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Safety and Traffic Operations at Cloverleaf Interchanges

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13. Abstract

The primary objective of this study was to assess the safety and traffic operational performance of cloverleaf interchanges in Louisiana and compare their performance with that of conventional diamond interchanges, suggesting appropriate countermeasures based on their current and predicted future performance. Data on peak-hour traffic volume, roadway geometry, and crash history from 2016-2021 were obtained and analyzed. The methodology included conducting microsimulation and traffic crash data analysis using PTV VISSIM and the FHWA's Surrogate Safety Assessment Model (SSAM). Eight interchanges were evaluated, including four cloverleaf interchanges (two with and two without Collector-Distributor [C-D] roads) and four diamond interchanges (two with roundabouts, one with signalized intersections, and one with stop-controlled intersections). The VISSIM model was calibrated and validated using traffic data (volume, travel time, and speed) from field observations and the Regional Integrated Transportation Information System (RITIS).

The findings from the microsimulation analysis indicated that cloverleaf interchanges handle high traffic volumes better than diamond interchanges but have greater safety concerns, particularly at weaving segments. Cloverleaf interchanges with C-D roads perform best under high traffic volumes (i.e., over 7000 vph total entering vehicles), while diamond interchanges with roundabouts were the best option in terms of traffic safety performance. At lower traffic volumes (i.e., below 5000 vph total entering vehicles), cloverleaf interchanges stand out in terms of operational efficiency, although diamond interchanges with roundabouts still offer the best safety performance. It was also found that increasing weaving lengths at cloverleaf interchanges without C-D roads significantly improves safety and traffic operational performance, whereas modifications to signal timings and the diameters of roundabouts at diamond interchanges have a modest impact on their performance. Through the analysis of eight interchanges, researchers found that most of the studied interchanges currently operate at an acceptable level of service, although improvements will be necessary at some interchanges in the future (e.g., after 10 and 20 years) to maintain this level.

A comprehensive crash data analysis was also conducted, and crash hot spots were identified at both cloverleaf and diamond interchanges using GIS. Additionally, safety performance functions (SPFs) were employed to predict crashes by incorporating traffic and highway parameters. Additionally, crash rates were estimated using the 2023 Louisiana DOTD guidelines. The results showed that most crashes at cloverleaf interchanges occurred at weaving segments, while at diamond interchanges, crashes primarily occurred at minor road intersections. These findings provide crucial insights for enhancing the safety and operational performance of these interchanges.

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Abstract

The primary objective of this study was to assess the traffic safety and operational performance of cloverleaf interchanges in Louisiana and compare their performance with that of conventional diamond interchanges, suggesting appropriate countermeasures based on their current and predicted future performance. Data on peak-hour traffic volume, roadway geometry, and crash history from 2016-2021 were obtained and analyzed. The methodology included conducting microsimulation and traffic crash data analysis using PTV VISSIM and the FHWA's Surrogate Safety Assessment Model (SSAM). Eight interchanges were evaluated, including four cloverleaf interchanges (two with and two without Collector-Distributor [C-D] roads) and four diamond interchanges (two with roundabouts, one with signalized intersections, and one with stop-controlled intersections). The VISSIM model was calibrated and validated using traffic data (volume, travel time, and speed) from field observations and the Regional Integrated Transportation Information System (RITIS).

The findings from the microsimulation analysis indicated that cloverleaf interchanges better handle high traffic volumes than diamond interchanges but have greater safety concerns, particularly at weaving segments. Cloverleaf interchanges with C-D roads perform best under high traffic volumes (i.e., over 7000 vph total entering vehicles), while diamond interchanges with roundabouts were the best option in terms of traffic safety performance. At lower traffic volumes (i.e., below 5000 vph total entering vehicles), cloverleaf interchanges stand out in terms of operational efficiency, although diamond interchanges with roundabouts still offer the best safety performance. It was found also that increasing weaving lengths at cloverleaf interchanges without C-D roads significantly improves traffic safety and operational performance, whereas modifications to signal timings and the diameters of roundabouts at diamond interchanges have a modest impact on their performance. Through the analysis of eight interchanges, it was found that most of the studied interchanges currently operate at an acceptable level of service, although improvements will be necessary at some interchanges in the future (e.g., after 10 and 20 years) to maintain this level.

A comprehensive crash data analysis was also conducted, and crash hot spots were identified at cloverleaf and diamond interchanges using GIS. Additionally, safety performance functions (SPFs) were employed to predict crashes by incorporating traffic and highway parameters. Additionally, crash rates were estimated considering the 2023

Louisiana DOTD guidelines. The results showed that most crashes at cloverleaf interchanges occurred at weaving segments, while at diamond interchanges, crashes primarily occurred at minor road intersections. These findings provide crucial insights for enhancing the safety and operational performance of these interchanges.

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Implementation Statement

This study performed an in-depth investigation using microsimulation and crash data analyses to compare the traffic safety and operational performance of various types of cloverleaf and diamond interchanges (e.g., with and without C-D roads, and with unsignalized, signalized, and stop-controlled intersections) under different traffic and geometric conditions. Accordingly, this study determined the most suitable interchange designs for various environments. It also assessed the performance of those eight interchanges for both current and future scenarios, providing researchers with insights into their future effectiveness and potential improvement strategies. Additionally, the study offers countermeasures to enhance the performance of the evaluated interchanges, aiding professionals in their efforts to improve traffic safety and operation at such sites.

Furthermore, the study analyzed crash data from 2016-2021, identifying the locations of hotspots and possible causes, while also suggesting appropriate countermeasures. This analysis can help professionals to better understand the critical locations of hotspots at cloverleaf and diamond interchanges and implement effective strategies to mitigate them. The study also pinpointed the primary contributing factors to these crashes and hotspots, facilitating a better understanding of the issues and prompting potential solutions. Moreover, it employed Safety Performance Functions (SPFs) and crash rate calculations for both cloverleaf and diamond interchanges, which can aid in predicting future crashes at such sites.

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Introduction

Interchanges are fundamental components of the modern highway network, playing a crucial role in facilitating the efficient movement of goods and people across vast distances. These complex structures allow for the seamless flow of traffic between different highways and roads without the need for stop-controlled or signalized intersections, thereby significantly enhancing traffic management and safety [1]. The United States has a substantial network of interchanges, with approximately 17,800 located on the Interstate Highway System and another 6,900 on other access-controlled highways. Notably, less than 35% of these interchanges are located in rural areas, highlighting their prominence in urban and suburban settings [2].

Among the myriad designs employed in the construction of these vital infrastructure elements, cloverleaf and diamond interchanges stand out due to their unique characteristics and widespread application. Cloverleaf interchanges (an example is shown in Figure 1), which constitute approximately 24% of all interchanges in the U.S., are particularly recognized for their loop ramps that facilitate left-turning movements without impeding the flow of through traffic. This design is advantageous in areas where space permits because it requires a larger footprint than other interchange types. Conversely, diamond interchanges (an example is shown in Figure 2), which represent approximately 62% of all interchanges in the U.S., are road junctions where a controlled-access highway crosses a minor road, using traffic control devices to manage turns onto the secondary roadway. They are favored for their economical layout and construction, making them suitable for both urban and rural environments [3].

Figure 1. Cloverleaf Interchange (Source: Google Map)

Figure 2. Diamond Interchange (Source: Google Map)

Despite their critical role in traffic management, cloverleaf interchanges present significant safety and operational challenges that necessitate careful examination of their current and future efficiency. The design of these interchanges often involves complex driving maneuvers, merging and diverging activities that can increase the likelihood of collisions and traffic congestion. For instance, the weaving sections—areas where traffic entering and exiting the freeway must cross paths within a short distance—are particularly problematic. These sections are often characterized by high speed differences and aggressive driving behaviors, thus elevating the risk of crashes [4].

To mitigate these risks, traffic safety analysts and engineers utilize a variety of analytical techniques and methodologies. One of the methods used to evaluate the performance of interchanges is microsimulation analysis (e.g., using PTV VISSIM). This is an advanced microsimulation software that has been extensively used in previous studies for this purpose. It offers a detailed and dynamic environment to simulate realistic road environments, driving behaviors, and traffic flows, all of which are crucial for analyzing complex cloverleaf and diamond interchanges. The software's ability to model various traffic scenarios and its flexibility in adjusting traffic inputs make it an invaluable tool in forecasting future traffic patterns and assessing potential safety improvements [5].

Additionally, crash data analysis of these interchanges (e.g., hot spot analysis) is crucial in pinpointing specific locations that exhibit high crash rates, thereby directing the focus of safety improvement measures. Safety performance functions (SPFs) are also widely used to predict the likelihood of crashes based on detailed evaluations of interchange design features, traffic density, and other relevant variables. These functions are crucial for quantifying safety concerns and formulating evidence-based interventions [6] [7].

Moreover, crash data analysis is integral to understanding the dynamics and specific factors contributing to traffic collisions at cloverleaf and diamond interchanges. This process involves examining crash reports and statistical evaluations to discern patterns and causal relationships. Such analyses not only help in assessing the effectiveness of existing safety features but also assist in designing future enhancements that could prevent collisions. By employing sophisticated statistical models and simulation tools, researchers can create nuanced strategies that address both the immediate and long-term safety challenges posed by these complex interchange configurations.

In summary, while cloverleaf and diamond interchanges are an essential part of the U.S. transportation infrastructure, their unique design and the complex driving maneuvers taking place within them necessitate conducting a comprehensive evaluation of their traffic safety and operational performance. Through targeted research, innovative engineering solutions, and robust safety assessments, the safety and operational efficiency of these interchanges can be significantly enhanced, ensuring they meet the evolving needs of modern traffic management and road safety standards.

The primary objectives of this study were to assess the traffic safety and operational performance of cloverleaf interchanges in Louisiana and compare their performance with that of traditional diamond interchanges. Additionally, the study conducted safety and traffic analyses of the current and predicted future performance of cloverleaf and diamond interchanges in Louisiana. Based on the current and predicted future performances, the study suggested countermeasures that may be implemented to mitigate the identified traffic safety and operational issues.

To achieve the above objectives, a microsimulation analysis was conducted using PTV VISSIM. Additionally, an in-depth evaluation of crash data was performed, which included calculating crash rates, conducting a hotspot analysis, and employing safety performance functions (SPFs).

Literature Review

The research team conducted a thorough review of prior relevant studies and reports examining the traffic safety and operational performance of both cloverleaf and diamond interchanges. The reviewed materials included journal articles, reports from Departments of Transportation and the Federal Highway Administration, and NCHRP reports and manuals. The findings of this task are divided into the following sections:

- Section 1: Safety at Cloverleaf Interchanges
- Section 2: Traffic Operations at Cloverleaf Interchanges
- Section 3: Safety at Diamond Interchanges
- Section 4: Traffic Operations at Diamond Interchanges
- Section 5: Safety at Other Interchanges
- Section 6: Traffic Operations at Other Interchanges
- Section 7: Hot Spot Analyses
- Section 8: Safety Performance Functions (SPFs)

Section 1: Safety at Cloverleaf Interchanges

Wang et al. (2017) explored the optimization of sight distances at the signalized ramp terminals of partial-cloverleaf interchanges to deter wrong-way entries. The study used 44 signalized ramps, leveraging aerial photography, street views, and GIS, along with a decade of crash data (2004-2013). It was found that stop line positioning between 40-60% at these interchanges minimized wrong-way driving (WWD) incidents, whereas positioning outside this range increased WWD risks. A minimum barrier distance of 21 meters at 60% stop line positioning was recommended. The study also suggested conducting further research on how driver sight distances are affected by vehicle speeds, nighttime conditions, and roadway geometry, utilizing this data to refine interchange design standards [8].

Atiquzzaman et al. (2022) modeled the risk of wrong way driving (WWD) at the exit ramp terminals of partial cloverleaf interchanges. Traffic crash data from 2009 to 2013 was used for the analysis. The study monitored seven locations for 48 hours and used Firth's penalized-likelihood logistic model. The results showed that geometric features and traffic control devices significantly influence WWD occurrences. Specifically, WWD likelihood increased with corner radii over 60 feet at entrance ramps and crossing medians but decreased with greater distances to the nearest access point and the presence of "Keep Right" signs. Higher traffic volumes reduced WWD risk at exit ramps but increased it at entrance ramps, while the presence of traffic signals lowered risk compared to unsignalized terminals. The study recommended modifications in intersection angles, channelizing islands, and median widths to impact WWD rates, but noted that the limited data call for further research [9].

Section 2: Traffic Operations at Cloverleaf Interchanges

Song et al. (2012) evaluated cloverleaf interchange capacity using microsimulation analysis by VISSIM. The study employed radar and digital cameras for traffic counts and used a multiple regression model. The findings indicated that capacity varied with the proportions of left- and right-turning vehicles and decreased when right-turners outnumbered left-turners. The study confirmed the method's effectiveness for cloverleaf interchanges, but its applicability to other types is uncertain. It also suggested reserving land for future expansions and noted limitations in extreme weather [10].

Mansourkhaki and Ghanad (2014) evaluated the optimization of loop ramp design for cloverleaf interchanges. The study compared two horizontal alignment methods: conventional spiral curves (CSC), which use a single radius based on ramp speed, and compound curves, which divide the loop ramp into three parts. These include the initial and final segments, both compound curves connecting to a central curve with a predefined radius. Differences in vertical alignment, especially in super-elevation runoff handling, were also noted. Using the Analytic Hierarchy Process (AHP), compound curves outperformed others in geometric design, driver comfort, and safety. Ant Colony Optimization (ACO) confirmed their superior efficiency and time savings. The study suggested extending this research nationally and internationally for more extensive validation [11].

Chattaraj and Subhashini (2015) analyzed traffic flow on cloverleaf interchanges and used a fuzzy logic model in MATLAB, focusing on variables such as radius of curvature, super-elevation, slope, and friction factor, with vehicle speed as the output. The study developed 24 rule sets through fuzzy clustering and excluded less significant variables, such as friction factor and super-elevation for up-ramps, from certain rules. The model, validated against empirical data, showed speed prediction errors of 8.75% for up-ramps and 6.85% for down-ramps. The results highlighted the influence of geometric factors on vehicle speeds at cloverleaf interchanges and recommended further research to improve traffic flow modeling [12].

Sutherland et al. (2018) used VISSIM to compare the operational effects of displaced partial cloverleaf interchanges (DPC) with PARCLO B-4Q. The study validated DPC simulations against empirical total travel time data and ran scenarios across various demand levels. Statistical tests confirmed that DPC generally offered shorter travel times, particularly with higher traffic demands. The research recommended DPC as a better alternative, advocating for further analysis of economic and design factors [13].

Janho et al. (2019) analyzed traffic congestion at a cloverleaf interchange in Dubai using Google Live Maps and data from Dubai's Road and Transport Authority, with projections for 2020 and 2035. They assessed congestion levels using the Highway Capacity Software (HCS), consistently finding a Level of Service (LOS) of 'F' during peak hours. The study proposed redesigns, including a semi-directional ramp and adding three or four ramps, with the four-ramp design elevating the LOS to 'A' and 'B' at peak times. However, it did not consider traffic lights, driver behavior, or vehicle types, recommending further simulation with VISSIM for a more thorough evaluation [14].

Molan and Hummer (2021) investigated parclo progressA, a modified partial cloverleaf interchange design that minimizes additional land use. This design features non-conflicting left turns from the freeway and two half signals instead of full signals. Utilizing PTV VISSIM, SSAM, and Synchro, traffic flow and safety were analyzed at 30 service interchanges, tailoring loop radii and ramp lengths for optimal speeds and transitions. Parclo progressA improved travel times through free-flow arterial movements and reduced stops; it also showed safety enhancements, though pedestrian performance was similar to conventional designs. The design required higher construction costs due to a wider bridge. The study highlighted the need for further research into driver behavior and traffic management to fully evaluate the design's real-world effectiveness [15].

Section 3: Safety at Diamond Interchanges

Claros et al. (2017) investigated the safety of diverging diamond interchanges (DDIs) by analyzing conventional diamond interchanges (CDIs) between 2009 and 2013. The study examined crash data from 3,000 collisions, collected before (36-51 months) and after (12-51 months) after the conversions from CDIs to DDIs, with daily traffic volumes averaging 11,000 vehicles. The study, which focused on crashes within a 500 meter radius of ramp terminals and used the Empirical Bayes method, found that DDIs reduced fatal and injury crashes by 55%, property damage only crashes by 31.4%, and total crashes by 37.5%. These results suggested significant safety enhancements with DDIs over CDIs, prompting the recommendation for further comparative studies to assess safety across different interchange types [16].

Atiquzzaman and Zhou (2018) analyzed wrong-way driving (WWD) risks on full diamond interchanges, examining 128 exit ramps with WWD incidents and 428 ramps without such incidents in Illinois and Alabama from 2009-2013. They developed predictive models using Firth's penalized likelihood logistic regression, focusing on ramp geometry, traffic control, and volume. Findings indicated that obtuse intersection angles and non-traversable medians reduce WWD risks, while lower ramp and higher minor road traffic increase them. Visible WWD signage and signalized ramp terminals were effective in reducing WWD. The study found urban interchanges more prone to WWD than rural ones and suggested using a broader dataset for future research [17].

Nye et al. (2019) evaluated the national-level safety of diverging diamond interchanges (DDIs). The study analyzed the safety impacts of 26 diverging diamond interchanges (DDIs) across 11 states, using crash data from three years pre-construction and two years post-construction. Using the Comparison Group (CG) method, researchers observed a 36.7% reduction in overall crashes with a crash modification factor (CMF) of 0.633. The results also showed decreases in angle and rear-end collisions, an increase in sideswipe collisions, and significant reductions in both fatal-and-injury collisions (CMF of 0.461) and property damage only (PDO) crashes (CMF of 0.695). Both daytime and nighttime crashes decreased, indicating enhanced safety through DDI implementation. The study recommended further research to evaluate long-term effects and regional variations in outcomes [18].

Meuleners and Roberts (2020) studied driver behavior at diverging diamond interchanges (DDIs) using a driving simulator with participants unfamiliar with DDIs but experienced in driving. The study, which included both pre-study pilot and exit interviews, utilized Fisher's Exact Test and r-ANOVA to assess driving errors and violations compared to standard intersections. Results showed slower driving speeds at DDIs and a higher incidence of red-light violations (44%) due to inadequate signage. The primary university student participants suggested improvements in signage and road markings. The study, limited by its non-representative sample and lack of diverse vehicle types, recommended better driver education on DDIs regarding speed and red-light compliance [19].

Abdelrahman et al. (2021) investigated the systematic safety evaluation of diverging diamond interchanges based on nationwide implementation data. The study compared the safety of 80 diverging diamond interchanges (DDIs) and 240 conventional diamond interchanges (CDIs) across 24 states, analyzing five years of crash data. Variables including traffic volume, speed limits, and crossover distances were accessed using three analytical methods: before-and-after with comparison group, empirical before-and-after, and cross-sectional. The results showed that DDIs reduced crash rates by 8% to 68% compared to CDIs, with greater crossover distances correlating to fewer crashes and higher speeds to more crashes. The study suggested further research on crash modification factors (CMFs) to understand changes in driving behavior over time [20].

Section 4: Traffic Operations at Diamond Interchanges

Song and Yang (2012) compared the operational performance of Diamond Interchanges with one intersection (DIO) in China to those with two intersections (DIT) in the U.S. They observed that DIOs feature wider left-turn angles and larger radii than the sharper, smaller-radius turns in U.S. DITs. Additionally, DIOs operate with simpler, single-signal cycles, while DITs use complex, multi-phase signaling. The study found DITs safer at conflict points but did not conclusively determine a superior design, recommending further analysis using tools like VISSIM due to limited data [21].

Jin et al. (2013) studied traffic organization at a diamond interchange using VISSIM. The study analyzed peak hour conditions on an eight-lane, two-way setup with service roads. It found average delays of 17.9 seconds, queue lengths of 12 seconds, and parking times of 29.25 seconds. The findings, supported by an entropy-based congestion method, indicated that upstream bus stations improved flow by reducing weaving, despite poor ride quality at high densities. The findings recommended lane changes and coordinated signals to manage traffic, particularly suggesting diamond interchanges for low traffic or expandable areas and called for further research on optimal interchange distances to minimize traffic problems [22].

Yang et al. (2014) developed a signal optimization model for diverging diamond interchanges (DDIs), considering isolated and adjacent intersection scenarios. The model, which accounted for traffic patterns with high through and left-turn volumes, set parameters such as 40 mph free flow speed and 120-second cycle length. VISSIM simulations showed that optimized signals reduced delays by nearly 20% at isolated DDIs and under 5% at DDIs with adjacent intersections. The study highlighted the need for further research to improve evaluation tools for DDIs' impact on traffic delays and nearby roads [23].

Leong et al. (2015) used VISSIM microsimulation to evaluate Diverging Diamond Interchanges (DDIs) against traditional diamond interchanges, focusing on peak-hour performance across five junctions within an expressway network. The study revealed that DDIs, due to their unique geometry, reduced traffic delays and travel times by facilitating better coordination between ramp and through movements. It also noted the safety advantages of DDIs, such as fewer conflict points and safer left turns without crossing opposing traffic. The research underscored the necessity of clear signage for optimal DDI operation. Despite these benefits, further studies were recommended to fully gauge DDI's efficiency amid evolving interchange designs [24].

Section 5: Safety at Other Interchanges

Baratian-Ghorghi et al. (2014) studied WWD fatal crashes in the U.S. from 2004-2011 using the Fatality Analysis Reporting System (FARS), finding that the top 10 states contributed to over 50% of these incidents, with urban areas accounting for 57%. The analysis showed that 58% of WWD crashes were linked to impairment from alcohol or drugs. The study recommended engineering solutions to prevent WWD and stricter DUI enforcement, suggesting that states with lower WWD rates could serve as models for others [25].

Pour-Rouholamin and Zhou (2015) examined how geometric design and access management at interchanges can mitigate wrong-way driving (WWD), highlighting exit ramps as crucial areas. They found that ramp design, angles, and sections significantly affect WWD occurrences. Key preventive measures included sharp exit angles, raised medians, elevated islands, and reduced radii for better visibility and intersection clarity [26].

Jalayer et al. (2016) evaluated GPS devices' accuracy in preventing wrong-way driving (WWD) at interchanges using five GPS devices and apps. The study measured distances between access points and exit ramps and tracked right-turn commands. The results showed a high risk of GPS misleading drivers when the distance was less than 350 feet, with critical errors occurring between 100-200 feet. The study recommended incorporating specific GPS alerts for short distances, such as 'no right turn' or 'left at next intersection', to reduce WWD risks. Given the widespread use of GPS for navigation, enhancing GPS features was deemed a cost-effective solution to mitigate these issues [27].

Tagar and Pulugurtha (2021) studied predictor variables influencing merging speed and lane-change related crashes by interchange type in urban areas. Crash data were collected from the Highway Safety Information System (HSIS) for five years (2011-2015), focusing on 96 merging lanes with 2,251 reported crashes. Using multinomial logistic regression, it was found that 41% of crashes occurred at cloverleaf interchanges, 23% at diamond interchanges, and 35% at other types of interchanges. Factors increasing crash likelihood included high traffic on ramps, single-lane ramps, and large speed differentials, with cloverleaf interchanges faring better on wider freeways. Diamond interchanges were more prone to crashes with higher freeway traffic volumes. The study recommended further research into road design and vehicle technology to improve crash prediction [28].

Gu et al. (2022) analyzed factors influencing interchange crashes in Florida by examining driver characteristics, roadway features, and environmental conditions using data from 2014. Utilizing logistic regression and Support Vector Machine models, the study found that cloverleaf and direct connection interchanges had higher fault probabilities, particularly in poor weather and among impaired drivers. In contrast, diamond interchanges were deemed safer. Risk factors included high traffic volume and variable speed limits. The recommendations emphasized improving road safety features such as better signage and shoulder paving, adopting connected vehicle technology for older drivers, and expanding the research scope to include other states [29].

Section 6: Traffic Operations at Other Interchanges

Yang et al. (2012) examined the capacity of Type A weaving segments on urban expressways in China using traffic data and VISSIM simulations. The study, which assessed segments ranging from under 150 meters to 600 meters, found that weaving vehicles significantly affected congestion more than non-weaving vehicles. Initial results showed that capacity depended on the volume ratio and segment length, but further analysis pointed to the influence of non-weaving segments. The findings, aligning closely with field data, suggested the need for additional research to fine-tune capacity estimates [30].

Alzoubaidi et al. (2021) compared the operational efficiency of the Super Diverging Diamond Interchange (SDDI) and other innovative interchanges like the Conventional Diamond (CDI), Diverging Diamond (DDI), and others using Colorado Department of Transportation data. Traffic studies from 5:15-6:15 PM with 8% heavy vehicles were modeled in Autodesk AutoCAD Civil 3D and analyzed in VISSIM, with signal timings refined by Synchro. The results indicated the Folded Diamond (FDI) was most efficient in 2018 and the Ramp Crossover (RCI) in 2038, while the CDI offered the shortest pedestrian times. The SDDI underperformed against expectations [31].

Mehrara et al. (2021) evaluated the safety performance of the offset diamond interchange (ODI) compared to parcloA and diamond interchange (DI) using VISSIM and the Surrogate Safety Assessment Model (SSAM). Traffic signals were optimized with Synchro and integrated into from VISSIM, assessing pedestrian performance, geometric configurations, traffic characteristics, and driver behaviors. Simulations used Weidemann 74 and 99 models, with parameters validated against probe vehicle data. The results showed that ODI had fewer severe conflicts and better performance than DI but was outperformed by parcloA in pedestrian safety and travel times [32].

Jim et al. (2022) compared the traffic operation and safety of a left hook interchange to a traditional cloverleaf using PTV VISSIM and SSAM. Both had similar speeds and geometric features, with vehicle speeds of 70 mph for cars and 60 mph for trucks, and traffic volumes up to 18,000 vehicles per hour. The left hook design, which included four partial loops for left turns, showed similar travel times of around 71-72 seconds but had higher conflict rates and fewer rear-end collisions than the cloverleaf due to less stopping. Both designs had comparable construction costs, and further studies were suggested to evaluate the left hook design's potential superiority [33].

Section 7: Hotspot Analyses

Lee et al. (2019) explored the identification of crash severity spatial patterns using hotspot analyses. Traffic crash data (2011-2017) from Lincoln, Nebraska was used. The study employed network-based local spatial autocorrelation and kernel density estimation methods to detect high and low severity crash clusters. The findings revealed distinct spatial distributions for different severity levels, with severe crashes predominantly on highways and minor crashes in urban settings. The study highlighted the importance of integrating crash severity into hotspot analyses for more informed decision-making in traffic safety management [7].

Zahran et al. (2019) evaluated road traffic accident hotspots on Jalan Tutong in Brunei using three GIS-based methods: Network Kernel Density Estimation (KDE), Getis-Ord G_{i*}, and Spatial Traffic Accident Analysis (STAA). The study used RTA data from 2012 to 2016 and was performed with ESRI ArcGIS software. Results showed that KDE effectively identified high-accident areas, while Getis-Ord G_{i*} found no significant clusters, suggesting limitations on linear road networks. STAA offered a detailed risk assessment by factoring in accident frequency, severity, and socioeconomic costs, thereby identifying critical hotspots [34].

Afolayan et al. (2022) analyzed crash hotspots on the Lokoja-Abuja-Kaduna highway in Nigeria using GIS, with data from 2013 to 2017. The study utilized mean center analysis, kernel density estimation, and Getis-Ord G_{i*} statistics to pinpoint high-risk areas, revealing significant hotspots in 2013, 2014, and 2017, while 2015 and 2016 showed more random patterns. The findings underscored GIS's effectiveness in hotspot identification and advocated for continuous data analysis to refine safety interventions according to dynamic local conditions [35].

Section 8: Safety Performance Functions (SPFs)

Montella et al. (2014) examined the impact of highway design consistency on road safety. Geometric, traffic, and crash data from Italy's A16 motorway were used. The study developed Safety Performance Functions (SPFs) that included variables like operating speed consistency, side friction, and tangent lengths. Using generalized linear modeling with a negative binomial distribution, it was found that design consistency significantly affects crash frequency and severity. The study underscores the importance of consistent design in reducing crash risks and suggests several road design and speed management adjustments to meet safety standards and driver expectations. [36].

Mehta et al. (2015) developed Safety Performance Functions (SPFs) to predict crashes on major highway bridges in Alabama, using data from local and national sources and employing negative binomial regression. The study created SPFs for both general and single-vehicle crashes and validated these models with statistical tests. It found that bridge length and traffic volume affected crash rates and noted a negative correlation between truck traffic percentage and crashes. The findings endorse using SPFs in bridge safety management and suggest extending the research to crash severity and different regions for customized safety assessments [6].

Choi et al. (2017) developed Safety Performance Functions (SPFs) and Crash Modification Factors (CMFs) for various types of expressway ramps. Crash data from 2007 to 2009 and negative binomial regression were used for analysis. These models quantified the influence of ramp design elements such as curvature, grade, and lane width on crash frequencies, proving effective in enhancing ramp safety. The accuracy of these models in predicting crashes was confirmed, advocating for their use in roadway design and other operational decisions in an effort to reduce crashes. The study also called for further research to improve these models and expand their applicability to different traffic conditions and road types [37].

Objectives

The primary objectives of this study were to:

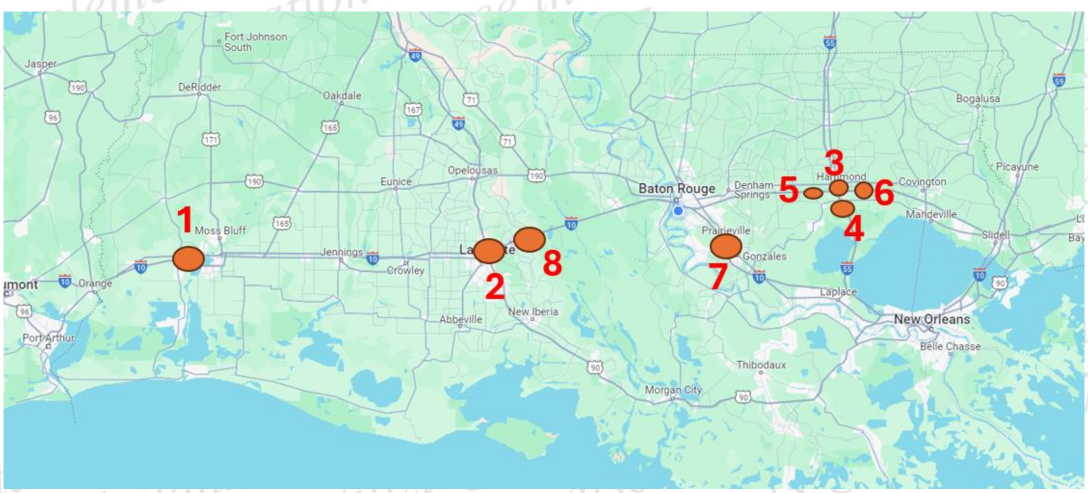
- Assess the safety and traffic operational performance of several cloverleaf interchanges in Louisiana and compare their performance with that of traditional diamond interchanges.
- Use the safety and traffic analysis of the current year to predict the future performance of cloverleaf and diamond interchanges in Louisiana.
- Suggest countermeasures or alternative interchange solutions that may be implemented if a cloverleaf or diamond interchange is not an appropriate alternative based on current and predicted future performance.

