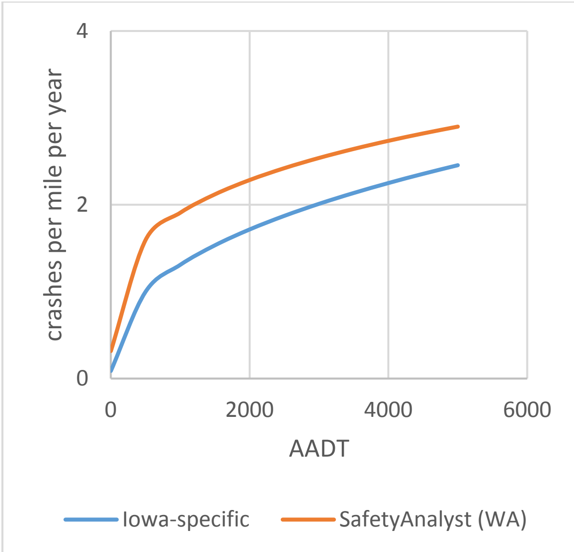


(d) Total crashes urban on-ramps

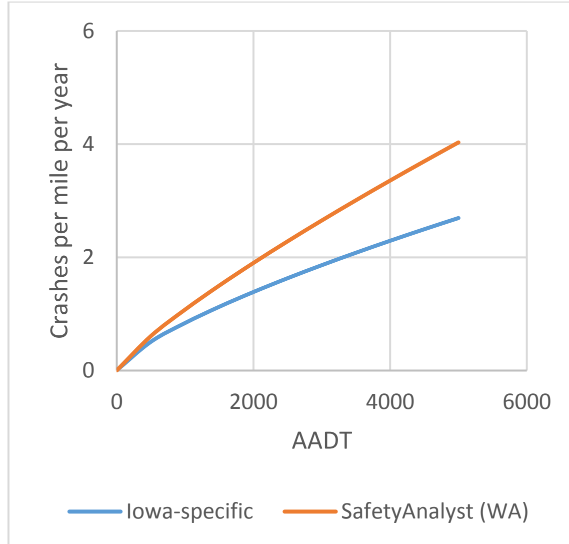
Figure 29: SPFs for Diamond Ramps

In figure 29(a), the SPFs from *SafetyAnalyst* for rural diamond off-ramps are similar to Iowa-specific SPFs until 1000 veh/day AADT when *SafetyAnalyst* underestimates expected crashes in comparison to the Iowa-specific SPFs. Figure 29 (b) shows SPFs developed for the rural diamond on-ramps, with the SPFs from *SafetyAnalyst* for all AADT ranges substantially underestimating total expected crashes in comparison to the Iowa-specific SPFs. A similar trend is observed in figure 29 (c) for total crashes at the urban diamond off–ramps, and in figure 29 (d) for the total crashes at the urban diamond on-ramps. These trends may be further validation for the development of state-specific SPFs or calibrating the national default *SafetyAnalyst* SPFs to local conditions (Lu et al., 2014).