Scope

The project scope included conducting a comprehensive analysis of a sample of cloverleaf and diamond interchanges in Louisiana to assess their safety and operational performance, along with a comparison between cloverleaf and diamond interchanges. Eight interchanges (four cloverleaf and four diamond interchanges) were selected for the evaluation. The four cloverleaf interchanges included two with C-D roads and two without C-D roads. The four diamond interchanges included one interchange with unsignalized intersections on the minor road, one interchange with signalized intersections on the minor road, and two interchanges with roundabouts on the minor road. These eight interchanges were selected based on feedback from the Project Review Committee. Figure 3 illustrates the location of the eight interchanges used in the analysis.

This project aimed to utilize comprehensive microsimulation and crash data analysis to determine which interchange design performs better in terms of traffic safety and operation under various traffic conditions, to predict their future performance with evolving traffic demands, and to suggest countermeasures to improve their performance as needed.

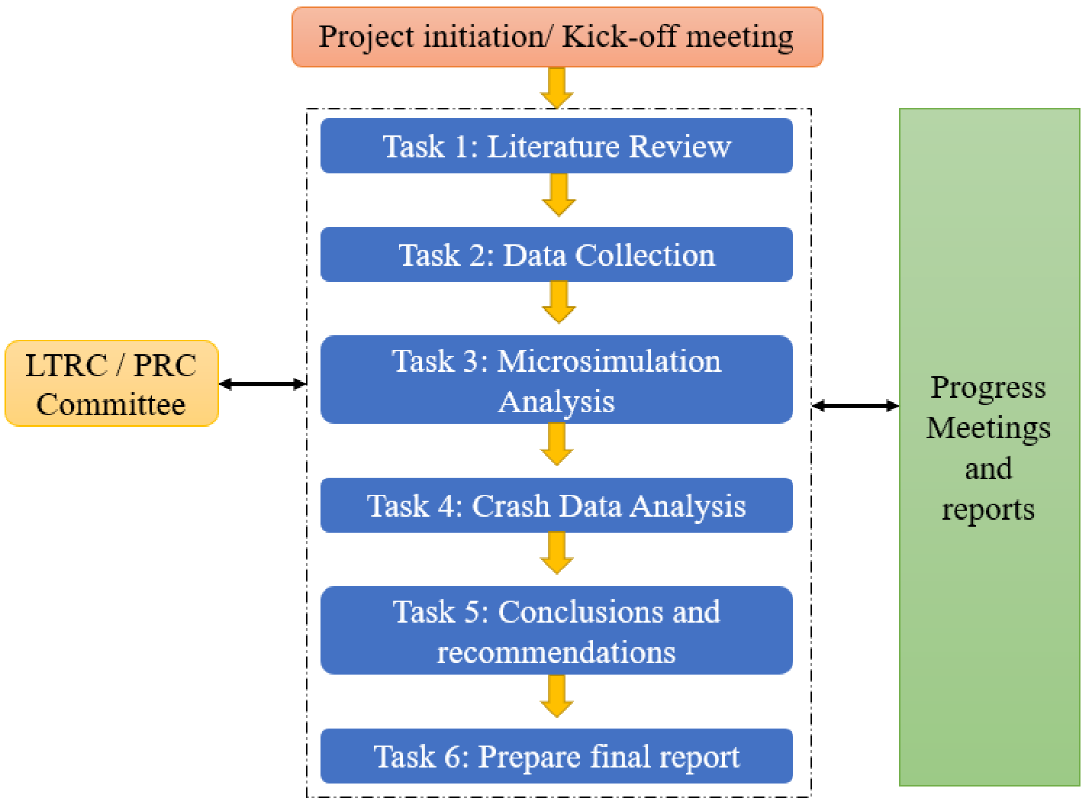
Figure 3. Locations of the eight interchanges under investigation



Methodology

This section outlines the methodology used in this project and details each task completed to meet the project objectives. Figure 4 illustrates the overall methodology for this study.

Figure 4. Overall Research Methodology



Task 1: Literature Review

In this task, an in-depth literature review was conducted, as discussed in the preceding section. The review focused on assessing the traffic safety and operations at cloverleaf and diamond interchanges. It also included an examination of microsimulation analysis, using VISSIM to evaluate interchange efficiency. The review also aimed to identify hotspot evaluation techniques and the development of Safety Performance Functions.

Task 2: Data Collection

Study Area

The study area includes a total of eight interchanges in Louisiana, consisting of four cloverleaf and four diamond interchanges. Among the cloverleaf interchanges, two have collector-distributor (C-D) roads, while the other two do not. Regarding the diamond interchanges, two have double roundabouts, one has a signalized intersection, and one has a stop-controlled intersection.

The eight interchanges analyzed in this study were:

- Interchange 1 (Cloverleaf with C-D roads), Location: I-10/Louisiana 108 near Lake Charles
- Interchange 2 (Cloverleaf without C-D roads), Location: I-10/I-49 at Lafayette
- Interchange 3 (Cloverleaf without C-D roads), Location: I-12/I-55 at Hammond
- Interchange 4 (Cloverleaf with C-D roads), Location: I-55/LA 22 at Ponchatoula
- Interchange 5 (Diamond with stop-controlled intersections), Location: I- 12/Pumpkin Center Rd near Hammond
- Interchange 6 (Diamond with double roundabouts), Location: I-12/SW Railroad Ave at Hammond
- Interchange 7 (Diamond with signalized intersections), Location: I-10/LA 73 at Dutch Town
- Interchange 8 (Diamond with double roundabouts), Location: I-10/Louisiana 347 at Grand Point Highway

Detailed locations of all eight interchanges are provided in Appendix A. After selecting the study area, three types of data were collected for the detailed analysis.

<u>Step 1: Collecting traffic data</u>

Traffic data required to calibrate and validate the microsimulation VISSIM model included turning traffic volumes, travel times, and speed data. Field traffic count was collected by a third-party firm (Quality Counts). Traffic data collection was performed during the morning peak hours (6-10am) and evening peak hours (3-7pm) at all eight interchanges under investigation. The data was collected over three weekdays (Tuesday, Wednesday, and Thursday). Traffic count included passenger cars and heavy vehicles. Travel time and speed data were collected from the Regional Integrated Transportation Information System (RITIS).

Calculation of growth rate for predicting traffic volumes after 10 and 20 years

There are several methods for forecasting future traffic count in transportation modeling and planning, including:

- Constant Rate
- Ratio-Trend Method
- Cohort Survival Method
- Compound Annual Rate Method
- Economic-Based Method

Among these various methods, the Compound Annual Method was used for forecasting the future traffic count required for this study, as recommended by Louisiana DOTD. The Compound Annual Method needs future and present traffic counts to calculate the traffic growth rate of roadways. Future traffic was evaluated using historical data plotted in scatterplot. The historical data for forecasting was obtained from traffic count data (https://ladotd.public.ms2soft.com/tcds/tsearch.asp?loc=ladotd). The nearby point of interchange that had historical data was obtained and used to calculate the growth rate.

Step 2: Collecting roadway geometric data

Roadway Geometric Data of all eight interchanges were collected from Google Maps and Google Earth. This data included the number of lanes, lane widths, shoulder widths, median types and widths, and the lengths of acceleration and deceleration lanes.

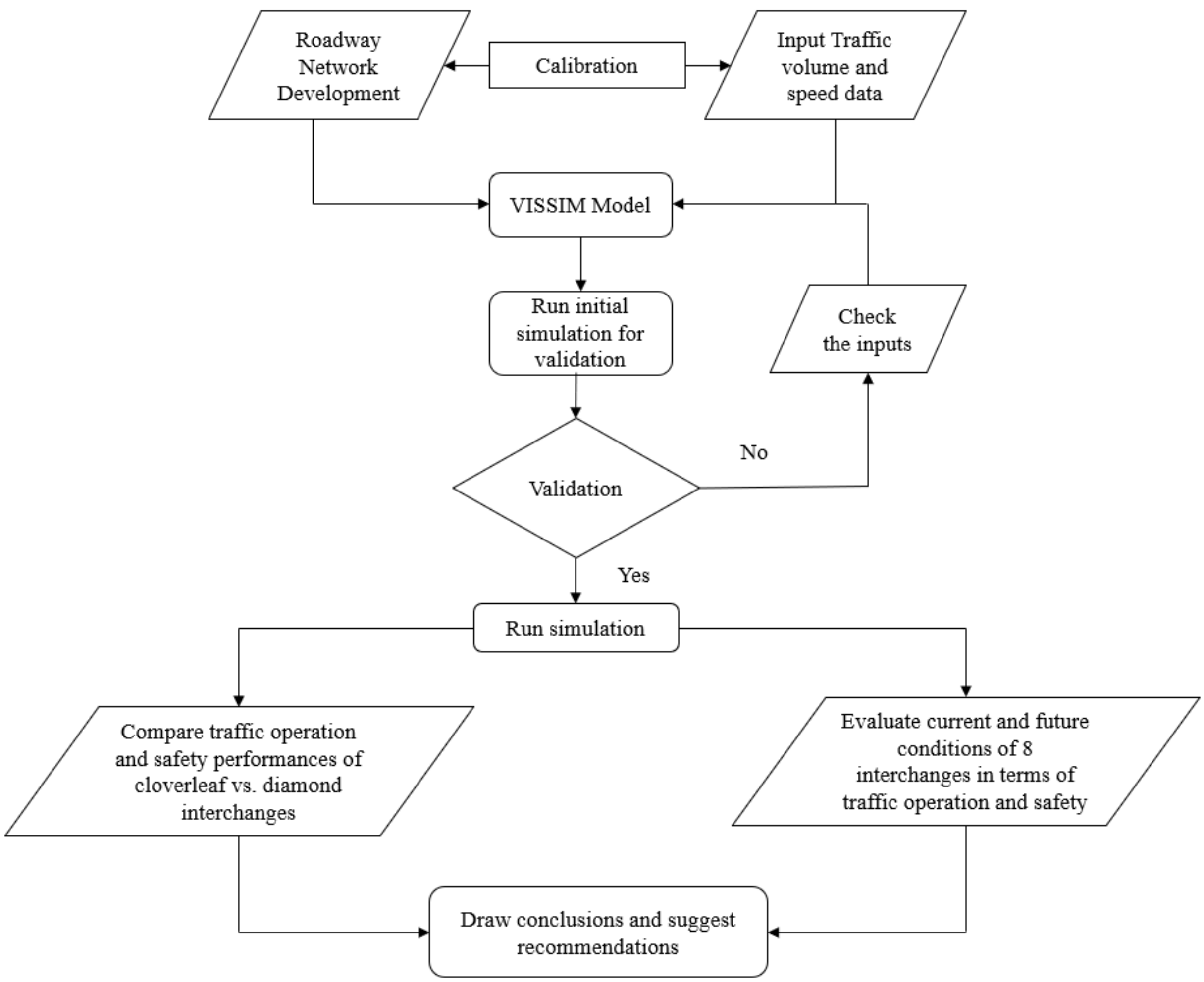
Step 3: Collecting crash data

Crash data required for the eight interchanges were provided by LTRC. The obtained crash data were from the years 2016 to 2021. The total number of crashes that occurred at the eight interchanges during these six years was 4,031. After obtaining the crash data, GIS was used to identify the exact locations of those crashes on the eight interchanges. Crashes that were not located on the influence areas of the eight interchanges were removed and disregarded.

Task 3: Microsimulation Analysis

Figure 5 illustrates the overall methodology used to conduct the microsimulation analysis in this study. It began with the development of the roadway network, followed by model calibration and the input of essential traffic volume and speed data. The VISSIM model was used to run initial simulations for validation. After validation, further simulations were conducted to analyze and compare the interchanges. Additionally, a thorough evaluation of current and future conditions for each of the eight interchanges was performed. Based on the analysis, conclusions were drawn, and recommendations were suggested.

Figure 5. Overall Methodology of Microsimulation Analysis



The microsimulation analysis for the study was conducted using PTV VISSIM, which is widely recognized as one of the standard tools for microscopic traffic and transport planning [38]. The steps followed to develop the VISSIM model that simulates the study area under investigation in this study were:

Step 1: Creation of the roadway networks

The initial phase of the microsimulation analysis using VISSIM involved creating the roadway network. In this process, links and connectors were designed to accurately model the interchanges. Vehicle routes, priority rules, and desired speed decisions were established.

Step 2: Calibration and validation of the VISSIM model

Before running simulations to obtain results, it was essential to calibrate and validate the VISSIM model using at least two parameters: traffic volume and speed/travel times, as recommended by the Washington State Department of Transportation [39] and the Louisiana Department of Transportation and Development [40]. Accordingly, the VISSIM model was calibrated using traffic volume and speed. The model was also validated against two criteria: throughput and travel time. The specifics of this validation are detailed below.

Throughput Validation:

$$GEH = \sqrt{\frac{2(m-c)^2}{(m+c)}}$$

Where,

m = output traffic throughput volumes from the simulation model (veh/h/ln), and
c = traffic throughput volumes based on field data (veh/h/ln)

Travel Time Validation:

Free Flowing: $\Delta = \dfrac{1}{\frac{1}{t} - \frac{4.4}{L}} - t,$

Interrupted Flow: $\Delta = \dfrac{1}{\frac{1}{t} - \frac{0.1*5280S}{3600L}} - t$

Where,

Δ = Allowable Travel Time Variation (+/- seconds),
t = Real World Travel Time (seconds),
L = Length (feet), and
S = Free Flow Speed (mph);

Posted Speed may be used for FFS if unknown. The results of the detailed validation are provided in the Appendix.

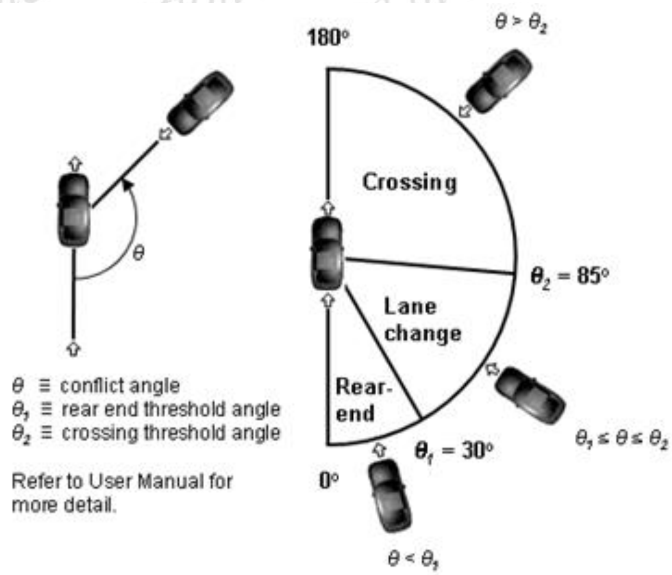
<u>Step 3: Performance of simulation runs for the results</u>

To assess the traffic operation at each of the interchanges under investigation, four measures of effectiveness were determined: vehicle delay, queue results, travel time, and LOS, following the guidelines outlined in the Highway Capacity Manual (HCM) 2010. The analysis of all eight interchanges were conducted for current conditions, as well as for two additional projected scenarios after 10 years and 20 years.

<u>Step 4: Evaluation of safety performance using SSAM</u>

The Surrogate Safety Assessment Model (SSAM) is a tool developed by FHWA to automatically identify, classify, and evaluate traffic conflicts in the vehicle trajectory data output from microscopic traffic simulation models. SSAM uses some surrogate safety measures including Time-to-Collision (TTC), Post-Encroachment Time (PET), and conflict angle to define traffic conflicts. In this analysis, the software's default values, which were calibrated and recommended by FHWA, were used: TTC (1.5s) and PET (5s). The rear-end and crossing angle were set to 30° and 80°, respectively, which is shown in Figure 6. The trajectory files were imported to SSAM, and analysis was performed for each scenario to calculate total conflicts.

Figure 6. Conflict angle as given by SSAM software



Task 4: Crash Data Analysis

The primary objectives of this task were to:

- Identify hotspot locations of traffic crashes at diamond and cloverleaf interchanges using GIS through roadway network and segment screening analysis.
- Suggest countermeasures based on the location of the hotspots.
- Employ safety performance functions (SPFs) for diamond and cloverleaf interchanges to predict the crash rate at those sites using future traffic volume.

Figure 7 illustrates the overall methodology used to conduct the crash data analysis. Initially, crash data was collected, imported into ArcGIS Pro, and organized for spatial analysis. After cleaning the data, crash hotspots were identified and analyzed. Safety Performance Functions (SPFs), based on the Highway Safety Manual (HSM), were employed to estimate the expected number of crashes at the eight interchanges under investigation. Crash rates were estimated following Louisiana DOTD guidelines. Finally, conclusions were drawn, and safety recommendations were provided to improve the identified hotspots and enhance road safety at the eight interchanges.

Hotspot Locations and Countermeasures

The following steps were implemented to achieve the objective of this sub-task:

Filtering the crash data

Crash data received from LTRC were filtered using ArcGIS Pro. Crashes that were not located on the influence areas of the eight interchanges were disregarded and removed from further consideration. Table 1 shows the distribution of the number of crashes at each interchange along with their Average Annual Daily Traffic (AADT) counts. It should be noted that the obtained crash data were not reviewed for accuracy.

Figure 7. Overall Methodology Used in Crash Data Analysis

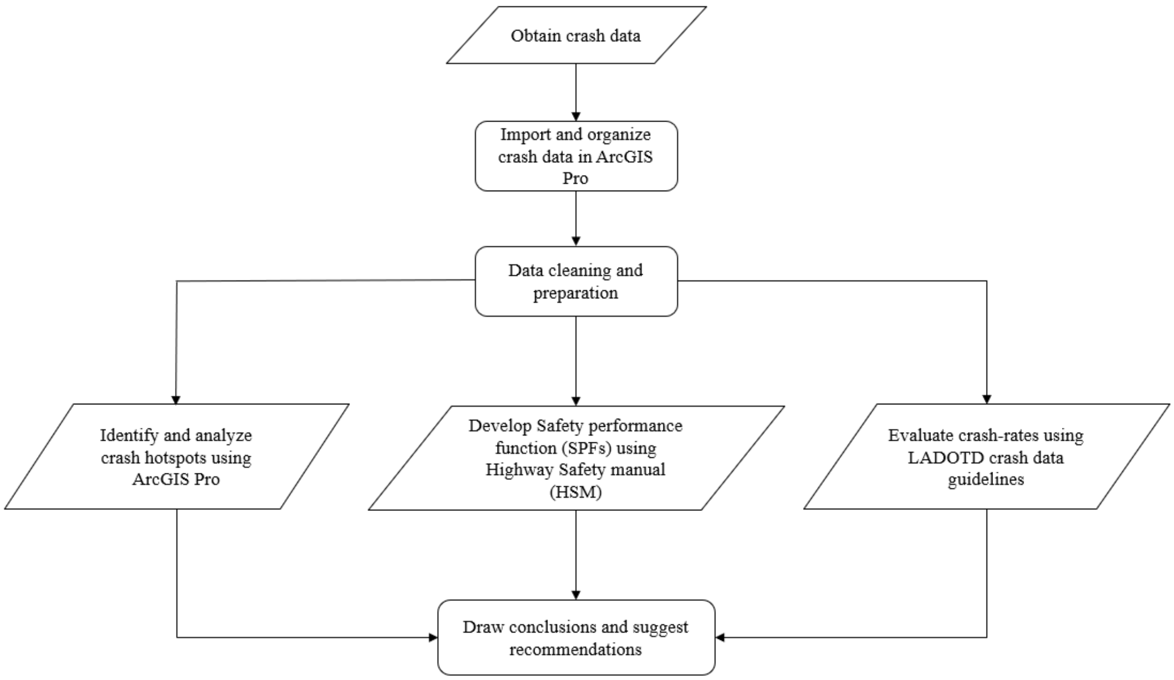
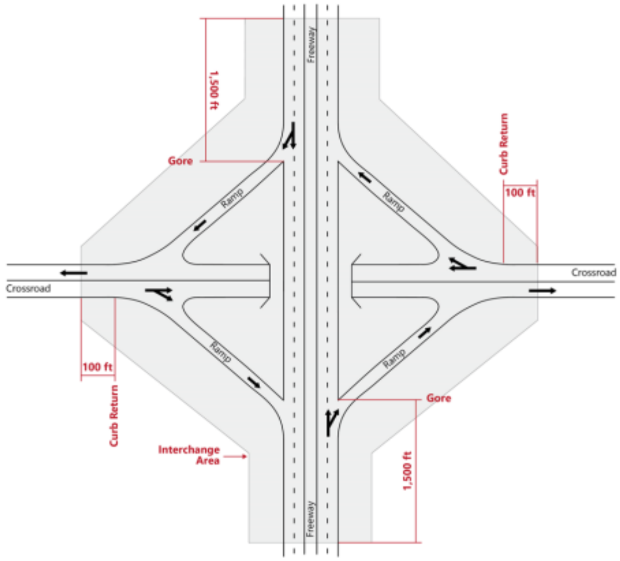


Table 1. Total crash data of all eight interchanges (2016-2021) after filtering

Interchange number			Total crashes	Traffic Volume (AADT)	
				Major Road	Minor Road
Cloverleaf Interchanges	1	With C-D roads	719	109,353	16,820
	2	Without C-D roads	613	70,250	72,710
	3	Without C-D roads	598	74,007	65,643
	4	With C-D roads	313	39,470	14,867
Diamond Interchanges	5	Unsignalized intersection	130	80,520	5,900
	6	Roundabouts	527	74,007	18,547
	7	Signalized intersection	890	90,510	24,670
	8	Roundabouts	241	62,470	8,270

Figure 8 illustrates a typical example of the influence areas of an interchange. For freeways, the impact zone extends 1500 feet from the physical gore point. For minor roads not classified as freeways, the influence area is defined as 100 feet from the physical gore point [41]. In this study, we followed these guidelines while allocating crashes on the influence areas of the interchange.

Figure 8. Influence areas of an interchange (Source: FHWA-HRT-23-041 [41])



<u>Methodology for calculating hotspots</u>

Hotspots, also known as "black spots" or high crash locations, are specific areas on a highway segment where the number of crashes exceeds the anticipated frequency, surpassing a certain level of statistical significance [42].

The hotspot location of traffic crashes can be identified using Kernel Density Estimation (KDE) and Getis-Ord G_{i*} statistic, which are two of the most popular methods utilized in previous studies. The Getis-Ord G_{i*} statistics provide information about high crash location through statistical significance.

<u>Kernel Density Estimation (KDE)</u>

The kernel density method is a non-parametric technique for density estimation that helps assess the likelihood of crashes and risk levels in specific areas. It involves overlaying a symmetrical surface on each data point and calculating the distance to a reference location using a mathematical function. The values from all surfaces are summed at the reference location. This process repeats for each data point, placing a kernel over each observation. The combined kernels produce a density estimate for the distribution of accident points [43]. The basic expression of kernel density function is provided in the Appendix.

Getis-Ord G_i^* Statistic

Kernel Density Estimation identifies clusters in data, but it is uncertain whether these clusters arise randomly or through underlying spatial processes.

To clarify, we use the Getis-Ord G_i^* statistic to objectively detect and evaluate significant patterns. This method precisely identifies high and low value clusters and uses associated p and z values to measure their statistical significance. This allows for flexibility in setting confidence levels at 99%, 95%, or 90% [44].

G-statistics, as developed by Getis and Ord, provide a global measure of spatial autocorrelation (SA), whereas the G_i^* statistic offers a local SA measure, enabling finer identification of clusters with varying densities [45]. The basic expression of Getis G_i^* Ord statistic is provided in the Appendix.

Safety Performance Functions and crash rate

Safety Performance Functions (SPFs) serve as fundamental components in identifying the connections between factors influencing crash risk and the occurrence of crashes [46]. The crash high risk factors on roadways include Average Annual Daily Traffic (AADT) and segment length.

In this study, it was not possible to calibrate and develop local SPFs for Louisiana conditions due to the reasons listed below.

According to the FHWA-SA-14-004 [47] report, which is shown in Table 2,

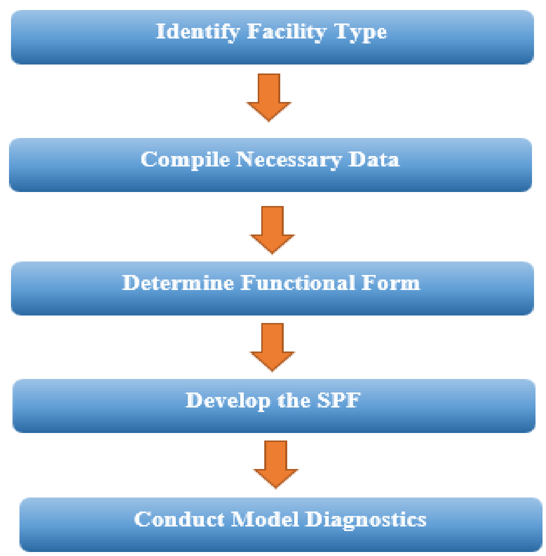
- To calibrate SPFs, 30-50 sites are required.
- To develop SPFs, 100-200 intersections or 100-200 miles are required.

Table 2. Sample size required to calibrate and develop SPFs, as recommended by FHWA.

Process	Sample needed
Calibrate SPF	• 30-50 sites; at least 100 crashes per year for total group. • At least three years of data are recommended
Develop SPF	• 100-200 intersections or 100-200 miles; at least 300 crashes per year for total group. • At least three years of data are recommended.

However, in this study, SPFs are developed by following the equations in the Highway Safety Manual (HSM). Figure 9 shows the detailed steps of developing SPFs. The process begins by identifying the facility type—roadway segments, intersections, or ramps. Data are compiled to determine the analysis level for project-level assessments, network screening, or engineering treatment evaluations. Network screening highlights areas that need safety improvements, and project-level analysis uses a before-and-after study approach to assess design changes. SPFs are typically created using negative binomial regression to model the relationship between crash frequency and site characteristics. The Safety Analyst software facilitates this process by integrating advanced safety management and economic analysis tools, enhancing road safety investments and user safety [47] [48].

Figure 9. Steps in developing SPFs.



The most popular count data models for rare events are Poisson and negative binomial regression models [47] [49]. Poisson distribution restricts the mean and variance to be equal, potentially resulting in over-dispersion of the data. To address this issue, a practical approach is to employ a negative binomial regression model when modeling crash counts. The detailed equations used in this process are provided in the Appendix.

Using the properties of negative binomial regression, the equation is used to develop SPFs, which are given by the Highway Safety Manual (HSM). The detailed equations used are provided in the Appendix.

Model diagnostics are crucial to confirm the accuracy of the Safety Performance Functions (SPFs), using statistical metrics like R-squared, mean absolute deviance (MAD), mean prediction bias (MPB), mean squared prediction error (MSPE), and the cumulative residual plot (CURE) [48]. These tests check how closely SPF predictions align with actual crash data, reflecting the model's ability to capture crash patterns. The SPFs were developed using six years of crash data (2016-2021), which involved calculating average crash counts for various road sections and obtaining road lengths from Google Maps and traffic volume data from the Louisiana Transportation Research Center (LTRC).

<u>Comparison of safety between four cloverleaf interchanges</u>

To compare the safety performance of the four cloverleaf interchanges and four diamond interchanges under investigation in this study, the number-rate of crashes was estimated, as recommended by Louisiana DOTD [50]. The formulas for the number-rate of road segments and intersections are:

For roadway segments, the equation is:

$$Rs = (C * 10^6)/(L * AADT * D)$$

Where,
Rs = segment crash rate,
C = crash count (crashes),
D = analysis days (days),
L = segment length (miles), and
AADT = annual average daily traffic (vehicles/day).

For intersections, the equation is:

$$Ri = (C * 10^6)/(EV * D)$$

Where,
Ri = intersection crash rate,
C = crash count (crashes),
D = analysis days (days), and
EV (Entering Vehicles) = average vehicles entering the intersection each day from all approaches (vehicles/day).

Discussion of Results

Task 3: Microsimulation Analysis

Interchange 1 (Cloverleaf with C-D roads)

Figure 10 illustrates the layout of Interchange 1, which is a cloverleaf with C-D roads. This interchange is located at the junction of I-10 and Louisiana 108 near Lake Charles.

Figure 10. Layout of Interchange 1

Table 3 presents the Levels of Service (LOS) of Interchange 1 currently, after 10 years, and after 20 years. The results indicate that currently and in 10 years, Interchange 1 has an acceptable level of service (LOS = D or better). However, after 20 years, the westbound (WB) and northbound (NB) directions will experience unacceptable levels of service (LOS = F and E, respectively). Given this projection, improvements will be necessary after 20 years. Additionally, Table 4 presents the safety results for Interchange 1 currently, after 10 years, and after 20 years, showing a projected increase in safety issues (e.g., an increase in the number of conflicts) for future conditions.

Table 3. Traffic Operation Results of Interchange 1

Approach	Level of Service (LOS)		
	Present	10 years	20 years
EB	B	B	C

Approach	Level of Service (LOS)		
	Present	**10 years**	**20 years**
WB	B	C	F
NB	B	D	E
SB	A	A	A

Table 4. Traffic Safety Results of Interchange 1

Scenarios	Conflicts Count				
	Crossing conflicts	**Rear-end conflicts**	**Lane change conflicts**	**Total conflicts**	**Percentage increase**
Current	1227	970	351	2548	
10 Years	2091	4074	745	6910	171%
20 Years	2525	8464	807	11796	363%

As shown in Figure 11, to improve the traffic safety and operation performance of Interchange 1 in the future, several modifications were suggested and tested, including:

- Addition of an extra lane on both the off-ramp and on-ramp, as indicated with the red line in Figure 11.
- Addition of an extra lane in the northbound direction before and after the interchange, as indicated with the yellow line in Figure 11.

Figure 11. Recommended modifications after 20 years for Interchange 1

The results indicate that after modifications, the LOS for both WB and NB will improve to LOS B. Additionally, the total conflicts at the interchange will decrease from 11,796 to 8,336, an approximate 29% reduction. Details of the projected results of the modifications are provided in the Appendix.

<u>Interchange 2 (Cloverleaf without C-D roads)</u>

Figure 12 illustrates the layout of Interchange 2, which is a cloverleaf without C-D roads. This interchange is located at the junction of I-10 and I-49 in Lafayette.

Figure 12. Layout of Interchange 2

Tables 5 and 6 present the traffic safety and operation results for Interchange 2, indicating that it currently operates at an acceptable level of service. However, projections after 10 and 20 years show that the interchange will experience unacceptable levels of service (LOS = E and F). Additionally, the total conflict count is expected to increase significantly after 10 and 20 years. Therefore, modifications will be necessary to maintain acceptable efficiency in the future.

Table 5. Traffic Operation Results of Interchange 2

Approach	Level of Service (LOS)		
	Present	10 years	20 years
EB	C	E	F
WB	B	D	F
NB	C	E	F
SB	B	F	F

Table 6. Traffic Safety Results of Interchange 2

Scenario	Conflicts Count				
	Crossing conflicts	Rear-end conflicts	Lane change conflicts	Total Conflicts	Percentage increase (%)
Current	2748	1736	5093	9577	
10 Years	11244	8716	12812	32772	242%
20 years	12060	13742	14165	39967	317%

Therefore, the following modification, shown in Figure 13 below, was suggested to improve traffic safety and operation performance after 10 years.

- Addition of an extra lane in the EB and WB sections only, as indicated in the red lines in Figure 13.

The results indicate that adding an extra lane in both the EB and WB directions will maintain an acceptable level of service and reduce the total conflict count by 72%. The details of the result of this modification are provided in the Appendix.

Additionally, another modification, shown in Figure 14 below, was suggested and evaluated to improve traffic safety and operation performance after 20 years.

- Addition of extra lane in the EB, WB, and NB sections, as indicated with the red line in Figure 14.
- Converting the loop ramp connecting EB to NB into semi-directional ramp, as indicated with the yellow arc in Figure 14.

Figure 13. Recommended modification after 10 years for Interchange 2

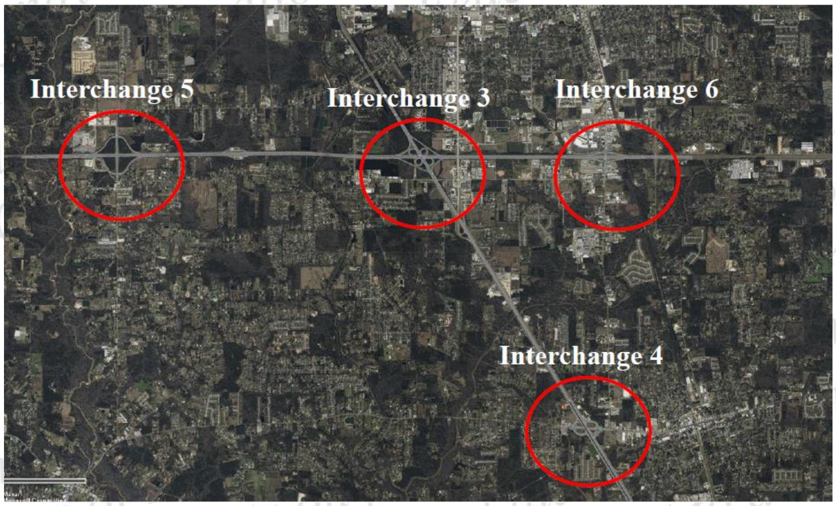
Figure 14. Recommended modification after 20 years at Interchange 2

The results demonstrate that the suggested modifications can maintain an acceptable level of service and significantly reduce the total conflict count from 39,967 to 7,064, an approximate 82% reduction. The details of the result of the modifications are provided in the Appendix.

Interchanges 3, 4, 5 & 6

As shown in Figure 15, Interchanges 3, 4, 5, and 6 are located near one another and have therefore been developed and evaluated using a single VISSIM model.

Figure 15. Layout of Interchanges 3, 4, 5, & 6



Interchange 3 (Cloverleaf without C-D roads)

Figure 27 illustrates the layout of Interchange 3, which is a cloverleaf without C-D roads. This interchange is located at the junction of I-10 and I-55 in Hammond. Table 7 presents the traffic operation results of Interchange 3 currently, after 10 years, and after 20 years. The findings indicate that currently, Interchange 3 has an acceptable level of service. However, projections indicate that it will experience an unacceptable LOS after 10 and 20 years. Therefore, modifications to Interchange 3 will be necessary after 10 and 20 years.

Figure 16. Layout of Interchange 3

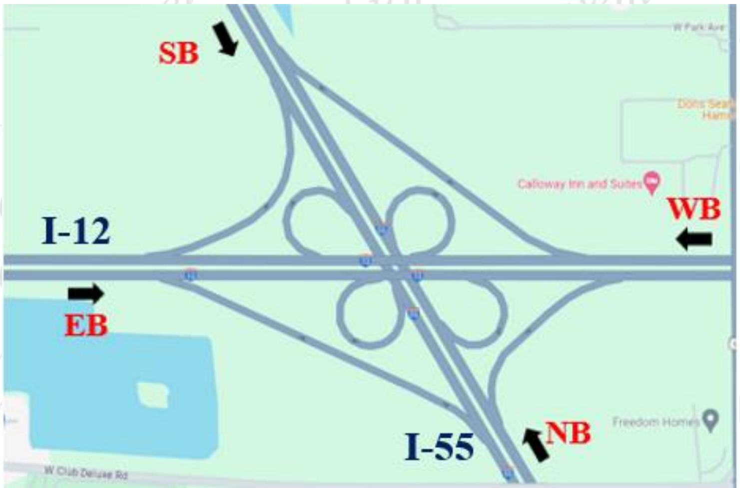


Table 7. Traffic Operation Results of Interchange 3

Approach	Level of Service (LOS)		
	Present	10 years	20 years
EB	B	C	F
WB	C	D	D
NB	C	E	F
SB	A	B	D

Therefore, the following modification, shown in Figure 17, was suggested and tested to improve traffic safety and operation performance after 10 years.

- Converting the loop ramp connecting EB to NB into a semi-directional ramp, as indicated with the yellow arc in Figure 17.

The results indicate that the modifications will enhance the level of service to an acceptable level after 10 years (LOS = C or better). The details of the result of the modifications are provided in the Appendix.

Figure 17. Recommended modification after 10 years for Interchange 3

Additionally, other modifications, shown in Figure 18, were suggested and evaluated to improve traffic safety and operation performance after 20 years.

- Addition of lane in EB and WB, as indicated with the red line in Figure 18.
- Converting the loop ramp connecting EB to NB into semi-directional ramp, as indicated with the yellow arc in Figure 18.

Figure 18. Recommended modification after 20 years for Interchange 3

The results indicate that the modifications will enhance the level of service to an acceptable level after 20 years. The details of the result of the modifications are provided in the Appendix.

Interchange 4 (Cloverleaf with C-D roads)

Figure 19 illustrates the layout of Interchange 4, which is a cloverleaf with C-D roads. This interchange is located at the junction of I-10 and LA 22 in Ponchatoula.

Figure 19. Layout of Interchange 4

Table 8 presents the traffic operation results of Interchange 4 currently, after 10 years, and after 20 years. The results indicate that Interchange 4 will continue to operate at an acceptable level of service for the next 20 years. Therefore, no modifications are required.

Table 8. Traffic Operation Results of Interchange 4

Approach	Level of Service (LOS)		
	Present	10 years	20 years
EB	B	B	B
WB	C	C	D
NB	B	B	C
SB	A	A	A

Interchange 5 (Diamond with stop-controlled intersections)

Figure 20 illustrates the layout of Interchange 5, which is a diamond with stop-controlled intersections. This interchange is located at the junction of I-12 and Pumpkin Center Rd near Hammond.

Figure 20. Layout of Interchange 5

Table 9 presents the traffic operation results currently, after 10 years, and after 20 years. The results indicate that Interchange 5 maintains an acceptable level of service in the current and 10-year scenarios. However, after 20 years, it will experience an unacceptable LOS at the SB direction (LOS = F), necessitating modifications.

Table 9. Traffic Operation Results of Interchange 5

Approach	Level of Service (LOS)		
	Present	**10 Years**	**20 Years**
EB	B	B	B
WB	A	A	B
NB	A	A	A
SB	A	A	F

Therefore, the following modification, shown in Figure 21, was suggested and evaluated to improve traffic safety and operation results after 20 years

- Stop-controlled intersections should be replaced by signalized intersections.

Figure 21. Recommended modifications after 20 years for Interchange 5

The results reveal that the suggested modifications will improve the level of service to an acceptable level over the next 20 years. The detailed results of the suggested modifications are provided in the Appendix.

<u>Interchange 6 (Diamond with double roundabouts)</u>

Figure 22 illustrates the layout of Interchange 6, which is a diamond with double roundabouts. This interchange is located at the junction of I-12 and LA 22 in Hammond. Table 10 presents the traffic operation results of Interchange 6 currently, after 10 years, and after 20 years. It shows that Interchange 6 experiences an acceptable level of service in both the current and 10-year scenarios. However, projections indicate that after 20 years, the LOS will become unacceptable in the WB and SB directions (LOS = E and F, respectively), requiring modifications.

Figure 22. Layout of Interchange 6

Table 10. Traffic Operation Results of Interchange 6

Approach	Level of Service (LOS)		
	Present	10 years	20 years
EB	B	C	C
WB	B	B	E
NB	A	A	A
SB	A	D	F

Based on these results, several modifications, shown in Figure 23, were suggested and evaluated to improve traffic safety and operation after 20 years.

- Converting roundabouts on minor road to signalized intersections.
- Addition of one extra lane in EB and WB, as indicated with the red lines in Figure 23.
- Addition of frontage roads along EB and WB, as indicated with the yellow lines in Figure 23.

Figure 23. Recommended modification after 20 years for Interchange 6

The results indicate that the suggested modifications will improve the level of service to an acceptable level. The details of the results of the modifications are provided in the Appendix.

Overall safety result of Interchanges 3, 4, 5, & 6

Interchanges 3, 4, 5, and 6 were evaluated collectively in a single model. Therefore, the Surrogate Safety Assessment Model (SSAM) provides safety results for these four interchanges together. Table 11 displays the safety results for these four interchanges currently, after 10 years, and after 20 years, as well as before and after improvements. The result demonstrates that the suggested modifications will improve safety by reducing the total conflicts.

Table 11. Safety Results of Interchanges 3, 4, 5 and 6

Scenario	Current	10 years		20 years	
		Before improvement	After improvement	Before improvement	After improvement
Type of conflicts	Conflicts Count				
Crossing conflicts	4851	10626	6060	19073	9528
Rear-end conflicts	6850	14217	6784	36496	11548
Lane change conflicts	3715	6699	3308	8571	6152
Total Conflicts	15416	31542	16152	64140	27228

Interchange 7 (Diamond with signalized intersections)

Figure 24 illustrates the layout of Interchange 7, which is a diamond interchange with signalized intersections. This interchange is located at the junction of I-10 and LA 73 in Dutch Town.

Figure 24. Layout of Interchange 7

The results indicate that Interchange 7 has an unacceptable level of service in all the three scenarios, as seen in Table 12. Therefore, modifications are required for each scenario to improve the LOS. The total conflict count is also high in all scenarios, as seen in Table 13.

Table 12. Traffic Operation Results of Interchange 7

Approach	Level of Service (LOS)		
	Present	10 years	20 years
EB	C	F	F
WB	C	E	F
NB	D	E	E
SB	E	F	F

Table 13. Traffic Safety Results of Interchange 7

	Conflict Count			
Scenario	**Crossing Conflicts**	**Rear-end conflicts**	**Lane change conflict**	**Total conflicts**
Present	2328	14466	2545	19339
10 years	3935	31062	4450	39447
20 years	4290	35763	4481	44534

Therefore, several modifications, shown in Figure 25, are suggested and evaluated to improve the current traffic safety and operation performances.

- Addition of frontage road on the north side of the interchange, as indicated with the yellow line in Figure 25.
- Addition of one extra lane in NB and SB section, as indicated with the red line in Figure 25.

Figure 25. Recommended current modifications for Interchange 7

The results indicate that the suggested modifications will improve the current level of service to an acceptable level of service. The total conflict count will also be reduced by 70% after modifications. The details of the results are provided in the Appendix.

Additionally, further modifications, shown in Figure 26, were suggested and evaluated to improve the traffic operation and safety performance after 10 years.

- Addition of one-way frontage road in NB and SB roadways, as indicated with the yellow line in Figure 26.
- Addition of lane in NB and SB roads, as indicated with the red line in Figure 26.
- Addition of lane in EB road after Intersection 2, as indicated with the red line in Figure 26.

Figure 26. Recommended modifications after 10 years for Interchange 7

The results indicate that the modifications will improve the level of service to an acceptable level over 10 years. The total conflict count will also be reduced by 74% after modifications. The details of the results of these modifications are provided in the Appendix.

Moreover, several modifications, shown in Figure 27, were suggested and evaluated to improve traffic safety and operation results after 20 years.

- Addition of a one-way frontage road in NB and SB roadways, as indicated with the yellow line in Figure 27.
- Addition of lanes in NB and SB roads, as indicated with the red line in Figure 27.
- Addition of lanes in EB and WB, as indicated with the red line in Figure 27.

The results indicate that the modifications mentioned above will improve the level of service to acceptable levels over 20 years. The total conflict count will also be reduced by 68% after modifications. The details of the results of these modifications are provided in the Appendix.

Figure 27. Recommended modifications after 20 years for Interchange 7

Interchange 8 (Diamond with double roundabouts)

Figure 28 illustrates the layout of Interchange 8, which is a diamond interchange with double roundabouts. This interchange is located at the junction of I-10 and LA 347 on Grand Point Highway.

Figure 28. Layout of Interchange 8

Tables 14 and 15 present the traffic safety and operation results for Interchange 8 currently, after 10 years, and after 20 years. The results indicate that Interchange 8 will maintain an acceptable level of service and traffic safety performance over the next 20 years, indicating that no modifications are necessary.

Table 14. Traffic Operation Results of Interchange 8

Location	Level of Service (LOS)		
	Present	10 years	20 years
EB	A	B	B
WB	A	B	B
NB	A	C	C
SB	A	B	D

Table 15. Traffic Safety Results of Interchange 8

	Conflicts Count			
Scenario	Crossing conflicts	Rear-end conflicts	Lane change conflicts	Total Conflicts
Present	491	468	487	1446
10 years	648	702	662	2012
20 years	801	1202	841	2844

Results of the comparison between Cloverleaf and Diamond Interchanges

In addition to analyzing the traffic safety and operation performances of each interchange in the study area, the following four configurations were developed and evaluated for two interchanges—Interchanges 1 and 4, which are both cloverleaf interchanges with C-D roads—to better compare the performance of cloverleaf and diamond interchanges, shown in Figures 29 through 32.

- Cloverleaf interchange with C-D roads: This type of cloverleaf interchange features additional roadways designed to enhance traffic flow by separating high-speed freeway traffic from vehicles entering and exiting, as shown in Figure 29.
- Cloverleaf interchange without C-D roads: This typical cloverleaf design omits C-D roads, which may result in shorter weaving distances and a higher number of traffic conflicts points, as shown in Figure 30.

- Diamond interchange with Signalized intersections: Includes direct, signalized crossings that regulate traffic flow on the intersecting local road, as shown in Figure 31.
- Diamond interchange with Roundabout intersections: Utilizes roundabouts at the interchange's minor road instead of traffic signals to enhance traffic efficiency and safety, as shown in Figure 32.

Figure 29. Cloverleaf interchange with C-D roads

Figure 30. Cloverleaf interchange without C-D roads

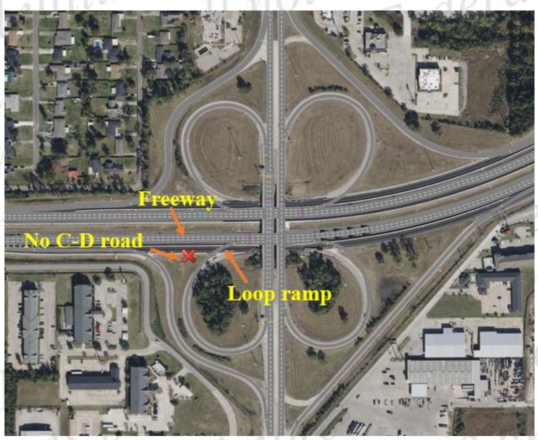
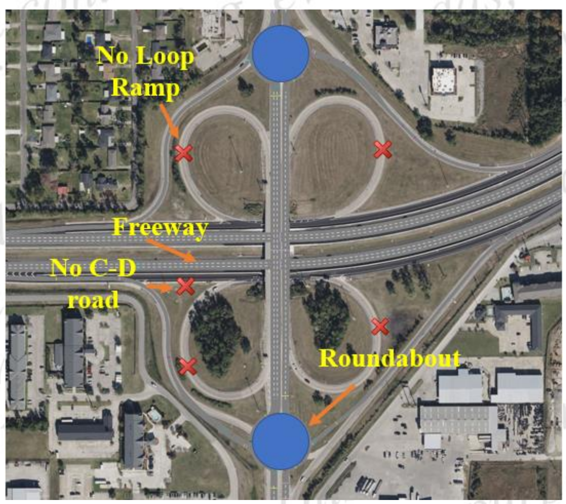


Figure 31. Diamond interchange with signalized intersections

Figure 32. Diamond interchange with double roundabouts



<u>Comparing safety and traffic performance of four interchange configurations at Interchange 1</u>

Tables 16 and 17 present the traffic volume (current, 10 years and 20 years) and roadway characteristics of Interchange 1.

Table 16. Traffic Volumes of Interchange 1

Location	Present	10 years	20 years
EB	2431	3100	3742
WB	2534	3231	3901
NB	1672	2284	2584
SB	999	1365	1544
Total	7636	9980	11770

Table 17. Roadway Characteristics of Interchange 1

Location	Number of lanes
EB	3
WB	3
NB	2
SB	2
Onramp and Offramp	1
Loop ramp	1

Notes:

- *The existing weaving length for the cloverleaf interchange is 360 ft.*
- *The traffic signal for the diamond interchange with signalized intersections is 90 seconds.*
- *The inscribed circular diameter for the diamond interchange with double roundabout intersections is 240 ft.*

Table 18 illustrates a comparison between the level of service (delay) and safety results (total conflicts) of four configurations that were tested at the interchanges (cloverleaf with C-D roads, cloverleaf without C-D roads, diamond with signalized intersections, and diamond with roundabouts). Considering traffic performance, the findings indicated that the cloverleaf interchanges with C-D roads perform better than all other interchange configurations at higher traffic volumes (total entering volume > 7000 vph). Regarding traffic safety, it was found that diamond interchanges with roundabouts outperformed the performance of other configurations.

Table 18. Traffic operation and safety results of various configurations at Interchange 1

Interchange Type	Level of Service (Delay)			Total conflicts		
	Current	10 years	20 years	Current	10 years	20 years
Cloverleaf with C-D roads	A (6.36)	D (31.82)	F (51.31)	2147	5699	10947
Cloverleaf without C-D roads	A (6.15)	E (46.94)	F (102.0)	2349	13494	27380
Diamond with signalized intersections	B (14.85)	D (47.81)	F (81.25)	956	4013	8835
Diamond with roundabouts	B (12.01)	E (38.02)	F (57.73)	912	3502	5323

Examining impacts of various weaving lengths for cloverleaf interchanges (with C-D roads)

The weaving segment is one of the most critical segments at cloverleaf interchanges [51]. Therefore, it is important to determine the impact of various weaving lengths for cloverleaf interchanges with C-D roads. Table 19 shows the results of cloverleaf interchanges with C-D roads by considering various weaving lengths. It was found that there is no significant impact on the level of service and safety by increasing weaving lengths at cloverleaf interchanges with C-D roads.

Table 19. Impacts of various weaving lengths on traffic operation and safety for cloverleaf interchanges with C-D roads at Interchange 1

	Weaving Length	Level of Service (Delay)			Total Conflicts		
		Current	10 years	20 years	Current	10 years	20 years
Cloverleaf interchange with C-D roads	360ft	A (6.36)	D (31.82)	F (51.31)	2147	5699	10947
	460ft	A (6.57)	D (33.96)	F (51.2)	2211	6055	11111
	560ft	A (6.26)	D (33.22)	F (50.31)	2132	5893	10922

Examining impacts of various weaving lengths for cloverleaf interchanges (without C-D roads)

In the same way, it is important to examine the impact of various weaving lengths for cloverleaf interchanges without C-D roads. Table 20 shows the traffic safety and operation results of cloverleaf interchange without C-D roads. It was found that there is a significant impact on the level of service and safety by increasing weaving lengths. This suggests that the operational efficiency of cloverleaf interchanges without C-D roads can be increased by increasing the weaving lengths.

Table 20. Impacts of various weaving lengths on traffic operation and safety for cloverleaf interchanges without C-D roads at Interchange 1

	Weaving Length	Level of Service (Delay)			Total Conflicts		
		Current	10 years	20 years	Current	10 years	20 years
Cloverleaf interchange without C-D roads	360ft	A (6.15)	E (46.94)	F (102.0)	2349	13494	27380
	460ft	A (6.29)	B (14.67)	F (59.57)	2097	4493	11030
	560ft	A (5.58)	B (12.37)	F (58.26)	2078	4351	10729

Examining impacts of different traffic signal timing for diamond interchanges with signalized intersections on minor roads

Different traffic signal timings result in varying traffic safety and operational performance. Therefore, it is important to investigate the impact of different traffic signal timings on traffic safety and operation. Table 21 shows the results when adopting different traffic signals at signalized intersections on the minor road of the diamond interchanges. It was found that there was a slight positive impact on the level of service and safety by increasing traffic signal cycle lengths.

Table 21. Impacts of signal cycle times on traffic operation and safety of diamond interchanges at Interchange 1

Signal cycle time	Level of Service (Delay)			Total Conflicts		
	Current	10 years	20 years	Current	10 years	20 years
70s	B (14.87)	D (50.67)	F (89.18)	972	5484	10304
90s	B (14.85)	D (47.81)	F (81.25)	956	4013	8835
110s	B (16.09)	D (39.82)	E (76.14)	1010	3560	7961

<u>Examining impacts of different inscribed circle diameters (ICDs) of roundabouts at diamond interchanges</u>

According to FHWA, different ICDs of roundabouts have different recommended speed and traffic capacities [52]. Therefore, it is important to explore the impact of different ICDs of roundabouts at diamond interchanges on traffic safety and operation. Table 22 presents the level of service and safety results when varying the inscribed circle diameters (ICDs) of diamond interchanges with roundabouts. It was found that increasing the ICDs has a slight positive impact on both the levels of service and safety. However, it should be noted that larger roundabouts may encourage drivers to increase their speed inside the roundabout, potentially enhancing flow but negatively impacting traffic safety. In this analysis, this scenario was not calibrated in the VISSIM model due to the lack of available field data. In this case, increasing the size of the roundabouts yielded positive results.

Table 22. Impacts of various ICDs on traffic operation and safety of diamond interchanges at Interchange 1

ICD	Level of service (Delay)			Safety (Total Conflicts)		
	Current	10 years	20 years	Current	10 years	20 years
160ft	B (12.31)	E (41.16)	F (77.57)	940	4483	7980
200ft	B (12.18)	E (38.23)	F (71.8)	928	3572	6650
240ft	B (12.01)	E (38.02)	F (57.73)	912	3502	5323

<u>Examining impacts of lower traffic volume scenario at Interchange 1</u>

This analysis was conducted to explore the type of interchange (cloverleaf with collector-distributor (C-D) roads, cloverleaf without C-D roads, diamond with signalized intersections, or diamond with roundabouts) that is best suited for areas with low traffic volumes. Table 23 shows the lower traffic volume considered for the analysis, which is 60% of the current total traffic volume. Table 24 presents the levels of service and safety results for Interchange 1 when operating at lower traffic volumes (under 5000 vehicles per hour). The results show that both cloverleaf configurations, with and without C-D roads, demonstrated superior traffic operation compared to diamond interchanges. In terms of traffic safety, however, diamond interchanges with roundabouts continued to outperform the other configurations.

Table 23. Lower traffic volumes considered at Interchange 1

Location	Traffic Volume
EB	1459
WB	1520
NB	1003
SB	599
Total	4582

Table 24. Traffic operation and safety results of Interchange 1 considering lower traffic volumes

Interchange Type	Level of Service (Delay)	Total Conflicts
Cloverleaf with C-D roads	2.54 (A)	685
Cloverleaf without C-D roads	2.11 (A)	603
Diamond with signalized intersections	9.82 (A)	284
Diamond with roundabout intersections	3.73 (A)	205

Comparing traffic and safety performance of four interchange configurations at Interchange 4

To verify the results obtained in the last section, the team compared the traffic safety and operational performances of the four different configurations mentioned earlier at Interchange 4 considering its traffic volumes and geometric features (e.g., number of lanes, etc). Figure 33 shows the layout of Interchange 4, which is a cloverleaf with C-D roads.

Figure 33. Layout of Interchange 4

Table 25 and Table 26 show the traffic volumes (current, after 10 years, and after 20 years) and roadway characteristics of Interchange 4.

Table 25. Traffic volumes at Interchange 4

Location	Current	10 Years	20 Years
EB	676	847	936
WB	1179	1477	1632
NB	1755	2152	2552
SB	1274	1562	1853
Total	4884	6038	6973

Table 26. Roadway characteristics of Interchange 4

Location	Number of lanes
EB	2
WB	2
NB	2
SB	2
Onramp and Offramp	1

Location	Number of lanes
Loop ramp	1

Notes:

- *The existing weaving length for the cloverleaf interchange is 550 ft.*
- *The traffic signal for the diamond interchange with signalized intersections is 90 seconds.*
- *The inscribed circular diameter for diamond interchange with double roundabout intersections is 220 ft.*

Table 27 shows the levels of service and safety results for each of the four configurations under investigation. Regarding traffic operation (in terms of level of service) and safety (in terms of total conflicts), the results indicate that the cloverleaf interchange without C-D roads outperforms the other three interchange configurations. This is due to the lower traffic and weaving volumes, which facilitate easier maneuvers at the weaving segments. However, the diamond interchanges experience reduced traffic safety and operation. This result contradicts the results of the evaluation of the four configurations at Interchange 1, presented in the preceding section.

Table 27. Traffic operation and safety results for various configurations at Interchange 4

Interchange Type	Level of Service (Delay)			Total Conflicts		
	Current	10 years	20 years	Current	10 years	20 years
Cloverleaf with C-D roads	B (11.71)	C (24.87)	D (34.74)	1917	4264	5320
Cloverleaf without C-D roads	A (9.48)	B (13.95)	C (19.76)	1323	2157	3127
Diamond with signalized intersections	B (19.76)	C (31.10)	F (100.41)	1274	2238	7094
Diamond with roundabout intersections	B (11.27)	F (54.80)	F (140.56)	1261	3047	7875

Therefore, a detailed analysis was conducted to explore this issue and try to explain the possible reasons for such inconsistent results. The investigation identified an uneven distribution of left-turn traffic volumes between Interchanges 1 and 4 as the primary cause.

The following section provides a detailed discussion on left-turn traffic at Interchanges 1 and 4. Figure 34 illustrates the left-turn traffic movement at Interchange 1, while Table 28 presents left-turn traffic volumes for the current, 10 year, and 20 year scenarios. The left-turn traffic volumes at two locations within Interchange 1 are nearly equal.

Figure 34. Left turn traffic movements at Interchange 1

Table 28. Volume of left turn movements at Interchange 1

	Movement	Current	10 years	20 years
Volume of left turn movements	Movement 1	188	240	289
	Movement 2	234	298	360
	Total	422	538	649

Figure 35 shows the left turn movement of traffic at Interchange 4, while Table 29 presents the left traffic volumes for the current, 10 year, and 20 year scenarios. As shown in Table 29, the left turn traffic varies significantly at two locations within Interchange 4, which is unlike the case at Interchange 1, as shown in Table 28. This could explain the difference in results between Interchanges 1 and 4.

Figure 35. Left turn traffic movements at Interchange 4

Table 29. Volume of left turn movements at Interchange 4

	Movement	Current	10 years	20 years
Volume of left turn movements	Movement 1	372	456	541
	Movement 2	180	221	262
	Total	552	677	803

Examining impacts of having different weaving lengths for Cloverleaf with C-D roads and without C-D roads at Interchange 4

Table 30 presents the outcomes of changing the weaving length for cloverleaf interchanges with C-D roads at Interchange 4. The data indicates that varying the weaving length has no substantial effect on improving traffic safety and operation performance at cloverleaf interchanges with C-D roads.

Table 30. Impacts of various weaving lengths on traffic operation and safety for cloverleaf interchanges with C-D roads at Interchange 4

Cloverleaf interchange with C-D roads	Level of Service (Delay)			Total Conflicts		
	Current	10 years	20 years	Current	10 years	20 years
450 ft weaving segment	B (12.03)	C (24.62)	D (35.23)	1930	4297	5409
550 ft weaving segment	B (11.71)	C (24.87)	D (34.74)	1917	4264	5320
650 ft weaving segment	B (11.83)	C (23.98)	D (33.65)	1878	4161	5290

Similarly, Table 31 presents the results of varying weaving lengths at cloverleaf interchanges without C-D roads. In this case, Interchange 4 has lower traffic volume. Notably, after 20 years, the interchange still maintains a Level of Service of C without increasing the weaving length. The result indicates that there is no significant change in LOS by increasing the weaving length. This suggests that at lower traffic volumes, modifying the weaving length in cloverleaf interchanges without C-D roads does not significantly impact performance.

Table 31. Impacts of various weaving lengths on traffic operation and safety for cloverleaf interchanges without C-D roads at Interchange 4

Cloverleaf interchange without C-D roads	Level of Service (Delay)			Total Conflicts		
	Current	10 years	20 years	Current	10 years	20 years
450 ft weaving segment	A (9.72)	B (14.11)	C (19.97)	1428	2285	3321
550 ft weaving segment	A (9.48)	B (13.95)	C (19.76)	1323	2157	3127
650 ft weaving segment	A (9.36)	B (13.64)	C (19.65)	1312	2002	3066

<u>Examining impacts of higher traffic volume at Interchange 4</u>

To examine whether the results of Interchange 4 will change or remain the same with higher traffic volume, the research team tested one additional scenario considering higher traffic volume from Interchange 1. This analysis was done to check the performance of Interchange 4 when there is an even distribution of left turn traffic and higher traffic volume.