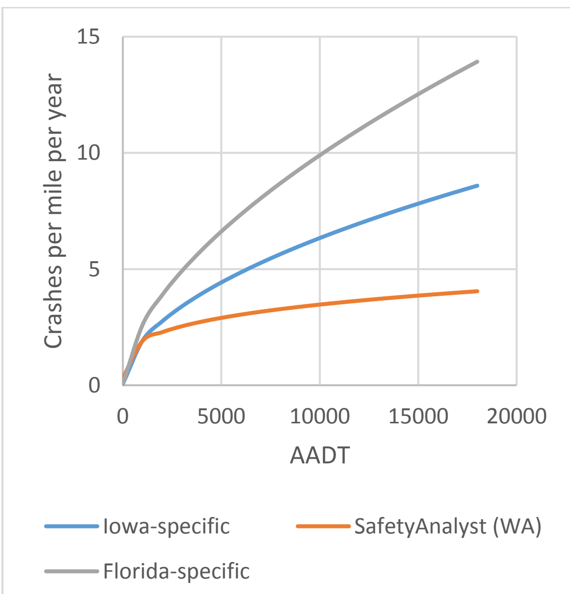
Figure 30 compares Iowa-specific SPFs for parclo loop ramps with Florida-specific and national SPFs from *SafetyAnalyst*. In figure 30(a), which shows SPFs developed for the total crashes on rural parclo off-ramp loops, the SPFs from *SafetyAnalyst* are observed to overestimate expected crashes compared to Iowa-specific SPFs for this site subtype. In figure 30(b), which are SPFs developed for the total crashes on urban parclo on-ramp loops, *SafetyAnalyst* again provides and overestimation in comparison to Iowa-specific SPFs. In the case of rural parclo off-ramp loops (figure 30(a)) and urban parclo on-ramp loops (Figure 30(b)), the Iowa-specific results are compared to only those from *SafetyAnalyst* as Florida-specific results are not available for this site subtype. However, in figure 30(c), which shows SPFs developed for the total crashes on urban parclo off-ramp loops, the default national *SafetyAnalyst* SPFs substantially underestimate the expected total crashes for this site subtype when compared to the Iowa-specific SPFs and Florida-specific SPFs. As shown in figure 30(d), after 4000 veh/day AADT, both the Florida-specific and the national default *SafetyAnalyst* SPFs are observed to predict more crashes on urban parclo on-ramp loops when compared to the Iowa-specific SPFs developed during course of this study for this site subtype.



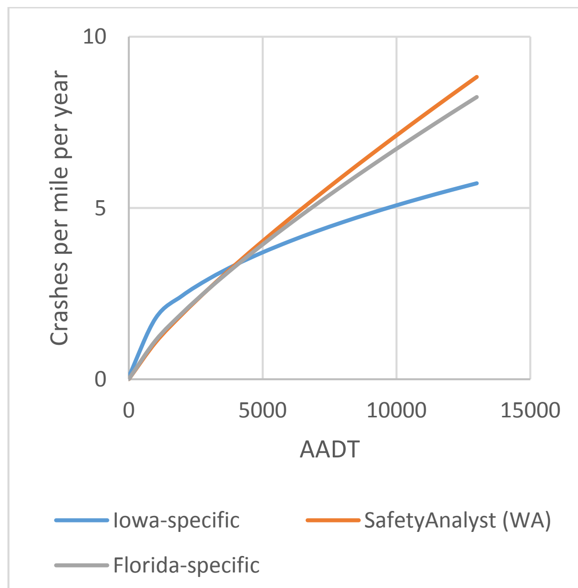
(a)Total crashes rural off-ramps
*Florida-specific SPF unavailable



(b) Total crashes urban on-ramps
*Florida-specific SPF unavailable



(c) Total crashes urban off-ramps



(d) Total crashes urban on-ramps

Figure 30: SPFs for Parclo Ramps

CHAPTER 6: CONCLUSION

Freeway interchanges represent areas of potential safety concerns. From an agency standpoint, it is critical that a data-driven approach can be utilized to inform subsequent planning and design decisions related to interchanges, particularly in light of the high costs associated with such facilities. While programs such as ISAT and ISATe provide a framework for such analyses, it is important to note these tools are based on data from only four states and must be calibrated for use in other jurisdictions.

Furthermore, recent research suggests the development of jurisdiction-specific safety performance functions (SPFs) generally results in significantly improved performance compared to SPFs that are locally calibrated. Considering these issues, this study provides a series of Iowa-specific SPFs that allow for proactive evaluation of the safety performance of interchange facilities. Separate empirical models are developed for both the interchange mainline, as well as the associated ramp facilities. The resulting analysis tools allow for a systematic approach that can be used to analyze interchanges throughout Iowa. This will aid in predicting crashes, as well as contrasting various interchange design alternatives. To this end, an additional benefit of jurisdiction-specific SPFs is that additional roadway geometric and operational characteristics can be used to support such analyses.

At the onset of the study, an exploratory crash analysis was conducted to examine safety trends for various interchange configurations. This analysis resulted in general crash rates for various common interchange configurations. These high-level trends provide general insights as to the safety performance of various interchange alternatives. They also motivate the need for more detailed investigations to distinguish the reasons for these differences in crash rates.

The diamond interchange, which is also the most common interchange configuration in Iowa, showed the lowest crash rate per interchange. This is likely due to the basic nature of the diamond interchange, which is applicable in most rural and suburban settings. In contrast, unconventional/hybrid interchanges, as well as the only SPUI in the state of Iowa, showed higher crash rates in comparison to other interchange configurations. This provides statistical support to longstanding guidance, which recommends simplicity and consistency of interchange design across a jurisdiction (Mulinazzi, 1973).

The primary contribution of this research was the development of Iowa-specific SPFs for mainline interstates and ramp facilities. Interestingly, those mainline facilities with higher speed limits experienced fewer crashes on average than those with lower posted speed limits. It is important to acknowledge this finding may simply be reflective of the fact that speed limits are generally reduced in more urbanized areas due to increased ramp density, space constraints, and other factor that make such roadways susceptible to higher crash rates.

In contrast, crashes were significantly reduced on interchange ramps with the lowest advisory speeds. However, higher advisory speeds were associated with more crashes. Consequently, additional efforts are warranted to better understand the speed-safety relationship.

On-ramps and freeway-to-freeway ramps were also associated with lesser crash risks. Signalized ramp terminals also showed elevated crash risk, particularly at the off-ramps if the terminal was signalized. Multi-vehicle rear-end crashes contribute a substantial amount to crashes occurring at signals (Xuedong, 2005). Although, in the case of the ramp terminals, this may need further investigation as many signalized ramp terminals may also be experiencing a high AADT at the terminal legs.

The SPFs developed were also compared with those from *SafetyAnalyst*, as well as results of similar research in the state of Florida. For the interchange mainline, the Iowa-specific SPFs were generally similar to those developed in Florida, and markedly different from the default SPFs from *SafetyAnalyst*. In some cases, the predicted crashes from Iowa-specific analysis were more than double that from *SafetyAnalyst* which provides strong support to the recommendation in the *SafetyAnalyst* User's Manual for calibration of the national SPFs. The findings also provide further support for the continued development of localized SPFs that truly reflect jurisdiction-specific roadway conditions.

6.1 Limitations

Ultimately, this research provides some important insights into those factors affecting the safety performance of interchange facilities. However, additional work is warranted to better understand the nature of these relationships. One limitation that arose within the context of this study is reflective of the Geographic Information Management System (GIMS) currently in use by the Iowa DOT. The GIMS database does not easily allow for a directional analysis. Consequently, the mainline analysis considers total crashes in both directions. Consequently, it is difficult to isolate the individual impacts of different approaches to hybrid interchanges, which include combinations of various ramp types.

The dataset is also somewhat limited in terms of the relative frequency of novel interchanges, such as SPUIs and diverging diamonds, which have both been installed on a limited basis in Iowa to date. Consequently, insights as to the relative performance of these and other interchange configurations should rely to a greater degree on existing guidance from outside of Iowa.

6.2 Future Research

Moving forward, there are a variety of additional analyses that could provide additional insights as to interchange safety. First, a more detailed directional analysis has the potential to provide important guidance as to how safety performance of the mainline is affected by interchange configurations. A detailed analysis of how the differences between the operating speed of the mainline and ramps influence crash frequencies may be useful in speed setting on such facilities. However, such an investigation may require access to the police crash report narratives or diagrams as the current GIMS database and the codes from the police crash report were found to be insufficient for conducting such analyses during this study.

Future research efforts are also warranted to examine the impacts of various geometric design elements, such as ramp radius, the length of acceleration/deceleration lanes, and others. For newer interchange design configurations, operational research may provide important short-term guidance as to potential safety issues associated with SPUIs, diverging diamonds, and other novel designs that cannot be assessed solely using crash data based upon the limited number of such locations.

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