Table 32 presents the outcomes for the current level of service and safety when the traffic volume from Interchange 1 was considered at Interchange 4. Regarding traffic operation, the results indicate that both types of cloverleaf interchanges (with and without C-D roads) performed better than the other two types of diamond interchanges (with signalized intersections and with roundabouts). With respect to traffic safety, it was found that diamond interchanges with roundabouts remained the best configuration, outperforming the performance of the other three configurations.

Table 32. Traffic operation and safety results at Interchange 4 considering higher traffic volumes

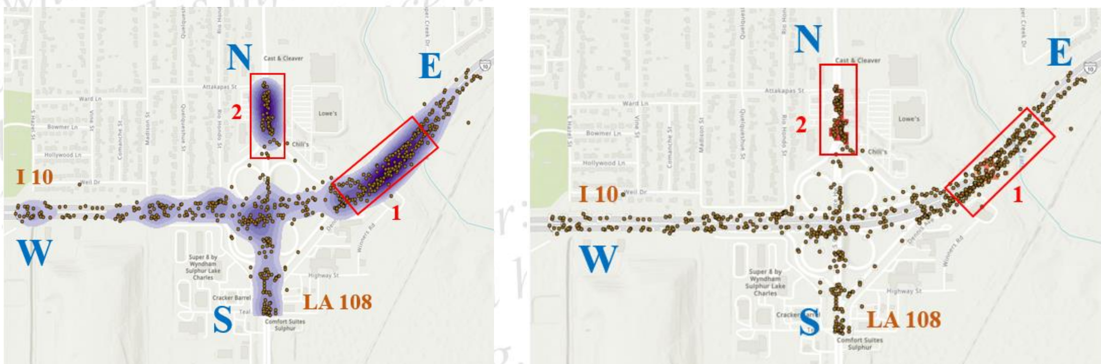
Interchange Type	Level of service in current scenario	Safety results in current scenario
Cloverleaf with C-D roads	B (14.67)	2695
Cloverleaf without C-D roads	B (13.31)	3017
Diamond with signalized intersections	C (33.96)	2405
Diamond with roundabouts	C (24.87)	2278

Task 4: Crash Data Analysis

Interchange 1 (Cloverleaf interchange with C-D roads)

Figure 36 illustrates the distribution of crashes and hotspots at Interchange 1, using both KDE and the Getis-Ord G_{i^*} statistic. The figure on the left presents the results from the KDE analysis, while the figure on the right shows the findings from the Getis-Ord G_{i^*} statistic.

Figure 36. Distribution of crashes and hotspots at Interchange 1



The locations of hotspots from KDE and Getis-Ord G_{i^*} statistics are:

- Merging and diverging segments at the East part of Interchange 1.
- Merging and diverging segments at the North part of Interchange 1.

Table 33 presents the primary contributing factors of all crashes and hotspots at Interchange 1, based on crash data analysis. The results indicate that violations are the most significant contributing factor, followed by movement prior to the crash. Crashes due to road surface and roadway condition occurred less frequently (i.e., only 1% and 4% of crashes, respectively). Further analysis of the types of violations and movement prior to crashes was done to provide more comprehensive insights into their root causes.

Table 33. Primary contributing factors of crashes at Interchange 1

Primary contributing factors of crashes	All crashes	Crashes at Hotspots
Violations	76%	80%
Movement prior to crash	15%	17%
Road Surface	1%	0%
Roadway Condition	4%	0%

Violations

Table 34 presents the distribution of violation types at Interchange 1. The findings reveal that the most common violation type is careless operation, accounting for 27.4% of the total number. Following too closely is the second most frequent violation type, at 13.5%. Exceeding the safe speed limit is another common violation, making up 10.6% of the total number. Failure to yield accounts for 9.6% of the violations, and turning from the wrong lane represents 6.7% of the total. 6.3% of the crashes at Interchange 1 had no violations. Finally, other unspecified violations constitute 8.6% of all violations.

Table 34. Violation types at Interchange 1

Violation Type	Percentage (%)
Careless operation	27.4
Following too closely	13.5
Exceeding safe speed limit	10.6
Failure to yield	9.6
Turned from wrong lane	6.7
No violations	6.3
Others	8.6

Movement Prior to Crash

Table 35 presents the distribution of vehicles' movements prior to crashes at Interchange 1. The findings reveal that "proceeding straight ahead" was the movement prior to approximately 54.6% of crashes at this site. Changing lanes on a multi-lane road was the second most common movement prior to crashes, accounting for 16.7% of the total. Entering the freeway from a ramp involved 8.5% of crashes, while leaving the freeway via an off-ramp accounted for 4.4%. Making a left turn was the movement prior to approximately 2.8% of all crashes. Both running off the road (i.e., not while making a turn at an intersection) and other or unknown movements each represented 2.5% of the total crashes.

Table 35. Movement prior to crash at Interchange 1

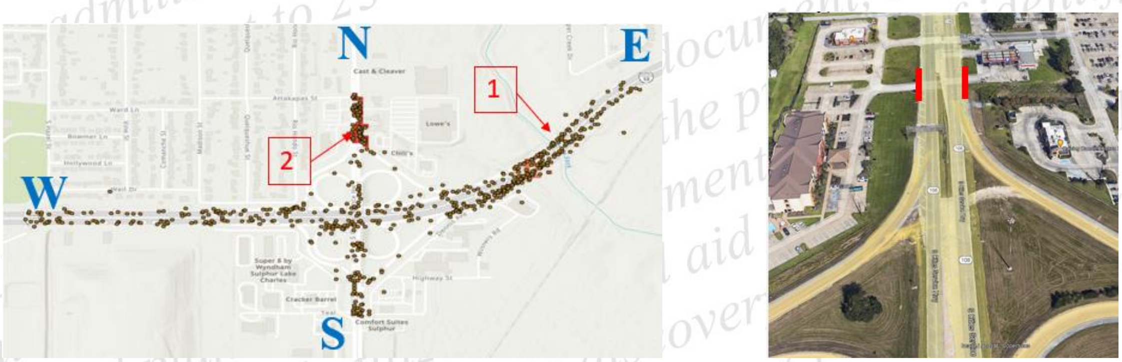
Movement prior to crash	Percentage (%)
Proceeding straight ahead	54.6
Changing lanes on multi-lane road	16.7
Entering freeway from ramp	8.5
Leaving freeway via off ramp	4.4
Making left turn	2.8
Ran off road (Not while making turn at intersection)	2.5
Others or unknown	2.5

It should be noted that the primary contributing factors of crashes, along with violation type and movement prior to crash, at the other seven interchanges were investigated, and the results were like those of Interchange 1 (shown in Tables 33, 34, and 35). Due to the page number constraints of the report, these results are available in the Appendix.

Possible reasons for the hotspots at Interchange 1, shown in Figure 37, include:

- The presence of an S-curve at the east part of the interchange.
- The presence of an intersection and driveways close to the merging and diverging sections at the north part of the interchange.

Figure 37. Locations and possible reasons for hotspots at Interchange 1



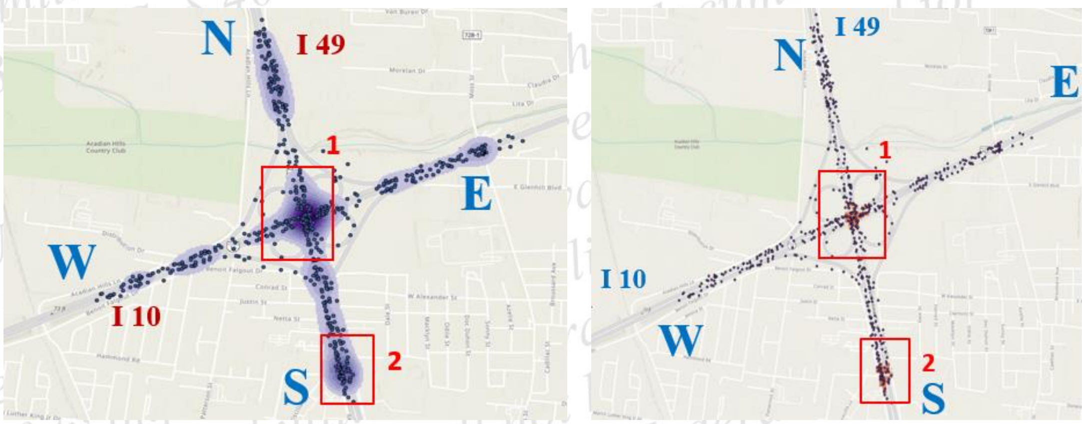
The following countermeasures were suggested to improve these hotspots:

- It is recommended to close the driveways close to the on/off ramps of the north part of the interchange, in addition to adding a taper/acceleration lane at the end of the off-ramp merging with the minor road and adding a taper/deceleration lane at the beginning of the on-ramp on the minor road.

Interchange 2 (Cloverleaf interchange without C-D roads)

Figure 38 illustrates the distribution of crashes and hotspots at Interchange 2, using both KDE and the Getis-Ord G_{i*} statistic. The figure on the left presents the results from the KDE analysis, while the figure on the right shows the findings from the Getis-Ord G_{i*} statistic.

Figure 38. Distribution of crashes and hotspots at Interchange 2



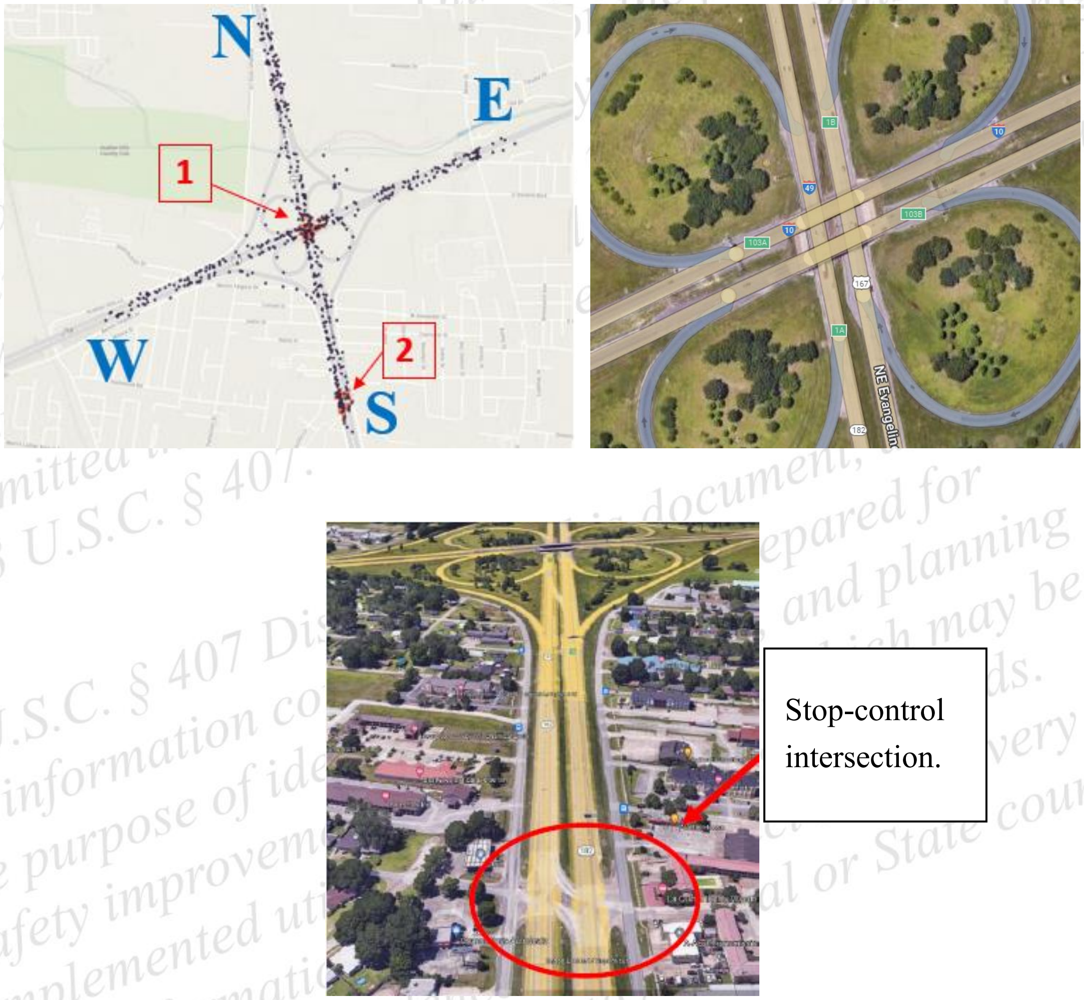
As shown in Figure 38, the location of hotspots at Interchange 2 included:

- Weaving segments
- The south part of interchange, near the stop-control intersection

Possible reasons for the hotspots at Interchange 2, shown in Figure 39, include:

- The presence of short weaving lengths with high traffic volumes on roadways (the weaving length for EB and WB is 600 ft., and the weaving length for NB and SB is 605 ft.)
- The distance between the off-on ramps on the minor road of the interchange and intersection is short

Figure 39. Locations and possible reasons for hotspots at Interchange 2



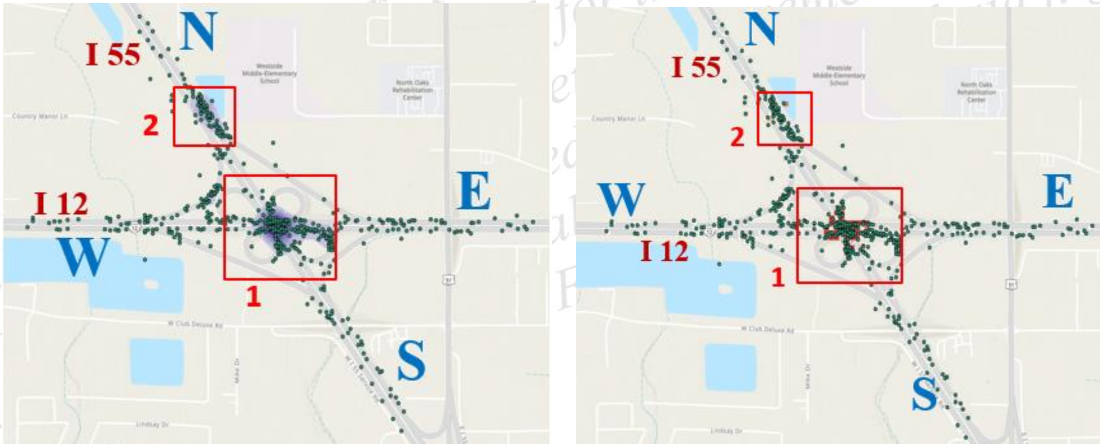
The following countermeasures were suggested to improve these hotspots at Interchange 2:

- Adding C-D roads or a semi-directional ramp
- Replacing the stop-controlled intersection with a signalized intersection, as shown in Figure 39

<u>Interchange 3 (Cloverleaf interchange without C-D roads)</u>

Figure 40 illustrates the distribution of crashes and hotspots at Interchange 3, using both KDE and the Getis-Ord G_i* statistic. The figure on the left presents the results from the KDE analysis, while the figure on the right shows the findings from the Getis-Ord G_i* statistic.

Figure 40. Distribution of crashes and hotspots at Interchange 3



As shown in Figure 40, the location of hotspots at Interchange 3 included:

- Weaving segments
- The merging segment at the north part of the interchange

Possible reasons for the hotspots at Interchange 3, shown in Figure 41, include:

- Having short weaving segments between two loop ramps

The following countermeasures were suggested to improve these hotspots at Interchange 3:

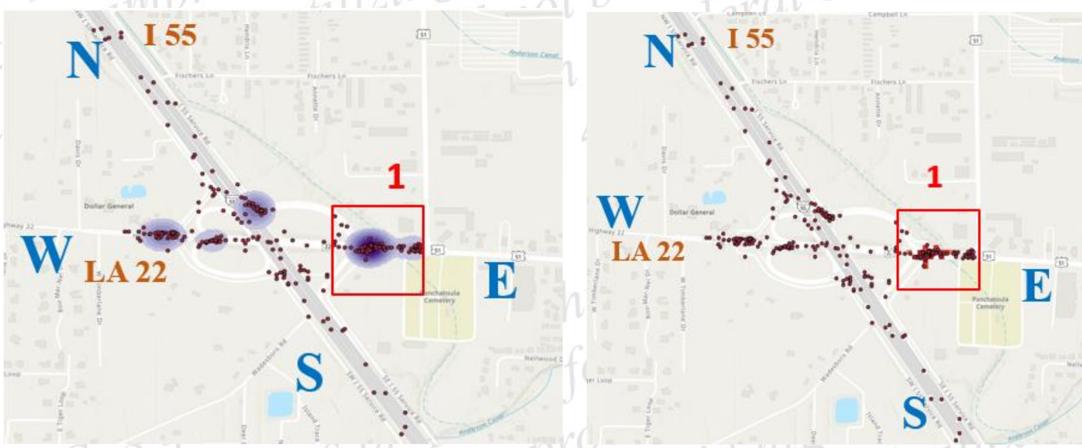
- A C-D road can be adopted at the weaving segments
- A semi-directional ramp can be adopted to reduce the number of loop ramps

Figure 41. Possible reasons for hotspots at Interchange 3

Interchange 4 (Cloverleaf interchange with C-D roads)

Figure 42 illustrates the distribution of crashes and hotspots at Interchange 4, using both KDE and the Getis-Ord G_i^* statistic. The figure on the left presents the results from the KDE analysis, while the figure on the right shows the findings from the Getis-Ord G_i^* statistic.

Figure 42. Distribution of crashes and hotspots at Interchange 4



As shown in Figure 42, the location of hotspots at Interchange 4 included:

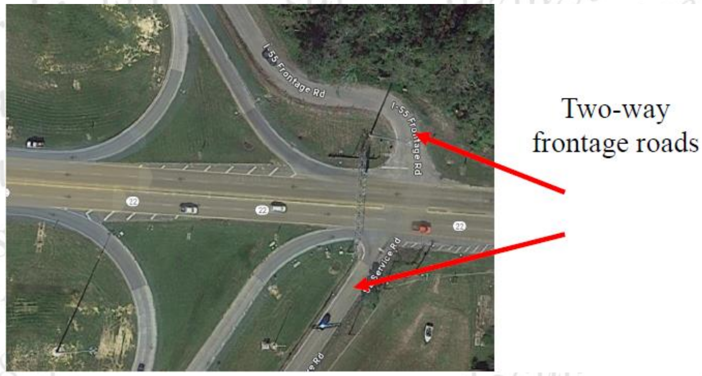
- Merging and diverging segments at the east part of the interchange (both directions of LA 22)

Possible reasons for the hotspots at Interchange 4, shown in Figure 43, include:

- The presence of two-way frontage roads near the off ramps and on ramps

- The presence of an uncontrolled intersection

Figure 43. Possible reasons for hotspots at Interchange 4



The following countermeasures were suggested to improve these hotspots at Interchange 4:

- Two-way frontage roads can be converted into one-way frontage roads
- The distance between the on ramp/off ramp and frontage roads should be increased

<u>Interchange 5 (Diamond interchange with stop-controlled intersections)</u>

Figure 44 illustrates the distribution of crashes and hotspots at Interchange 5, using both KDE and the Getis-Ord G_{i*} statistic. The figure on the left presents the results from the KDE analysis, while the figure on the right shows the findings from the Getis-Ord G_{i*} statistic.

Figure 44. Distribution of crashes and hotspots at Interchange 5

As shown in Figure 44, the hotspots at Interchange 5 were located at the two stop-controlled intersections on the minor road of the diamond interchange.

Possible reasons for the hotspots at Interchange 5, as shown in Figure 45, include:

- The presence of stop-controlled intersections
- The presence of right-turn slip-lanes

The following countermeasure was suggested to improve these hotspots at Interchange 5:

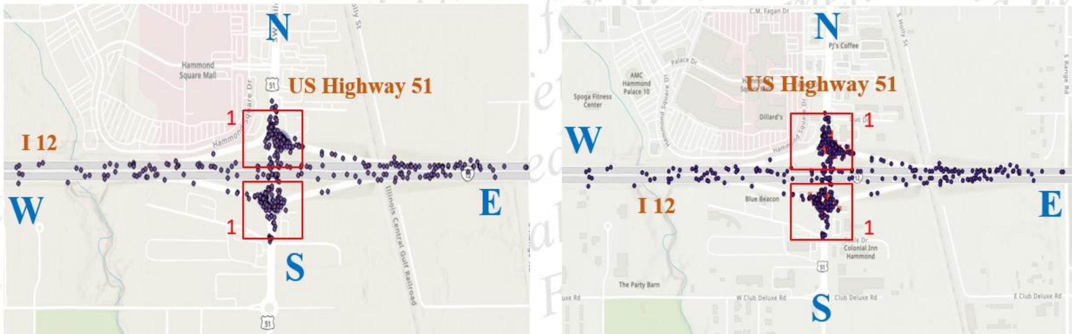
- Stop controlled intersections may need to be converted to signalized intersections or roundabouts

Figure 45. Possible reasons for hotspots at Interchange 5

<u>Interchange 6 (Diamond interchange with double roundabouts)</u>

Figure 46 illustrates the distribution of crashes and hotspots at Interchange 6, using both KDE and the Getis-Ord G_i^* statistic. The figure on the left presents the results from the KDE analysis, while the figure on the right shows the findings from the Getis-Ord G_i^* statistic.

Figure 46. Distribution of crashes and hotspots at Interchange 6



As shown in Figure 46, the location of hotspots was primarily at the roundabouts of Interchange 6.

Possible reasons for the hotspots at Interchange 6, as shown in Figure 47, may include the failure to yield with traffic in the roundabouts.

The following countermeasures were suggested to improve these hotspots at Interchange 6:

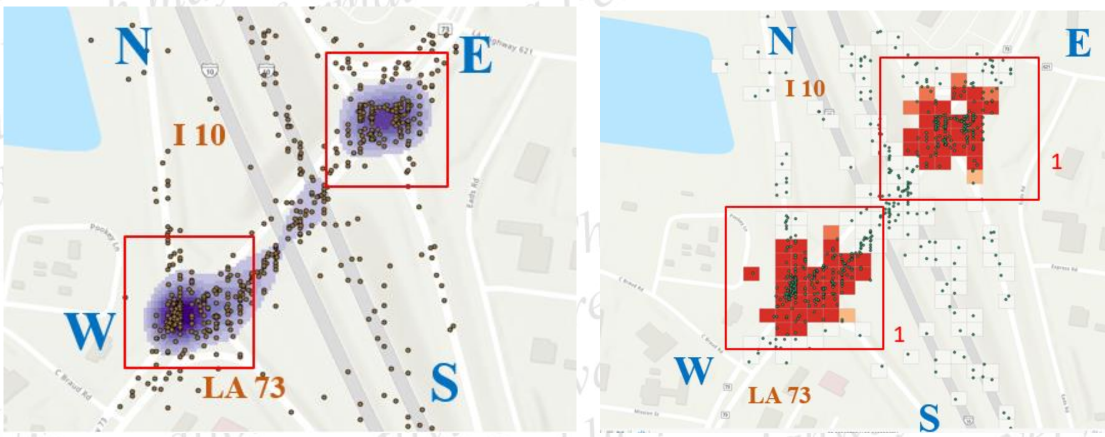
- Dedicated lanes for the right-turn movements can be provided to separate them from other movements at the roundabouts
- Driver awareness about driving at roundabouts should be improved to enhance drivers' understanding about the priority rules at roundabouts

Figure 47. Possible reasons for hotspots at Interchange 6

Interchange 7 (Diamond interchange with signalized intersections)

Figure 48 illustrates the distribution of crashes and hotspots at Interchange 7, using both KDE and the Getis-Ord G_i^* statistic. The figure on the left presents the results from the KDE analysis, while the figure on the right shows the findings from the Getis-Ord G_i^* statistic.

Figure 48. Distribution of crashes and hotspots at Interchange 7



As shown in Figure 48, the location of hotspots at Interchange 7 was primarily at signalized intersections on the minor road.

Possible reasons for the hotspots at Interchange 7, as shown in Figure 49, include:

- The short distance between nearby signalized intersections (less than 400 ft.)
- The absence of dedicated lanes for merging vehicles

The following countermeasures were suggested to improve these hotspots at Interchange 7:

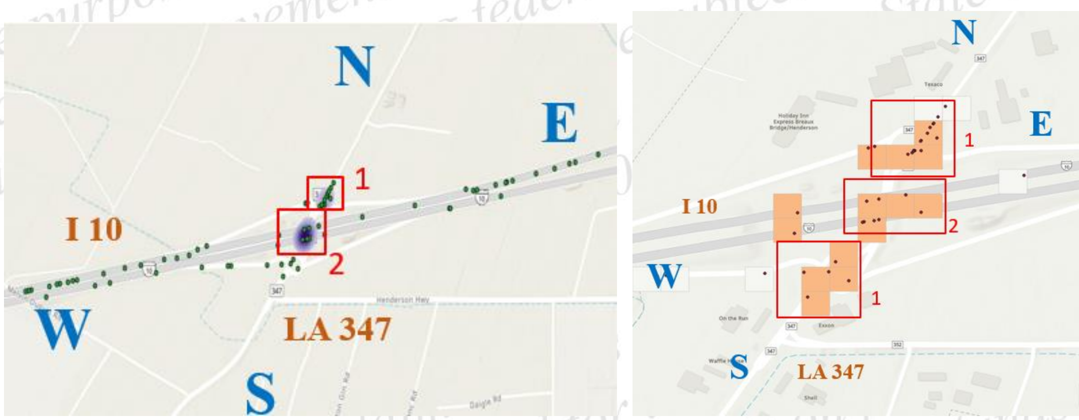
- Consider adding dedicated lanes for merging vehicles with the minor roads
- Right-turn slip-lanes can be removed to provide greater distance between nearby signalized intersections

Figure 49. Possible reasons for hotspots at Interchange 7

<u>Interchange 8 (Diamond interchange with double roundabouts)</u>

Figure 50 illustrates the distribution of crashes and hotspots at Interchange 8, using both KDE and the Getis-Ord G_{i*} statistic. The figure on the left presents the results from the KDE analysis, while the figure on the right shows the findings from the Getis-Ord G_{i*} statistic.

Figure 50. Distribution of crashes and hotspots at Interchange 8



As shown in Figure 50, the location of hotspots at Interchange 8 were primarily at both roundabouts, and between the two roundabouts.

Possible reasons for the hotspots at Interchange 8, as shown in Figure 51, included:

- The presence of sharp curves at the end of both off ramp and beginning of roundabouts
- The presence of one lane roundabouts
- Drivers' failure to yield with the traffic in the roundabouts

Figure 51. Possible reasons for hotspots at Interchange 8

The following countermeasure was suggested to improve these hotspots at Interchange 8:

- The radius of the sharp curves can be increased

Analysis of Severity and Manner of Collision

The severity level and collision manner were evaluated for the eight interchanges under investigation. Across all cases, property damage only (non-injury) crashes were more prevalent (approximately 77%), whereas fatal (0.4%) and severe crashes (0.5%) were insignificant. It should be noted that fatal crashes occurred primarily at the merging and diverging area for both cloverleaf and diamond interchanges.

In terms of collision manner, rear-end (44.2 %), sideswipe collisions in the same direction (21.0%), and non-collision with motor vehicles (20.6 %) were the most common types of crashes. Please refer to Appendix K for a more detailed analysis regarding the severity and manner of collision.

Safety Performance Functions (SPFs) and comparison between interchanges

Development of SPFs and crash number-rate at four cloverleaf interchanges

Table 36 presents a detailed overview of crash data, traffic volume, and segment lengths for four distinct cloverleaf interchanges. The data showed the average annual number of crashes, Annual Average Daily Traffic (ADT), and the lengths of major road, minor road, and ramp segments for each interchange from 2016 to 2021. Notably, the table also includes the effective segment length for major road segments, providing a refined measure for safety analysis.

Table 36. Details of data required for developing SPFs at cloverleaf interchanges

Cloverleaf interchanges	Types of road segments	# of observed crashes	AADT	Segment length (mi)	Effective segment length (mi)
Interchange 1 (with C-D roads)	Major Road	74	109,353	1.08	0.465
	Minor Road	27	16,820	0.46	
	Ramps	19	41,133	1.84	
Interchange 2 (without C-D roads)	Major Road	39	70,250	1.23	0.265
	Minor Road	52	72,880	1.19	
	Ramps	11	61,743	2.62	
Interchange 3 (without C-D roads)	Major Road	43	74,007	1.12	0.445
	Minor Road	30	65,643	1.06	
	Ramps	27	58,098	2.06	
Interchange 4 (with C-D roads)	Major Road	6	39,470	1.03	0.51
	Minor Road	29	14,867	0.36	
	Ramps	17	20,366	1.68	

Note: The major road of all cloverleaf interchanges is a freeway. As per the HSM 2014, the segment length should be effective length. The effective length of the major road (freeway) is calculated using the equations given above.

Table 37 presents findings from a negative binomial regression model analyzing crash frequencies at cloverleaf interchanges in Louisiana. For the major road, significant coefficients include the intercept (-22.88) and log_AADT (2.44), with strong statistical support from z-values (-3.29 for intercept, 3.94 for log_AADT) and low p-values (0.00099 for intercept, 8.06E-05 for log_AADT), indicating a robust relationship between traffic volume and crashes. Conversely, the minor road segment shows a decrease in crash frequency with increased traffic volume, as evidenced by a negative coefficient for log_AADT (-0.42) and significant statistics (z-value of -3.04, p-value of 0.0023). This pattern suggests that crashes are less frequent at freeway minor roads compared to other segments. However, the ramps segment reveals less definitive relationships between traffic metrics and crashes, with higher p-values (0.50, 0.44, and 0.39) for the intercept (-18.68), log_AADT (2.25), and AADT (-6.7E-05), respectively.

Model fit varies by segment: The major road exhibits strong predictive performance with an R-squared of 0.90, the minor road segment shows a moderate fit with an R-squared of 0.50, indicating the need for further refinement, and the ramps segment has an R-squared of 0.071, suggesting that the model shows weak predicted performance for the ramps.

Table 37. Output of negative binomial regression of cloverleaf interchanges

Segment	Coefficient	Estimate	Std. Error	z value	p value	R-squared
Major Road	Intercept	-22.88	6.9538	-3.29	0.00099*	0.9
	log_AADT	2.44	0.6208	3.94	8.06E-05*	
Minor Road	Intercept	8.33	1.4711	5.66	1.49E-08*	0.5
	log_AADT	-0.42	0.1396	-3.04	0.0023*	
Ramps	Intercept	-18.68	27.99	-0.66	0.50	0.071
	log_AADT	2.25	2.967	0.75	0.44	
	AADT	-6.7E-05	7.97E-05	-0.84	0.39	

Table 38 illustrates the crash rates at four cloverleaf interchanges, with and without collector-distributor (C-D) roads. Interchange 1, which has C-D roads, experiences a crash rate of 2.61 crashes per year per mile. Similarly, Interchange 4, also with C-D roads, records a crash rate of 2.62 crashes per year per mile. On the other hand, Interchanges 2 and 3, which lack C-D roads, show lower crash rates of 1.95 and 1.96 crashes per year per mile, respectively. The two cloverleaf interchanges with C-D roads have a freeway intersecting with arterial roads, whereas the other two cloverleaf interchanges without C-D roads have two freeways intersecting with each other. This pattern indicates that interchanges with C-D roads, particularly where the freeway intersects with non-freeway minor roads, tend to have higher crash rates compared to those where both the major road and minor road are freeways. It should be noted that these results are based on analyzing the four cloverleaf interchanges only.

Table 38. Crash rate calculation using number rate for cloverleaf interchanges

Cloverleaf Interchanges	Crash rate
Interchange 1 (with C-D roads)	2.61

Cloverleaf Interchanges	Crash rate
Interchange 2 (without C-D roads)	1.95
Interchange 3 (without C-D roads)	1.96
Interchange 4 (with C-D roads)	2.62

Note: All four cloverleaf interchanges were treated as intersections for crash rate calculation.

Development of SPFs and calculation of crash rate at four diamond interchanges

Table 39 presents a detailed overview of crash data, traffic volume, and segment lengths for four diamond interchanges. The data showed the average annual number of crashes, Annual Average Daily Traffic (AADT), and the lengths of major road, minor road, and ramp segments for each interchange from 2016 to 2021. Notably, the table also includes the effective segment length for major road segments, providing a refined measure for safety analysis.

Table 39. Details of data required for developing SPF for diamond interchanges

Diamond Interchanges	Road Segment Type	Crash Count	AADT	Segment length (mi)	Effective segment length (mi)
Interchange 5 (with stop-controlled intersections)	Major Road	15	80,520	1.13	0.40
	Ramps	2	11,729	1.46	
Interchange 6 (with double roundabouts)	Major Road	28	74,007	1.06	0.63
	Ramps	21	29,631	0.86	
Interchange 7 (with signalized intersections)	Major Road	52	90,510	1.16	0.57
	Ramps	41	35,994	1.19	
Interchange 8 (with double roundabouts)	Major Road	21	62,470	1.06	0.57
	Ramps	7	13,634	0.99	

Note: The major road of all cloverleaf interchanges is a freeway. As per the HSM, the segment length should be effective length. The effective length of the major road (freeway) was calculated using the equations given above.

Table 40 shows the result of negative binomial regression analysis of the diamond interchanges. The result indicates a significant positive relationship between traffic volume, as measured by the logarithm of Average Annual Daily Traffic (log_AADT), and crash rates on major road segments, with a coefficient of 2.509 ($p = 0.00167$) and an R-squared value of 0.89, highlighting that 89% of the variance in crash rates on the major road can be explained by this model. However, for ramp segments, while the model has a high explanatory power with an R-squared value of 0.98, the coefficients for log_AADT (4.382) and AADT (-9.720E-05) are not statistically significant (p-values of 0.112 and 0.425, respectively), suggesting that the relationship between traffic volume and crash rates on ramps is less clear and may be influenced by other factors not captured in this model. This disparity highlights the detailed impact of traffic volume on crash rates, with a clear positive correlation on major road segments but an ambiguous relationship on ramps. This can be attributed to the limited number of sites assessed, which fails to accurately capture the precise relationship between predicted crashes and AADT at ramps.

Table 40. Output of negative binomial regression model of diamond interchanges

Segment	Coefficient	Estimate	Std. Error	z value	p value	R-squared
Major Road	Intercept	-24.277	8.997	-2.698	0.00697*	0.89
	log_AADT	2.509	0.798	3.144	0.00167*	
Ramps	Intercept	-38.96	24.67	-1.579	0.114	0.98
	log_AADT	4.382	2.758	1.589	0.112	
	AADT	-9.720E-05	1.217E-04	-0.798	0.425	

<u>Comparing the safety of four diamond interchanges using number-rate</u>

Table 41 presents an analysis of crash rates across the four diamond interchanges under investigation in this study. The interchange featuring stop-controlled intersections (Interchange 5) showed the lowest crash rate of 0.7 crashes per year per mile. Conversely, interchanges utilizing roundabouts (Interchange 6 and Interchange 8) and signalized intersections (Interchange 7) showed higher crash rates of 2.6, 1.55, and 3.52 crashes per year per mile, respectively. However, it should be noted that these conclusions are drawn from a limited sample of only four diamond interchanges.

Table 41. Crash rate calculation using number-rate of diamond interchanges

Diamond Interchanges	Crash rate
Interchange 5 (stop-controlled intersections)	0.7
Interchange 6 (Roundabouts)	2.6
Interchange 7 (Signalized intersections)	3.52
Interchange 8 (Roundabouts)	1.55

Note: All four diamond interchanges were treated as intersections for crash rate calculation.

Comparing the overall safety between cloverleaf and diamond interchanges from crash rate given by Louisiana DOTD

When comparing the overall crash rate at eight different locations across Louisiana (four diamond interchanges and four cloverleaf interchanges), it can be concluded that the diamond interchange with signalized intersections (Interchange 7) showed the highest crash rate (3.52 crashes per year per mile).

Note: Comprehensive data analysis was conducted on the crash data. However, we acknowledge the limitations inherent in the dataset. Specifically, most data elements in the Louisiana crash dataset are between 70% and 80% accurate, with location data being only 82% accurate at the 0.05-mile threshold. This limitation should be considered when interpreting the results of our analysis.

Conclusions

This study conducted a comprehensive evaluation of traffic safety and operational performances of cloverleaf and diamond interchanges. A total of eight interchanges were analyzed, with an equal split of four cloverleaf and four diamond interchanges. Initially, a comparison between cloverleaf and diamond interchanges (cloverleaf with C-D roads, cloverleaf without C-D roads, diamond with signalized intersections, and diamond with double roundabouts) was carried out based on microsimulation analysis using PTV VISSIM, considering various scenarios including high and low traffic volumes, different traffic signals, lengths of weaving segments, various inner circle diameters (ICDs) of roundabouts, and varying left turn volumes. Additionally, the traffic safety and operation of all eight interchanges were assessed in their current condition as well as in two projected future scenarios, after 10 and 20 years. Countermeasures were suggested for those interchanges that did not meet an acceptable level of service. Following this, a crash analysis was conducted using ArcGIS Pro to identify hotspots and propose potential countermeasures. Safety Performance Functions (SPFs) were employed to predict the number of crashes at both cloverleaf and diamond interchanges and compare them to the observed crashes. Finally, crash rates were estimated for all interchanges under investigation following the 2023 Louisiana DOTD crash data analysis guidelines. The results of each objective of this research are summarized below.

Results of Objective 1: Assess the safety and operational performances of cloverleaf interchanges in Louisiana compared to that of traditional diamond interchanges

- The findings indicated that while most of the eight interchanges under investigation showed an acceptable level of service, some of them may need to be improved in the future (i.e. after 10 or 20 years) to continue providing an acceptable level of service.
- According to the results of the microsimulation investigation, cloverleaf interchanges are more suitable for managing heavy traffic volumes than diamond interchanges. However, they posed significant safety concerns (e.g., more conflict points), particularly at the weaving segments.
- When traffic volumes are high (i.e., entering volume > 7000 vph), it was found that cloverleaf interchanges with C-D roads outperform all other interchanges in terms of traffic operations. Diamond interchanges with roundabouts on the minor road outperform other interchange configurations in terms of traffic safety.

- At lower traffic volumes (i.e., entering volume < 5000 vph), it was found that both cloverleaf interchanges with and without C-D roads perform better than diamond interchanges in terms of traffic operation. Regarding traffic safety, the results revealed that diamond interchanges with roundabouts still outperform the performance of other configurations at low traffic volumes.
- Furthermore, it was discovered that increasing the weaving lengths of cloverleaf with C-D roads has no significant effect on improving traffic safety and operational performances. On the other hand, extending the weaving lengths of cloverleaf interchanges without C-D roads significantly improved traffic safety and operation.
- For diamond interchanges with double roundabouts, it was found that increasing the inscribed circle diameters (ICDs) showed a slight positive impact on both the levels of service and safety at higher traffic volumes.
- Additionally, for diamond interchanges with signalized intersections, it was found that altering the traffic signal timing showed a slight positive impact on both the levels of service and safety at higher traffic volumes.

<u>Results of Objective 2: Employ safety and traffic analysis to predict future performance of cloverleaf and diamond interchanges in Louisiana</u>

- According to the results of the microsimulation analysis, it was found that most of the interchanges are currently operating at an acceptable level of service, with the exception of Interchange 7, a diamond interchange with signalized intersections.
- The results showed that some of the interchanges need modification after 10 years, and most of the interchanges need modification after 20 years.
- Interchange 4 (cloverleaf with C-D roads) and Interchange 8 (diamond with roundabouts) do not need modifications.

<u>Results of Objective 3: Suggest countermeasures/alternative interchange solutions that should be implemented if a cloverleaf or diamond interchange is not an appropriate alternative based on their predicted future performance</u>

<u>Considering Results of Microsimulation Analysis</u>

- The research team suggested several countermeasures to improve traffic safety and operational performance for those interchanges that are currently operating, or will be operating in the future, under unacceptable levels of service. For example, the team evaluated the effectiveness of adding an extra lane at some interchanges to enhance traffic safety and operation, which proved to be an effective countermeasure in most scenarios. Additionally, they suggested the following recommendations to further improve traffic safety and operation.

- For cloverleaf interchanges without C-D roads, implementing a semi-directional ramp with an extra lane addition on the freeway was effective in maintaining acceptable level of service over 20 years when the weaving volumes are high.
- It was found that considering one-way frontage roads instead of two-way frontage roads is effective in handling large traffic volumes.
- For stop-controlled intersections, when traffic volume is high, it is recommended to replace it with signalized intersections to maintain acceptable levels of service and safety.
- When traffic volume increases, it was found that using C-D roads is effective in maintaining an acceptable level of service.

The detailed recommendations of countermeasures for the eight interchanges currently, after 10 years, and after 20 years have been provided in the Appendix.

Considering Results of Crash Data / Hotspots Analysis

- For cloverleaf interchanges without C-D roads, it was found that most of the crashes occurred at the weaving segments. Therefore, increasing the weaving lengths or adding C-D roads, where possible, can help improve safety performance.
- For cloverleaf interchanges with C-D roads, off ramps and on ramps are close to the two-way frontage roads where most crashes occurred. Therefore, it is recommended to keep frontage roads at a greater distance from the on ramps and off ramps to enhance safety.
- For diamond interchanges with signalized or stop-controlled intersections, it was found that most of the crashes occur at those intersections. It is recommended to reduce the number of access points, and thus the number of conflict points, close to these intersections and ensure that there are enough acceleration and deceleration lanes. Additionally, traffic compliance studies may be conducted to determine the reasons for crashes at these intersections.

- For diamond interchanges with stop-controlled intersections on the minor road, it was found that crashes primarily occurred at these intersections. Therefore, it is recommended to replace stop-controlled with signalized intersections.
- For diamond interchanges with double roundabouts, the majority of crashes happen at the roundabouts. Therefore, it is recommended to improve drivers' awareness through campaigns to enhance their understanding of the priority rules at roundabouts.

Recommendations

The following recommendations can be drawn from this study:

- Implementing longer weaving sections for cloverleaf interchanges without C-D roads can mitigate safety concerns associated with increasing conflict points at weaving segments. This could potentially reduce crash rates in these areas.
- Where feasible, it is recommended to add C-D roads to existing cloverleaf interchanges that currently lack them. The results of this study indicated that cloverleaf interchanges with C-D roads perform better in terms of traffic safety and operation at high traffic volumes.
- Optimize traffic signal timing at the signalized intersections of diamond interchanges to help improve both traffic safety and operation, reducing both delays and the likelihood of collisions.
- Conduct further future research and data collection to develop local Safety Performance Functions (SPFs) tailored to Louisiana's conditions.
- Plan and execute infrastructure improvements based on identified hotspots and crash data analysis to specifically target areas with high crash rates or operational inefficiencies.
- Enhance driver awareness and understanding regarding priority and right of way rules while approaching and driving on roundabouts.
- Collect more data, such as driving behaviors, to calibrate in VISSIM for more accurate results.

Acronyms, Abbreviations, and Symbols

Term	Description
AADT	Annual Average Daily Traffic
C-D roads	Collector Distributor roads
EB	Eastbound
EMS	Emergency Medical Services
GIS	Geographic Information System
KDE	Kernel Density Estimation
LR	Loop Ramp
NB	Northbound
SB	Southbound
SSAM	Surrogate Safety Assessment Model
vph	Vehicles per hour
WB	Westbound

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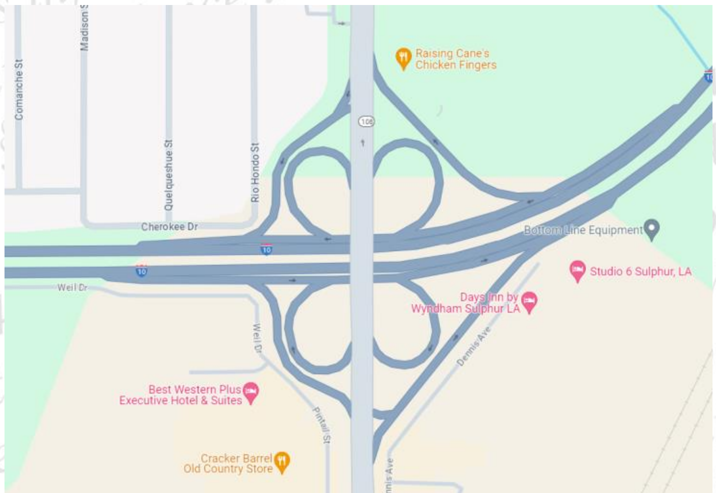
Appendix

Appendix A: Study Area

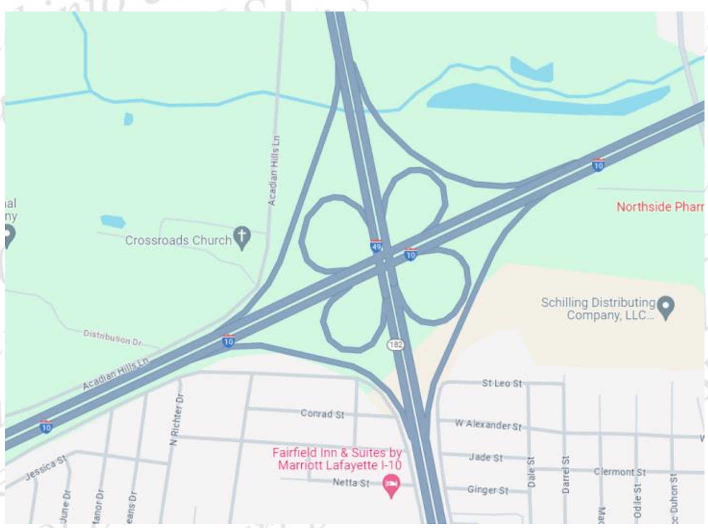
Interchange 1 (Cloverleaf with C-D roads)
Location: I-10/Louisiana 108 near Lake Charles

Figure 52. Interchange 1 (Cloverleaf with C-D roads)



Interchange 2 (Cloverleaf without C-D roads)
Location: I-10/I-49 at Lafayette

Figure 53. Interchange 2 (Cloverleaf without C-D roads)

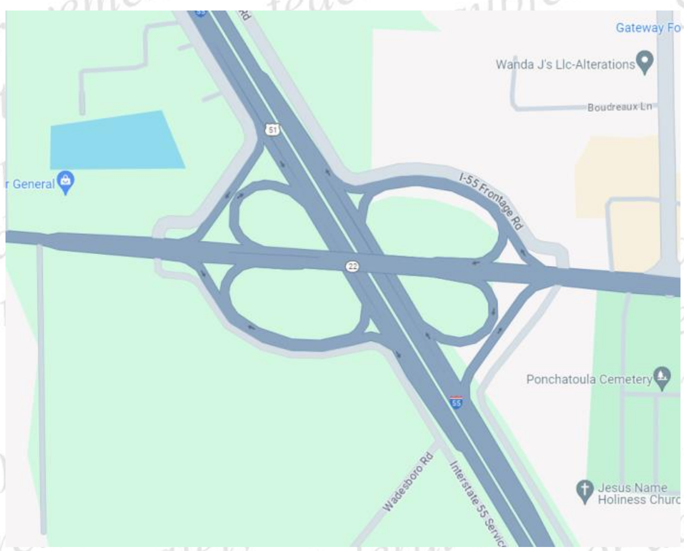


Interchange 3 (Cloverleaf without C-D roads)
Location: I-12/I-55 at Hammond

Figure 54. Interchange 3 (Cloverleaf without C-D roads)

Interchange 4 (Cloverleaf with C-D roads)
Location: I-55/LA 22 at Ponchatoula

Figure 55. Interchange 4 (Cloverleaf with C-D roads)



Interchange 5 (Diamond with stop-controlled intersections)
Location: I-12/Pumpkin Center Rd at Hammond

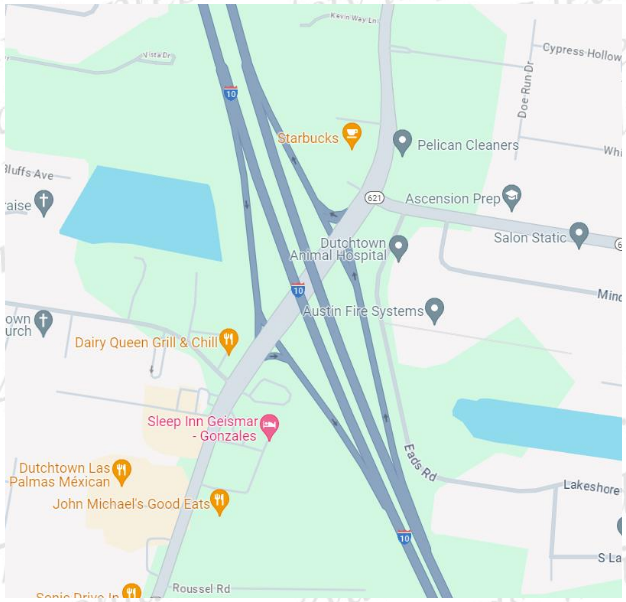
Figure 56. Interchange 5 (Diamond with stop-controlled intersections)

Interchange 6 (Diamond with double roundabouts)
Location: I-12/ SW Railroad Ave at Hammond

Figure 57. Interchange 6 (Diamond with double roundabouts)

Interchange 7 (Diamond with signalized intersections)
Location: I-10/ LA 73 at Dutch Town

Figure 58. Interchange 7 (Diamond with signalized intersections)



Interchange 8 (Diamond with double roundabouts)
Location: Louisiana 347 at Grand Point Highway

Figure 59. Interchange 8 (Diamond with double roundabouts)

Appendix B: Detail Calculation of Growth Rate

The equation used to calculate growth rate is:

$$i = (\frac{F}{P})^{1/n} - 1$$

Where,
F = target year AADT,
P = current year AADT,
i = growth rate, and
n = number of years between target and current years.

The growth rate was calculated for both major and minor roads to gain accuracy in forecasting volumes.

Table 42 shows the growth rate in all the major and minor roads in all eight interchanges.

Table 42. Growth rate of all eight interchanges

Forecasting years	10 years		20 years	
	Minor road (%)	Major road (%)	Minor road (%)	Major road (%)
Interchange 1	3.17	2.46	2.2	2.18
Interchange 2	1.01	1.31	1.06	1.2
Interchange 3	2.06	1.97	1.89	1.82
Interchange 4	2.28	2.06	1.64	1.89
Interchange 5	1.97	1.97	1.82	1.82
Interchange 6	0.29	1.87	0.6	1.59
Interchange 7	2.66	2.42	1.95	2.01
Interchange 8	1.98	1.13	1.56	1.07

Appendix C: Validation Results of VISSIM Model

Throughput and travel time are the two criteria used for the validation of the model.

Tables 43 and 44 present the throughput and travel time validation for Interchange 1. For throughput, all of the evaluated locations are validated. For travel time, all locations are validated except for WB and LR11.

Interchange 1:

Table 43. Throughput validation of Interchange 1

Location	Field data	Simulated data	GEH statistics	Allowable	Status
1. EB	2431	2414	0.3	<5	Met
2. WB	2534	2492	0.8	<5	Met
3. NB	1672	1661	0.3	<5	Met
4. SB	999	1003	0.1	<5	Met
R7	57	52	0.7	<5	Met
R8	981	972	0.3	<5	Met
R9	473	459	0.6	<5	Met
R10	369	370	0.1	<5	Met
LR11	318	305	0.7	<5	Met
LR12	188	178	0.7	<5	Met
LR13	144	121	2.0	<5	Met
LR14	234	223	0.7	<5	Met

Table 44. Travel time validation of Interchange 1

Location	Field Data (sec)	Length (L) in ft	Lower range	Higher range	Simulated Data (sec)	Status
1. EB	76.7	7920.0	73.3	80.1	78.4	Met
2. WB	75.9	8025.6	72.6	79.2	79.3	Not Met
3. NB	55.6	2698.5	50.0	61.1	52.9	Met
4. SB	50.5	2679.3	45.9	55.0	50.1	Met

Location	Field Data (sec)	Length (L) in ft	Lower range	Higher range	Simulated Data (sec)	Status
R7	32.8	940.0	26.8	38.7	32.1	Met
R8	25.7	1363.9	23.3	28.0	26.3	Met
R9	36.6	1280.2	31.3	41.9	38.7	Met
R10	21.6	1100.0	19.6	23.7	21.5	Met
LR11	17.8	825.4	15.9	19.7	21.5	Not Met
LR12	22.2	816.6	19.2	25.2	21.5	Met
LR13	20.5	889.2	18.2	22.9	21.7	Met
LR14	23.6	871.9	20.4	26.8	23.8	Met

Interchange 2:

Tables 45 and 46 present the throughput and travel time validation for Interchange 2. For throughput, all of the evaluated locations are validated except loop ramp3. For travel time, all of the evaluated locations are validated except for EB and offramp1.

Table 45. Throughput validation of Interchange 2

Location	Field data	Simulated data	GEH statistics	Allowable	Status
EB	2424	2412	0.24	<5	Met
WB	1785	1763	0.52	<5	Met
NB	2886	2890	0.07	<5	Met
SB	2087	2086	0.02	<5	Met
Offramp1	363	366	0.16	<5	Met
Onramp1	242	232	0.65	<5	Met
Offramp2	365	344	1.12	<5	Met
Onramp2	851	843	0.27	<5	Met
Loop ramp 1	285	283	0.12	<5	Met
Loop ramp2	969	897	2.36	<5	Met
Loop ramp3	537	281	12.66	<5	Not Met
Loop ramp4	290	285	0.29	<5	Met

Table 46. Travel time validation of Interchange 2

Location	Field Data	Length (L)	Allowable	Lower range	Higher range	Simulated Data	Status
EB	32.4	3229.44	1.50	30.9	33.9	36.5	Not Met
WB	32.4	3075.84	1.57	30.8	33.9	33.6	Met
NB	36.1	3092.37	1.95	34.1	38.0	37.8	Met
SB	34.1	3138.63	1.71	32.4	35.8	32.8	Met
Offramp1	26.0	2184.86	1.43	24.5	27.4	30.3	Not Met
Onramp1	28.1	2289.25	1.60	26.5	29.7	28.4	Met
Offramp2	28.5	1996.05	1.91	26.6	30.4	27.6	Met
Onramp2	31.4	2448.83	1.88	29.5	33.3	32.2	Met

Interchanges 3, 4, 5, & 6:

Tables 47 and 48 present the throughput and travel time validation for Interchanges 3, 4, 5, & 6. For throughput, all of the evaluated locations are validated. For travel time, all of the evaluated locations are validated except westbound.

Note: This model contains four interchanges; validation was done in different parts of the four interchanges.

Table 47. Throughput validation of Interchanges 3, 4, 5, & 6

	Location	Field data	Simulated data	GEH statistics	Allowable	Status
Interchange 3	SB	1870	1890	0.5	<5	Met
	Loop ramp1	523	481	1.9	<5	Met
	Loop ramp2	650	719	2.6	<5	Met
	Loop ramp3	490	450	1.8	<5	Met
	Loop ramp4	302	280	1.3	<5	Met
Interchange 4	EB	640	634	0.2	<5	Met
	WB	1255	1258	0.1	<5	Met
	NB	1575	1586	0.3	<5	Met
	Loop ramp1	302	150	10.1	>5	Not Met
	Loop ramp2	158	172	1.1	<5	Met
	Loop ramp3	181	160	1.6	<5	Met
	Loop ramp4	67	86	2.2	<5	Met

Interchange 5	EB	2668	2656	0.2	<5	Met
	NB	406	403	0.1	<5	Met
	SB	365	369	0.2	<5	Met
Interchange 6	WB	2572	2569	0.1	<5	Met
	NB	1169	1169	0.0	<5	Met
	SB	1392	1381	0.3	<5	Met
	Offramp1	504	604	4.2	<5	Met
	onramp1	851	756	3.4	<5	Met
	offramp2	772	781	0.3	<5	Met
	Onramp2	695	630	2.5	<5	Met

Table 48. Travel time validation of Interchanges 3, 4, 5, & 6

Location	Field Data	Length (L)	Allowable	lower range	Higher range	Simulated Data	Status
EB	338.916	35270.40	14.96	323.95	353.88	333.2	Met
WB	353.536	34056.00	16.92	336.61	370.46	388.9	Not Met
NB	257.05	24340.80	12.53	244.52	269.58	266.2	Met
SB	240.668	23918.40	11.15	229.52	251.82	230.8	Met
I5 NB/SB	68.576	2234.15	10.71	57.87	79.28	68.8	Met
I6 NB	14.542	361.55	3.13	11.42	17.67	14.4	Met
I6 SB	14.918	358.17	3.35	11.57	18.27	15.0	Met

Note: I6 NB/SB had one average travel time. EB, WB, NB and SB cover end to end in horizontal and vertical directions.

Interchange 7:

Tables 49 and 50 present the throughput and travel time validation for Interchange 7. For throughput, all of the evaluated locations are validated. For travel time, all of the evaluated locations are validated except NB and SB.

Table 49. Throughput validation of Interchange 7

Location	Field data	Simulated data	GEH statistics	Allowable	Status
EB	1157	1162	0.15	<5	Met
WB	1205	1213	0.23	<5	Met
NB	3783	3771	0.20	<5	Met
SB	4175	4162	0.20	<5	Met
Onramp1	449	462	0.61	<5	Met

Offramp1	822	886	2.19	<5	Met
Onramp2	956	979	0.74	<5	Met
Offramp2	1735	1601	3.28	<5	Met

Table 50. Travel time validation of Interchange 7

Location	Field Data (sec)	Length (L) in ft	Allowable	lower range	Higher range	Simulated Data	Status
EB	66.7	1392.22	17.80	48.87	84.48	68.3	Met
WB	100.5	1386.70	47.05	53.45	147.56	96.5	Met
NB	64.2	6124.80	3.10	61.06	67.26	83.2	Not Met
SB	54.8	5808.00	2.37	52.38	57.12	68.7	Not Met
Onramp1	17.1	1069.46	1.30	15.82	18.42	18.1	Met
Offramp1	58.5	1468.51	12.42	46.05	70.89	63.8	Met
Onramp2	17.3	1012.67	1.40	15.88	18.68	16.5	Met
Offramp2	133.0	1419.80	93.31	39.72	226.34	107.0	Met

Interchange 8:

Tables 51 and 52 present the throughput and travel time validation for Interchange 8. For throughput, all of the evaluated locations are validated. For travel time, all of the evaluated locations are validated.

Table 51. Throughput validation of Interchange 8

Location	Field data	Simulated data	GEH statistics	Allowable	Status
EB	1791	1768	0.55	<5	Met
WB	1711	1684	0.66	<5	Met
NB	505	507	0.09	<5	Met
SB	446	445	0.05	<5	Met
Offramp1	360	354	0.32	<5	Met
Onramp1	131	128	0.26	<5	Met
Offramp2	207	203	0.28	<5	Met
Onramp2	275	299	1.42	<5	Met

Table 52. Travel time validation of Interchange 8

Location	Field Data (sec)	Length (L) in ft	Allowable	lower range	Higher range	Simulated Data	Status
EB	85.4	8628.61	3.89	81.54	89.32	82.2	Met
WB	90.9	8858.27	4.30	86.62	95.22	93.6	Met

Location	Field Data (sec)	Length (L) in ft	Allowable	lower range	Higher range	Simulated Data	Status
Offramp1	28.1	1198.93	3.23	24.87	31.33	29.2	Met
Onramp1	22.3	1370.21	1.73	20.61	24.07	22.9	Met
Offramp2	30.5	1520.7	2.95	27.53	33.43	33.0	Met
Onramp2	24.7	1474.05	1.97	22.75	26.69	26.7	Met

Note: Field data of Northbound and Southbound was not available.

Appendix D: Level of Service Criteria

Table 53 shows the level of service criteria for the signalized intersection and unsignalized intersection, which is given by HCM 2010. Table 54 shows the level of service of the freeway, which is based on density. The level of service of all roadway segments is estimated based on the two tables.

Table 53. Level of service for signalized and unsignalized intersections (Exhibit 18-4 and Exhibit 19-1, HCM 2010 Vol 3)

LOS	Signalized Intersection	Unsignalized Intersection
A	≤10 sec	≤10 sec
B	10–20 sec	10–15 sec
C	20–35 sec	15–25 sec
D	35–55 sec	25–35 sec
E	55–80 sec	35–50 sec
F	>80 sec	>50 sec

Table 54. HCM freeway density criteria (Exhibit 10-7, HCM 2010 Vol 2)

Level of Service	Density (pc/mi/ln)
A	≤ 11
B	> 11-18
C	> 18-26
D	> 26-35
E	> 35-45
F	> 45 or Demand exceeds capacity

Appendix E: Modifications Suggested for Interchanges

Modifications suggested for Interchange 1 after 20 years

Tables 55 and 56 present the traffic operation and safety results before and after implementing the suggested improvements, respectively.

Table 55. Traffic operation results at Interchange 1 before and after implementing the suggested modifications after 20 years

Approach	Level of Service (LOS)	
	Before Improvement	After Improvement
EB	C	B
WB	F	B
NB	E	B
SB	A	B

Table 56. Traffic safety results at interchange 1 before and after implementing the suggested modifications after 20 years

Type of Conflicts	Conflicts Count	
	Before Improvement	After Improvement
Crossing	2525	3369
Rear-end	8464	4265
Lane change	807	702
Total conflicts	11796	8336

Modifications suggested for Interchange 2 after 10 years

Tables 57 and 58 show the traffic operation and safety results before and after implementing the suggested improvements at Interchange 2 after 10 years.

Table 57. Traffic operation results at Interchange 2
before and after implementing the suggested modifications after 10 years

Approach	Level of Service (LOS)	
	Before Improvement	After Improvement
EB	E	B
WB	D	A
NB	E	C
SB	F	B

Table 58. Safety results at Interchange 2
before and after implementing the suggested modifications after 10 years

	Conflicts Count	
Type of Conflicts	Before Improvement	After Improvement
Crossing conflicts	11244	3053
Rear-end conflicts	8716	1977
Lane change conflicts	12812	4011
Total conflicts	32772	9041

Modifications suggested for Interchange 2 after 20 years

Tables 59 and 60 present the traffic operation and safety results before and after implementing the suggested improvements at Interchange 2 after 20 years.

Table 59. Traffic operation results at Interchange 2
before and after implementing the suggested modifications after 20 years

Approach	Level of Service (LOS)	
	Before Improvement	After Improvement
EB	F	A
WB	F	B
NB	F	B
SB	F	B

Table 60. Traffic safety results at Interchange 2 before and after implementing the suggested modifications after 20 years

Type of Conflicts	Conflicts Count	
	Before Improvement	After Improvement
Crossing conflicts	12060	2922
Rear-end conflicts	13742	1844
Lane change conflicts	14165	2298
Total conflicts	39967	7064

Modifications suggested for Interchange 3 after 10 years

Table 61 presents the traffic operation results of Interchange 3 before and after implementing the suggested improvements after 10 years.

Table 61. Traffic operation results at Interchange 3 before and after implementing the suggested modifications after 10 years

Location	Before Improvement	After Improvement
EB	C	B
WB	D	B
NB	E	C
SB	B	A

Modifications suggested for Interchange 3 after 20 years

Table 62 presents the traffic operation results before and after implementing the suggested improvements after 20 years.

Table 62. Traffic operation results at Interchange 3 before and after implementing the suggested modifications after 20 years

Approach	Level of Service (LOS)	
	Before Improvement	After Improvement
EB	F	B

Approach	Level of Service (LOS)	
	Before Improvement	After Improvement
WB	D	C
NB	F	C
SB	D	A

Modifications suggested for Interchange 5 after 20 years

Table 63 shows the traffic operation results before and after implementing the suggested improvements after 20 years.

Table 63. Traffic operation results at Interchange 5 before and after implementing the suggested modifications after 20 years

Approach	Level of Service (LOS)	
	Before Improvement	After Improvement
EB	B	B
WB	B	B
NB	A	A
SB	F	C

Modifications suggested for Interchange 6 after 20 years

Table 64 shows the traffic operation results before and after implementing the suggested improvements at Interchange 6 after 20 years.

Table 64. Traffic operation results at Interchange 6 before and after implementing the suggested modifications after 20 years

Approach	Level of Service (LOS)	
	Before Improvement	After Improvement
EB	C	C
WB	E	C
NB	A	C
SB	F	C

Modifications suggested for Interchange 7 for current conditions

Tables 65 and 66 show the traffic operation and safety results before and after implementing the suggested improvements for current conditions.

Table 65. Traffic operation results at Interchange 7 before and after implementing the suggested modifications for current conditions

Approach	Level of Service (LOS)	
	Before Improvement	After Improvement
EB	C	C
WB	C	C
NB	D	B
SB	E	B

Table 66. Traffic safety results at Interchange 7 before and after implementing the suggested modifications for current conditions

Type of Conflicts	Conflicts Count	
	Before Improvement	After Improvement
Crossing	2328	1975
Rear-end	14466	2188
Lane change	2545	1638
Total conflicts	19339	5801

Modifications suggested for Interchange 7 after 10 years

Tables 67 and 68 show the traffic operation and safety results before and after implementing the suggested improvements at Interchange 7 after 10 years.

Table 67. Traffic operation results at Interchange 7 before and after implementing the suggested modifications after 10 years

Approach	Level of Service (LOS)	
	Before Improvement	After Improvement
EB	E	C

Approach	Level of Service (LOS)	
	Before Improvement	**After Improvement**
WB	F	D
NB	F	C
SB	E	C

Table 68. Traffic safety results at Interchange 7
before and after implementing the suggested modifications after 10 years

Type of Conflicts	Conflicts Count	
	Before Improvement	**After Improvement**
Crossing	3935	3630
Rear-end	31062	3735
Lane change	4450	2799
Total Conflicts	39447	10164

Modification suggested for Interchange 7 after 20 years

Tables 69 and 70 show the traffic operation and safety results before and after implementing the suggested improvements at Interchange 7 after 20 years.

Table 69. Traffic operation results at Interchange 7
before and after implementing the suggested modifications after 20 years

Approach	Level of Service (LOS)	
	Before Improvement	**After Improvement**
EB	F	D
WB	F	D
NB	F	C
SB	F	C

Table 70. Traffic safety results at Interchange 7
before and after implementing the suggested modifications after 20 years

	Conflicts Count	
Type of Conflicts	**Before Improvement**	**After Improvement**
Crossing	4290	5214
Rear-end	35763	5164
Lane change	4481	4027
Total Conflicts	44534	14405

Appendix F: Suggested countermeasures for all eight interchanges for current conditions, after 10 years, and after 20 years

Table 71 shows the suggested countermeasures for current conditions, after 10 years, and after 20 years using microsimulation analysis.

Table 71. Suggested countermeasures of all eight interchanges

Interchange		Modifications required for current conditions	Modifications required after 10 years	Modifications required after 20 years
1	Cloverleaf Interchange	Not required	Not required	i. Addition of extra lane in offramp from Westbound and on ramp merging to Eastbound. ii. Addition of lane in Northbound direction except in between onramp and offramp.
2	Cloverleaf Interchange	Not required	i. Addition of extra lane in EB and WB section only	i. Addition of one extra lane in EB and WB section only. ii. Converting the loop ramp connecting EB to NB into direct ramp.
3	Cloverleaf Interchange	Not required	i. Converting the loop ramp connecting EB to NB into direct ramp.	i. Converting the loop ramp connecting EB to NB into a semi-direct ramp. ii. Addition of one extra lane in EB and WB section.
4	Cloverleaf Interchange	Not required	Not required	Not required
5	Diamond interchange	Not required	Not required	i. Converting to signalized intersections
6	Diamond interchange	Not required	Not required	i. Converting Roundabouts to signalized intersections. ii. Addition of one lane in EB and WB. iii. Addition of frontage roads along EB and WB.

Interchange	Modifications required for current conditions	Modifications required after 10 years	Modifications required after 20 years
7	i. Addition of frontage road in North of the interchange. ii. Addition of lane in NB and SB section	i. Addition of one-way frontage road in NB and SB roadways. ii. Addition of lane in NB and SB roads. iii. Addition of lane in EB road after intersection 2	i. Addition of one-way frontage road in NB and SB roadways. ii. Addition of lane in NB and SB roads. iii. Addition of lane in EB and WB
8	Not required	Not required	Not required

Appendix G: Mathematical Expression of Kernel Density Estimation and Getis G_{i^*} Ord Statistics

Kernel Density Estimation:

When dealing with observations from an unknown probability density function, the kernel estimator can be described as follows:

$$f(x) = \frac{1}{nh} \sum_{i=1}^{n} \mathrm{K}\left(\frac{x - xi}{h}\right)$$

Where,

h = the smoothing parameter or bandwidth,

K= the kernel function (assigns weights to observations, depending on their distance from the point x where the density estimate is being calculated),

$\hat{f}$ = the estimator for the probability density function "f", and

n = number of events.

Consequently, the kernel estimator's effectiveness is influenced by both the bandwidth (h) and the kernel function (K). The selection of the smoothing parameter "h" is vital and should be tailored to the specific objectives of the estimation, given that for any chosen kernel "K", the performance of the kernel estimator is significantly impacted by the choice of "h" [53].

A basic expression of the G_{i*} statistic is provided by Songchitruksa [54]:

$$G_i^*(d) = \frac{\sum_{j=1}^{n} w_{ij}(\mathrm{d}) x_j}{\sum_{j=1}^{n} x_j}$$

Where,

G_{i*} = the spatial autocorrelation (SA) statistic for a specific event i among n events,

x_j = the value of variable x at event j across all n events, essentially measuring the Crash Severity Index (CSI) at a given location,

w_{ij} = weight value between event i and j that represents their spatial interrelationship,

and n = number of subdivisions of the regions.

The distribution of the G_{i*} statistic follows a normal pattern when the variable x's underlying distribution is also normal. For this analysis, the threshold distance, which determines how close one crash must be to another to be considered neighboring, was set at zero. This means that all features were treated as neighbors to every other feature across the study area. The standardized G_{i*} effectively acts as a Z-score, which can be linked to statistical significance in the following manner:

$$\mathrm{Z}(G_i^*) = \frac{\sum_{j=1}^{n} w_{ij}\, x_j - \bar{\mathrm{x}} \sum_{j=1}^{n} w_{ij}^2}{s\sqrt{\frac{n\sum_{j=1}^{n} w_{ij}^2 - (\sum_{j=1}^{n} w_{ij})^2}{n-1}}}$$

High absolute values of the G_{i*} statistic, whether positive or negative, indicate clusters of crashes with high-value and low-value events, respectively. Conversely, a G_{i*} statistic near zero suggests that the distribution of events is random [53].

Appendix H: Primary contributing factors of overall crashes and hotspots for all eight interchanges

Table 72 presents the primary contributing factors to all crashes and hotspots at Interchange 2, based on crash data analysis. Like Interchange 1, the results indicate that violations are the most significant contributors, followed by movement prior to the crash. Crashes due to road surface and roadway condition are less frequent.

Table 72. Primary contributing factors for overall crashes and hotspots at Interchange 2

Primary contributing factors of crashes	All crashes	Crashes at Hotspots
Violations	76%	83%
Movement prior to crash	18%	14%
Road surface	2%	1%
Roadway condition	1%	0%

Table 73 shows the primary contributing factors for overall crashes and hotspots at Interchange 3, based on crash data analysis. Violations are the primary cause of crashes at Interchange 3. Crashes due to movement prior to the crash, road surface, and roadway condition are less common.

Table 73. Primary contributing factors for overall crashes and hotspots at Interchange 3

Primary contributing factors of crashes	All crashes	Crashes at Hotspots
Violations	93%	95%
Movement prior to crash	1%	1%
Road surface	0%	0.5%
Roadway condition	1%	0.5%

Table 74 shows the primary contributing factors to overall crashes and hotspots at Interchange 4, based on crash data analysis. Like Interchanges 1, 2, and 3, the results indicate that violations are the most significant contributors, followed by movement prior to the crash. Crashes due to road surface and roadway condition are less frequent.

Table 74. Primary contributing factors for overall crashes and hotspots at Interchange 4

Primary contributing factors of crashes	All crashes	Crashes at Hotspots
Violations	76%	62%
Movement prior to crash	17%	31%
Road surface	0%	0%
Roadway condition	1%	0%

Table 75 shows the primary contributing factors to overall crashes and hotspots at Interchange 5, based on crash data analysis. Violations were the primary cause of crashes at Interchange 5. Crashes due to movement prior to the crash, road surface, and roadway condition were much less common.

Table 75. Primary contributing factors for overall crashes and hotspots at Interchange 5

Primary contributing factors of crashes	All crashes	Crashes at Hotspots
Violations	92%	100%
Movement prior to crash	0%	0%
Road surface	0%	0%
Roadway condition	2%	0%

Table 76 shows the primary contributing factors to overall crashes and hotspots at Interchange 6, based on crash data analysis. The results indicate that violations were the most significant contributing factor, followed by movement prior to the crash. Crashes due to road surface and roadway condition are less frequent.

Table 76. Primary contributing factors for overall crashes and hotspots at Interchange 6

Primary contributing factors of crashes	All crashes	Crashes at Hotspots
Violations	86%	87%
Movement prior to crash	11%	13%
Road surface	1%	0%
Roadway condition	0%	0%

Table 77 shows the primary contributing factors to overall crashes and hotspots at Interchange 7, based on an analysis of crash data. Violations are the primary cause of crashes at Interchange 7. Crashes due to movement prior to the crash, road surface, and roadway condition are less common.

Table 77. Primary contributing factors for overall crashes and hotspots at Interchange 7

Primary contributing factor	Overall crashes	Hotspots
Violations	93%	95%
Movement prior to crash	3%	3%
Road surface	0.2%	0%
Roadway condition	0%	0%

Table 78 shows the primary contributing factors to overall crashes and hotspots at Interchange 8, based on crash data analysis. The results indicate that violations were the most significant contributing factor, followed by movement prior to the crash. Crashes due to road surface and roadway condition were less frequent.

Table 78. Primary contributing factors for overall crashes and hotspots at Interchange 8

Primary contributing factor	Overall crashes	Hotspots
Violations	53%	53%
Movement prior to crash	21%	24%
Road surface	1.7%	0.7%
Roadway condition	4%	3%

Appendix I: Negative Binomial Regression and Safety Performance Function

Negative Binomial Regression:

The equation of negative binomial regression can be written as:

$$\lambda_i = f(\beta X_i) \times \exp(\varepsilon_i) \quad \text{(Equation 4.3 of [48])}$$

Where,

ε_i = gamma-distributed disturbance term.

If a log- linear model is assumed, then

$$\lambda_i = \exp(\beta X_i * \exp(\varepsilon_i) = \exp(\ \beta X_i + \varepsilon_i) \quad \text{(Equation 4.4 of [48])}$$

By adding the disturbance term, the variance has increased beyond the mean, and it can be demonstrated that the variance is now:

$$\text{VAR}(y_i) = E(y_i) + k * [E(y_i)]^2 \quad \text{(Equation 4.5 of [48])}$$

Cameron and Trivedi (1998) have referred to this version of the negative binomial regression model as the NB2 model. In the equation above, k represents the over-dispersion parameter. Some research, such as that by Hauer et al. (2002), opts to work with the inverse of the over-dispersion parameter instead of the parameter itself. Letting represents the inverse of the over-dispersion parameter means that $\Phi = 1/k$. Under this approach, equation 4.5 is rewritten as:

$$VAR(y_i) = E(y_i) + \frac{[E(y_i)]^2}{\Phi} \quad \text{(Equation 4.6 of [48])}$$

Safety Performance Functions (SPFs):

Using the properties of negative binomial regression, the equation to develop SPFs, which is given by Highway Safety Manual (HSM), is:

The equation to develop SPFs for the roadway segment (minor road) is:

$$N_{spfrd} = e^{a+b*\ln(AADT)+\ln(L)}$$
(Equation 11-9 in [49])

Where,
N_{spfrd} = base total number of roadway segment crashes per year,
AADT = annual average daily traffic (vehicles/day) on roadway segment,
L = length of roadway segment, and
a, b = regression coefficients.

The equation used to develop SPFs for the freeway segment (major road) is:

$$N_{spf,rps,x,my,z} = L^{*} * \exp(a + b * \ln[c * AADT_{fs})$$
(Equation 18-15 of [55])

with

$$L^{*} = Lfs - (0.5 * \sum_{i=1}^{2} \mathrm{Len,seg,i}) - (0.5 * \sum_{i=1}^{2} \mathrm{Lex,seg,i})$$
(Equation 18-16 of [55])

$N_{spf,rps,x,my,z}$ = Predicted average multiple-vehicle crash frequency of a freeway segment with base conditions, n lanes, and severity z (z = fi: fatal and injury, pdo: property damage only) (crashes/yr),
L* = effective length of freeway segments (mi),
L_{fs} = length of freeway segment (mi),
$L_{en,seg,i}$ = length of ramp entrance i adjacent to subject freeway segment (mi),
$L_{ex,seg,I}$ = length of ramp exit i adjacent to subject freeway segment (mi),
a, b = regression coefficients,
c = AADT scale coefficient, and
$AADT_{fs}$ = AADT volume of freeway segment (veh/day).

The equation used to develop SPFs for the ramp segment is:

$$N_{spf,rps,x,my,z} = L_r * \exp(a + b * \ln[c * AADT_r] + d[c * AADT_r])$$
(Equation 19-20 of [55])

Where,

$N_{spf,rps,x,mv,}$ = predicted average multiple-vehicle crash frequency of a ramp segments with base conditions, cross section x (x=nEN: n-lane entrance ramp, nEX: n-lane exit ramp), and severity z (z=fi:fatal and injury, pdo: property damage only) (crashes/yr),

L_r = length of ramp segments (mi),

$AADT_r$ = AADT volume of ramp segments (veh/day),

a,b,d = regression coefficients, and

c = AADT scale coefficient.

The equation used to develop SPFs for intersections is:

$$N_{spfint} = \exp[a + b * \ln(AADT_{maj}) + c * \ln(AADT_{min})]$$

(Equation 11-11 of [49])

Where,

N_{spfint} = SPF estimate of intersection-related expected average crash frequency for base conditions,

$AADT_{maj}$ = AADT (vehicles per day) for major-road approaches,

$AADT_{min}$ = AADT (vehicles per day) for minor-road approaches, and

a, b, c = regression coefficients.

Appendix J: Predicted vs observed crashes at cloverleaf interchanges and diamond interchanges

Cloverleaf interchanges

Table 79 compares the predicted and observed crashes at four cloverleaf interchanges across major roads, minor roads, and ramps. Interchange 1 shows an overestimation, with predicted crashes 19% higher than observed. In contrast, Interchanges 2 and 3 exhibit underestimations, with predictions falling short by 10% and 7%, respectively. For Interchange 4, the discrepancy between predicted and observed crashes is a mere 0.4%. These results indicate variability in the accuracy of the traffic prediction model at different sites, demonstrating that the SPF model can effectively predict crashes at cloverleaf interchanges.

Table 79. Predicted and observed crashes at the four cloverleaf interchanges

	Major Road	Minor Road	Ramps	Predicted Total	Observed Total	Difference (%)
Interchange 1	91	30	22	143	120	19%
Interchange 2	30	43	19	92	102	-10%
Interchange 3	36	40	17	93	100	-7%
Interchange 4	10	25	17	54	52	0.4%

Diamond interchanges

Table 80 compares the predicted and observed crashes at four diamond interchanges, revealing discrepancies in accuracy across cases. The Interchange 5 prediction overestimated the crash rate by 47%, with actual crashes significantly fewer than expected. Interchange 6 also experienced an overestimation, albeit to a lesser extent, with 10% lower observed crashes than predicted crashes. Conversely, Interchanges 7's and 8's observed crashes surpassed predicted crashes by 9% and 21%, respectively, indicating underestimation in the forecasts. The SPF model gave satisfactory results at three of the interchanges, although it did not provide accurate crash predictions at one interchange. This showed that the obtained SPF model can still be used to predict crashes at other diamond interchanges.

Table 80. Predicted and observed crashes at the four diamond interchanges.

	Major Road	Ramps	Predicted Total	Observed Total	Difference (%)
Interchange 5	15	2	25	17	-47%
Interchange 6	28	21	53	48	-10%
Interchange 7	52	41	85	93	9%
Interchange 8	21	7	22	28	21%

Appendix K: Severity level and manner of collision at all eight interchanges

Interchange 1 (Cloverleaf with C-D roads)

Analysis of Severity

Table 81 presents the total count and percentage of crashes categorized by severity level. The data reveals that most crashes, 79.42%, result in no injuries (Severity E). Conversely, only a small fraction of crashes resulted in fatalities (Severity A) or incapacitating/severe injuries (Severity B), with percentages of 0.28% and 1.39%, respectively. Non-incapacitating/moderate injuries (Severity C) account for 4.45% of the crashes, while possible injuries or complaints (Severity D) represent 14.46%.

Table 81. Severity levels at Interchange 1

Severity	Coding	Count	Percentage
Fatal	A	2	0.28
Incapacitating/severe	B	10	1.39
Non-incapacitating/moderate	C	32	4.45
Possible/complaint	D	104	14.46
No injury	E	571	79.42
Total		719	100

Figure 60 shows the distribution of severity data at Interchange 1, including all severity levels. Figure 61 illustrates the distribution of injury crashes at Interchange 1, excluding non-injury crashes.

Figure 60. Distribution of all severity levels at Interchange 1

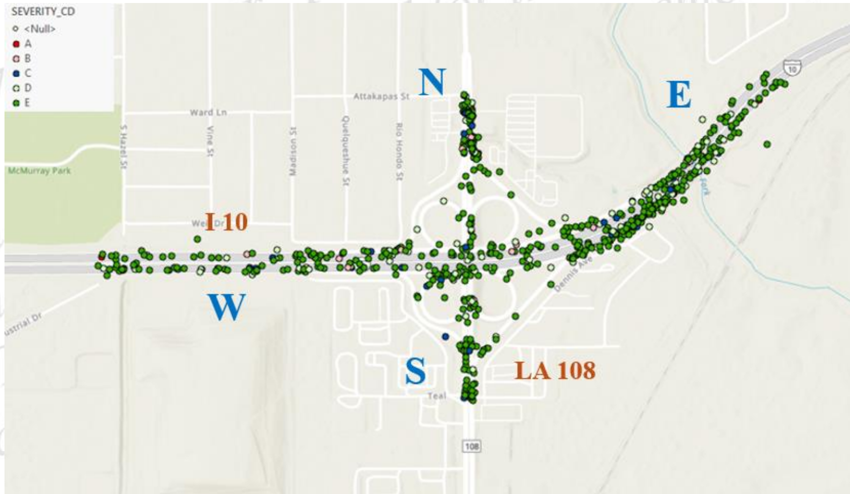


Figure 61. Distribution of severity levels excluding non-injury crashes at Interchange 1

Note: The legend for the type of severity is provided at the top left of the figure.

Analysis of Manner of Collision

Table 82 shows the distribution of traffic collisions at Interchange 1 according to the manner of collision. As shown in the table, the majority of collisions are rear-end crashes (41.31%), followed by sideswipe-same direction (26.84%) and non-collision with motor vehicles (15.99%). Other types, such as head-on, right angle, and various left and right turn-related incidents, each represent less than 2% of crashes at Interchange 1.

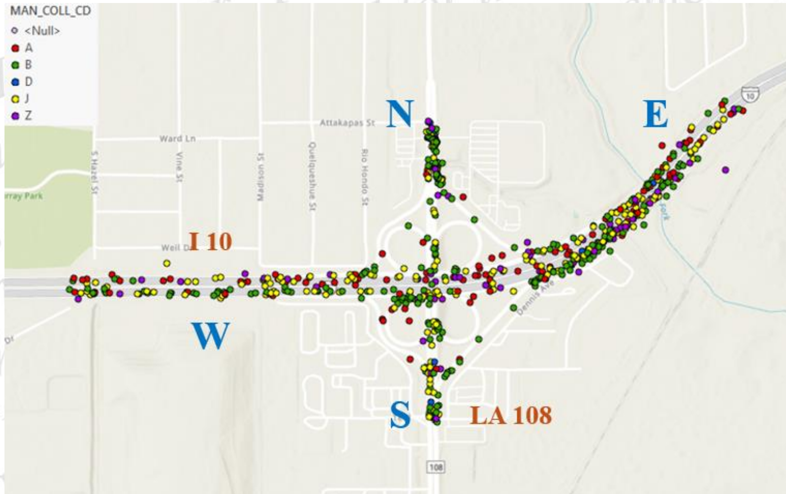
Table 82. Manner of collision at Interchange 1

Manner of collision	Coding	Count	Percentage
Non-collision with motor vehicle	A	115	15.99
Rear-end	B	297	41.31
Head-on	C	3	0.42
Right angle	D	15	2.09
Left turn - angle	E	2	0.28
Left turn - opposite direction	F	12	1.67
Left turn - same direction	G	5	0.70
Right turn - same direction	H	11	1.53
Right turn - opposite direction	I	0	0.00
Sideswipe - same direction	J	193	26.84
Sideswipe - opposite direction	K	2	0.28
Other	Z	64	8.90
Total		719	100

Figure 62 illustrates the distribution of various collision types at Interchange 1, focusing on the five most frequent collision manners. The concentration of these collisions is particularly high at specific hotspots within the interchange.

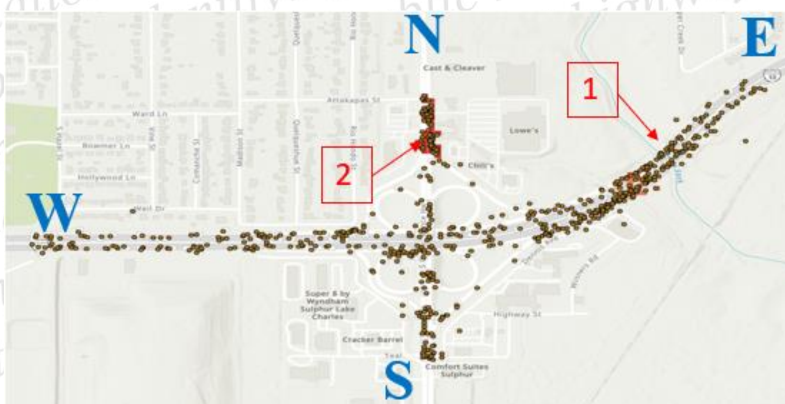
Figure 63 highlights the hotspot locations for collisions at Interchange 1. Points 1 and 2 mark the hotspot areas. A closer analysis of the types of collisions within these hotspots reveals that rear-end collisions are the most common. This is followed by a significant number of sideswipe-same direction collisions.

Figure 62. Distribution of manner of collision (top five highest by count) at Interchange 1



Note: The legend for the manner of collision is provided at the top left of the figure.

Figure 63. Distribution of crashes and hotspots at Interchange 1



Interchange 2 (Cloverleaf without C-D roads)

Analysis of Severity

Table 83 presents the total count and percentage of crashes categorized by severity level at Interchange 2. The data reveals that most crashes, 71.29%, are property damage only (Severity E). Conversely, only a small fraction of crashes resulted in fatalities (Severity A) or incapacitating/severe injuries (Severity B), with percentages of 0.49% and 0.98%, respectively. Non-incapacitating/moderate injuries (Severity C) account for 5.55% of the crashes, while possible injuries or complaints (Severity D) represent 21.70%.

Table 83. Severity levels at Interchange 2

Severity	Coding	Count	Percentage
Fatal	A	3	0.49
Incapacitating/severe	B	6	0.98
Non-incapacitating/moderate	C	34	5.55
Possible/complaint	D	133	21.70
No injury	E	437	71.29
Total		613	100

Figure 64 shows the distribution of crashes according to severity level at Interchange 2. In addition, Figure 65 illustrates the distribution of injury-related crashes at Interchange 2, excluding non-injury crashes.

Figure 64. Distribution of all severity levels at Interchange 2

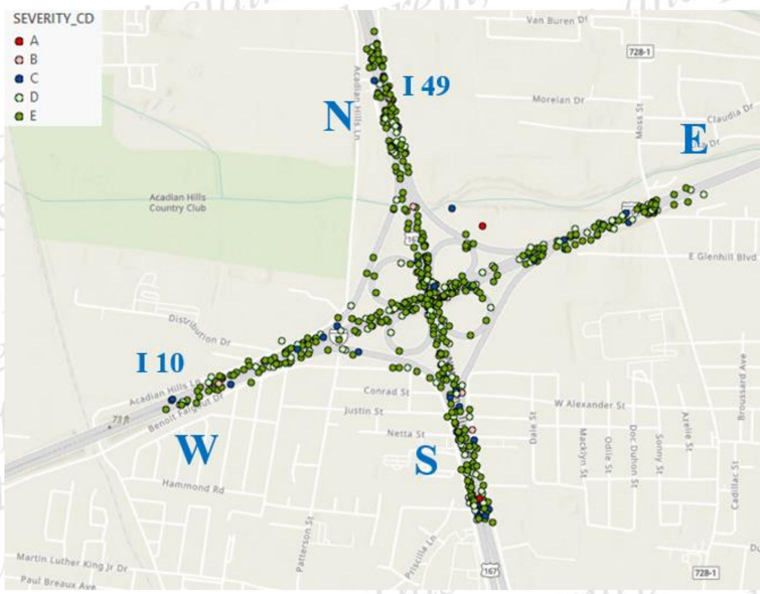
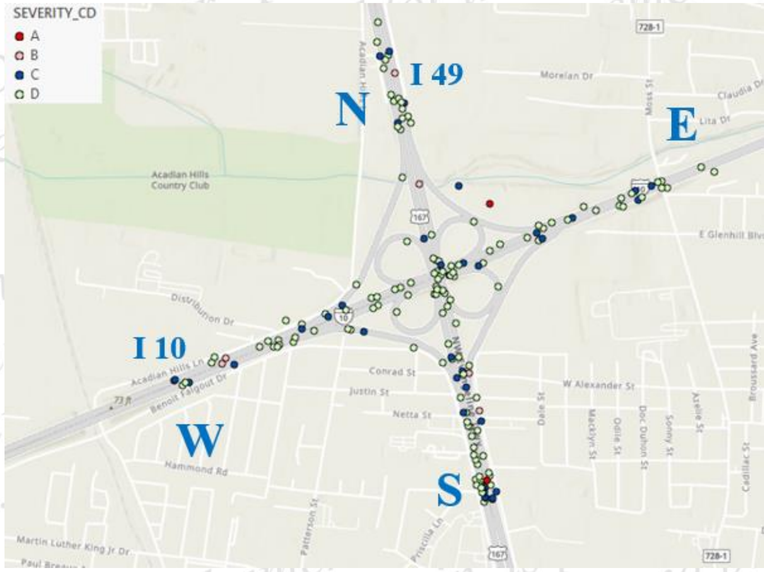


Figure 65. Distribution of severity levels excluding non-injury crashes at Interchange 2



Analysis of Manner of Collision

Table 84 shows the distribution of traffic collisions at Interchange 2 according to the manner of collision. The majority of collisions are rear-end (43.23%), followed by sideswipe-same direction (31.65%) and non-collision with motor vehicles (15.33%). Other types, such as head-on, right angle, and various left and right turn-related incidents, each represent less than 2% of the total.

Table 84. Manner of collision at Interchange 2

Manner of collision	Coding	Count	Percentage
Non-collision with motor vehicle	A	94	15.33
Rear-end	B	265	43.23
Head-on	C	1	0.16
Right angle	D	15	2.45
Left turn - angle	E	6	0.98
Left turn - opposite direction	F	1	0.16
Left turn - same direction	G	1	0.16
Right turn - same direction	H	4	0.65
Right turn - opposite direction	I	0	0.00
Sideswipe - same direction	J	194	31.65
Sideswipe - opposite direction	K	2	0.33
Other	Z	30	4.89
Total		613	100

Figure 66 illustrates the distribution of various collision types at Interchange 2, focusing on the five most frequent collision manners. The concentration of these collisions is particularly high at specific hotspots within the interchange.

Figure 67 highlights the hotspot locations for collisions at Interchange 2. Red boxes mark the hotspots areas. A closer analysis of the types of collisions within these hotspots reveals that rear-end collisions are the most common. This is followed by a significant number of sideswipe-same direction and non-collision with motor vehicle collisions.

Figure 66. Distribution of manner of collision (top five highest by count) at Interchange 2

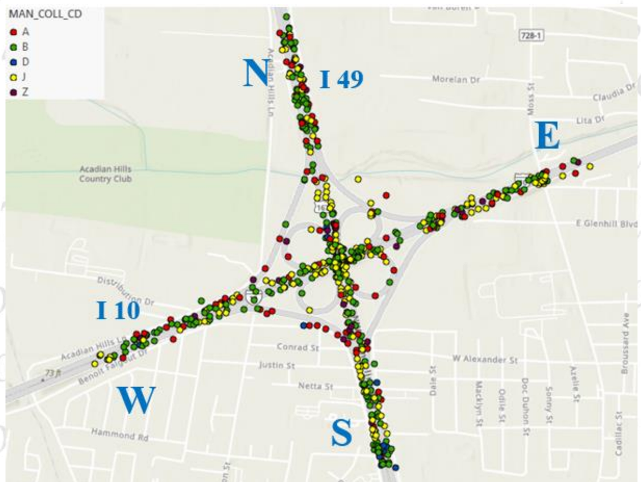
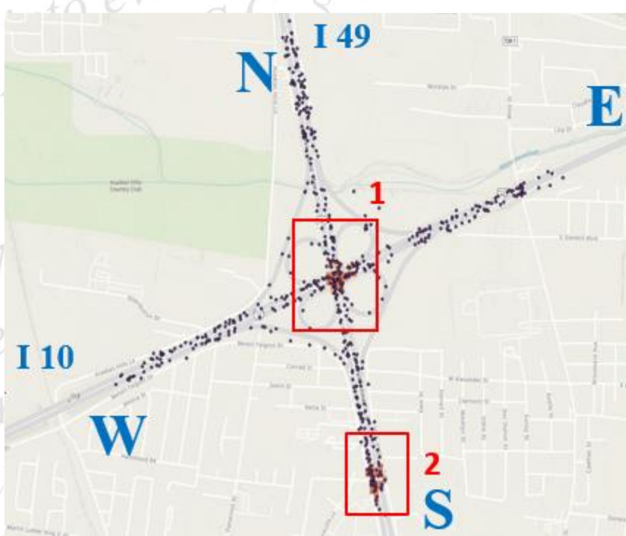


Figure 67. Distribution of crashes and hotspots at Interchange 2



Interchange 3 (Cloverleaf without C-D roads)

<u>Analysis of Severity</u>

Table 85 presents the distribution of crashes at Interchange 3 categorized by severity level. The data reveals that most crashes, 84.45%, resulted in no injuries (Severity E). Conversely, only a small fraction of crashes resulted in fatalities (Severity A) or incapacitating/severe injuries (Severity B), with percentages of 0.33% each. Non-incapacitating/moderate injuries (Severity C) account for 3.01% of the crashes, while possible injuries or complaints (Severity D) represent 11.87%.

Table 85. Severity levels at Interchange 3

Severity	Coding	Count	Percentage
Fatal	A	2	0.33
Incapacitating/severe	B	2	0.33
Non-incapacitating/moderate	C	18	3.01
Possible/complaint	D	71	11.87
No injury	E	505	84.45
Total		598	100

Figure 68 shows the distribution of crashes at Interchange 3 considering all severity levels, while Figure 69 illustrates the distribution of injury crashes at Interchange 3, excluding non-injury crashes.

Figure 68. Distribution of all severity levels at Interchange 3

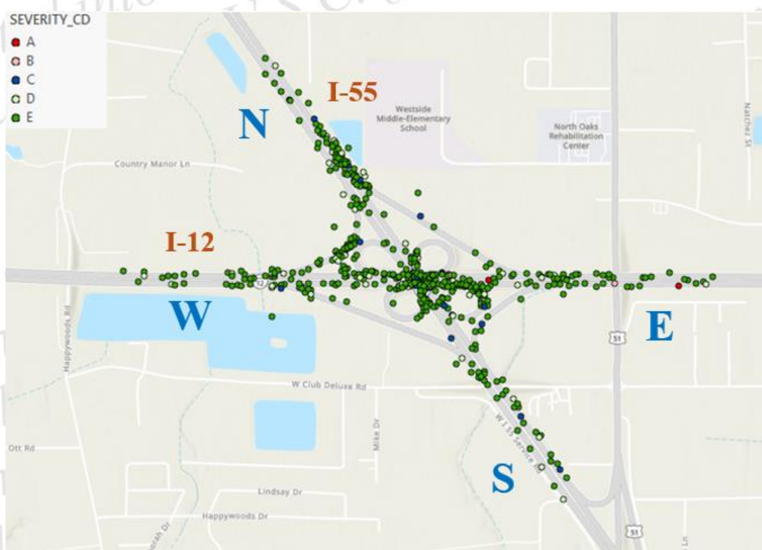
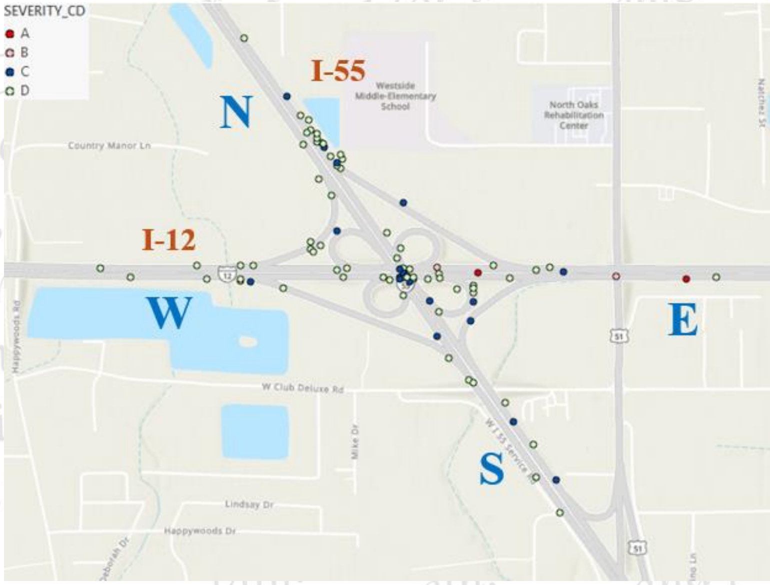


Figure 69. Distribution of severity levels excluding non-injury crashes at Interchange 3



<u>Analysis of Manner of Collision</u>

Table 86 shows the manner of collisions at Interchange 3. The majority of crashes are non-collisions with motor vehicles, accounting for 49.67%. Rear-end collisions represent the next most common type at 26.25%, followed by sideswipe collisions in the same direction at 21.24%. Other types of collisions, such as head-on, right angle, and right turn in the same direction, each represent 1% or less of the total, with left turn and opposite direction collisions being notably absent at 0%.

Table 86. Manner of collision at Interchange 3

Manner of collision	Coding	count	Percentage
Non-collision with motor vehicle	A	297	49.67
Rear-end	B	157	26.25
Head-on	C	1	0.17
Right angle	D	6	1.00
Left turn - angle	E	0	0.00
Left turn - opposite direction	F	0	0.00
Left turn - same direction	G	0	0.00
Right turn - same direction	H	6	1.00
Right turn - opposite direction	I	0	0.00

Manner of collision	Coding	count	Percentage
Sideswipe - same direction	J	127	21.24
Sideswipe - opposite direction	K	0	0.00
Other	Z	4	0.67
Total		598	100

Figure 70 illustrates the traffic collisions at Interchange 3 according to the manner of collision, focusing on the five most frequent collision manners. Figure 71 highlights the hotspot locations for collisions at Interchange 3. Red boxes mark the hotspot areas. A closer analysis of the types of collisions within these hotspots reveals that non-collisions with motor vehicles are the most common. This is followed by a significant number of rear-end and sideswipe-same direction collisions.

Figure 70. Distribution of manner of collision (top five highest by count) at Interchange 3

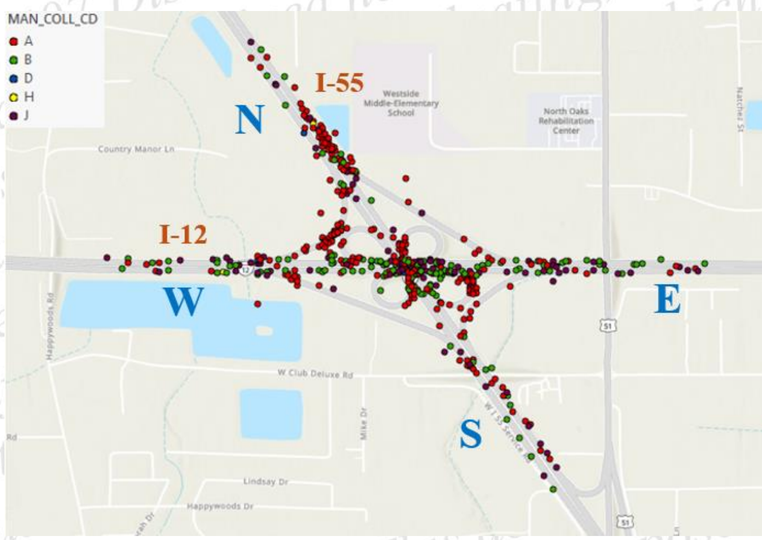
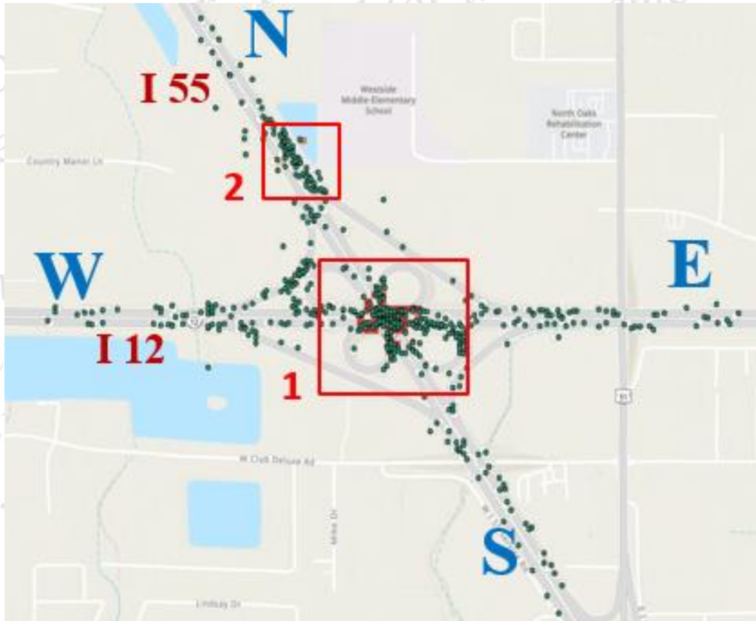


Figure 71. Distribution of crashes and hotspots at Interchange 3



Interchange 4 (Cloverleaf with C-D roads)

<u>Analysis of Severity</u>

Table 87 presents the distribution of crashes at Interchange 4 according to severity level. The results reveal that most crashes, 79.55%, resulted in no injuries (Severity E). Additionally, fatal crashes (Severity A) are rare, accounting for only 0.64%, and there are no incapacitating/severe injuries (Severity B) reported. Non-incapacitating/moderate injuries (Severity C) make up only 5.11% of the crashes, while possible injuries or complaints (Severity D) constitute 14.70% of the total.

Table 87. Severity levels at Interchange 4

Severity	Coding	Count	Percentage
Fatal	A	2	0.64
Incapacitating/severe	B	0	0.00
Non-incapacitating/moderate	C	16	5.11
Possible/complaint	D	46	14.70
No injury	E	249	79.55
Total		313	100

Figure 72 shows the distribution of crashes at Interchange 4, according to all severity levels. Figure 73 illustrates the distribution of injury related crashes at Interchange 4, excluding non-injury incidents.

Figure 72. Distribution of all severity levels at Interchange 4

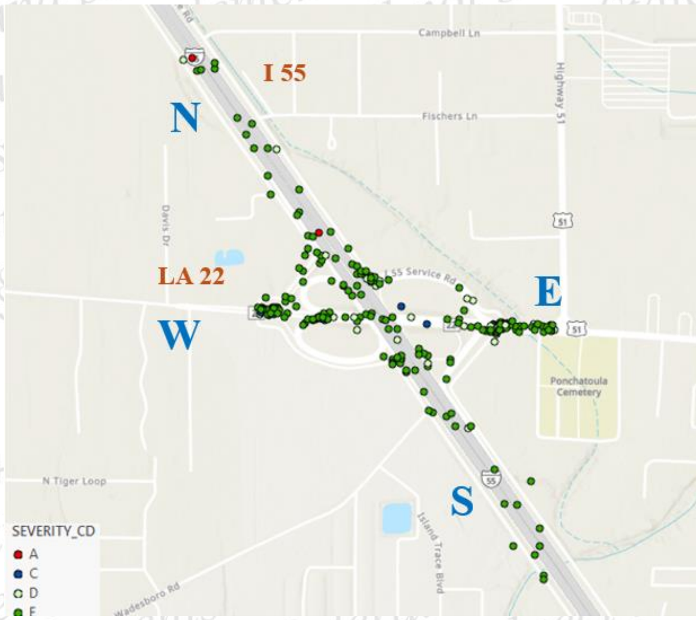
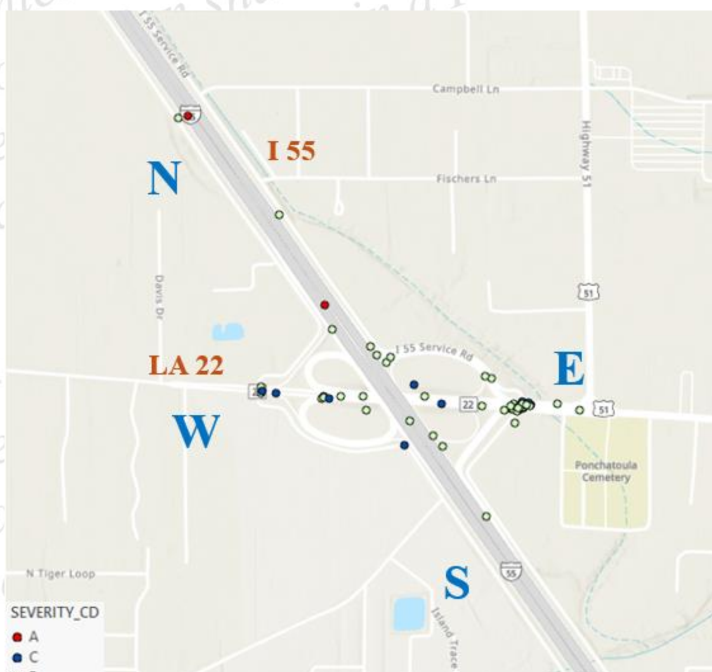


Figure 73. Distribution of severity levels excluding non-injury crashes at Interchange 4



Analysis of Collision Manner

Table 88 shows the manner of collisions at Interchange 4. The majority of collisions are rear-end (49.84%), followed by non-collision with motor vehicles (15.34%) and sideswipe-same direction (13.42%). Other types, such as head-on (0.96%), right angle (5.43%), and various left and right turn-related incidents, each represent less than 5% of the total.

Table 88. Manner of collision at Interchange 4

Manner of collision	Coding	Count	Percentage
Non-collision with motor vehicle	A	48	15.34
Rear- end	B	156	49.84
Head-on	C	3	0.96
Right angle	D	17	5.43
Left turn - angle	E	4	1.28
Left turn - opposite direction	F	7	2.24
Left turn - same direction	G	9	2.88
Right turn - same direction	H	9	2.88
Right turn - opposite direction	I	1	0.32
Sideswipe - same direction	J	42	13.42
Sideswipe - opposite direction	K	2	0.64
Other	Z	15	4.79
Total		313	100

Figure 74 illustrates the distribution of various collision types at Interchange 4, focusing on the five most frequent collision manners. In addition, Figure 75 highlights the hotspot locations for collisions at Interchange 4. Red boxes mark the hotspot areas. A closer analysis of the types of collisions within these hotspots reveals that rear-end collisions are the most common. This is followed by a significant number of non-collisions with motor vehicles and sideswipe-same direction collisions.

Figure 74. Distribution of manner of collision (top five highest by count) at Interchange 4

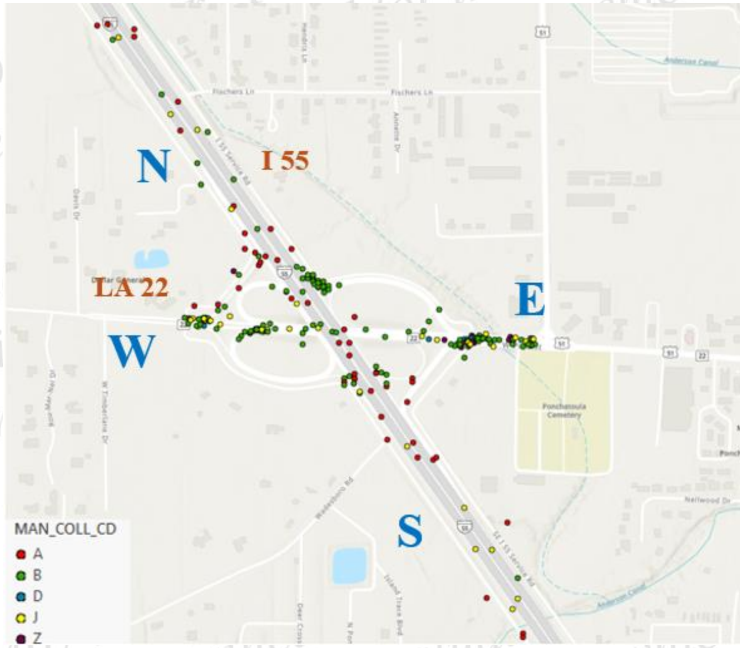
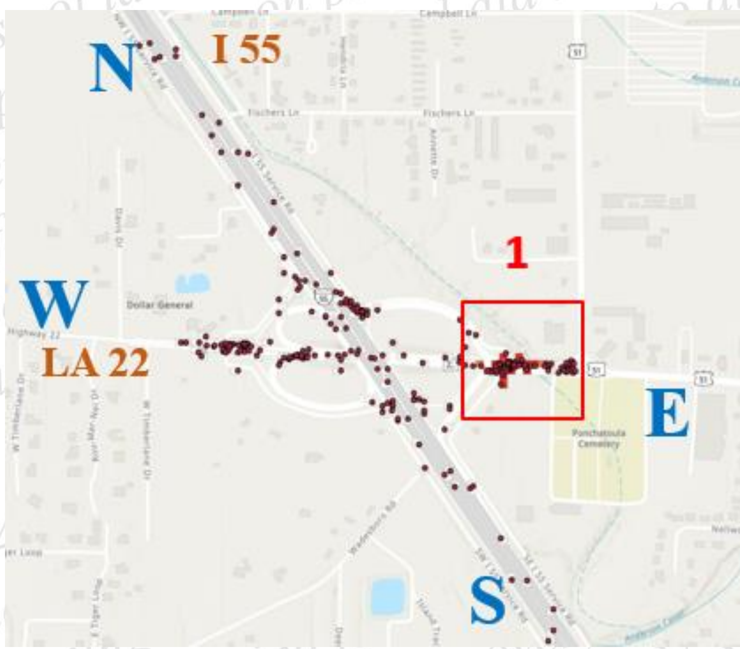


Figure 75. Distribution of crashes and hotspots at Interchange 4



Interchange 5 (Diamond with signalized intersections)

Analysis of Severity

Table 89 presents the distribution of crashes according to severity level at Interchange 5. The results indicate that the majority of crashes, 76.74%, resulted in no injuries (Severity E). On the other hand, fatal crashes (Severity A) are rare, accounting for only 0.78%, and there are no incapacitating/severe injuries (Severity B). Non-incapacitating/moderate injuries (Severity C) make up 3.10% of the crashes, while possible injuries or complaints (Severity D) constitute 19.38% of the total.

Table 89. Severity levels at Interchange 5

Severity	Coding	Count	percentage
Fatal	A	1	0.78
Incapacitating/severe	B	0	0.00
Non-incapacitating/moderate	C	4	3.10
Possible/complaint	D	25	19.38
No injury	E	99	76.74
Total		129	100

Figure 76 shows the distribution of crashes at Interchange 5, according to all severity levels, while Figure 77 illustrates the distribution of injury crashes at Interchange 5, excluding non-injury incidents.

Figure 76. Distribution of all severity levels at Interchange 5

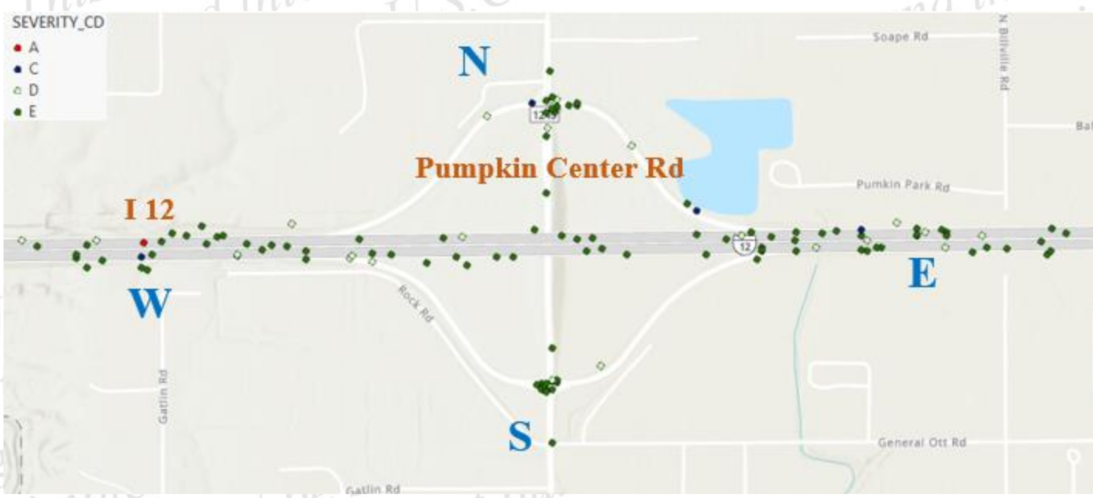
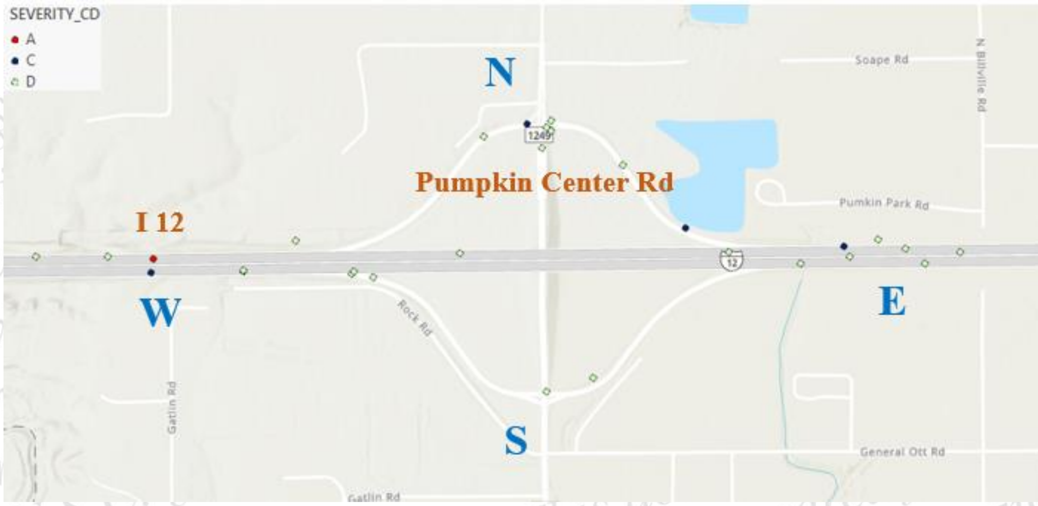


Figure 77. Distribution of severity level excluding non-injury crashes at Interchange 5



<u>Analysis of Manner of Collision</u>

Table 90 shows the manner of collisions at Interchange 5. The majority of collisions are rear-end (41.09%), followed by non-collision with motor vehicles (24.81%) and sideswipe-same direction (20.16%). Other types, such as head-on (0.78%), right angle (4.65%), and various left and right turn-related incidents, each represent less than 5% of the total.

Table 90. Manner of collision at Interchange 5

Manner of collision	Coding	Count	Percentage
Non-collision with motor vehicle	A	32	24.81
Rear-end	B	53	41.09
Head-on	C	1	0.78
Right angle	D	6	4.65
Left turn - angle	E	0	0.00
Left turn - opposite direction	F	3	2.33
Left turn - same direction	G	2	1.55
Right turn - same direction	H	0	0.00
Right turn - opposite direction	I	1	0.78

Sideswipe - same direction	J	26	20.16
Sideswipe - opposite direction	K	0	0.00
Other	Z	5	3.88
Total		129	100

Figure 78 illustrates the distribution of various collision types at Interchange 5, focusing on the five most frequent collision manners. In addition, Figure 79 highlights the hotspot locations for collisions at Interchange 5. Red boxes mark the hotspot areas. A closer analysis of the types of collisions within these hotspots reveals that rear-end collisions are the most common. This is followed by a significant number of non-collisions with motor vehicle and sideswipe-same direction collisions.

Figure 78. Distribution of manner of collision (top five highest by count) at Interchange 5

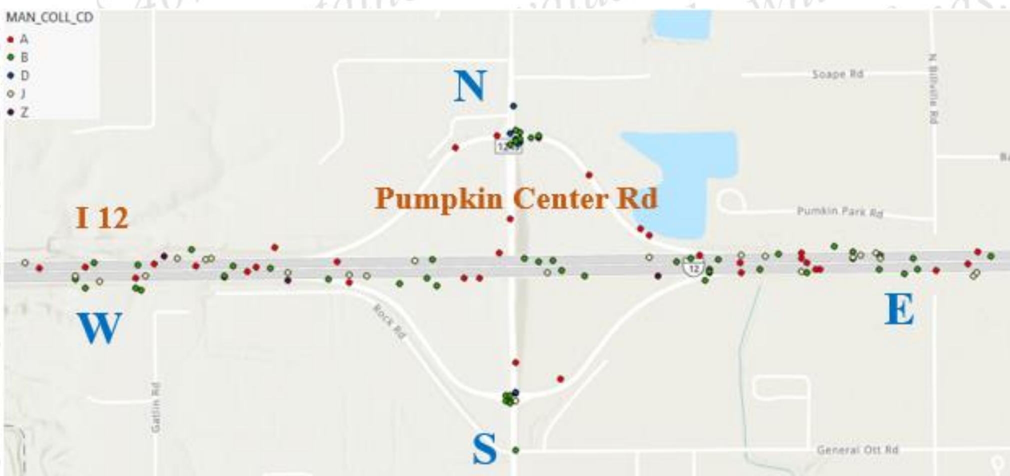
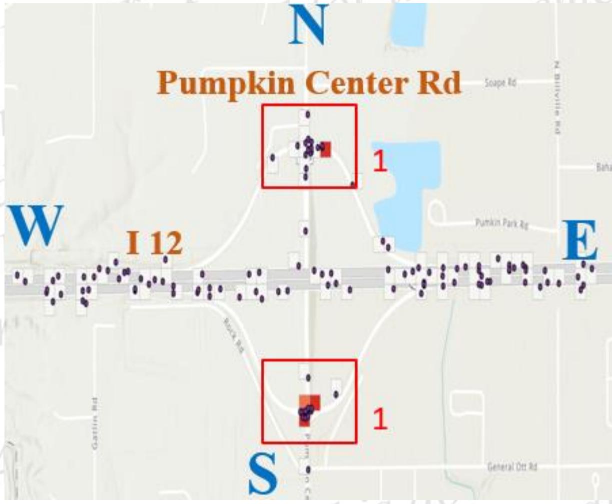


Figure 79. Distribution of crashes and hotspots at Interchange 5



Interchange 6 (Diamond with roundabout intersections)

<u>Analysis of Severity</u>

Table 91 presents the distribution of crashes per severity level at Interchange 6. The findings reveal that the majority of crashes, 82.16%, resulted in no injuries (Severity E). Fatal crashes (Severity A) are rare, accounting for 0.38%, and there are no incapacitating/severe injury crashes (Severity B). Non-incapacitating/moderate injury crashes (Severity C) make up 2.09% of the total, while crashes with possible injuries or complaints (Severity D) constitute 15.37%.

Table 91. Severity level at Interchange 6

Severity	Coding	Count	Percentage
Fatal	A	2	0.38
Incapacitating/severe	B	0	0.00
Non-incapacitating/moderate	C	11	2.09
Possible/complaint	D	81	15.37
No injury	E	433	82.16
Total		527	100

Figure 80 shows the distribution of crashes at Interchange 6, according to all severity levels, while Figure 81 illustrates the distribution of injury crashes at Interchange 6, excluding non-injury incidents.

Figure 80. Distribution of all severity levels at Interchange 6

Figure 81. Distribution of severity levels excluding non-injury crashes at Interchange 6

Analysis of Manner of Collision

Table 92 shows the distribution of crashes according to the manner of collisions at Interchange 6. The majority of collisions are rear-end (51.42%), followed by sideswipe-same direction (27.70%) and non-collision with motor vehicles (7.97%). Other types, such as right angle (3.04%), left turn-related incidents (3.80% combined), and various right turn-related incidents (2.28% combined), each represent a small fraction of the total. There are no head-on collisions or sideswipe-opposite direction collisions reported.

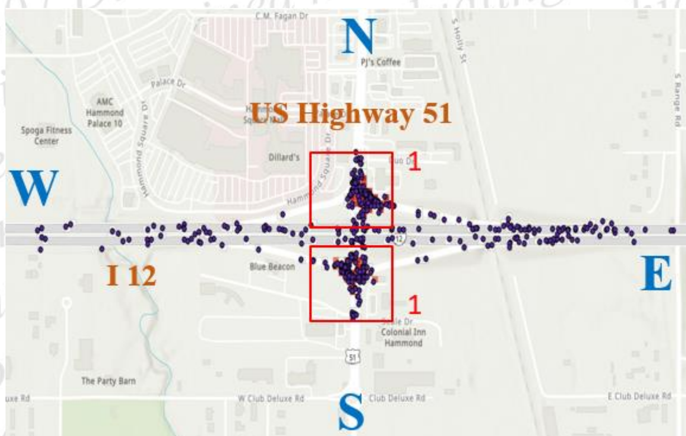
Table 92. Manner of collision at Interchange 6

Manner of collision	Coding	Count	Percentage
Non-collision with motor vehicle	A	42	7.97
Rear-end	B	271	51.42
Head-on	C	0	0.00
Right angle	D	16	3.04
Left turn - angle	E	5	0.95
Left turn - opposite direction	F	0	0.00
Left turn - same direction	G	15	2.85
Right turn - same direction	H	11	2.09
Right turn - opposite direction	I	1	0.19
Sideswipe - same direction	J	146	27.70
Sideswipe - opposite direction	K	0	0.00
Other	Z	20	3.80
Total		527	100

Figure 82 illustrates the distribution of various collision types at Interchange 6, focusing on the five most frequent collision manners. In addition, Figure 83 highlights the hotspot locations for collisions at Interchange 6. Red boxes mark the hotspot areas. A closer analysis of the types of collisions within these hotspots reveals that rear-end collisions are the most common. This is followed by a significant number of sideswipe-same direction and non-collisions with motor vehicle collisions.

Figure 82. Distribution of manner of collision (top five highest by count) at Interchange 6

Figure 83. Distribution of crashes and hotspots at Interchange 6



Interchange 7 (Diamond with signalized intersections)

<u>Analysis of Severity</u>

Table 93 presents the distribution of crashes per severity level at Interchange 7. The results indicate that the majority of crashes, 78.76%, resulted in no injuries (Severity E). Fatal crashes (Severity A) are very rare, accounting for 0.11%, while incapacitating/severe injury crashes (Severity B) make up 0.34%. Non-incapacitating/moderate injury crashes (Severity C) constitute 1.46% of the total, and crashes with possible injuries or complaints (Severity D) represent 19.33%.

Table 93. Severity levels at Interchange 7

Severity	Coding	Count	Percentage
Fatal	A	1	0.11
Incapacitating/severe	B	3	0.34
Non-incapacitating/moderate	C	13	1.46
Possible/complaint	D	172	19.33
No injury	E	701	78.76
Total		890	100

Figure 84 shows the distribution of crashes at Interchange 7, including all severity levels, while Figure 85 illustrates the distribution of injury crashes at Interchange 7, excluding non-injury crashes.

Figure 84. Distribution of all severity levels at Interchange 7

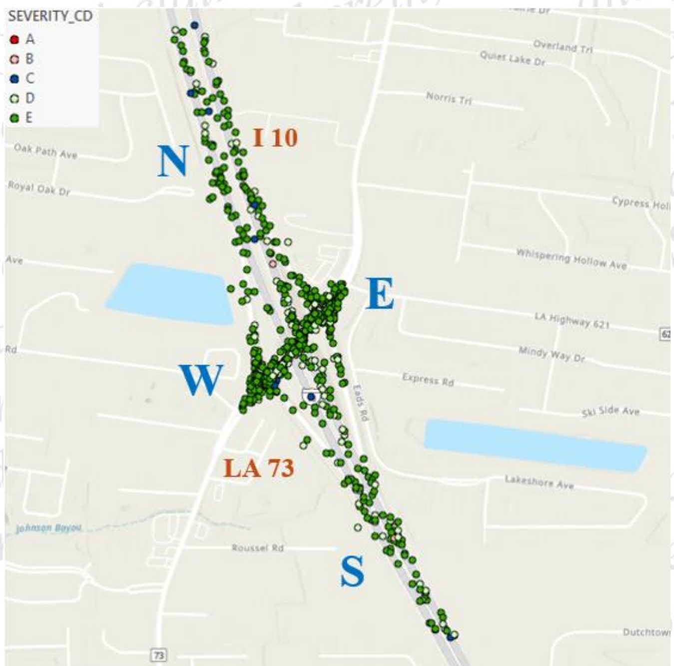


Figure 85. Distribution of severity levels excluding non-injury crashes at Interchange 7

Analysis of Manner of Collision

Table 94 shows the distribution of crashes according to the manner of collision at Interchange 7. The majority of collisions are rear-end (65.51%), followed by sideswipe - same direction (11.80%) and non-collision with motor vehicles (8.88%). Other types, such as head-on (0.34%), right angle (5.39%), and various left and right turn-related incidents (each less than 6%), represent smaller fractions of the total.

Table 94. Manner of collision at Interchange 7

Manner of collision	Coding	Count	Percentage
Non-collision with motor vehicle	A	79	8.88
Rear-end	B	583	65.51
Head-on	C	3	0.34
Right angle	D	48	5.39
Left turn - angle	E	0	0.00
Left turn - opposite direction	F	52	5.84
Left turn - same direction	G	7	0.79
Right turn - same direction	H	7	0.79

Right turn - opposite direction	I	1	0.11
Sideswipe - same direction	J	105	11.80
Sideswipe - opposite direction	K	0	0.00
Other	Z	5	0.56
Total		890	100

Figure 86 illustrates the distribution of various collision types at Interchange 7, focusing on the five most frequent collision manners. Figure 87 highlights the hotspot locations for collisions at Interchange 7. Red boxes mark the hotspot areas. A closer analysis of the types of collisions within these hotspots reveals that rear-end collisions are the most common. This is followed by a significant number of sideswipe-same direction collisions and non-collisions with motor vehicles.

Figure 86. Distribution of manner of collision (top five highest by count) at Interchange 7

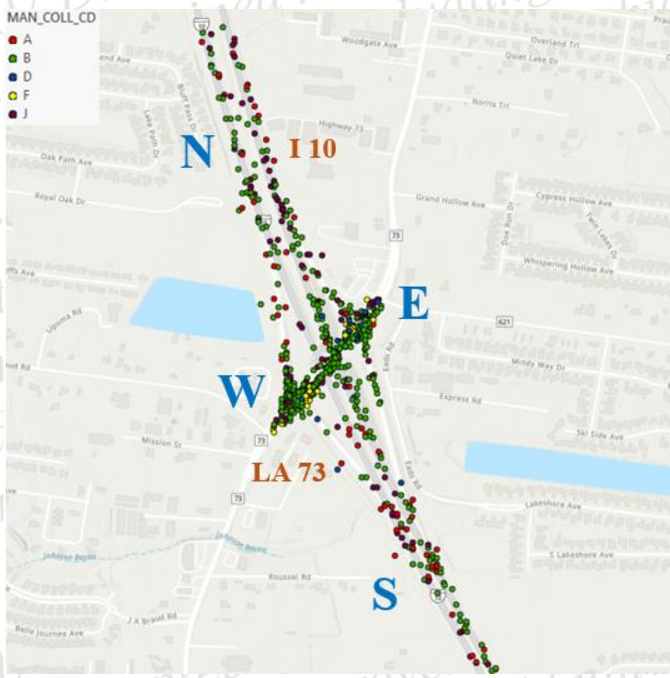
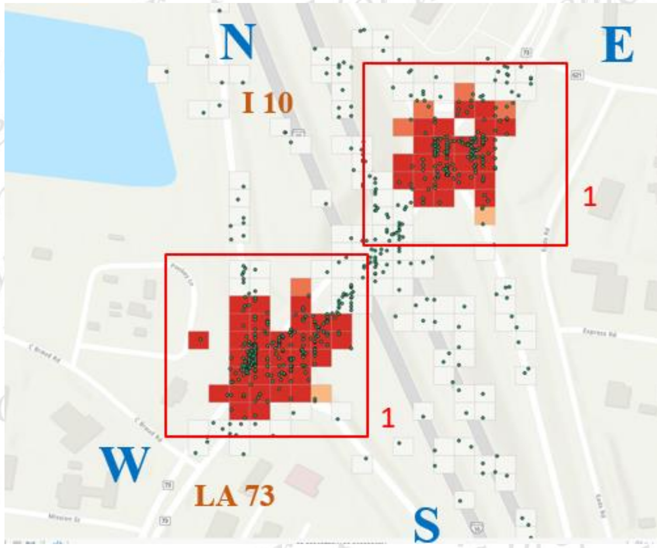


Figure 87. Distribution of crashes and hotspots at Interchange 7



Interchange 8 (Diamond with roundabout intersections)

<u>Analysis of Severity</u>

Table 95 presents the distribution of crashes categorized by severity level at Interchange 8. The findings reveal that almost two thirds of crashes, 63.49%, resulted in no injuries (Severity E). Fatal crashes (Severity A) are very rare, accounting for 0.41%, while incapacitating/severe injury crashes (Severity B) make up 1.24%. Non-incapacitating/moderate injury crashes (Severity C) constitute 6.22% of the total, and crashes with possible injuries or complaints (Severity D) represent 28.63%.

Table 95. Severity levels at Interchange 8

Severity	Coding	Count	Percentage
Fatal	A	1	0.41
Incapacitating/severe	B	3	1.24
Non-incapacitating/moderate	C	15	6.22
Possible/complaint	D	69	28.63
No injury	E	153	63.49
Total		241	100

Figure 88 shows the distribution of crashes at Interchange 8, according to all severity levels, while Figure 89 illustrates the distribution of injury crashes at Interchange 8, excluding non-injury crashes.

Figure 88. Distribution of all severity levels at Interchange 8

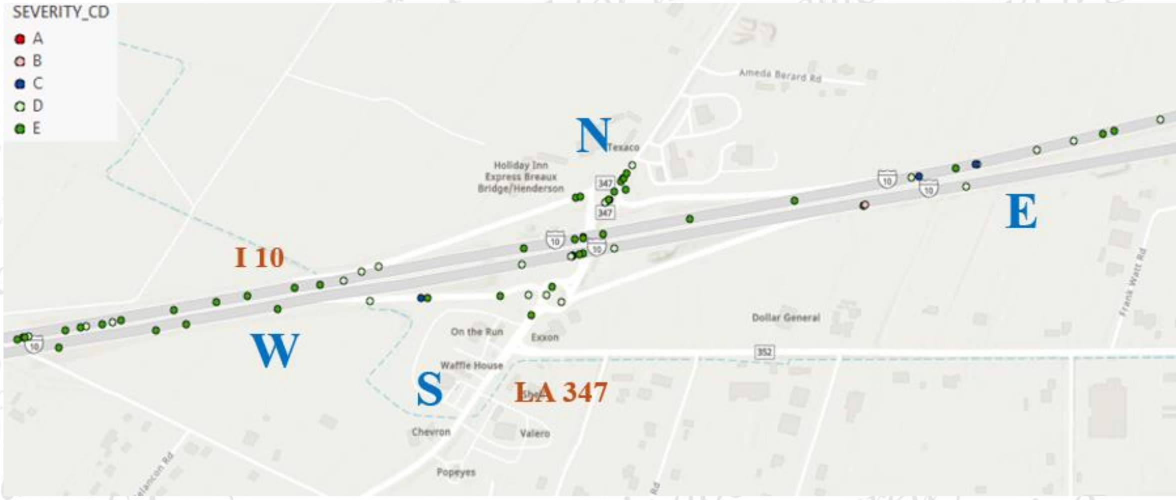
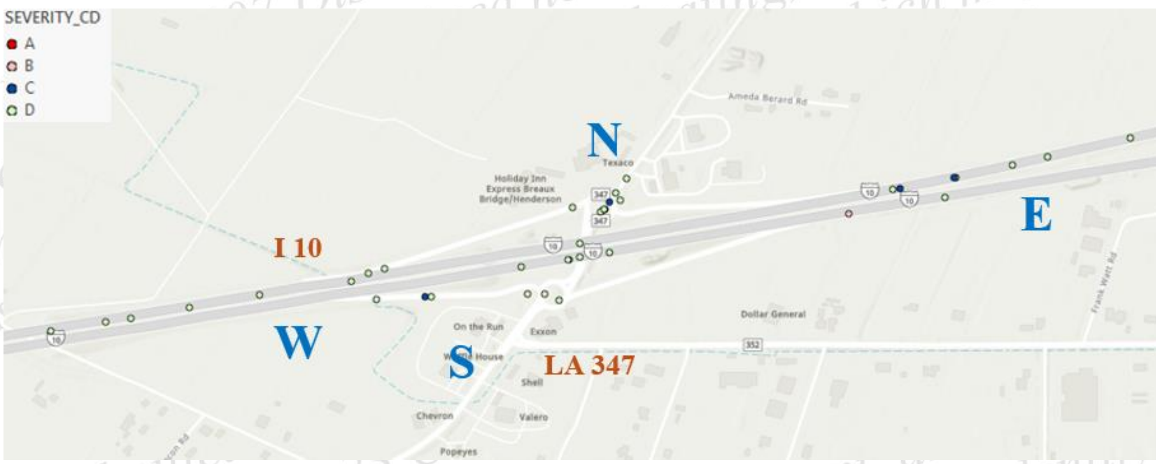


Figure 89. Distribution of severity levels excluding non-injury crashes at Interchange 8



<u>Analysis of Manner of Collision</u>

Table 96 shows the distribution of crashes according to the manner of collisions at Interchange 8. The larger proportion of collisions is rear-end (35.27%), followed by non-collision with motor vehicles (26.97%) and sideswipe-same direction (14.94%). Other types, such as head-on (0.41%), right angle (7.05%), and various left and right turn-related incidents (each less than 3%), represent smaller fractions of the total. Additionally, other collision types account for 4.15% of the total incidents.

Table 96. Manner of collision at Interchange 8

Manner of collision	Coding	Count	Percentage (%)
Non-collision with motor vehicle	A	65	26.97
Rear-end	B	85	35.27
Head-on	C	1	0.41
Right angle	D	17	7.05
Left turn - angle	E	4	1.66
Left turn - opposite direction	F	7	2.90
Left turn - same direction	G	6	2.49
Right turn - same direction	H	7	2.90
Right turn - opposite direction	I	0	0.00
Sideswipe - same direction	J	36	14.94
Sideswipe - opposite direction	K	3	1.24
Other	Z	10	4.15
Total		241	100

Figure 90 illustrates the distribution of various collision types at Interchange 8, focusing on the five most frequent collision manners. In addition, Figure 91 highlights the hotspot locations for collisions at Interchange 8. Red boxes mark the hotspot areas. A closer analysis of the types of collisions within these hotspots reveals that rear-end collisions are the most common. This is followed by a significant number of non-collisions with motor vehicles and sideswipe-same direction collisions.

Figure 90. Distribution of manner of collision (top five highest by count) at Interchange 8

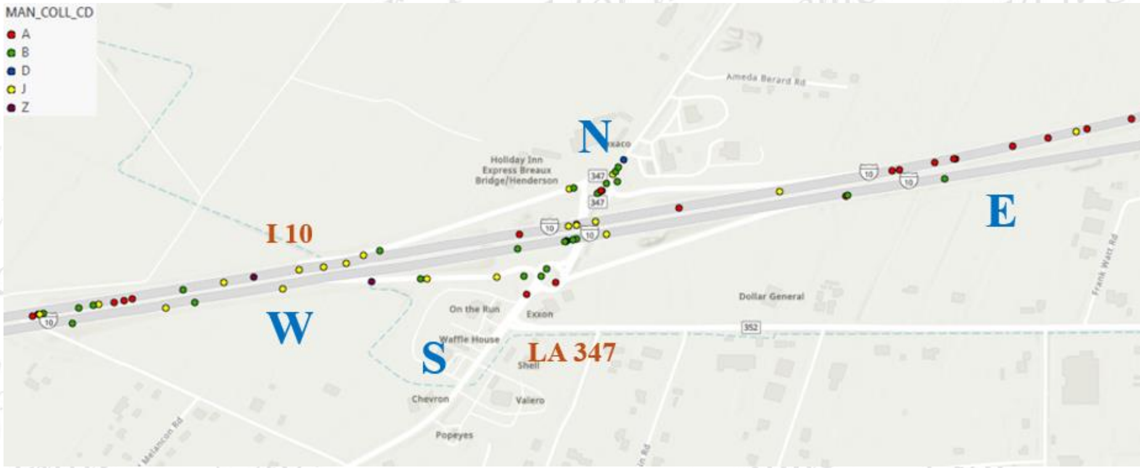
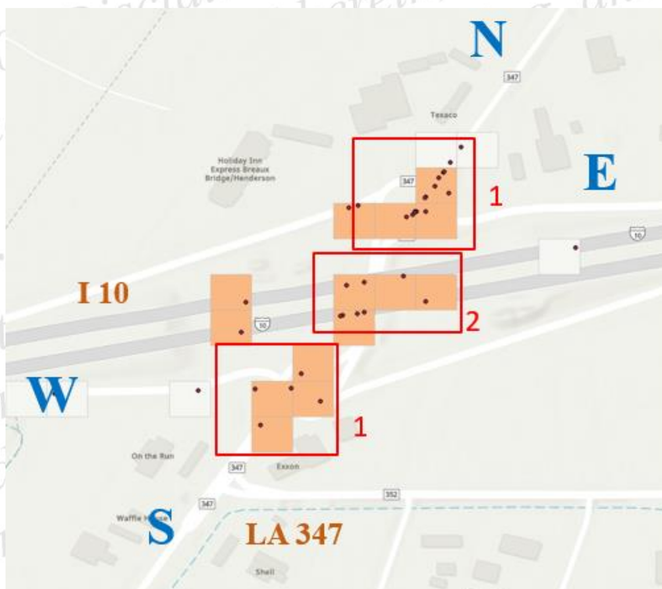


Figure 91. Distribution of crashes and hotspots at Interchange 8



Appendix L: Violations and movements prior to crashes

Interchange 2 (Cloverleaf without C-D roads)

Violations

Table 97 presents the distribution of violation types at Interchange 2. The results show that the most common violation type is careless operation, accounting for 38.0% of violations at Interchange 2. Other violation types included exceeding safe speed limit (6.9%), turning from the wrong lane (5.7%), and exceeding stated speed limit (3.1%). Other unspecified violations make up 12.7%, while 8.2% of the crashes involve no violations at all. Finally, 14.2% of the violations were classified as unknown.

Table 97. Violation types at Interchange 2

Violation type	Percentage (%)
Careless operation	38.0
Exceeding safe speed limit	6.9
Turned from wrong lane	5.7
Exceeding stated speed limit	3.1
Other (not specified)	12.7
No violations	8.2
Unknown	14.2

Movement Prior to Crashes

Table 98 presents the distribution of vehicle movements prior to crashes at Interchange 2. The findings reveal that the majority of crashes, 51.3%, occurred while vehicles were proceeding straight ahead. Changing lanes on a multi-lane road is the second most common movement before a crash, accounting for 21.2%. Other or unknown movements are involved in 9.5% of crashes. Leaving the freeway via an off-ramp accounts for 4.8% of the cases, while slowing to stop is reported in 3.7% of the crashes. Entering the freeway from an on-ramp is the movement type reported in about 3.3% of crashes and running off the road (not while making a turn at an intersection) represents 1.8% of the total crashes.

Table 98. Movement prior to crashes at Interchange 2

Movement prior to crash	Percentage (%)
Proceeding Straight ahead	51.3
Changing lanes on multi-lane road	21.2
Other or unknown	9.5
Leaving freeway via off ramp	4.8
Slowing to stop	3.7
Entering freeway from on ramp	3.3
Ran off road (Not while making turn at intersection)	1.8

Interchange 3 (Cloverleaf without C-D roads)

Violations

Table 99 presents the distribution of violation types at Interchange 3. The findings reveal that the most common violation is careless operation, accounting for 52.4% of crashes at Interchange 3. Following too closely is the second most frequent violation, with 11.9%. Cutting in and improper passing is another type of violation, reported in about 11.9% of crashes at Interchange 3. Exceeding the stated speed limit and exceeding the safe speed limit represent 5.5% and 4.5% of violations, respectively. Turning from the wrong lane is noted in 4.4% of the cases, and other unspecified violations make up 2.3% of violations at Interchange 3.

Table 99. Violation types at Interchange 3

Violation type	Percentage (%)
Careless operation	52.4
Following too closely	11.9
Cutting in, improper passing	11.9
Exceeding stated speed limit	5.5
Exceeding safe speed limit	4.5

Violation type	Percentage (%)
Turned from wrong lane	4.4
Other	2.3

Movement Prior to Crashes

Table 100 presents the distribution of vehicle movements prior to crashes at Interchange 3. The findings reveal that the majority of crashes, 39.9%, occurred while vehicles were running off the road (not while making a turn at an intersection). Changing lanes on a multi-lane road is the second most common movement before a crash, accounting for 16.4%. Entering the freeway from an on-ramp is involved in 15.4% of crashes, while proceeding straight ahead accounts for 14.8%. Leaving the freeway via an off-ramp is noted in 6.1% of the cases. Slowing to stop represents 3.5% of the crashes, and being stopped accounts for 1.0% of the total crashes.

Table 100. Movement prior to crashes at Interchange 3

Movement prior to crash	Percentage (%)
Ran off road (Not while making turn at intersection)	39.9
Changing lanes on multi-lane road	16.4
Entering freeway from on ramp	15.4
Proceeding straight ahead	14.8
Leaving freeway via off ramp	6.1
Slowing to stop	3.5
Stopped	1.0

Interchange 4 (Cloverleaf with C-D roads)

Violations

Table 101 presents the distribution of violation types at Interchange 4. The findings reveal that the most common violation type is careless operation, accounting for 29.4% of the total. Following too closely is the second most frequent violation, with 19.5%. Failure to yield is another violation type, making up 16.6% of violations. Exceeding the safe speed limit accounts for 8.6% of the violations, while other unspecified violations constitute 4.5%. Unknown violations are noted in 3.8% of the cases, and exceeding the stated speed limit represents 3.5% of the total violations.

Table 101. Violation types at Interchange 4

Violation type	Percentage (%)
Careless operation	29.4
Following too closely	19.5
Failure to yield	16.6
Exceeding safe speed limit	8.6
Other	4.5
Unknown	3.8
Exceeding stated speed limit	3.5

Movement Prior to Crashes

Table 102 presents the distribution of vehicle movements prior to crashes at Interchange 4. The findings reveal that the largest percentage of crashes, 55.9%, occurred while vehicles were proceeding straight ahead. Running off the road (not while making a turn at an intersection) was the second most common movement before a crash at 9.1%. Changing lanes on a multi-lane road is involved in 7.7% of crashes, while making a left turn accounts for 6.3% of crashes. Leaving the freeway via an off-ramp was the movement prior to 4.2% of crashes at Interchange 4. Finally, both stopping and making a right turn represent 3.5% of the total crashes each.

Table 102. Movement prior to crashes at Interchange 4

Movement prior to crash	Percentage (%)
Proceeding straight ahead	55.9
Ran off road (Not while making turn at intersection)	9.1
Changing lanes on multi-lane road	7.7
Making left turn	6.3
Leaving freeway via off ramp	4.2
Stopped	3.5
Making right turn	3.5

Interchange 5 (Diamond interchange with stop-controlled intersections)

Violations

Table 103 presents the distribution of violation types at Interchange 5. The findings reveal that the most common violation was careless operation, accounting for 30.8% of violations. Following too closely was the second most frequent violation, representing 20.0% of violations. Cutting in and improper passing made up 10.8% of violations, while exceeding the safe speed limit and failure to yield each account for 6.2% of violations. Exceeding the stated speed limit and unknown violations each represent 5.4% of the total violations.

Table 103. Violation types at Interchange 5

Violation type	Percentage (%)
Careless operation	30.8
Following too closely	20.0
Cutting in, improper passing	10.8
Exceeding safe speed limit	6.2
Failure to yield	6.2
Exceeding stated speed limit	5.4
Unknown	5.4

Movement Prior to Crashes

Table 104 presents the distribution of vehicle movements prior to crashes at Interchange 5. The findings reveal that the largest percentage of crashes, 38.3%, occurred while vehicles were proceeding straight ahead. Running off the road (not while making a turn at an intersection) and changing lanes on a multi-lane road were the second most common movements before a crash, each accounting for 21.7% of crashes. Making a left turn was involved in 5.0% of crashes. Both backing and slowing to stop represent 3.3% of the crashes, as does entering the freeway from an on-ramp.

Table 104. Movement prior to crashes at Interchange 5

Movement prior to crash	Percentage (%)
Proceeding straight ahead	38.3
Ran off road (Not while making turn at intersection)	21.7
Changing lanes on multi-lane road	21.7
Making left turn	5.0
Backing	3.3
Slowing to stop	3.3
Entering freeway from on ramp	3.3

Interchange 6 (Diamond interchange with roundabouts)

Violations

Table 105 presents the distribution of violation types at Interchange 6. The findings reveal that the most common violation type was careless operation, accounting for 26.0% of violations. Following too closely was the second most frequent violation type, representing 21.4% of violations. Failure to yield made up about 14.2% of violations. Exceeding the safe speed limit accounts for 8.7% of violations, while turning from the wrong lane constitutes 7.2% of violations. Unknown violations accounted for 4.9% of the cases, and 4.4% of crashes had no violations.

Table 105. Violation types at Interchange 6

Violation type	Percentage (%)
Careless operation	26.0
Following too closely	21.4
Failure to yield	14.2
Exceeding safe speed limit	8.7
Turned from wrong lane	7.2
Unknown	4.9
No violations	4.4

Movement Prior to Crashes

Table 106 presents the distribution of vehicle movements prior to crashes at Interchange 6. The findings reveal that almost two thirds of crashes (63.8%) at Interchange 6 occurred while vehicles were proceeding straight ahead. Changing lanes on a multi-lane road was the second most common movement before crashes, accounting for 10.3% of crashes. Other or unknown movements were involved in about 6.1% of crashes, while leaving the freeway via an off-ramp accounted for 3.3% of crashes. Both running off the road (not while making a turn at an intersection) and making a right turn represented 2.3% of crashes. Finally, backing was reported in about 1.9% of crashes at Interchange 6.

Table 106. Movement prior to crashes at Interchange 6

Movement prior to crash	Percentage (%)
Proceeding straight ahead	63.8
Changing lanes on Multi-lane road	10.3
Other or unknown	6.1
Leaving freeway via off ramp	3.3
Ran off road (Not while making turn at intersection)	2.3
Making right turn	2.3
Backing	1.9

Interchange 7 (Diamond interchange with signalized intersections)

Violations

Table 107 presents the distribution of violation types at Interchange 7. The findings reveal that the most common violation type was careless operation, accounting for 29.9% of violations. Following too closely was the second most frequent violation type, with 29.0%. Exceeding the safe speed limit made up about 9.7% of violations. Failure to yield accounts for 7.4% of the violations, while unknown violations constitute 5.1%. Disregarding traffic control is noted in 3.4% of the cases, and other unspecified violations make up 2.5% of the total.

Table 107. Violation types at Interchange 7

Violation type	Percentage (%)
Careless operation	29.9
Following too closely	29.0
Exceeding safe speed limit	9.7
Failure to yield	7.4
Unknown	5.1
Disregarded traffic control	3.4
Other	2.5

Movement Prior to Crashes

Table 108 presents the distribution of vehicle movements prior to crashes at Interchange 7. The findings reveal that over half of crashes at Interchange 7 (55.2%) occurred while vehicles were proceeding straight ahead. Slowing to stop was the second most common movement before a crash, accounting for 10.0% of crashes. Making a left turn and changing lanes on a multi-lane road were the movements prior to about 9.0% and 6.6% of crashes at Interchange 7, respectively. Running off the road (not while making a turn at an intersection) represents 6.3% of crashes. Leaving the freeway via an off-ramp was reported in 3.6% of the crashes, and entering the freeway from an on-ramp accounted for 2.0% of crashes.

Table 108. Movement prior to crashes at Interchange 7

Movement prior to crash	Percentage (%)
Proceeding straight ahead	55.2
Slowing to stop	10.0
Making the left turn	9.0
Changing lanes on multi-lane road	6.6
Ran off road (Not while making turn at intersection)	6.3
Leaving freeway via off ramp	3.6
Entering freeway from on ramp	2.0

Interchange 8 (Diamond interchange with roundabouts)

Violations

Table 109 presents the distribution of violation types at Interchange 8. The findings reveal that the most common violation type was careless operation, accounting for 26.8% of violations. Following too closely was the second most frequent violation, representing 12.1% of violations. Failure to yield made up about 10.9% of violations. Exceeding the safe speed limit and issues with vehicle condition each account for 4.2% of violations. No violations were noted in 18.0% of crashes while other unspecified violations constitute 6.7% of crashes.

Table 109. Violation types at Interchange 8

Violation type	Percentage (%)
Careless operation	26.8
No violations	18.0
Following too closely	12.1
Failure to yield	10.9
Other	6.7
Exceeding safe speed limit	4.2
Vehicle condition	4.2

Movement Prior to Crashes

Table 110 presents the distribution of vehicle movements prior to crashes at Interchange 8. The findings reveal that over half of crashes at Interchange 8 (54.5%) occurred while vehicles were proceeding straight ahead. Making a left turn was the second most common movement before a crash, accounting for 8.4% of crashes. Running off the road (not while making a turn at an intersection) was reported in about 7.7% of crashes, while entering traffic from a private lane or driveway accounted for 5.6% of crashes. Both backing and changing lanes on a multi-lane road were reported in about 3.5% of the cases each. Other or unknown movements made up 2.8% of the total crashes.

Table 110. Movement prior to crashes at Interchange 8

Movement prior to crash	Percentage (%)
Proceeding straight ahead	54.5
Making left turn	8.4
Ran off road (Not while making turn at intersection)	7.7
Entering traffic from private lane or driveway	5.6
Backing	3.5
Changing lanes on multi-lane road	3.5
Other or unknown	2.8